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Rock glaciers, fault gouge and asphalt Hard particles in a nonlinear creeping matrix

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Abstract

Composite materials that are composed of hard particles within a soft matrix demonstrate two significantly different deformation behaviours. There is a threshold at which the particle interaction of the hard particles becomes the dominating process. For a low volumetric content of the hard particles, the strain rate of the composite is equal to the strain rate of the soft material reduced by a factor that is a linear function of the volume fraction of the hard particles f. The factor is thought to be material dependent. A value of $1-(5/3)f$ was found for the frozen soil under investigation. At large solids fractions creep deformatins are mostly eliminated by dilatancy. Due to the limited tensile strength of the pore ice, the large strain strength of the composite only depends on the strength of the unfrozen soil.

Keywords: Creep; Soil strength; Rock glacier; Frozen ground

1. Introduction

Rock glaciers are apparent geomorphological features of a permafrost environment. They occur in many mountainous regions, such as high in the Swiss Alps, and are complex inhomogeneous mixtures of ice with varying proportions of rock fragments (Giardino et al., 1987; Martin and Whalley, 1987; Barsch, 1996). Even though only a limited number of published data on the internal structure of rock glaciers is available (Whalley et al., 1994; Elconin and La Chapelle, 1997; Berthling et al., 1998; Arenson et al., 2002) a large spatial and temporal variations of the compositions can be noticed. Some rock glaciers show upper layers that may consist almost of pure ice, and the lower layers of mostly rock. Variations in temperature with time and depth further complicate the system *rock glacier*. The active layer at the top may experience temperatures well below freezing during the winter and well above zero centigrade during warm summers. The seasonal

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variations diminish with depth and are no longer recordable at a depth of about 15 m, depending on various factors, such as the location (e.g. Vonder Mühll and Haeberli, 1990). The mean annual temperatures get warmer with increasing depth. The base of the rock glacier does not have to be identical with the permafrost basis and temperatures above freezing may occur within this lower, generally blocky layer. This phenomenon might be typical for very old rock glaciers that experienced colder temperatures and where the permafrost base is slowly moving upwards (Vonder Mühll et al., 2003). In addition to the two constituents mentioned, air pockets form a third place, and there may be a fourth phase of unfrozen water. This can be present as flowing water in unfrozen channels or as adsorbed water around fine particles. In particular at temperatures close to the melting point of ice, the amount of unfrozen water can be very significant (Williams, 1967; Anderson and Tice, 1972). The engineering importance of rock glaciers rests on their possible involvement in natural hazard formation (after Strozzi et al., 2004):

- rockfall caused by continuous debris transport of active rock glaciers;
- potential source of debris flow due to steady mass transport by creep (Hoelzle et al., 1998; Kääb, 2000);
- triggering catastrophic slides due to reduction in strength as the ice gets warmer possibly provoked by climate change (Haeberli, 1992; Zimmermann and Haeberli, 1992; Haeberli et al., 1993, 1997; Davies et al., 2001).

The mechanical behaviour of ice is reasonably well understood and is dominated by creep at low deformation rates and by fracture at high rates (e.g. Palmer et al., 1983; Sanderson, 1988; Cole, 2001; Schulson, 2001). Assemblages of rock fragments are equally understood and are characterised by friction and dilation familiar in soil mechanics (Bolton, 1986). Active rock glaciers are a mixture of mainly ice and rock fragment, and if the mechanics are to be understood we need tractable constitutive models that idealise the leading properties.

At low solid particle (rock) concentrations, the mixture behaves as 'dirty' ice (e.g. Hooke et al., 1972). The rock fragments do not contact each other, but they hinder creep because they act as essentially undeformable inclusions. However, a small concentration of solid particles may result in smaller ice crystals and tests showed that the creep rates are slightly higher than those observed for pure ice (Weaver and Morgenstern, 1981). At higher rock concentrations, the fragments come into contact, and deformation then requires relative sliding and rotation between the fragments, accompanied by deformation of the ice in the pore spaces. Using data presented by Gougnour and Andersland (1968), Ting et al. (1983) proposed a failure mechanism map, which shows that various mechanisms are at play simultaneously, depending on the volume fraction of the Ottawa sand under investigation. At a temperature of -7 °C a significant increase in the peak strength was noted for volumetric ice contents below about 40%, where structural hindrance due to particle interaction plays a decisive role. Similar observation are reported more recently in Yasufuku et al. (2003) for direct shear tests on sand, and in Arenson and Springman (2005a) or Arenson et al. (2004) for triaxial shear test.

A comparable material occurs in earthquake faults, where in general a finite layer of damaged rock can be found rather than a sharp planar contact between walls of intact rock (e.g. Sammis et al., 1987; Ben-Zion and Sammis, 2003; Storti et al., 2003). Rock particles are fragmented, ground and compacted during the development of the fault to a heterogeneous material called fault gouge or fault breccia, where larger grains are included in a matrix of fine-grained powder or clayey material (e.g. Scholz, 1990). A distinction is usually made between granular gouges and clay gouges (Vrolijk and van der Pluijm, 1999). The latter types are of interest here, since the matrix seems to be of viscous nature that explain creep observation in such gouges (e.g. Thompson et al., 1997).

Another material of this type is paving asphalt, which is composed of rock fragments in a continuous matrix of a binder, blended at temperatures between 135 and 163 °C (Dongré et al., 1996; Read and Whiteoak, 2003). Bitumen, the most popular asphalt binder, is a highly viscous residue of crude oil, obtained by removing most of its volatile components (Hunter, 2000; Read and Whiteoak, 2003). A number of studies have demonstrated the effect of all kinds of particles on the viscosity and the elastic properties of filled polymer systems (overview: Shenoy, 1999). The size of the filler and its distribution, the filler concentration, the filler type, the filler agglomerates or the filler surface treatment changes the viscous properties of the mixture. In general all mechanical properties are significantly less dependent on the volume fraction of the filler within the low and very low filler loading range. At a higher volume fraction of the filler the changes in the rheology are much more pronounced. Similar materials are encountered in food processing (cf. Fellows, 1988; Aguilera and Stanley, 1999).

This paper first discusses the classical model of two-phase materials before considering a possible generalisation that can be applied to the various materials mentioned above.

2. The Terzaghi hypothesis

Terzaghi (1923) introduced the concept of effective stress, which is extensively applied in soil mechanics, often uncritically. In its usual form it is applied to two-phase materials, in which one phase is composed of solid particles in contact with each other, and the other phase is a fluid. A pore pressure is defined as the pressure that is in equilibrium with the fluid in the pore space. Thus, for example, if we insert into water-saturated sand a tubular probe that terminates in a porous stone, with pores too large for the sand particles to enter, the pore pressure is that water pressure in the probe at which water neither flows into or out of the probe. The stress averaged over the two phases together is called the total stress. For a saturated soil, the effective stress σ_{ij}^{e} is expressed as

$$\sigma_{ij}^{\mathrm{e}} = \sigma_{ij} - u\delta_{ij} \tag{1}$$

in which σ_{ij} = the total stress, u = the pore pressure, and δ_{ij} = the Kronecker's delta.

Terzaghi contended that the deformation of the soil depends only on the effective stress, i.e. the stresses between the solid particles. If the soil is deformed, that deformation determines the effective stress, and the pore pressure can alter independently. However, if the deformation includes a volume change that alters the volume of the pore space, then fluid has to move into or out of the pores. If the fluid moves, soil mechanics calls the deformation *drained*. If the fluid does not move, either because the boundaries are sealed or because the viscosity is too high or the time too short, the pore pressure changes in such a way as to make the effective stress consistent with a constant-volume deformation: soil mechanics calls that *undrained*. The changes in pore pressure may provoke flows within the material. Locally those flows include shear deformations as the fluid moves through the constricted channels between the particles. On a larger scale the velocity of the flows depend on the gradient of pore pressure, on the permeability, and on the viscosity of the fluid.

Terzaghi's principle of effective stress is an assertion about constitutive equations. It is not law of mechanics, and counter-examples can be found in literature (Biot, 1941, 1955; Rice and Cleary, 1976; Rice, 1977).

3. A generalisation

A more general model of the frozen soil is to divide the total stress σ_{ij} into an effective stress σ_{ij}^{e} and an ice stress σ_{ij}^{i}, so that formally

$$\sigma_{ij}^{\mathrm{e}} = \sigma_{ij} - \sigma_{ij}^{\mathrm{i}}. \tag{2}$$

The ice stress corresponds to the pore pressure in the Terzaghi model, but unlike the pore pressure it will not generally be hydrostatic.

If the solid particle fraction is small, the particles are not (or only rarely) in contact, and the effective stress is therefore zero. The relation between the ice stress and strain is, however, modified by the presence of the fragments, in the way discussed below.

4. Creep deformation at low solids fractions

The easier regime to understand is the low solids fraction regime at comparatively low stresses, where the ice deforms in hindered creep. The stress–strain rate relation for pure ice can be idealised as power-law creep, where the strain rate is proportional to the nth power of the stress. Glen (1955) found n to be about 3, which was confirmed by various other authors and is generally accepted (e.g. Hooke et al., 1980; Sinha, 1982; Cole, 1987). A relation between strain rate and stress for general states of stress is

$$\dot{\varepsilon}_{ij} = B(s_{kl} \cdot s_{kl})^{(n-1)/2} \cdot s_{ij} \tag{3}$$

where s_{ij} is the deviatoric stress and the repeated subscript summation convention is applied. This generalisation reduces to linear viscosity as n tends to 1, and as n tends to infinity it becomes the stress/strain rate relation for a rigid/plastic incompressible von Mises material (Malvern, 1969) that obeys the associated flow rule.

The parameter B is a function of the temperature, and if solid fragments are present B is also a function of the volumetric solid fraction f. The effect of temperature would be expected to be the same for a rock/ice mixture as for pure ice. This was also assumed by Ladanyi (2002) when presenting a factor that calculates the uniaxial compression strength of a sand/ice mixture as a function of the strength for pure ice up to f=0.65.

The effect of f can be estimated by applying the bound theorems for creep. Martin (1966) showed that an upper bound on the deformation rate for a creeping structure can be obtained from any statically admissible stress field that satisfies equilibrium and is in equilibrium with external loads, and then integrating the corresponding dissipation rate $\sigma_{ij}\dot{\epsilon}_{ij}$ over the whole structure: the strain rates $\dot{\epsilon}ij$ do not need to be compatible. Lower bounds can be found from any velocity field that satisfies any displacement boundary conditions: the corresponding stresses do not need to be in equilibrium. Hill and Power (1956) had previously applied the same ideas to linear problems. Palmer (1967) then used the bound theorems to determine the mean and surface velocities of glaciers with parabolic cross-sections.

This idea can now be applied to a composite of ice and undeformable solid fragments under a stress state σ_{ij}. It is assumed that the ice is frozen to the fragments, neglecting a possible layer of absorbed water at the particle surface, so that the interfaces can transmit the same stress as the ice itself. The ice volume fraction is $1-f$ and the solids volume fraction is f. A uniform stress field with the same stress σ_{ij} throughout both ice and the solid is certainly statically admissible. Applying the upper bound theorem, there is no dissipation in the solid fraction, and in the ice the dissipation corresponds to the stress σ_{ij}. Compared to pure ice, the dissipation is multiplied by $1-f$. An upper bound on the effect of solids is therefore obtained by multiplying the strain rate for pure ice by $1-f$.

Triaxial compression tests on sand–ice mixtures are presented by Gougnour and Andersland (1968) under varying confining pressures. They then compared different compositions at the same strain rate and temperature. Their measurements of strength can be interpreted as creep tests at constant strain rate: the test induces a stress (maximum) at which the sample creeps fast enough to keep up with the imposed strain rate. For a fixed strain rate the stress increases with increasing f, as we expect.

From the measurements of stress for the same strain rate but different f we can estimate a strain rate at the same stress for different f by applying Eq. (3). Fig. 1 replots in this way Gougnour and Andersland's (1968) data from Figs. 3, 6(b) and 7(b) in their paper. The tests were carried out at a temperature of -12.03 °C, and strain rates of 1.33×10^{-4} min^{-1} and 2.66×10^{-4} min^{-1}, respectively. The exponent for creep n is taken as 3, and the stress difference between the axial compressive and the cell pressure is 1 MPa.

The data show that the strain rate decreases nearly linearly with f, and that it approaches zero when f reaches 0.6. At this point the mixture is far from dilute, and most of the solid sand particles are in contact with each other. It corresponds to a voids ratio of 0.67. The simple upper bound underestimates the effect of the particles, even at low values of f.

The same analysis has been applied to a second data set from triaxial creep tests on cores from a Swiss rock glacier at temperatures between -4.45 and -3.74 °C (Arenson, 2002; Arenson and Springman, 2005a,b). A complication is that most of the samples included significant fractions of air and the enhanced dependency of creep on changes in the temperature at the presented range, and therefore samples with an air fraction greater than 12% have been excluded. The data in Fig. 2 are determined by using the average from up to four creep stages carried out on a single sample. The results show more scatter than Fig. 1 but the same trend in the dependence on f can be noted.

Yasufuku et al. (2003) carried out shear box tests on sand/ice mixtures. The tests imposed a uniform

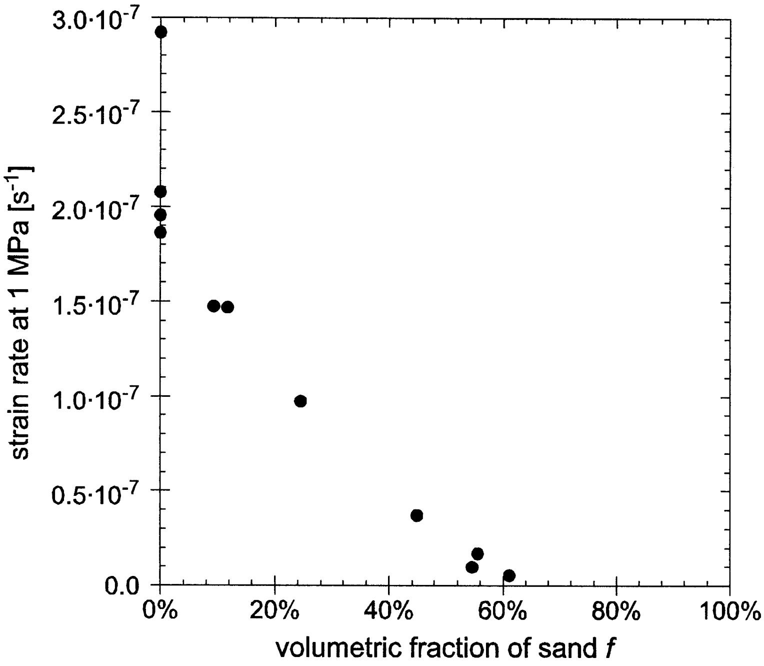

Fig. 1. Strain rates at a reference stress of 1 MPa as a function of the volumetric fraction of solids *f*; data from Gougnour and Andersland (1968).

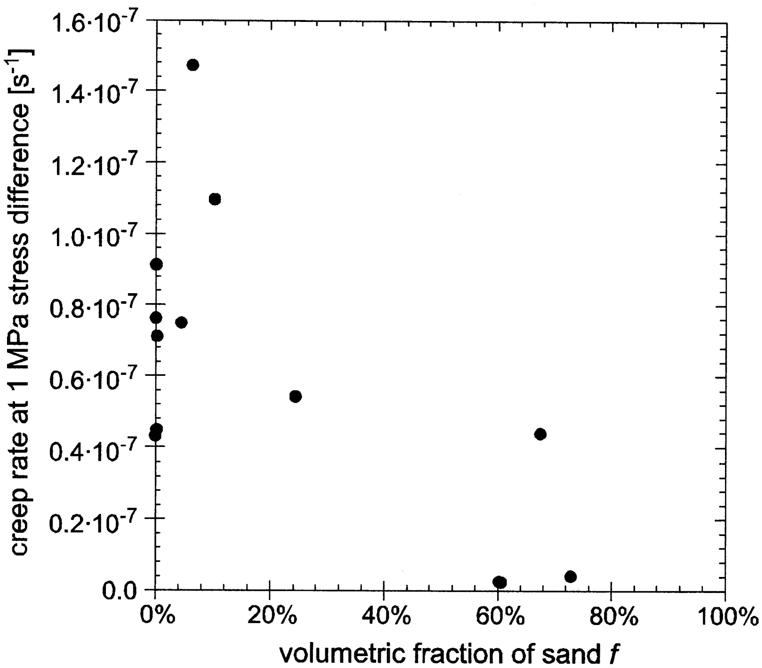

Fig. 2. Strain rates at a reference stress of 1 MPa as a function of the volumetric fraction of solids *f*; data from Arenson and Springman (2005a).

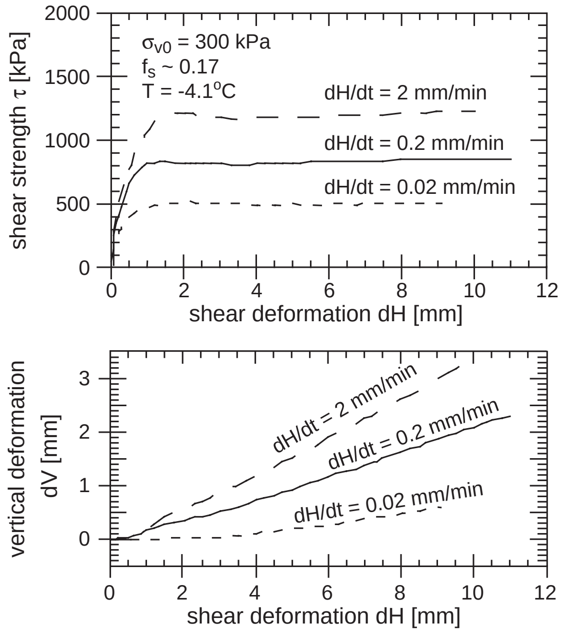

Fig. 3. Dependency of deformation rate (dH/dt) on strength τ and dilatancy bahaviour (dV) due to shear (Yasufuku et al., 2003).

relative horizontal displacement rate between the two halves of the box, under a constant vertical load applied by a piston. The mean shear stress and the vertical displacement of the piston were measured during the test. Interpretation of their results is complicated by the fact that deformation within a shear box is far from uniform, and that the horizontal stresses are not measured. Fig. 3, redrawn from Yasufuku et al. (2003) Fig. 4, plots mean shear stress τ (Fig. 3a) and relative vertical displacement dV (Fig. 3b) againts horizontal displacement dH for three relative velocities, in tests at -4.1 °C with a sand fraction f of 0.17 or 0.18 and vertical stress of 0.3 MPa. A 10-fold increase in relative velocity from 0.02 to 0.2 mm/min multiplies the quasi-steady shear by about 1.65, and a further 10-fold increase to 2 mm/min multiplies the stress by about 1.46.

If the behaviour of the system is governed by the constitutive Eq. (3), one would not expect those ratios to be $10^{1/n}$ unless the horizontal stresses were equal to the vertical stress. That is not the case: defining axes 1 in the horizontal shear direction, 2 in the vertical

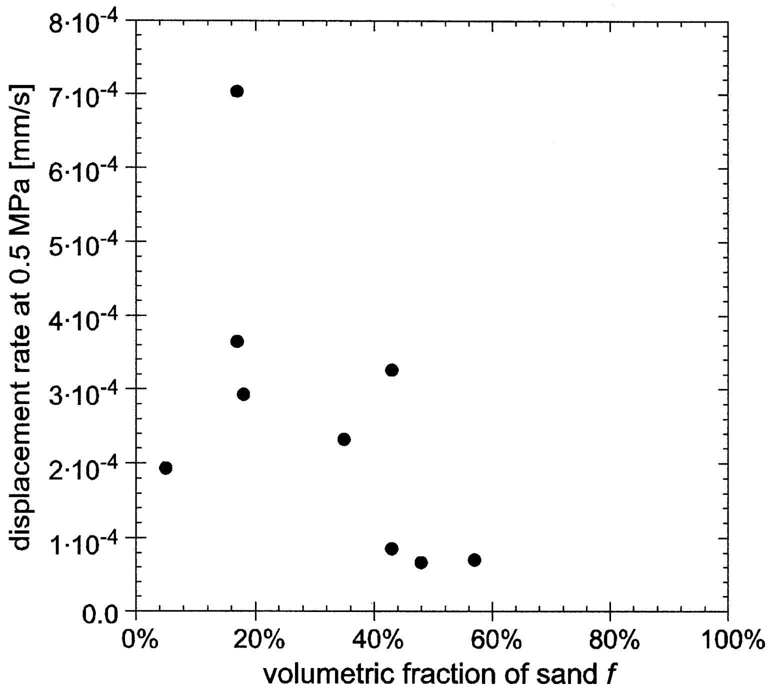

Fig. 4. Strain rates at a reference stress of 1 MPa as a function of the volumetric fraction of solids f; data from Yasufuku et al. (2003).

direction, and 3 in the transverse direction, the strain rate $\dot{\varepsilon}_{22}$ is positive (extensional), whereas $\dot{\varepsilon}_{11}$ and $\dot{\varepsilon}_{33}$ must be negative (assuming that the volumetric strain rate is zero). Inverting Eq. (3),

$$s_{ij} = B^{-1/n}(\dot{\varepsilon}_{kl} \cdot \dot{\varepsilon}_{kl})^{(1-n)/2n} \cdot \dot{\varepsilon}_{ij} \tag{4}$$

and therefore, if we have two loading cases identified by superscripts I and II, and we know all the strain rates and one of the stress components, say s_{12}, we can determine n from

$$\frac{s_{12}^{\mathrm{I}}}{s_{12}^{\mathrm{II}}} = \left(\frac{\dot{\varepsilon}_{kl}^{\mathrm{I}}\dot{\varepsilon}_{kl}^{\mathrm{I}}}{\dot{\varepsilon}_{kl}^{\mathrm{II}}\dot{\varepsilon}_{kl}^{\mathrm{II}}}\right)^{(1-n)/2n} \cdot \frac{\dot{\varepsilon}_{12}^{\mathrm{I}}}{\dot{\varepsilon}_{12}^{\mathrm{II}}}. \tag{5}$$

Applying this to the data presented in Yasufuku et al. (2003) at deformation rates of 0.02 and 0.2 mm/min, respectively, and assuming that $\dot{\varepsilon}_{11}$ and $\dot{\varepsilon}_{33}$ are equal, n is found to be 4.37. The calculation does not require the actual thickness of the zone in which deformation takes place, but does assume it to be the same in both cases.

Fig. 4 applies the same procedure as before, and plots the relative horizontal displacement rate against f for a stress of 0.5 MPa, taking n as 4.37. Though the scatter is quite significant, again the displacement rate becomes small as f approaches 0.6. The scatter may be attributed to the inhomogeneous deformation in a shear box.

5. Deformation processes at large solids fractions

If the solids fragment fraction is larger, so that most of the particles are in contact with neighbouring particles, structural hindrance is the dominating process and the effective stress is non-zero.

Most particulate materials respond to shear deformation by dilating, so that the particles move apart and the pore space between the particles increases (Bolton, 1986). If the pore spaces are filled with a compressible materials such as air, or a mixture of air and water, the pore pressure in the fluid drops. If the pore space is filled with a relatively incompressible fluid such as water, and if the boundaries are not sealed so that additional fluid can flow in from the outside, fluid flows in so to fill the increased pore space. The pore pressures remain constant. If, on the other hand, the boundaries are sealed, or if there is no time for additional fluid to flow inwards, for example due to a low permeability, the pore pressure drops and the effective stress increases. The effective stress may increase to a level at which dilatancy does not occur, because the influence of the shear is balanced by the more compressive effective stress. Ultimately cavitation in the pore space can occur if the pore pressure drops far enough: this happens during rapid undrained deformation of sand in cutting processes (Palmer, 1999).

In the current approach the pore spaces are filled with ice, and the shear deformation creates a tendency for the solid phase to dilate. Either

(1) the ice phase has to flow so that it continues to fill the pore spaces, or
(2) the ice fractures internally so that the additional space is taken up by fractures filled with the air or water vapour, or
(3) the ice breaks away form the particles, again creating additional voids filled with the air or water vapour.

Case 1 is extremely unlikely unless the deformation is so slow that the ice is able to creep from outside to keep the pore space filled. Cases 2 and 3 correspond to the cavitation case when pore space is filled with a relatively incompressible fluid. They are much more likely than case 1. Dilatation tends to create hydrostatic tension in the pore ice, but ice is very weak in tension (e.g. Currier and Schulson, 1982; Cole and Gould, 1989). The tensile strength in the pores is of the order of the fracture toughness divided by the square root of the pore diameter. Taking the fracture toughness as 0.1 MPa $\mathrm{m}^{1/2}$ (cf. Table 2 in Dempsey, 1991), and assuming the pore diameter in a rock glacier as 0.1 m, the tensile stress at which fracture is initiated is in the order of about 0.3 MPa. In other words, frozen soils with low ice contents get an increase in strength due to the tensile strength of ice that prevents the solid particles from dilating. As soon as the tensile stresses in the pore ice reaches a value in the region of 0.5 MPa, cracks develop in the pore ice dilation is no longer suppressed. The material behaviour might be similar to the behaviour of unfrozen soil. In consequence, the large strain or long term strength of dense frozen granular materials is similar to their unfrozen large strain strength.

6. Non-frozen materials

The authors are aware of only very few data for other materials to demonstrate the analogy of the proposed approach. Martínez-Boza et al. (2001) published tests that demonstrate the influence of mineral oil, resin and polymer concentrations on the steady-state viscous flow of model synthetic binders. A significant increase in the zero-shear-rate limiting viscosity was noted for volumetric resin contents above about 40%. The addition of polymers changes the rheology of the binder and the data presented can not be analysed in a similar way anymore. Some experimental data for asphalt mixtures are published by Shashidhar and Shenoy (2002). The data demonstrate that the threshold where the stiffness of the mixtures increases significantly can be found at a volume fraction of the filler between about 15% and 25% depending on the aggregates used.

Fig. 5 replots data presented in Han (1974) Fig. 2 for polypropylene melts filled with calcium carbonate at a temperature of 200 °C. A creep parameter $n=2.09$ was calculated for the polypropylene matrix, and a reference stress of 1 kPa was chosen. The trend is similar to the ice-rich material described above. However, it has to be noted that the decreases in the strain rate for the chosen reference stress is less linear. On the one hand, the scatter of the data at low volumetric fraction of the filler is quite significant and on the other hand, chemical reactions between the filler and the matrix may influence the strength of the composite, which does not occur in frozen soils or fault gouge materials.

7. Conclusion

Rock glaciers are a composite of ice, solid particles, air and water. Considering the two most prominent constituents, ice and solids, a threshold seems to exist at which the time dependent deformation behaviour changes significantly. When the volumetric fraction of solid particles f is less than 0.6, creep controls the deformation and occurs within the ice matrix. A simple relation was found where the strain rate is the strain rate for pure ice at the same stress

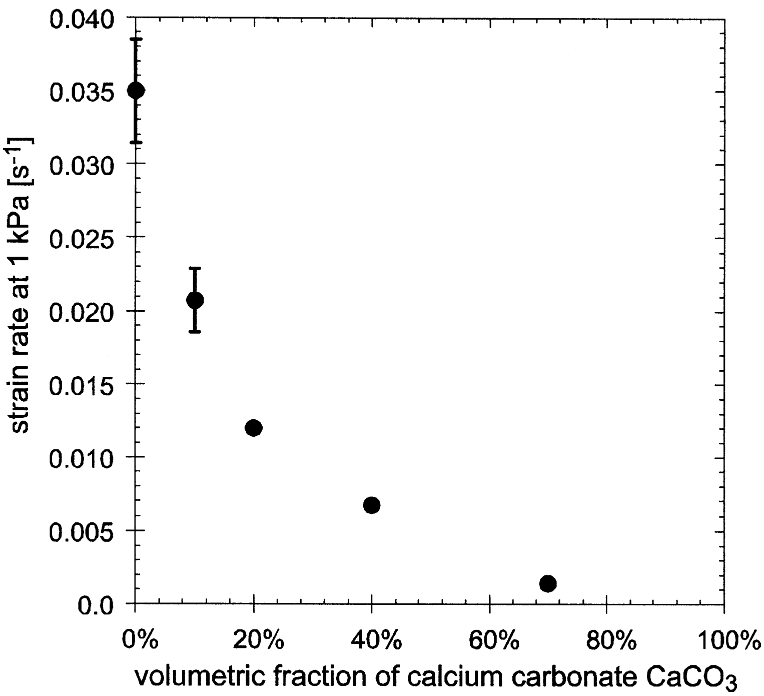

Fig. 5. Strain rates at a reference stress of 1 kPa as for polypropylene as a function of the volumetric fraction of the calcium carbonate filler; data from Han (1974). $T=200$ °C, $n=2.09$. The error bars indicate the scatter of the data at low volume fractions.

and temperature multiplied by $1-(5/3)f$. The behaviour was confirmed for various laboratory investigations presented in literature. The relationship is linear for the interval of volumetric ice content between 0 and 40%, and does, however, not account for the phenomenon reported for dirty ice (Weaver and Morgenstern, 1981; Arenson and Springman, 2005b).

For frozen soils where the particles are not dispersive distributed anymore, the creep deformations are eliminated for the most part due to dilatancy. The large strain strength of the material only depends on the strength of the unfrozen material, since tensile cracks develop in the ice at relative low tensile stresses.

Eve though only a few hard data available, the same concept can be applied for other material where hard particles are included in nonlinear creeping matrix. The threshold might be slightly different, but the authors assume that even for such materials the critical value, where creep takes over the material behaviour lays at about 60% volumetric fraction of the matrix. Further studies are, however, strongly recommended.

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