Frozen soil properties modification in the context of climate change

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ABSTRACT

One of the expected effects of global warming is the gradual melting of permanently frozen ground, known as permafrost. The melting of permafrost will significantly impact soil material properties and (pore) fluid flow, potentially causing instability of infrastructures and triggering natural hazards such as landslides, rock falls, or debris flows. The overall objective of this research is to experimentally quantify the effect of thawing on the geomechanical strength of a reconstituted soil. More specifically, it is intended to qualify the initial frozen state and compare it to the state after thawing. The proposed experimental study was carried out in three steps. Firstly, soil samples were identified by the usual parameters such as water content, bulk density, solid particle density, triaxial test etc. Permafrost temperatures recorded by PERMOS and PermaFrance in the Alps (Swiss and French) are of the order of -3 to -4.5°C at a depth of -10m. This is why, for the second step, the shear tests were carried out at a slightly lower temperature than that recorded in situ. Artificial samples were sheared at a temperature of -5°C in our temperature controlled triaxial press in order to determine the soil's strength parameters (cohesion and internal angle of friction). Thirdly, same tests were carried out at a temperature of +5°C in order to thaw the soil completely before the shearing. In total, three tests for each temperature were compared and discussed. The expected results aim at a better understanding and quantification of soil strength reduction after the thawing phase. As many infrastructures are now built on permafrost, such as roads, ski infrastructures, power lines or alpine chalets, they will be affected by this phenomenon in the near future. A better understanding of (geo)mechanical consequences might facilitate risk analysis, evaluation and mitigation.

Keywords: Frozen soil, Temperature Controlled Triaxial test, shear strength

1. Introduction

The thaw of permafrost has many harmful impacts for the community: an increased risk of natural hazards, but also a drastic variation of soil strength over the thawing process, which may cause the loss of service ability or even failure of engineering facilities.

Permafrost areas have been widely identified in Alaska, Greenland, Canada, Siberia or the Arctic, but alpine regions such as Switzerland are also concerned at altitudes where the average air temperature is low enough to keep the ground frozen. The issue of global warming has encouraged research into the thermo-hydro-mechanical consequences of a future thaw around the world. Zhang & al. conducted studies on the response of Canadian permafrost after a rise of aerial temperature. The simulation results showed that, at a depth of 0.2m, a permafrost temperature increase of 2.1°C, respectively 5.1°C was expected, for and aerial temperature increase of 2.8°C, respectively 7°C (Zhang, et al., 2008). Several predictions of global warming tend to an increase of air temperature to 2°C to 4°C in 2100 (Hipp, et al., 2011; Guo, et al., 2012; Lyon, et al., 2022). The temperature of the permafrost will follow the increase of the air temperature. This rise in the soil's temperature progressively shrinks the extent of permafrost and increases the danger areas.

The permafrost degradation due to the global warming effect has a large incidence on natural hazards incurring in mountains such as debris flows, landslides, rock falls, etc. (Haeberli, 1992; Isaksen, et al., 2001; Isaksen, et al., 2002; Etzelmüller, et al., 2003; Isaksen, et al., 2011; Farbrot, et al., 2013; Blikra & Christiansen, 2014; Myhra, et al., 2017; Frauenfelder, et al., 2018; Matthews, et al., 2018). Risk maps have been drawn up in Russia, based on the effects of global warming on permafrost (Perov, et al., 2017). Other studies (Saemundsson, et al., 2018; Sattler, et al., 2011; Stoffel, et al., 2011; Damm & Felderer, 2013; Bardou, et al., 2011) pointed out the impact of degrading permafrost on the triggering of debris flows. The evolution of the rocky glacier dynamics due to the rise of the temperature (Arenson, et al., 2002). can explained the frequency- and amplitude increase of the debris flows. Similar studies have been conducted in China (Ding, et al., 2019) and in the Norwegian sub-arctic permafrost (Hilger, et al., 2021). This degradation is clearly discernible in mountainous countries. For example, in Switzerland we observe the gradual permafrost thawing, thus increasing its instability. This instability can manifest by very impressive phenomena such as the collapse of the Piz Cengalo in 2017 causing the death of eight people. The consequences of the thaw can have repercussions involving people's lives, which is why it is important to study and understand this phenomenon.

The study of permafrost in Switzerland is an active field of research, mainly since the ‘70s. A triggering fact was the instability of the ice-saturated scree slopes of the Grande Dixence hydroelectric plant. The study of the phenomenon is very complex and covers many aspects. Chamberlain & Gow have studied in 1979 the evolution of the permeability of four fine-grained soils after a cycle of freeze / thaw. The results are surprising as it was found that the void ratio decreases and the vertical permeability increases. The process depends on the type of soil and no definite relationships have been established (Chamberlain & Gow, 1979). Scientists have also conducted studies on the formation of ice lenses during freezing (Dysli, 1993) and on the evolution of isotherms during freezing, the heat transfers, the water flow during the formation of the ice lenses, the evolution pressures / capillary suctions or the swelling of the ground

Arenson, Johanson and Springman have proposed mathematical formulations to describe the thermomechanical behavior of ice-rich frozen soils at temperatures close to 0°C. They highlighted how ice content influences the apparent cohesion and the angle of internal friction. Moreover, apparent cohesion seems to be influenced by the temperature and the applied compression strain rate (Arenson & Springman, 2005). However, only limited analyses of apparent cohesion and friction angle have been carried out and further work is required.

This paper describes a set of triaxial shear tests that were performed at a temperature of -5°C and at a temperature of +5°C after a cycle of freezing / thawing. The samples were prepared artificially in order to have controlled and repeatable conditions. The focus of the investigation presented in this paper was to determine the effect of the temperature on the cohesion and the internal friction angle evolution when samples of fine soil pass from a frozen state to a thawed state.

2. Current state of the applicants’ research

The loss of soil bearing capacity after thawing was the subject of a preliminary study via a Bachelor's thesis in civil engineering at the School of Engineering and Management of Vaud (HEIG-VD) in 2017.

Direct shear tests at the Casagrande’s box and oedometric tests were carried out on reconstituted samples before a freezing and after a thawing phase. The soil used in this study came from the washing sludge of the La Poissine gravel located on the Neuchâtel Lake shore. In a first step, a classification of the soil was performed. Analyses have identified the soil as a silt ML (USCS classification). Then, the student established a protocol to prepare artificially samples under repeatable conditions. These samples were used to carried out direct shear tests and oedometric tests. In order to freeze the soil, samples were put in a climatic chamber at a temperature of -15°C during 18 days. Samples were thawed at room temperature (approximately 20°C) during a day.

Direct shear tests were performed on specimens for the three states of the soil: before freezing, during freezing and after thawing. For technical reasons, oedometric tests were carried out only for the states before freezing and after thawing. The time allowed for this diploma work was limited to 10 weeks, reducing the number of results. In addition, many samples were destroyed during thawing (about 40%), which further reduced the number of usable tests. A total of 3 oedometer tests and 24 direct shear tests yielded usable results. Due to time restrictions, shortcuts had to be used to get the results in time.

We were able to show that the shear strength was reduced by 20 to 40% after thawing and that the compressibility increased by 41% (Torche & Prina Howald, 2017; Prina Howald & Torche, 2020). This is a clear indication of the potential negative effect of thawing on infrastructures.

3. Soil properties and test program

3.1. Classification

The test soil used for this study was collected from the washing sludge of a lacustrine gravel near the Neuchâtel Lake (Figure 1). Soil physical properties were measured according to Swiss standards VSS and listed in table 1. The soil is composed by 3% of sand, 69% of silt and 28% of clay. According to the USCS classification, the soil is a clay loam CL. (Figure 2). The frost sensitivity is very high: G4. According to the compaction test, the maximum dry density and optimum water content were 1.831 Mg/m3 and 14.4% respectively.

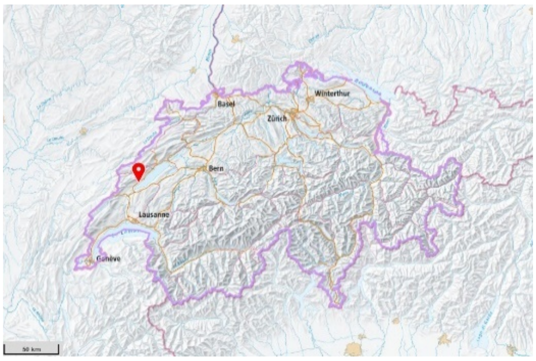

Figure 1. Localisation of the gravel in Switzerland

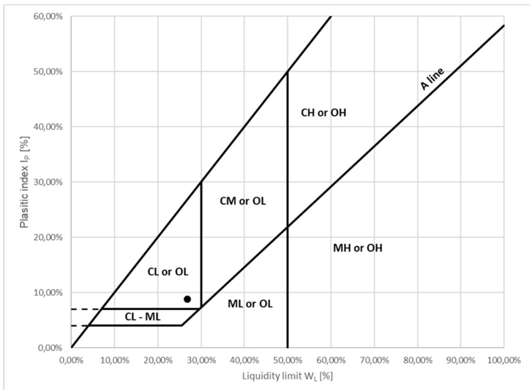

Figure 2. USCS classification

Table 1. Physical properties of tested clay loam

Sand [%]	Silty [%]	Clay [%]	Liquid limit [%]	Plastic limit [%]	Specific gravity [Mg/m^3]
3.0	69.4	27.6	27.2	18.5	2.74

3.2. Sample preparation

The samples were prepared artificially under repeatable conditions in order to limited the natural heterogeneity of the soil. Thus, the variations observed between tests were only due to the effect of temperature and not to an error in samples preparations. All samples were compacted with a water content of 20%, which correspond to a degree of saturation of 85% approximately. The saturation of samples was made during the triaxial test. Water was mixed with powder soil to achieve the target water content with a mechanical mixing machine. Then the soil was stored in a plastic bag until the water was homogenously redistributed. The soil was compacted in a A-type Proctor mould (L=119 mm and Ø=101mm) with an energy compaction of 0.6MJ/m^3. This energy is used for the standard compaction test. Our compacter is set up for this energy, that is why we chose 0.6Mj/m^3 compaction energy in order to limit the human error during the confection of the samples. After extraction from the Proctor mould, the samples were cut in order to obtain a cylinder of 100mm length and 50mm diameter (Figure 3).

Figure 3. Sample preparation

Several tests were performed on artificial samples before freezing, including a CU+u triaxial test, to determine the reference properties of the specimens and are listed in table 2.

Table 2. Reference properties

γ [Mg/m^3]	w [%]	Sr [%]	e_0 [-]	c_u [kPa]	φ_u [°]
2.01	20.1	86.4	0.639	23.9	22.4

3.3. Test plan

To quantify experimentally the effect of thawing on a frozen soil's properties, triaxial compression tests with a temperature control were carried out. Three triaxial tests at a temperature of -5°C were performed in our triaxial press and three tests were performed at a temperature of 5°C. As the samples were frozen, no water circulation can take place during compression test, that is why a consolidated and drained (CD) test was impossible to carry out. The choice of the test type was therefore made for a consolidated undrained test (CU) for frozen sample and a consolidated undrained test with pore water pressure measurement (CU+u) for the thawed samples. In order to limit the results' variation due to the test parameters, the shear speed was fixed to 0.02mm/min. It was the same speed as the initial triaxial test. To begin the test, samples were saturated (Sr>95%). The cell pressure is gradually increase to 660 kPa while maintaining an effective pressure of 10 kPa. Then, samples were consolidated during 24 hours. The back pressure was maintained at 650 kPa and the cell pressure was increased to 750 kPa, 850 kPa, 1050 kPa and 1250 kPa. As these tests were innovative, we decided to use four samples per tests and we have taken the three best Mohr circles for the exploitation of the results. After the consolidation step, samples were frozen at -5°C temperature during 24 hours. The freezing system is pretty simple. The coil surrounding the sample is connected to a thermal system (Figure 4).

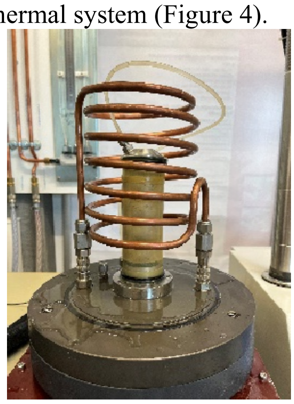

Figure 4. System assembly

When the thermal system is switched on, a liquid flow through the closed system and cools or heats the liquid in the cellule. It is composed of water and antifreeze to maintain a liquid state during the cooling of the system and to allow the application of the cell pressure. A thermal sensor is integrated into the system to monitor the temperature of the containment fluid. The temperature of the containment fluid drops from 20°C to -5°C in approximately 1h15. After 24 hours at -5°C, the samples were sheared in the press. During the shear phase, the chamber pressure was maintained constant. The shear strength was measured every 0.1% of deformation.

For the tests after thawing, the samples were frozen following the same procedure as for the frozen tests, then thawed at +5°C during 12 hours before shearing.

After exploitation of the shear strength curves, we were able to determine the cohesion and the internal friction angle of the soil to compare the state during and after freezing.

4. Test results

4.1. Triaxial test at -5°C

Reporting and interpreting the stress-strain response of the samples require some choices to describe the results. Indeed, the interpretation of the data is not straightforward as the samples were frozen during the triaxial test. A standard triaxial test interpretation is not possible for this type of test due to the ice in the specimens. Since water in the samples was in ice state, no pore water pressure can be measured. That is why, the calculations were made in total stress. Moreover, we can assume that the sample was totally frozen and can be considered as an impervious rock. Thus, we have considered the cell pressure as the effective pressure.

The failure criterion adopted in this study was the peak of deviatoric stress if it was clearly visible. Otherwise, values of deviatoric stress for a displacement of 15% were taken. Some deviatoric stress curves did not show a clear increase upon loading (Figure 5). This effect is due to the engage system of the triaxial press and was considered when measuring the peak of deviatoric stress.

As triaxial tests on frozen ground are rare, we decided to carry out four shear tests per test in case one point was totally inconsistent with the others. This choice proved to be very useful, because as expected, some points were discarded in order to obtain consistent results. Figures 5, 6 and 7 show the curves of deviatoric stresses. Table 3 summarises the raw results of the three triaxial tests at -5°C.

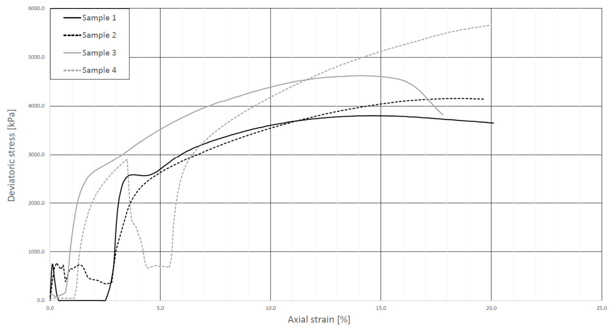

Figure 5. Deviatoric curves, test 1 at -5°C

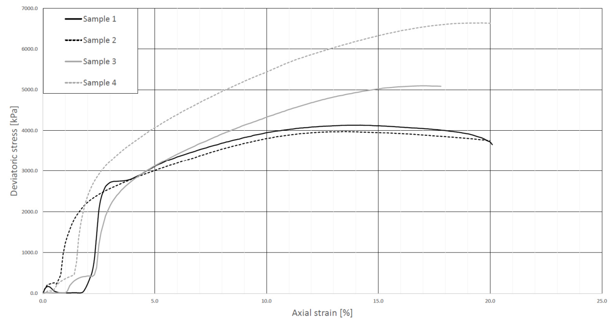

Figure 6. Deviatoric curves, test 2 at -5°C

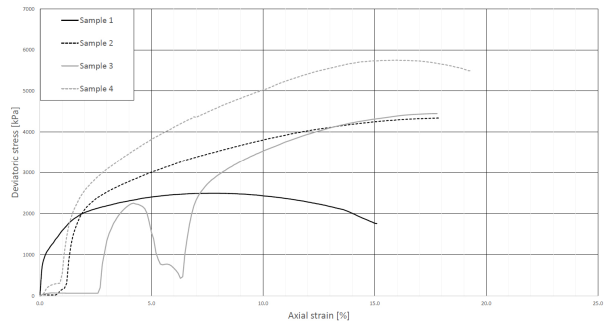

Figure 7. Deviatoric curves, test 3 at -5°C

Table 3. Raw results of tests at -5°C

		Effective pressure [kPa]	Deviatoric stress peak [kPa]	Axial strain peak [%]
Test 1	sample 1	750.8	3741.5	17.5
	sample 2	850.9	4148.5	17.8
	sample 3	1050.4	4559.0	15.7
	sample 4	1250.3	5262.3	16.1
Test 2	sample 1	750.7	4055.6	16.8
	sample 2	850.5	3932.9	15.7
	sample 3	1050.4	5096.8	17.2
	sample 4	1250.3	6487.9	16.4
Test 3	sample 1	750.8	2497.2	8.0
	sample 2	850.4	4303.1	16.2
	sample 3	1050.4	4449.5	17.8
	sample 4	1250.4	5479.1	16.0

Table 4. Table 1 Raw results of test at 5°C

		Effective pressure [kPa]	Deviatoric stress peak [kPa]	Axial strain peak [%]
Test 1	sample 1	99.5	295.4	17.0
	sample 2	199.7	479.4	15.7
	sample 3	399.9	845.8	19.6
Test 2	sample 1	99.8	268.8	15.6
	sample 2	199.7	467.1	15.5
	sample 3	399.8	857.5	16.3
Test 3	sample 1	99.7	262.0	16.3
	sample 2	199.8	424.0	15.7
	sample 3	399.9	808.2	17.1

4.2. Triaxial test at +5°C

The exploitation of these tests was more usual. Triaxial tests which were made at +5°C were easier to interpret. Indeed, water inside the specimens returned to a liquid phase and the pore water sensor was able to record useable data. As for the tests at -5°C, deviatoric stress peaks were taken at 15% of displacement (plus correction) if no peaks were observed. Figures 8, 9 and 10 show the curves of deviatoric stresses and table 4 summarises the raw results of the three triaxial tests at +5°C.

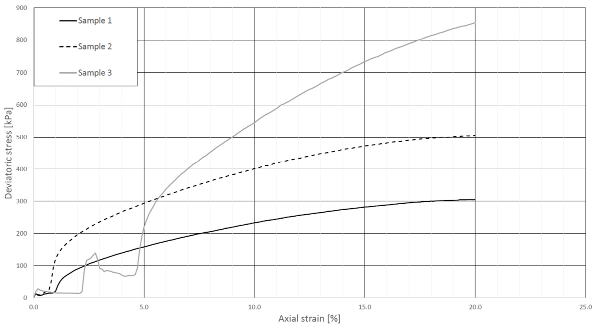

Figure 8. Deviatoric curves, test 1 at 5°C

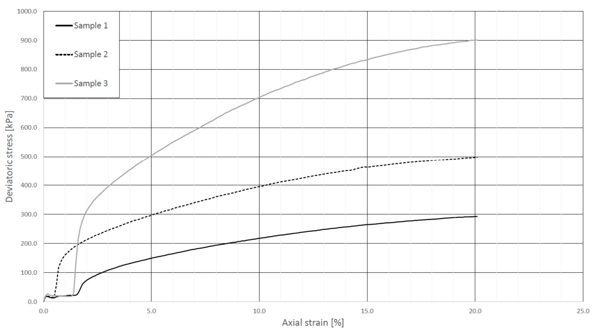

Figure 9. Deviatoric curves, test 2 at 5°C

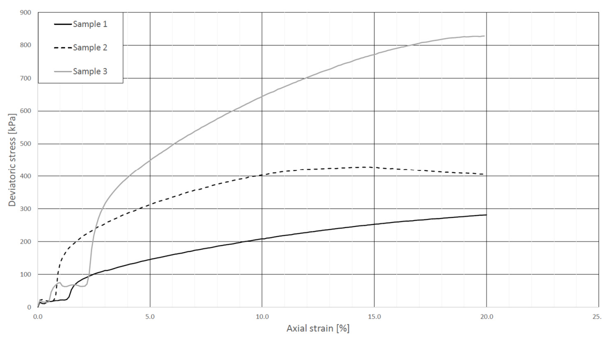

Figure 10. Deviatoric curves, test 3 at 5°C

5. Discussion

Although the temperature of the samples was negative, the behavior of the soils was not brittle when the shear speed was slow (0.02mm/min). Relatively few specimens showed a clearly visible shear plan. The most highlighted shear plan was the sample 3 from test 1 at -5°C (figure 11). The other specimens showed predominantly a failure in barreling mode.

As mentioned before, the exploitation of the freezing triaxial test results was difficult. Figures 12, 13 and 14 are the representation of Mohr's circles for the three tests carried out at a temperature of -5°C. As mentioned in chapter 4, we have considered the frozen samples as a single-phase block, which implies that the cell pressure is equal to the effective pressure of the sample. For each test, sigma 3 is equal to the confining pressure and sigma 1 is the sum of sigma 3 and the deviatoric stress peak.

The difficulty arises in drawing the Mohr-Coulomb line. Indeed, the alignment of Mohr's circles is not perfect for tests 2 and 3 at -5°C and it was relatively difficult to draw a tangent line to Mohr's circles. In contrast, the first test at -5°C was pretty easy to exploit. The Mohr-Coulomb line was tangent to the four circles as it shows in figure 12.

Figure 11. Sample 3 of test 1 at -5°C

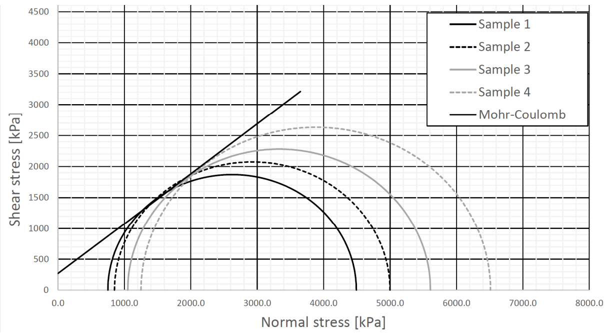

Figure 12. Test 1 at a temperature of -5°C

It can be seen in figure 13 that the Mohr's circle drawn with the results of sample 2 is smaller than the circle of sample 1. There were therefore two possible Mohr-Coulombs lines, the first being the tangent to circles 1, 3 and 4 and the second being the tangent to circles 2, 3 and 4. The second solution was discarded because it gave a soil cohesion value below 0 kPa, which is impossible. We can therefore conclude that the shearing of sample 2 encountered a problem that led to a low shear strength.

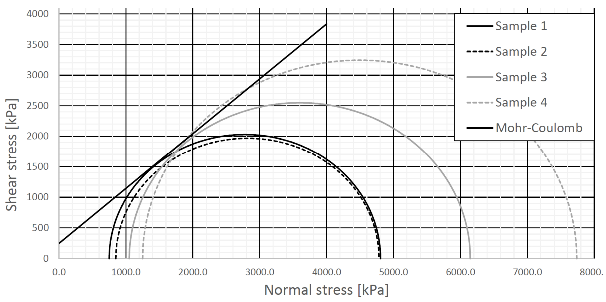

Figure 13. Test 2 at a temperature of -5°C

The exploitation of the 3rd test at -5°C was the most difficult to do. In order to visualize the problem two Mohr's Coulomb lines have been drawn in figure 14. A first Mohr-Coulomb line was drawn from circles of samples 1, 3 and 4. Although the line was perfectly tangent to the three circles, this line has been discarded for the following reasons. Firstly, the deviatoric stress curve of sample 1 was odd. The deviatoric stress peak of this sample has been achieved for 8% axial strain rate. This value is abnormally low compared to all other tests (approximately two times lower), especially since no clear failure is visible. It is likely that the sample had a design flaw, leading to low shear strength. Secondly, the circle from the results of the third sample is clearly not usable. We can see on figure 14 that the Mohr's circle is smaller than sample 2 for a higher effective stress. This is probably due to the pre-failure visible on figure 7 at 4.5% of deformation. Thirdly, the first Mohr-Coulomb line was not correct because it led to a negative cohesion. That is why we have drawn a second Mohr-Coulomb line tangent to the circles 2 and 4. This line led to a realistic result.

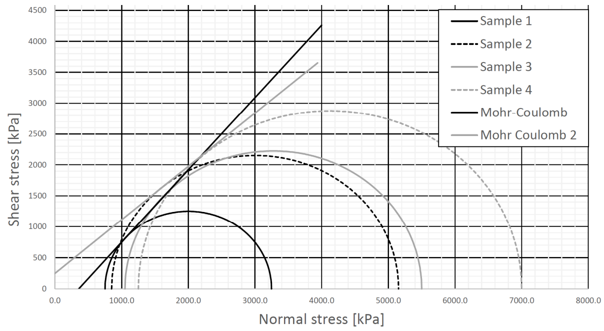

Figure 14. Test 3 at a temperature of -5°C

As a reminder, the cohesion is the ordinate at the origin of the Mohr-Coulomb line and the angle of internal friction is the slope of the line. Table 5 summarize the results of the tests at -5°C. The three tests gave very similar results of cohesion and internal friction angle. As expected, the cohesion of the soil increased considerably during the freezing phase. Compared to its initial state, the cohesion of the soil is 10.6 times higher and the internal friction angle is 1.8 times higher. The soil's changing of phase has a great incidence on its mechanical properties.

Table 5. Summary of cohesion and friction angle during freezing

	Cohesion [kPa]	Internal friction angle [°]
Test 1	268.97	38.89
Test 2	245.47	41.91
Test 3	246.95	40.84
Mean	253.80	40.55
Standard deviation	13.16	1.53

The exploitation of the test after the cycle of freezing at -5°C and after thawing at +5°C did not present major difficulties. The alignment of the Mohr-Coulomb lines is almost perfect on all three tests (figure 15, 16 and 17). On top of that, no anomalies have been detected on the deviatoric curves.

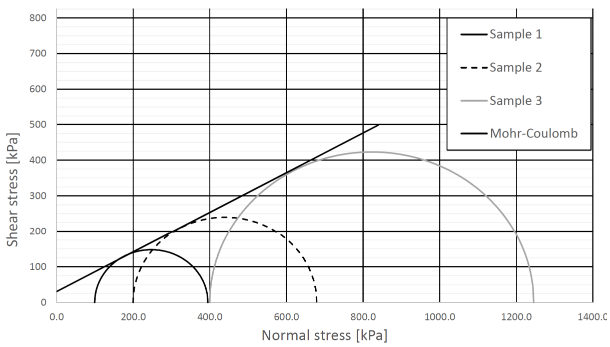

Figure 15. Test 1 at a temperature of 5°C

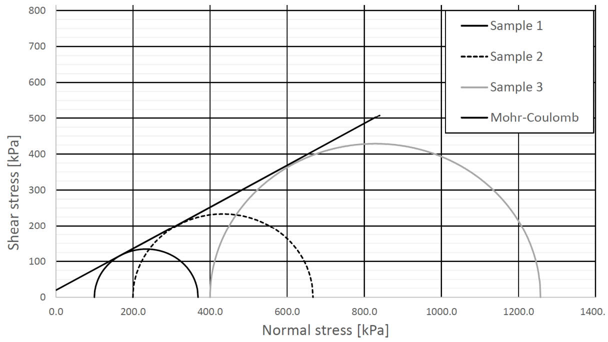

Figure 16. Test 2 at a temperature of 5°C

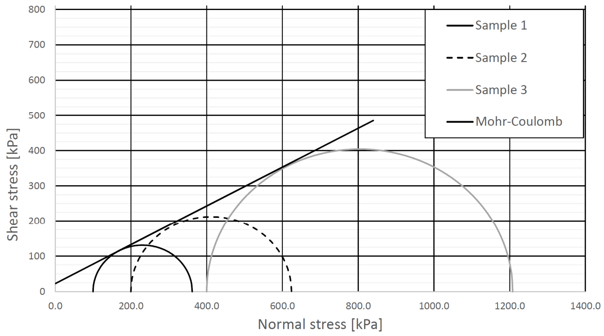

Figure 17. Test 3 at a temperature of 5°C

Table 6 summarizes the results of the tests at +5°C. As expected, the results decreased after the specimens thawed. During the thawing process, the cohesion returned to a close value of its initial state. Nevertheless, it is slightly lesser than the initial state (-0.3kPa). We can, however, question the significance of this decrease. Moreover, the standard deviation of the cohesion is quite high and covers the difference between the two values.

Although the effect of one freeze / thaw cycle does not seem to affect significantly the cohesion value, the same is not true for the internal friction angle. Strong changes in the value of the internal friction angle can be identified in all states. When the sample is frozen, friction angle increases by about 81% compared to its initial state. Then during thawing, friction angle decreases by 27.4%. However, after the freeze-thaw cycle, the value of friction angle remains about 30% higher than its pre-freeze state. The results obtained by the six tests (3 during freezing, 3 after thawing) are very close and the standard deviations are low. A phenomenon in the soil occurs during the freezing phase that permanently changes the initial internal friction angle. We can make several hypotheses as to its origin. A probable rearrangement of the solid skeleton during the gel phase is one of them. To determine more precisely the cause of this phenomenon, further studies are necessary.

It should also be noted that when the soil goes from the frozen to the thawed state, its mechanical properties decrease very sharply (11-fold decrease in cohesion and 30% decrease in friction angle). These changes lead to a sharp decline in the bearing capacity of the soil. The loss of bearing capacity also depends on other factors that are specific to each infrastructure (type of foundation, depth of foundation etc.) and therefore cannot be accurately quantified in a general way.

Table 6. Summary of cohesion and friction angle after thawing

	Cohesion [kPa]	Internal friction angle [°]
Test 1	30.54	29.12
Test 2	17.64	30.39
Test 3	22.60	28.84
Mean	23.59	29.45
Standard deviation	6.51	0.83

6. Conclusion

This study aimed to experimentally quantify the effect of thawing on a reconstituted soil's geomechanical strength. For this purpose, a classification of soil geomechanical parameters was first established to serve as a reference point. Then, three triaxial shear tests at a temperature of -5°C were performed. This temperature was chosen to be slightly lower than that measured in permafrost monitoring boreholes in the Alps (Switzerland and France). Three additional tests were carried out at a temperature of 5°C after being frozen for 24 hours at a temperature of -5°C.

These tests highlighted the importance of soil condition (frozen/thawed) on soil cohesion. Indeed, cohesion varies by a factor of 11 between these two states. Nevertheless, the value of the cohesion seems to return to its initial state after thawing. Soil cohesion seems to depend only on the state of the soil (frozen/thawed) but it would be interesting to perform additional tests at different temperatures to confirm this hypothesis.

The variation of the soil's friction angle is more subtle. These variations are less marked than for the cohesion. When the soil thaws, friction angle decreases by about 30% of its value. It should be noted that this angle increases sharply when the soil is frozen (+81% compared to its initial state). The most surprising thing is that the value of friction angle does not return to its initial state, unlike cohesion. This may indicate two things:

- Either the freeze-thaw cycle causes a fundamental change in soil properties. The soil cannot return to its original mechanical properties.
- Either the mechanical parameters of the soil are strongly temperature dependant.
- Or a combination of these two facts.

This study should also be completed with additional tests at other temperatures to verify these hypotheses.

Understanding the impacts of permafrost thaw is complex. That is why, further studies of this project are strongly recommended. The more studies are carried out on the subject, the more we will be able to understand the impact of global warming on permafrost, and therefore the impact on human kind and their environment.

References

Arenson, L., Hoelzle, M. & Springman, S., 2002. Borehole deformation measurements and internal structure of some rock glacier in Switzerland. Permafrost and periglacial processes, Volume 13, pp. 117-135. https://doi.org/10.1002/ppp.414

Arenson, L. & Springman, S., 2005. Mathematical descriptions for the behaviour of ice-rich frozen soils at temperatures close to 0 °C. Canadian Geotechnical Journal, pp. 431-442. https://doi.org/10.1139/t04-109

Bardou, E., Favre-Bulle, G., Ornstein, P. & Rouiller, J.-D., 2011. Influence of the connectivity with permafrost on the debris-flow triggering in high-alpine environment. Engineering Geology and the Environment, Volume 10, pp. 13-21.https://doi.org/10.4408/IJEGE.2011-03.B-002

Blikra, L. H. & Christiansen, H. H., 2014. A filed-based model of permafrost-controlled rockslide deformation in northern Norway. Geomorphology, Volume 208, pp. 34-49. https://doi.org/10.1016/j.geomorph.2013.11.014

Chamberlain, E. & Gow, A., 1979. Effect of Freezing and Thawing on the Permeability and Structure of Soils. Developments in Geotechnical Engineering, Volume 26, pp. 73-92. https://doi.org/10.1016/B978-0-444-41782-4.50012-9

Damm, B. & Felderer, A., 2013. Impact of atmospheric warming on permafrost degradation and debris flow initiation - a case study from the eatern European Alps. *Quaternary Science journal,* 62(2), pp. 136 - 149. https://doi.org/10.3285/eg.62.2.05

Ding, Y. et al., 2019. Global warming weakening the inherent stability of glaciers and permafrost. *Science bulletin,* Volume 64, pp. 245 - 253. https://doi.org/10.1016/j.scib.2018.12.028

Dysli, 1993. *Where does the water go during ice lenses thaw?.* Anchorage, s.n., pp. 45-50.

Etzelmüller, B., Berthling, I. & Sollid, J. L., 2003. Aspects and concepts on the geomorphological significance of Holocene permafrost in southern Norway. *Geomorphology,* Volume 52, pp. 87-104. https://doi.org/10.1016/S0169-555X(02)00250-7

Farbrot, H., Isaksen, K., Etzelmüller, B. & Gisnas, K., 2013. Ground thermal regime and permafrost distribution under a changing climate in Northern Norway. *Permafrost and periglacial processes,* Volume 24, pp. 20-38. https://doi.org/10.1002/ppp.1763

Frauenfelder, R., Isaksen, K., Matthew, J. L. & Noetzli, J., 2018. Ground thermal and geomechanical conditions in a permafrost-affected high-latitude rock avalanche site (Polvartinden, northern Norway). *The cryosphere,* Volume 12, pp. 1531-1550. https://doi.org/10.5194/tc-12-1531-2018

Guo, D., Wang, H. & Li, D., 2012. A projection of permafrost degradation on the Tibetan Plateau during the 21st century. *Journal of Geophysical research,* Volume 117. https://doi.org/10.1029/2011JD016545

Haeberli, W., 1992. Construction, Environmental problems and natural hazards in periglacial mountain belts. *Permafrost and periglacial processes,* Volume 3, pp. 111 - 124. https://doi.org/10.1002/ppp.3430030208

Hilger, P. et al., 2021. Permafrost as a first order control on long-term rock.slope deformation in (Sub-)Arctic Norway. *Quaternary Sciences Reviews,* Volume 251. https://doi.org/10.1016/j.quascirev.2020.106718

Hipp, T., Etzelmüller, B., Farbrot, H. & Schuler, T. V., 2011. Modelling the temperature evolution of permafrost and seasonal frost in southern Norway during the 20th and 21st century. *The cryosphere discussions,* Volume 5, pp. 811 - 854 https://doi.org/10.5194/tcd-5-811-2011

Isaksen, K. et al., 2011. Degrading mountain permafrost in Southern Norway : spatial and temporal varaibility of mean ground temperatures, 1999-2009. *Permafrost and Periglacial Processes,* Volume 22, pp. 361-377. https://doi.org/10.1002/ppp.728

Isaksen, K. et al., 2002. Mountain permafrost distribution in Dovrefjell and Jorunheimen, southern Norway, based on BTS and DC resistivity tomography data. *Norwegian journal geography,* 56(2), pp. 122-136. https://doi.org/10.1080/002919502760056459

Isaksen, K., Holmlund, P., Sollid, J. L. & Harris, C., 2001. Three deep alpine-permafrost boreholes in Svalbard and Scandinavia. *Permafrost and periglacial processes,* Volume 12, pp. 13-25. https://doi.org/10.1002/ppp.380

Lyon, C. et al., 2022. Climate change research and action must look beyond 2100. *Global Change Biology,* Volume 28, pp. 349-361. https://doi.org/10.1111/gcb.15871

Matthews, J. A. et al., 2018. Small rock-slope failures conditioned by Holocene permafrost degradation: a new approach and conceptual model based on Schmidt-hammer exposure-age dating, Jotunheimen, southern Norway. *An international journal of quaternary research,* Volume 47, pp. 1144-1169. https://doi.org/10.1111/bor.12336

Myhra, K. S., Westermann, S. & Etzemüller, B., 2017. Modelled distribution and temporal evolution of permafrost in steep rock walls along a latitudianal transect in Norway by CryoGrid 2D. *Permafrost and Periglacial Processes,* Volume 28, pp. 172-182. https://doi.org/10.1002/ppp.1884

Perov, V., Chernomorets, S. & Budarina, O., 2017. Debris flow hazards for mountain regions of Russia: regional features and key events. *Springer,* pp. 199 - 235. https://doi.org/10.1007/s11069-017-2841-3

Prina Howald, E. & Torche, J., 2020. Global warming and loss of bearing capacity of permafrost : an experimental study on the effects of freezing/thawing cycles on a silty soil. *Global Journal of Earth Science and Engineering,* Issue 7, pp. 1-21.

Saemundsson, P. et al., 2018. The triggering factors of the Moafellshyrna debris slide in northern Iceland : intense precipitation, earthquake activity and thawing of mountain permafrost. *Science of the total environment,* Volume 621, pp. 1163 - 1175. https://doi.org/10.1016/j.scitotenv.2017.10.111

Sattler, K., Keiler, M., Zischg, A. & Schrott, L., 2011. On the connection between debris flow activity and permafrost degradation: a case study from the schnalstal, South Alps, Italy. *Permafrost and periglacial precesses,* Volume 22, pp. 254 - 265.https://doi.org/10.1002/ppp.730

Stoffel, M., Bollschweiler, M. & Beniston, M., 2011. Rainfall characteristics for periglacial debris flows in the Swiss Alps: past incidences - potential future evolutions. *Climatic Change,* Volume 105, pp. 263 - 280. https://doi.org/10.1007/s10584-011-0036-6

Torche, J. & Prina Howald, E., 2017. Influence of the climate change on the evolution of soil bearing capacity : an experimental study on the effects of freezing/thawing cycles. *IX Simposio Nacional sobre Taludes y Laderas Inestables,* pp. 295 - 306.

Zhang, Y., Chen, W. & Riseborough, D. W., 2008. Transient projections of permafrost distribution in Canada during 21st century under scenarios of climate change. *Global and Planetary Change,* Volume 60, pp. 443- 456. https://doi.org/10.1016/j.gloplacha.2007.05.003