PERMAFROST AND PERIGLACIAL PROCESSES
Permafrost and Periglac. Process. **15**: 261–271 (2004)
DOI: 10.1002/ppp.498

Effects of Volumetric Ice Content and Strain Rate on Shear Strength under Triaxial Conditions for Frozen Soil Samples

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ABSTRACT

A set of triaxial constant strain rate and constant stress tests was performed on artificially frozen soil samples to study the effect of the volumetric ice fraction, strain rate and also confining stress on the mobilized shear strength. In general, the peak shear strength increased with decreasing volumetric ice content and increasing strain rate. Pure ice samples, however, showed a peak shear strength that was higher than that for those containing about 80% ice by volume for a similar strain rate, although the loss in strength after this maximum was usually much more pronounced, reaching a lower large strain strength than that for frozen sands. The influence of the ice in samples with low ice contents was primarily noticeable at low strains, whereas the large strain behaviour was very similar to that of the unfrozen material. The tests showed the dependency of the mechanical failure and deformation mode of frozen soils on the loading conditions. It could be demonstrated further that large strains have a significant influence on the strength of frozen geo-materials and therefore efforts should be made to establish in situ strain states when analysing the stability of frozen slopes.

KEY WORDS: frozen soil; shear strength; creep; triaxial testing

INTRODUCTION

Rock glaciers are geomorphological phenomena formed in creeping permafrost and can be found in mountainous regions at high latitudes or altitudes. A recent compilation of available deformation characteristics in alpine rock glaciers showed the presence of a clear shear horizon in which up to 97% of the total deformation takes place (Arenson *et al.*, 2002). The stratigraphy profiles and the temperatures within these rock glaciers showed that at the depth of the shear zone, ice-rich frozen gravels and sands prevail, at a temperature close to the melting point of the ice. Although there are only limited data available and rock glaciers may vary significantly, there seems to be an indication that such conditions facilitate creep of frozen slopes (Arenson, 2002; Tart, 2003).

Even though rock glaciers in the Swiss Alps have been investigated intensively since the 1970s (see, e.g., Barsch, 1996), there are still many unanswered questions. In particular, there is a gap in knowledge concerning the mechanical processes within this frozen geo-material. The loss of strength of the frozen soil as an effect of increasing temperatures, as well as the formation of the characteristic surface structure, showing furrows and ridges, is not clear (Weber, 2003). Within an alpine environment, rock glaciers

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Received 6 January 2004
Revised 21 May 2004
Accepted 27 June 2004

can often be found at the lower boundary of permafrost and may therefore have temperatures close to 0°C (Vonder Mühll and Haeberli, 1990; Vonder Mühll *et al.*, 2003), which make them very sensitive to global warming. This might lead to stability problems on steep slopes and is of major concern in the densely populated regions of the Swiss Alps (Haeberli *et al.*, 1997).

Triaxial compression tests have widely been used to determine the mechanical properties of various geomaterials, including frozen soils (e.g. Goughnour and Andersland, 1968; Sayles, 1974; Parameswaran and Jones 1981; Ladanyi and Benyamina, 1995; Da Re *et al.*, 2003). This paper describes a set of triaxial shear and creep tests that were performed at a constant temperature of about -2°C. The samples were prepared artificially in order to have controlled and repeatable conditions. The data presented are part of a larger investigation on the thermo-mechanical behaviour of rock glacier material that was performed at the Institute for Geotechnical Engineering (IGT) in Zurich, Switzerland (Arenson, 2002; Johansen, 2002; Arenson *et al.*, 2003; Arenson and Springman, unpublished data). The focus of the investigation presented in this paper was the effect of volumetric ice contents and applied strain rates on the strength and deformation rate under natural stress conditions, representative of the in situ stress state in the preferential shear zone in a rock glacier.

SAMPLE PREPARATION

The samples were prepared artificially under repeatable conditions based on permafrost samples that were obtained from two rock glaciers in the upper Engadine, Swiss Alps (Arenson, 2002). Original solid soil grains from the sites were used and mixed with crushed ice and water in the correct proportions to achieve the desired mixture of ice and soil. Air is often found in significant percentages (c. 10%) in frozen soil from active rock glaciers. This has a major influence on the mechanical response (Arenson *et al.*, 2003), but was not an element considered in this study. The volumetric air content of the samples could not be controlled during the sample preparation and therefore they were nearly 100% ice saturated. Back analyses of the volume composition after melting showed that the samples had an average volumetric air content of 2% and a maximum of 4%. The solid grains were sieved to a silty, sandy well graded gravel (Figure 1) before mixing them with the ice and saturating the matrix with 0°C water. The sample was then frozen one-dimensionally from the bottom allowing drainage of excess water at the top. The cylindrically shaped samples had a diameter of 74 mm and a length of 200 mm. They were then stored in a freezer at a temperature of -18°C. Both ends of the sample were cut with a circular diamond saw to a length of about 150 mm before the specimen was placed into the triaxial cell. After the tests (Figure 2), the samples were measured again and thawed. The volume of the sample after the test was compared with the readings of the change in cell volume, to check the accuracy of the data.

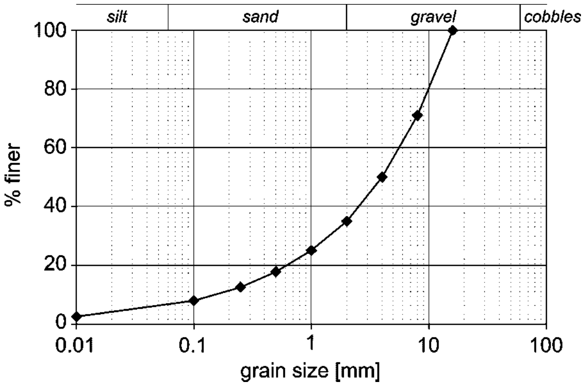


Figure 1 Grain size distribution of the soil tested derived from reconstituted cores of alpine permafrost (Muragl and Murtèl-Corvatsch rock glaciers).

TEST OVERVIEW

Two types of triaxial compression test were performed: control strain rate (CSR) and constant stress (creep) tests (CSC). All tests were carried out with the new triaxial test apparatus that has been designed and constructed at the IGT in Zurich (Arenson *et al.*, in preparation) (Figure 3). The loading frame, load cell and sample are placed in a pressure cell containing about 52 litres of cell liquid. The temperature of the cell fluid was then measured at up to three locations close to the sample and was held constant at about -2°C for all tests. During a test, the temperature of the cell liquid, and hence within the frozen sample, were in the range of $+/-0.05$°C due to the damping effect of the large volume of cell fluid in terms of changes in cold room temperature of $+/-1$°C.

Volume changes of the sample were determined indirectly during the test by measuring the change in volume of the cell fluid. After the test, the actual sample volume was measured and compared with the data derived from the cell fluid. Even though the data are not very precise, trends could be recorded very well.

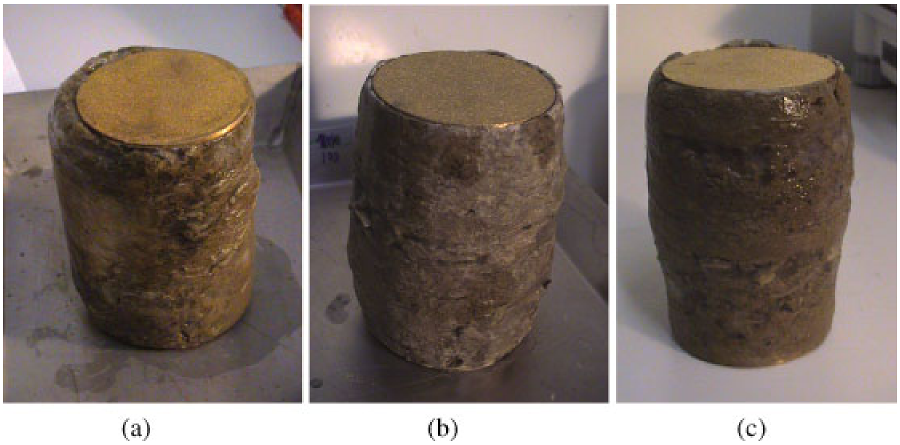


Figure 2 Various samples after a triaxial compression test: (a) $w_i = 80\%$, (b) $w_i = 50\%$, (c) $w_i = 50\%$. This figure appears in colour on the journal's website (www.interscience.wiley.com/journal/ppp).

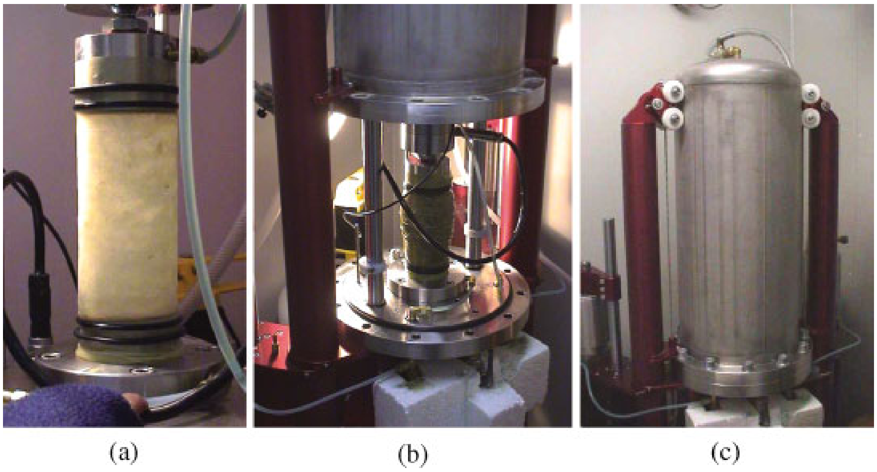


Figure 3 IGT triaxial test device. Open cell with a sample before (a) and after testing (b), and the closed cell during a test (c). This figure appears in colour on the journal's website (www.interscience.wiley.com/journal/ppp).

Triaxial testing requires two phases after the sample is mounted in the apparatus. First, the confining pressure is applied to the cell and the axial stress is held constant at the same level, initiating an isotropic consolidation phase. However, for frozen soils and ice, this phase not only causes excess pore pressures to dissipate, but also minor creep deformations to occur. The goal of this first phase is to establish in situ conditions. Secondly, the axial deformation is started, maintaining either a constant axial strain rate (CSR) or a constant axial load (CSC).

Constant Strain Rate Tests

A programme was set up to test the strength under different confining pressures σ_3 and for a range of strain rates $\dot{\varepsilon}$ as well as volumetric ice contents w_i. The range of the test conditions was chosen to represent natural conditions, i.e. in situ frozen soil conditions down to about 40 m depth. The strain rates were selected so that primarily ductile rather than brittle behaviour would be provoked. Table 1 gives an overview of all CSR tests carried out.

All tests were performed under undrained conditions, i.e. pore pressures were recorded at the top and the bottom of the samples. The measured pressures originate partially from air in the sample and probably from unfrozen water that may be present at the pressure and temperature under investigation (e.g. Williams, 1967). The pore pressure data can therefore not be used to calculate effective stresses, but they provide some indication of sample behaviour during shearing.

Table 1 Test matrix for constant strain rate triaxial compression tests.

Volumetric ice content w_i:	30%				50%			80%			100%		
Confining pressure [kPa]:	50	100	200	400	50	100	200	50	100	200	50	100	200
Strain rate:													
$0.0016\,h^{-1}$		×				×			×			×	
$0.016\,h^{-1}$	××	×	×	×	×	××	×	×	×	×	×	××	×
$0.16\,h^{-1}$		×				×			×			×	

Constant Stress Tests

Only five creep tests were performed successfully (Table 2) to study the effect of the volumetric ice content on creep deformations. The axial deviatoric stresses were chosen at about 70% of the peak shear strength of the corresponding constant strain rate test.

TEST RESULTS

Constant Strain Rate Tests

Table 3 presents a summary of the physical properties and the results of the strength test samples. Despite small variations, the distribution of density and strain rate was uniform throughout the individual test groups. As indicated within the table, several tests did not reach a residual state, i.e. a constant deviatoric stress at large strain. The straining, however, had to be stopped at those strains, since boundary effects start to influence the results at such large strains. The values recorded are therefore only an upper limit to the values expected in reality.

Figure 4 shows the peak and the residual shear strength for samples with different ice contents at three different strain rates. With increasing strain rate, the strength increases as expected. The results for $w_i = 100\%$ are very similar to the values presented by Cole (1987) for pure ice. Compared with earlier tests performed on frozen Ottawa sand (Sayles, 1974), however, the strength values are approximately one order of magnitude smaller. This comparison is very difficult, since the Ottawa sand samples were sheared at a temperature of −3.89°C and the average volumetric ice content was about 37%. Sayles (1974) proposed the following relationship for his data:

$$(\sigma_1 - \sigma_3)_c = 16.6 \times 10^6\ \dot{\varepsilon}^{0.1},$$

where $(\sigma_1 - \sigma_3)_c$ = peak deviatoric strength in N m^{-2} and $\dot{\varepsilon}$ = applied rate of strain per minute.

Using a similar approach, a relationship for the data presented has the form

$$(\sigma_1 - \sigma_3)_c = 4.6 \times 10^6\ \dot{\varepsilon}^{0.17} \qquad \text{lower boundary}$$

$$(\sigma_1 - \sigma_3)_c = 7.5 \times 10^6\ \dot{\varepsilon}^{0.17} \qquad \text{upper boundary}$$

where these lower and upper boundaries are defined to consider the variation in volumetric ice content from 29 to 100% (Figure 4). The low or the high ice contents represent neither the lower nor the upper boundary and therefore the equation cannot be expressed as a function of the ice content. Various other effects, such as strain rate, also influence the strength of the material.

Figure 5 shows the peak and residual shear strength against volumetric ice content for samples with different strain rates and confining pressures. Analysing the residual strength is problematical. As shown in Table 3, tests that were performed at low strain rates have not reached a residual state with constant deviatoric stress and no volume change, even at large strains. Two trends can be noted: (i) the strength is highly influenced by the volumetric ice content and (ii) the confining pressure shows only a minor influence on the strength within the range under investigation.

The influence of the fraction of sand on the strength has been reported by various authors (e.g. Goughnour and Andersland, 1968; Ting *et al.*, 1983). At a volume fraction of about 40% sand, structural hindrance starts

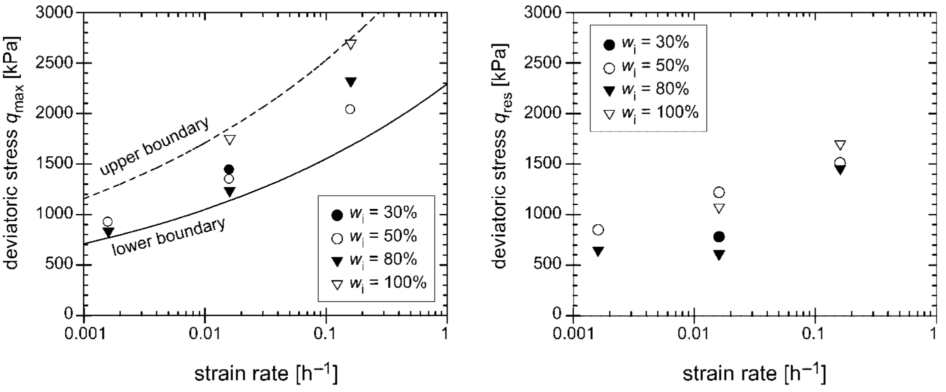
Table 2 Constant stress (creep) triaxial tests.

Test no.	Volumetric ice content [%]	Confining pressure [kPa]	Deviatoric stress q [kPa]
CSC 1	31.8	100	1000
CSC 2, CSC 3	50.9, 46.1	100, 200	900
CSC 4	79.7	100	900
CSC 5	100	100	1200

Table 3 Test results of constant strain rate triaxial compression tests.

Test no.	w_i	σ_3	$\dot{\varepsilon}$	Density	Deviatoric stress [kPa]		Axial strain ε at [%]	
	[%]	[kPa]	h^{-1}	[Mg/m^3]	Peak	Residual	Peak	Residual
CSR 1	32.2	50	0.016	2.06	1330	865	5.54	21.8
CSR 2	28.8	50	0.016	2.15	1706	1125	6.87	27.0
CSR 3	32.0	100	0.016	2.06	1438	780*	2.98	29.2
CSR 4	32.2	201	0.016	2.06	1277	960*	2.33	15.9
CSR 5	33.0	400	0.016	2.06	1849	—	15.9	—
CSR 6	47.6	51	0.016	1.81	1115	1085	2.46	20.9
CSR 7	46.5	100	0.016	1.84	1258	1125*	2.27	20.0
CSR 8	47.3	100	0.016	1.76	1344	1220	2.16	19.8
CSR 9	47.6	200	0.016	1.79	1289	1135	2.27	20.7
CSR 10	52.1	100	0.0016	1.74	919	850	2.71	15.4
CSR 11	52.7	100	0.160	1.71	2033	1510*	1.78	26.5
CSR 12	79.9	50	0.016	1.26	1521	1010*	1.55	29.0
CSR 13	77.9	101	0.016	1.25	1236	610*	1.71	25.9
CSR 14	79.5	201	0.016	1.23	1380	915*	2.20	25.7
CSR 15	81.5	101	0.0017	1.23	835	650*	2.42	15.9
CSR 16	81.8	100	0.164	1.23	2322	1455	1.64	14.8
CSR 17	100	50	0.016	0.90	1975	1145	1.37	13.2
CSR 18	99.0	100	0.016	0.89	1752	1075	1.74	22.3
CSR 19	98.5	200	0.016	0.89	1836	1135	1.61	21.5
CSR 20	99.7	100	0.164	0.90	2696	1700	1.98	25.6

* Constant value not reached.



(a) (b)

Figure 4 Deviatoric stress $q = \sigma_1 - \sigma_3$ versus strain rate for different ice contents and a confining pressure σ_3 of 100 kPa: (a) peak shear strength q_{max}; (b) residual shear strength q_{res}.

to change the mechanisms as sand particles come into contact with each other. An increase in strength is expected. Figure 5(a) confirms this finding. An increase can be noted for the peak strengths at volumetric ice contents of 70% and lower. In addition, an increase was recorded for higher ice contents, which can be explained by the weakening effect of dispersely distributed solid grains in the ice matrix (Arenson and Springman, submitted-a). The minimum strength must therefore be expected to be at volumetric ice contents of about 80%. However, strain rate, temperature and grain size may influence this point.

Three typical stress–strain curves are presented in Figure 6 to investigate the effect of strain rate on the deviatoric stress–strain response. The tests indicate that the deviatoric stress mobilized in the frozen soil increases rapidly at the beginning of the test, due to the stiffening effect of the pore ice, after which it

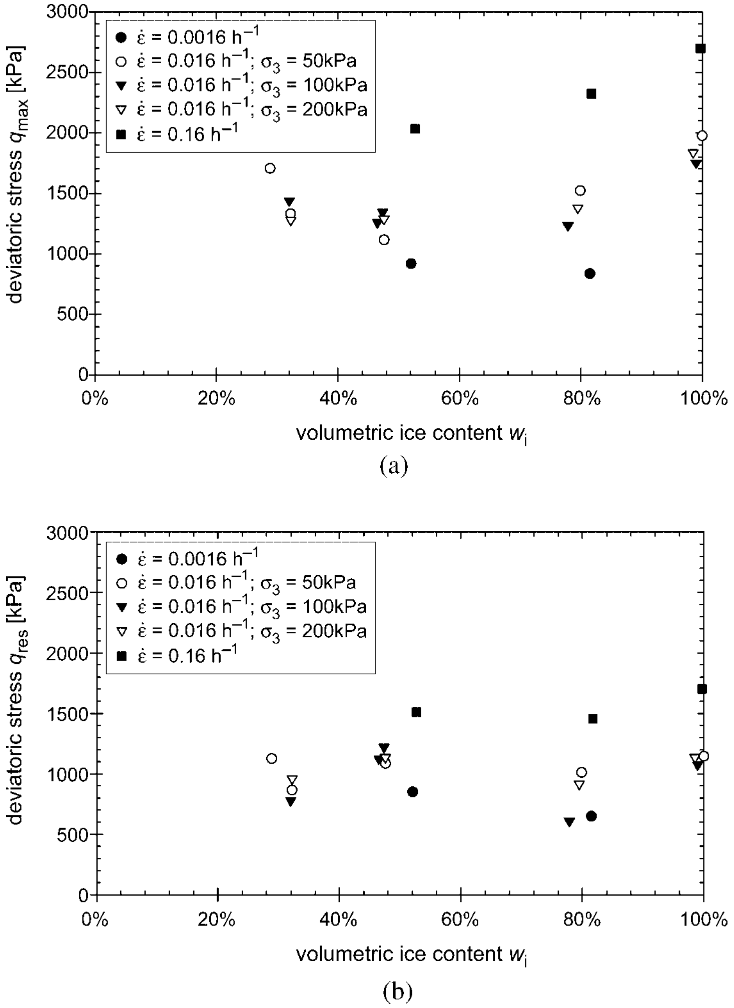


Figure 5 Deviatoric stress $q = \sigma_1 - \sigma_3$ versus volumetric ice content at different strain rates and confining pressures: (a) peak shear strength q_{max}; (b) residual shear strength q_{res}.

reaches peak strength and drops to a lower value due to bonds cracking between the soil and the ice. The strain softening, however, is a function of the applied strain rate. High strain rates result in a more brittle behaviour with high strength at small strains, whereas increasingly more ductile behaviour is induced as strain rates are reduced in that there is less difference between the peak and the residual (large strain) strength. The axial strain at which the maximum shear strength is reached is also a function of the strain rate (Figure 7), varying from 1.8 to 2.7% as the strain rate was dropped by two orders of magnitude from $0.16\,h^{-1}$. However, the initial gradient of the stress–strain curve seemed steeper initially for the test conducted at the lowest strain rates, which was unexpected and may have been due to seating effects of the sample against the platten.

Constant Stress Tests

Because of the limited time, only a few creep tests could be carried out and a comparison between these tests is very difficult. However, the authors have chosen to include them nonetheless, since the results demonstrate the effect of the ice content, which is comparable with the results from the constant strain rate tests.

Figure 8 shows that clearly recognizable minimum creep strain rates were achieved within the time span of this investigation for samples CSC 2 (volumetric solid contents $w_s = 48.9\%$), CSC 4 ($w_s = 18.9\%$) and CSC 5 ($w_s = 0\%$). The other two tests seem to remain in the primary creep phase for the whole test duration of about 300 h (CSC 1, $w_s = 66.3\%$) and 90 h (CSC 3, $w_s = 49.9\%$). Tests CSC 4 and CSC 5 showed

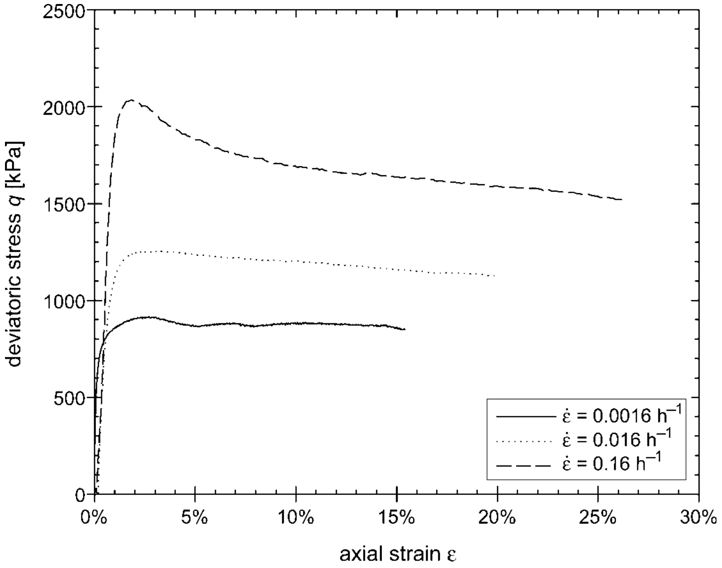


Figure 6 Stress–strain diagram for three different strain rates. $w_i = 50\%$, $\sigma_3 = 100$ kPa.

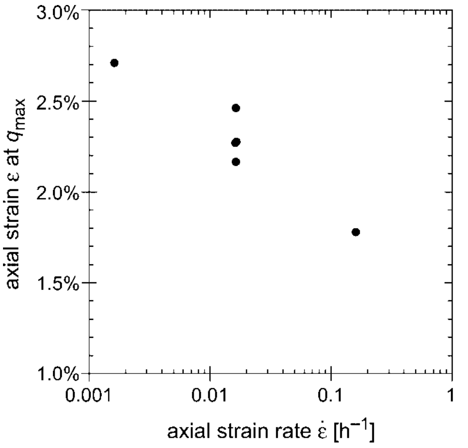


Figure 7 Axial strain at maximum deviatoric stress $q_{\max}$ for different strain rates; $w_i = 50\%$.

minimum creep strain rates of $8.9 \times 10^{-7}\,s^{-1}$ and $1.1 \times 10^{-6}\,s^{-1}$, indicating that there is only a minor difference in the creep behaviour between pure ice and a solid content of 20%. Test CSC 2, however, showed a minimum creep strain rate of about $2 \times 10^{-6}\,s^{-1}$, which seems rather high when compared with the other tests, and should therefore be considered carefully (Figure 9). It is not clear why the creep strain rate of this particular test was higher than the others; no significant difference in temperature or applied stress was observed. Even though steady state conditions might not have been reached completely for tests CSC 1 and CSC 3 by the end of the test period, at least these two end values represent an upper boundary to the minimum creep strain rate. Based on this, it can be shown that the minimum creep strain rate decreases with increasing solid content (Figure 9). The more solid particles are embedded in the ice matrix, the steeper the decrease in minimum strain rate. This is a clear indicator for the creep reducing effect of solid soil particles at low volumetric ice contents.

DISCUSSION AND CONCLUSION

A clear trend could be recorded whereby higher strain rates resulted in higher shear strengths. In general, it is found that the strength increases with decreasing ice content except for the samples with 100% ice, which showed the highest strength.

The confining pressure seems to have had little effect on the strength within the range of confining pressures tested, especially for ice-rich soils. As shown by Ting *et al.* (1983), soil strength and dilatancy effects start to influence the strength of a frozen soil at a volume fraction of sand of about 60%. Therefore, soils with low volumetric ice contents and high confining pressures showed behaviour similar to that of unfrozen soils due to interlocking of the particles, with increasing confinement also resulting in increasing shear strength. The total stresses at failure are shown for four different ice contents and a strain rate of $0.016\,h^{-1}$ in Figure 10. Even though only a few tests are available at each ice content, and significant scattering exists for all but the tests at

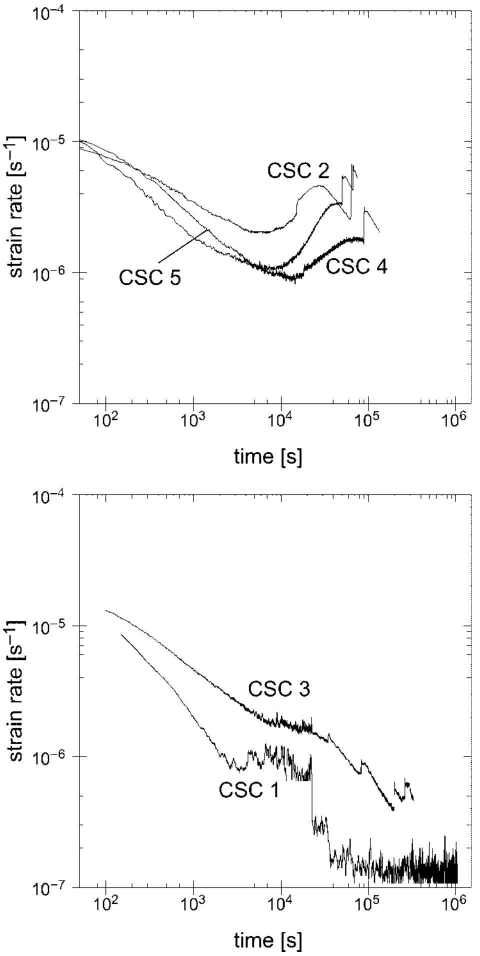


Figure 8 Creep strain rate with time. CSC 2, CSC 4 and CSC 5 show minimum creep strain rates with subsequent increase in creep strain rate, whereas CSC 1 and CSC 3 are still in the primary creep stage.

$w_i = 50\%$, the diagrams indicate that the effect of confinement reduces with increasing ice content. The gradient M^* of a linear regression in q–p space can be determined for $w_i = 50\%$ to be 0.91 ($r^2 = 0.64$), implying a mobilized angle of friction $\phi = 23.3°$ and a cohesion $c = 363$ kPa.

It is expected that the mobilized strength would become less sensitive to the confining pressure as the volumetric ice content increased. Friction can only be mobilized due to structural hindrance and therefore ϕ would decrease as w_i increased. On the other hand, only ice has a tensile strength and is therefore responsible for the cementing of the solid particles, i.e. cohesion is possible. With decreasing volumetric ice content, this effect becomes less important and c decreases too.

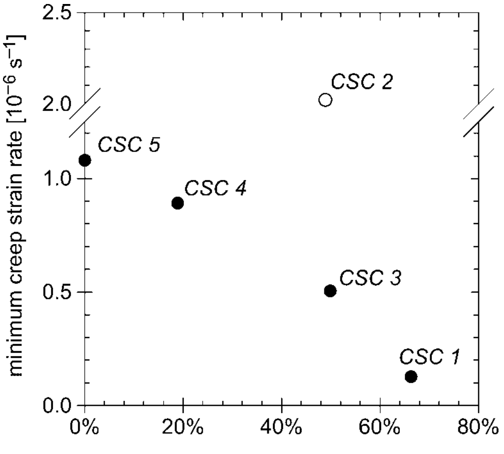


Figure 9 Minimum creep strain rates as a function of the volumetric solid content w_s.

'Soil strengthening' occurs due to structural hindrance of the solid particles after a small improvement in initial strength of the ice with increasing solid content. This results in an increase in strength during the test with increasing axial strain, which has also been described by other authors (e.g. Ladanyi, 1981; Ting *et al.*, 1983). This change in mechanism is further replicated in the stress–strain diagrams, as well as in the corresponding volume and pore pressure behaviour. The comparison between two strain rate tests at strain rates of 0.16 h^{-1} and confining pressures of 50 kPa are shown in Figure 11 for volumetric ice contents of 28.8 and 100%, respectively. The actual values for the volumetric strain and the average of the top and bottom pore pressures are not relevant, but their trends are indicative. The ice-poor soil shows behaviour that is typical for unfrozen geo-materials with two exceptions:

- Stiffer behaviour is observed at the beginning of shearing. This can be attributed to the cementing characteristics of the pore ice. With ongoing straining, the strengthening effect of the ice decreases.
- Volume change, which is similar to a drained test, even though the test was performed under nominally undrained conditions. Even if all the ice had melted, this would still only explain a volume increase of less than 3%. In consequence, pore air in the frozen soil may be responsible for the volume changes observed.

The 100% ice sample showed higher shear strength than the 29% ice sample. The peak is much more

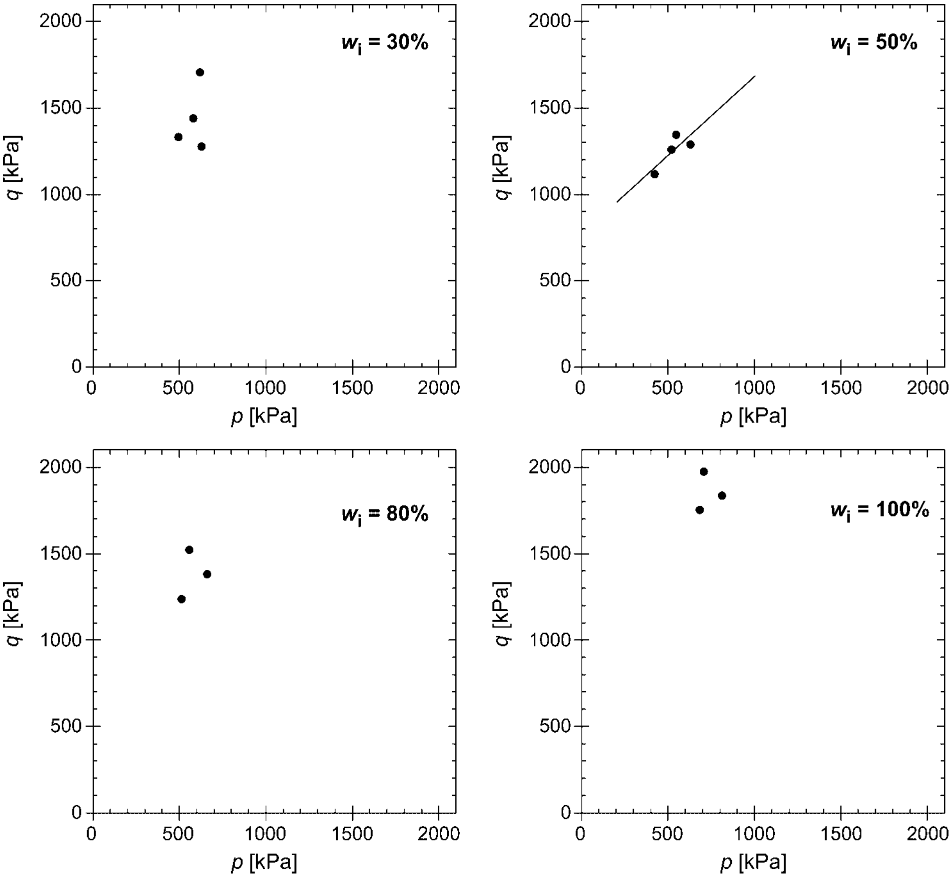


Figure 10 p–q diagrams showing values at failure for various volumetric ice contents and axial strain rates of 0.016 h^{-1}.

$$q = \sigma_1 - \sigma_3$$
$$p = \tfrac{1}{3}(\sigma_1 + 2\sigma_3)$$

pronounced, that is, there is a significant drop in strength after reaching a maximum. However, this sample showed hardly any change in volume and the pore pressures remained constant after a small increase at the beginning, which seems contradictory to a brittle behaviour in that either a sudden volume increase and/ or pore pressure decrease would have been expected. Another explanation might be that the sample failed along a single rather than a series of discontinuities.

The accuracy of the volumetric strain determined was limited by the resolution of the measurement (1.227 mm^3 over a total cell volume of 52 litres, relating to a sample volume of 0.645 litres) and by potential volume change, should the planned test conditions of constant confining stress and constant temperature not be achieved. Similarly, the pore pressure measurements were dependent upon the local pressure adjacent to the plattens and these were certainly not representative of a uniform state in the frozen sample at low strains. Nonetheless, the data obtained deliver trends and these show that frozen soils with a volumetric ice content lower than about 40% start behaving similarly to pure ice. After some straining, the ice matrix starts to fail and the behaviour approaches that of an unfrozen material, showing dilatancy. Although the pure ice sample reaches a higher peak strength, the shear strengths at strains above about 5%, are much lower than those of the sand-rich samples.

Triaxial shear tests on 'undisturbed' granular alpine permafrost samples from boreholes drilled using triple tube and air cooled techniques in rock glaciers in the Engadine (Arenson, 2002; Arenson and Springman, unpublished data) show similar results for temperatures at about −2.1°C and strain rates of 0.03 h^{-1}. These samples were ice rich and contained particles up to gravel sizes. Depending on the volumetric ice contents, values of q_{max} between 1137 kPa ($w_i = 73\%$,

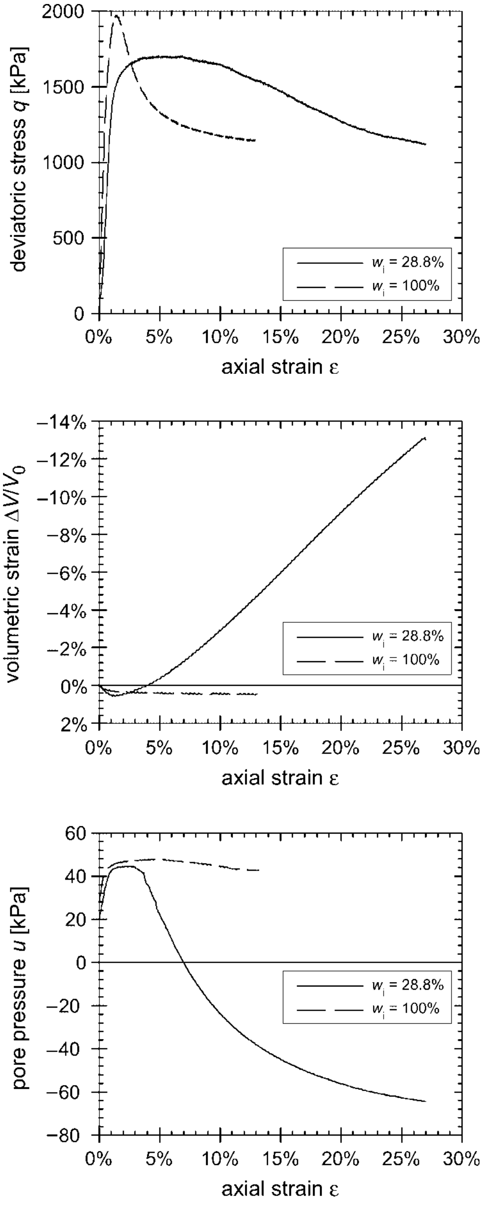


Figure 11 CSR triaxial compression tests at a confining pressure σ_3 of 50 kPa and strain rate of 0.016 h^{-1} for two volumetric ice contents w_i (CSR 2, CSR 17). Deviatoric stress q (a), volumetric strain $\Delta V/V_0$ (b), and pore pressures u (c).

$w_s = 20\%$) and 2148 kPa ($w_i = 56\%$, $w_s = 40\%$) were recorded. A direct comparison, however, is difficult since the majority of these alpine permafrost samples had higher air contents (up to 25%) than the artificially frozen samples. In addition, the strength and the mechanisms of failures obtained in frozen soil are dependent on the temperature. Lower temperature leads to higher strength. Small temperature differences in the soil samples and in the cold room used for the testing can explain the different values for strength found for similar tests (Table 3; Figure 10). On the other hand, small differences in the volumetric ice content might also have influenced the strength of the sample. Nevertheless, shear tests on artificial samples do represent similar mechanical responses to those from 'undisturbed' frozen granular soil samples gained from within two Alpine rock glaciers Muragl and Murtèl-Corvatsch (Arenson and Springman, unpublished data), and are therefore highly recommended for parameter studies.

The following conclusions can be drawn from the tests presented.

- The shear strength of ice-rich frozen soils increases exponentially with increasing strain rate.
- Decreasing volumetric ice contents result in increasing strength. However, the minimum strength was recorded at ice contents of 80–90%. It is assumed that dispersely distributed solid particles change the failure mechanisms of the ice and have a weakening effect, which is contrary to the findings of Ting *et al.* (1983).
- The axial strain at which the maximum strength is reached decreases with increasing strain rate. This is also an indicator for a change in the failure mechanism.
- Even though the peak strength of pure ice can be higher, the strength of samples that are riddled with solid particles might be higher at large strains. This is important when analysing strength of natural slopes of glaciers or permafrost, since ongoing deformation may put the strength mobilized into the large strain range.
- Samples have to be prepared carefully in order to achieve repeatable tests. The technique used for sample preparation has proven to be appropriate, but an air content of up to 4% was measured. Different methods for the sample preparation should be considered to control the air in the sample.
- It could be shown that samples prepared artificially from material obtained from thawed alpine permafrost cores represent the behaviour of undisturbed alpine permafrost very well. In contrast to investigations into the response of natural samples, it is possible to carry out repeatable tests. This is important since the recovery of undisturbed alpine permafrost samples is very expensive due to the access difficulties and cost of drilling and field work.

Unfortunately several aspects could not be investigated during this study. Based on the results presented, the authors suggest further investigation that concentrates on the volumetric response and sample composition. The volumetric change of the samples should be examined in detail with precise measurement techniques during shear. Such data may help in understanding the various mechanisms at play. In addition, the ice content versus shear strength should be examined in more detail, in particular at high ice contents. Finally, the influence of air voids in the sample should be studied. Such extended investigations on artificially frozen soil samples may help in developing a better understanding of rock glacier and general permafrost dynamics at temperatures close to the melting point of ice.

ACKNOWLEDGEMENTS

The authors would like to thank the laboratory staff of the Institute for Geotechnical Engineering at the Swiss Federal Institute of Technology in Zurich for their support during the experimental programme, especially Marco Sperl and Dusan Bystricky. The second author would like to thank Professor Niels Foged of the Department of Civil Engineering at the Technical University of Denmark for the opportunity to write her master's thesis in Switzerland. The support from the organization of the Eighth International Permafrost Conference and from PACE21 and Professor Charles Harris to attend the conference and to write this publication is highly appreciated. In addition, the authors are most grateful for the critical comments from the two anonymous reviewers. It is hoped that the resulting amendments have led to improvements in the manuscript.

REFERENCES

Arenson LU. 2002. *Unstable Alpine Permafrost: A Potentially Important Natural Hazard—Variations of Geotechnical Behaviour with Time and Temperature*. Doctoral Thesis No. 14801, Institute for Geotechnical Engineering. Swiss Federal Institute of Technology: Zurich.

Arenson LU, Almasi N, Springman SM. 2003. Shearing response of ice-rich rock glacier material. In *Proceedings of the Eighth International Conference on Permafrost*, Phillips M, Springman SM, Arenson LU (eds). Balkema: Lisse, Netherlands; 39–44.

Arenson LU, Hoelzle M, Springman SM. 2002. Borehole deformation measurements and internal structure of some rock glaciers in Switzerland. *Permafrost and Periglacial Processes* **13**: 117–135.

Barsch D. 1996. *Rockglaciers: Indicators for the Present and Former Geoecology in High Mountain Environments*. Springer: New York.

Cole DM. 1987. Strain-rate and grain-size effects in ice. *Journal of Glaciology* **33**: 274–280.

Da Re G, Germaine JT, Ladd CC. 2003. Triaxial testing of frozen sand: equipment and example results. *Journal of Cold Regions Engineering, ASCE* **17**: 90–118.

Goughnour RR, Andersland OB. 1968. Mechanical properties of a sand–ice system. *Journal of the Soil Mechanics and Foundations Division, ASCE* **94**: 923–950.

Haeberli W, Wegmann M, Vonder Mühll DS. 1997. Slope stability problems related to glacier shrinkage and permafrost degradation in the Alps. *Ecologae Geologicae Helveticae* **90**: 407–414.

Johansen MM. 2002. *Effect on Volumetric Ice Content and Strain Rates on Shear Strength and Creep Rate under Triaxial Conditions for Frozen Soil Samples*. Diploma Thesis, Institute for Geotechnical Engineering. Swiss Federal Institute of Technology: Zurich.

Ladanyi B. 1981. Mechanical behaviour of frozen soils. In *Mechanics of Structured Media, International Symposium on the Mechanical Behaviour of Structured Media*, Selvadurai APS (ed.). Elsevier: Amsterdam; 205–245.

Ladanyi B, Benyamina MB. 1995. Triaxial relaxation testing of a frozen sand. *Canadian Geotechnical Journal* **32**: 496–511.

Parameswaran VR, Jones SJ. 1981. Triaxial testing of frozen sand. *Journal of Glaciology* **27**: 95–147.

Sayles FH. 1974. *Triaxial Constant Strain Rate Tests and Triaxial Creep Tests on Frozen Ottawa Sand*, CRREL Technical Report 253. United States Army Corps of Engineers CRREL: Hanover, NH.

Tart RG. 2003. Heave and solifluction on slopes. In *Proceedings of the Eighth International Conference on Permafrost*, Phillips M, Springman SM, Arenson LU (eds). Balkema: Lisse, Netherlands; 1135–1140.

Ting JM, Martin RT, Ladd CC. 1983. Mechanisms of strength for frozen sand. *Journal of the Geotechnical Engineering Division, ASCE* **109**: 1286–1302.

Vonder Mühll DS, Arenson LU, Springman SM. 2003. Temperature conditions in two Alpine rock glaciers. In *Proceedings of the Eighth International Conference on Permafrost*, Phillips M, Springman SM, Arenson LU (eds). Balkema: Lisse, Netherlands; 1195–1200.

Vonder Mühll DS, Haeberli W. 1990. Thermal characteristics of the permafrost within an active rock glacier (Murtel/Corvatsch, Grisons, Swiss Alps). *Journal of Glaciology* **36**: 151–158.

Weber M. 2003. *Oberflächenstrukturen auf Blockgletschern*, Diplomarbeit. Department of Geography. University of Zurich: Zurich.

Williams PJ. 1967. Unfrozen water in frozen soils. *Publications of the Norwegian Geotechnical Institute* **72**: 37–48.