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On modeling cohesive ridge keel punch through tests with a combined finite-discrete element method

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**Abstract**

This paper introduces a technique for modeling partly consolidated ice rubble using a two-dimensional combined finite-discrete element method and an application of the technique on ice rubble punch through experiments. In the technique, each ice block within the rubble, the contact forces between the blocks, the block deformation, and the rubble freeze bonds are modeled. Simulations with various freeze bond strengths and block to block friction coefficients were performed. As a main simulation result, the close relationship between rubble deformation patterns and load records is demonstrated in detail. It is shown that the buoyant load component due to the rubble becoming detached from the surrounding rubble field and displaced during an experiment is of crucial importance when interpreting punch through experiment results. The consequences of simulation results on ice rubble material modeling are discussed.

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1. Introduction

Ice ridges are common features in northern seas. Two of the most important parts of ice ridges are the consolidated layer, which is comprised of frozen water and ice blocks, and the keel, which consists of ice rubble. The rubble can be a collection of loose blocks, but it is often partly consolidated, that is, it consists of ice blocks bonded together by freeze bonds, which resist the relative movement of the blocks. The freeze bonds are formed due to freezing or sintering (Ettema and Schaefer, 1986; Kuroiwa, 1961).

Punch through experiments are commonly used to test the properties of ice rubble. In a punch through experiment, a flat indenter platen penetrates the rubble mass while the force applied by the rubble on the indenter is measured. The indenter force-displacement records, together with the dimensions of the experimental set up, are then used to derive some of the material properties of the rubble.

The first punch through experiments were performed by Leppäranta and Hakala (1989, 1992) using a loading platform and concrete blocks. Since then, the experimental equipment has been improved and the method has been used in full scale as reported by, for example, Bruneau et al. (1998), Heinonen and Määttänen (2000, 2001a,b), Croasdale et al. (2001) and Heinonen (2004). For more detailed analysis of the behavior and failure mechanism of rubble in punch through tests, experiments in laboratory scale have been performed by, for example, Leppäranta and Hakala (1992), Bruneau et al. (1998), Azarnejad et al. (1999), Azarnejad and Brown (2001), Jensen et al. (2001), Lemee and Brown (2002), Serré (2011) and Polojärvi and Tuhkuri (2012). Liferov and Bonnemaire (2005) have reviewed the experimental work and modeling.

The modeling of punch through experiments using continuum models has been performed by a number of authors. These models have been successful in replicating full scale (Heinonen, 2004) and laboratory (Liferov et al., 2003; Serré, 2011) experiments, but have the disadvantage that the details about the rubble behavior have been smoothed out from the modeling results due to the continuum description of the rubble. One such detail has to do with the relation of rubble mass transfer to indenter load records as addressed in Polojärvi et al. (2012) in the case of non-cohesive rubble.

Hence, even if the ice rubble usually consists of multitude of ice blocks, it remains unclear whether or not there are enough blocks to describe the rubble as a continuum, and thus, if the continuum models can always reliably be used for rubble. This motivates the discontinuous approach, in which rubble is modelled block by block, used here. We believe, that this approach helps in gaining more understanding on the phenomena behind ice rubble behavior. This understanding can then be used not only in the estimation of ice loads or in the planning of future experiments, but also in making further improvements to the more commonly used continuum models.

This paper presents a technique for modeling partly consolidated ice rubble using a discontinuous approach, and modeling of punch through experiments using the technique. The traditional way of modeling discontinuum is the discrete element method (DEM), which dates back to Cundall and Strack (1979). In DEM, the individual particles are usually assumed rigid, and their deformation is taken into account in the inter-particle contact models. In the present study, however, the blocks within the keel are deformable and the combined finite-discrete element method (FEM-DEM) is used (Munjiza, 2004; Munjiza and Andrews, 2005).
2. Description of the simulations

The virtual punch through experiments were performed using the combined finite-discrete element method (FEM-DEM). In this method, the discrete elements representing the rubble keel blocks are meshed into finite elements. The finite elements are used to compute the block deformation and the contact forces between colliding blocks. In addition to contact forces, the deformation and the motion of the blocks is caused by inertial forces and cohesive forces, and by buoyant force, which is caused by water.

The simulations were explicit and central difference method was used to advance between the time steps. On each time step of a simulation, the following seven tasks are performed: (1) Determination of internal forces according to the displacement field for continuous material, (2) a neighbor search, (3) derivation of contact forces, (4) stress state check at freeze bonds, (5) calculation of the cohesive forces in the material points under failure process, (6) adding external forces, and (7) updating of node positions using Newton’s laws for the next time step.

2.1. Contact forces

The contact forces were derived using a penalty function and the potential contact force method (Munjiza, 2004; Munjiza and Andrews, 2000; Munjiza et al., 1995). In the potential contact force method, a potential $\varphi$ with continuous first partial derivatives with respect to spatial coordinates is defined for every point $P$ of each finite element area $\Gamma$. Further, $\varphi = \varphi(P)$ should vanish on finite element edge $S$ for a smooth collision response. Hence,

$$\varphi(P) > 0, \quad P \in \Gamma \quad \land \quad \varphi(P) = 0, \quad P \in S.$$

When triangular finite elements are used, an obvious choice for $\varphi(P)$ is the area coordinates.

The contact force, $f_{\varphi}$, applied to an infinitesimal area element, $d\Gamma_o$, penetrating into $\varphi$ is determined from the gradient of $\varphi$ as (Fig. 1a)

$$\frac{df_{\varphi}(P)}{d\Gamma_o} = -s\nabla\varphi(P),$$

where $s$ is a positive constant penalty term. The negative sign is due to the repulsive nature of the contact force. The contact force, $f_o$, due to $\varphi(P)$ is determined by integration over the overlap area, $\Gamma_o$, of two colliding elements. The integral is reduced to a computationally more efficient integral over the boundary of area $d\Gamma_o$ using a generalized version of Gauss’s theorem:

$$f_o = -s\int_{\Gamma_o} \nabla\varphi(P) d\Omega = -s\int_{\Gamma_o} \varphi(P) n \cdot df.$$

where $n$ is the unit outer normal of $S_o$ (Fig. 1b). The previous equation shows that the distributed load acting upon overlapping volume elements due to $\varphi$ is reduced to a force acting upon a single point on $S_o$.

Dissipation due to sliding friction is modelled using dynamic Coulomb friction. The frictional force, $f_{\mu}$, is solved using the following equation:

$$f_{\mu} = -\mu f_o \frac{|v_i - v_\varphi|}{|v_i - v_\varphi|} \cdot n,$$

where $\mu$ is the friction coefficient and $v_i - v_\varphi \cdot n$ is the tangential component of the relative velocity of contacting blocks at the point of contact.

2.2. Block deformation

Though large displacements of individual ice blocks are allowed in the simulations, the deformation of material elements within the continuous ice blocks was assumed to be small. The material behavior of the continuous ice blocks is thus assumed to be linear elastic. This assumption is justified because, rather than being dominated by the deformation of the individual ice blocks, the deformation of the ice rubble is dominated by inter-particle sliding and the movement of the blocks within the rubble (Heinonen, 2004; Sayed et al., 1992).

Furthermore, the material behavior of the blocks is assumed to be isotropic and plane strain state is assumed. The material damping of the blocks on the elastic regime is viscous. Internal forces due to the deformation of the blocks are solved using constant strain triangle elements with an explicit solution procedure implemented as presented in detail by Munjiza (2004).
2.3. Freeze bonds

The freeze bonds were modelled using initially rigid cohesive elements (Block et al., 2007; Camacho and Ortiz, 1996; Morris et al., 2006; Ortiz and Pandolfi, 1999; Sam et al., 2005). In this approach, the finite element mesh is initially continuous. Once the stress state in some point of the mesh (here in point belonging to a freeze bond) fulfills a predefined failure criterion, an energy dissipating cohesive crack growth process at that point begins. The approach was used here, as Fig. 2 illustrates: The simulated keel was meshed in its initial configuration with the mesh being continuous over the contacting surfaces of the blocks.

As Fig. 2 further shows, the edges of the finite elements connecting the blocks were defined as freeze bonds. The failure within the rubble was limited to only on the freeze bonds in order to study their effect on rubble failure. On each time step of a simulation, the stress state at each freeze bond point (a finite element mesh node belonging to a freeze bond) was monitored and compared to a failure criterion, as presented below. Once the failure criterion at a freeze bond point was reached, the point underwent a cohesive crack growth process, which is described below.

2.3.1. Failure criterion

The failure criterion for the freeze bonds is as follows:

\[ t_e \geq \sigma_{cr} \]  

(5)

where \( t_e \) is the effective traction at a bond point defined as

\[ t_e = \begin{cases} \sqrt{\beta^2 t^2 + t_n^2} & \text{if } t_n \geq 0 \\ \beta^{-1} |t_e| - \mu |t_n| & \text{if } t_n < 0. \end{cases} \]  

(6)

In the definition for \( t_e \), \( t_t \) and \( t_n \) are the tangential and normal components of the traction vector \( t \) at a bond point, respectively, and \( \mu \) is the friction coefficient. Furthermore, \( \beta \) is the shear stress factor, defined as \( \beta = \tau_{cr}/\sigma_{cr} \), where \( \tau_{cr} \) and \( \sigma_{cr} \) are the shear and tensile strengths of the bond, respectively. The failure criterion is for a mixed mode fracture, as the definition for \( t_e \) shows. In addition, for shear failure under compression, the criterion takes into account the contact friction, as discussed in the next section.

The traction vector, \( t \), is solved here similarly to Block et al. (2007) using the area weighted average of the elastic stresses of the elements connected to the bond point:

\[ t = \sum_{i=1}^{n} \left( \frac{A_i}{A} \right) \sigma_{i,n}. \]  

(7)

In the previous equation, \( A_i \) and \( \sigma_{i,n} \) are the area and elastic stress tensor, respectively, of the element \( i \), \( A \) is the total area of \( n \) elements connected to the point, and \( n \) is the bond surface normal.

2.3.2. Cohesive crack growth

When cohesive crack growth is initiated at a freeze bond point, the finite element mesh node is split into two nodes with initially equal nodal coordinates, \( x^+ \) and \( x^- \), as shown in Fig. 3a and b. The mass, \( m \), of the node before splitting is divided by the new nodes according to the elements connected to the node; hence \( m^+ + m^- = m \). It should be noted, that here lumped masses are assumed.

\[ \text{Fig. 3. An illustration of the bond splitting process: (a) An intact node belonging to the bond surface (between the dashed lines) with a normal } n, \text{ nodal coordinate } x, \text{ and mass } m \text{ is split into (b) two nodes with masses } m^+ \text{ and } m^- \text{ and the nodal coordinates } x^+ \text{ and } x^- \text{. Similarly, new crack surfaces } \Gamma^+ \text{ and } \Gamma^- \text{ are generated.} \]
During the cohesive crack growth, a nodal cohesive force, $f_c = f_c^0 = -f_c^t$, resisting the cohesive crack opening is applied to the newly generated nodes similarly to the work of Block et al. (2007).

The initial value, $f_c^0$, is derived as follows. The elements connected to node $x\rightarrow x^-$ are used to compute the internal force vector $F^{(\rightarrow)}$ for the stress state $t_e = \alpha_x$ (Eq. (5)). Then, $F^{(\rightarrow)}$ are used to solve the initial value $f_c^0$ from (Block et al., 2007; Papouli and Vavasis, 2003; Sam et al., 2005);

$$f_c^0 = \left| \dfrac{m}{m} \cdot \mathbf{r} + \dfrac{m}{m} \cdot \mathbf{r} \right|$$

As the distance between the nodes $-\rightarrow$ and $+\rightarrow$ increases during the crack growth, the value of cohesive force $|f_c|$ decreases following the linear softening law illustrated in Fig. 4. As the figure shows, $|f_c|$ is a function of effective crack opening displacement (Camacho and Ortiz, 1996; Ortiz and Pandolfi, 1999; Sam et al., 2005)

$$\delta_e = \sqrt{\delta^2 + (\delta_0)^2}$$

where $\delta_c$ and $\delta_0$ are the components of the cohesive crack opening displacement, $\delta_e = x^+ - x^-$, to the tangential and normal directions of the freeze bond, respectively. The operator $\sim$ in the definition is the Macaulay brackets; hence $\delta_e = 0$ if $\delta_e < 0$.

In Eq. (9), $\delta_e = \delta_0$ if $\delta_e < 0$ due to the fact that the contact of the crack surfaces is treated as phenomena independent of the actual cohesive model, which is similar to how Camacho and Ortiz (1996) and Ortiz and Pandolfi (1999) treated it. When the crack is closed, the contact forces between the contacting crack surfaces are solved, as presented in Section 2.1 for the blocks. Hence, in the case of relative sliding of the closed crack surfaces under compression, the cohesive crack growth is simultaneously resisted by both the frictional and cohesive forces. Due to this, the friction coefficient $\mu$ was included in the failure criterion through the definition of $\delta_e$ on the compressive side (see Eqs. (5) and (6)): If $\mu$ is not included in the failure criterion and the failure initiated under compression with $t_e = t_{in}$, then the cohesive crack growth would be arrested by friction until $t_e > \mu t_{in}$.

The cohesive crack growth can include loading, unloading and reloading phases, as illustrated in Fig. 4: in loading phase $\delta_e$ increases, in unloading it decreases, and in reloading again increases but has a value smaller than maximum effective crack opening displacement $\delta_m$ reached in preceding loading phase. The irreversibility of this crack growth process is ensured by solving the cohesive force value during these phases based on

$$f_c(\delta_e) = \begin{cases} f_c^0(\delta_e) & \text{for loading} \\ f_c^0 - \frac{\delta_e}{\delta_m} & \text{for un-/reloading.} \end{cases}$$

The direction of the cohesive force depends on components $\delta_e$ and $\delta_0$ of cohesive crack opening displacement $\delta$. When using these components, the tangential and normal components of the cohesive force are, respectively, solved from the following equation:

$$f_c(\delta_e) = \frac{\delta_e}{\delta_e}$$

As Fig. 4 further shows, $f_c(\delta_e)$ linearly decreases with an increasing $\delta_e$ and vanishes once $\delta_e$ reaches $\delta_0$. The value of $\delta_0$ is not a material parameter, but instead is derived from the fracture energy $G$: If the length of the newly formed crack surfaces associated with a bond point is $l$, the energy dissipated due to the cohesive crack growth process at the point should be equal to $Gl$ in 2D. When using the definition for work, linear softening law in Fig. 4, and requiring the dissipation in the cohesive crack growth to be equal to $Gl$, the value of $\delta_0$ is achieved from

$$\int_0^\infty f_c(\delta_e) d\delta_e = \frac{1}{2} f_c^0(\delta_e) = Gl \Rightarrow \delta_0 = \frac{2Gl}{f_c^0}$$

2.4. Virtual punch through experiments

The simulations were performed in two phases: (1) During the initial phase the initial configuration of the rubble was generated and (2) during the punch through phase the indenter penetrated the rubble. The first phase was performed using rigid blocks in a manner similar to that of Polojärvi and Tuhkuri (2009) and Polojärvi et al. (2012) to achieve different initial configurations for the ridges. During the first phase, the rigid blocks were released under water with random orientations and velocities and allowed to float until their kinetic energy had dissipated, and a quasi-static rubble pile had formed. At the end of the first phase, the rubble pile was meshed using Gmsh finite element mesh generator (Guezaine and Remacle, 2009) for the second phase of the simulation.

At the beginning of the second phase, the buoyant force and gravitation were applied to the whole rubble mass, consolidated layer, and indenter platen. The buoyant force acting on the indenter platen was subtracted from the indenter force records. Before any indenter motion, it was ensured that the system was in balance. The boundary conditions and the simulation domain for the punch through phase of the simulations are illustrated in Fig. 5, with the values for the dimensions in the figure shown in Table 1. The domain width $w$ was 50 m. The simulations showed that the domain was substantially wider than the width of the area of moving rubble during indenter penetration.

The block dimensions were randomly varied within each simulated ridge so that the block thickness and length varied between 0.2 and 0.4 m and 0.6 and 1.8 m, respectively, with the lower bound for thickness chosen based on Heinonen (2004). This random generation of the blocks resulted into aspect ratio distribution with a mean at $\sim 1:3.5$. The ridge keel thickness, $h$, and the consolidated layer thickness given in Table 1 were chosen after full scale punch through experiments reported and analysed by (Heinonen, 2004). In his experiments, rubble thicknesses varied between 2.2 and 5 m (mean value 3.8 m), with consolidated layer thickness varying between 0.6 and 1.4 m (mean value 1 m). The rubble was freeze bonded to the consolidated layer. The indenter width, $w_l = 4$ m, used here was always at least ten times higher than the block thickness, similarly
to the experiments by Heinonen (2004). The consolidated layer and the indentor were elastic. For simplicity, the material parameters for the consolidated layer and the indentor were the same as for the rubble blocks.

The freeze bond strength was not constant through the rubble depth but instead decreased linearly towards the bottom of the rubble, where the bond shear and tensile strength values were −10% of those on the top. Simple linear relation was used, as experimental data indicates, that the layer of partially consolidated rubble does not necessarily reach through the rubble thickness (Croasdale et al., 2001; Høyland and Løset, 1999; Leppäranta and Hakala, 1989, 1992; Timco et al., 2000).

A number of freeze bond strength values were used in the simulations. As Table 1 shows, the freeze bond shear strength on top of the rubble \(\tau_{cr}\) varied between 5 and 100 kPa. The lowest \(\tau_{cr}\) values used are close to those measured in a laboratory (Ettema and Schaefer, 1986; Repetto-Llamazares et al., 2011a, 2011b), while the higher ones are close to those measured in field studies (Shafrova and Høyland, 2008) or achieved by a rough scaling of the laboratory experiments (Ettema and Schaefer, 1986; Lifterov, 2005). Unfortunately there is a lack of data on the freeze bond tensile strengths \((\sigma_{tc})\), hence 10 kPa on top of the rubble was used here.

Furthermore Table 1 shows the elastic modulus, \(E = 2\) GPa, used in the simulations; it was chosen based on the values 1 to 5 GPa for the strain modulus of first-year sea ice suggested by Timco and Weeks (2010). The friction coefficient values \(\mu = 0.05\) and 0.3 were used based on values \(\mu \approx 0.02 – 0.7\), which are reported in the literature for ice (Frederking and Barker, 2002; Lishman et al., 2009; Pritchard et al., 2011). Fracture energy for the freeze bonds, \(G = 15\) J m\(^{-2}\), was chosen after that of the first-year sea ice (Dempsey et al., 1999). Similar to the freeze bond strength, \(G\) decreased linearly towards the bottom of the rubble. Porosity \(\eta\), which defines the ratio between the area of the voids and the area of solid material within the rubble, was measured in the initial rubble configuration. Hence, \(\eta\) was not predetermined, but, instead, was a result of the first simulation phase described above.

The indentor motion was displacement driven, with the displacement being controlled on top of the indentor plate. The indentor was accelerated to its final velocity of 0.1 ms\(^{-1}\) during the first two seconds of its motion to avoid a load due to inertia of the rubble. A number of simulations with various slower indentor velocities were used to verify, that a peak load that appeared at the beginning of the indentor penetration was not due to the inertia of the rubble pile, but, instead, due to its quasi-static response. Lowering the indentor velocity to 0.01 ms\(^{-1}\) while keeping the acceleration time constant decreased the load at the initial peak by only 3%, hence it was concluded that the indentor force was virtually only due to static load.

### 3. Results and analysis

We performed the simulations on four different ridge geometries, hereafter referred to as Ridges 1–4, which had been generated during phase 1 of the simulations (see Section 2.4). We used various freeze bond strengths and two friction coefficients for the simulations. In all the simulations, the strength of the freeze bonds decreased linearly towards the bottom of the rubble, as described in Section 2.4. For brevity, this is not mentioned in the text below and all of the freeze bond strength values provided refer to their maximum values on top of the rubble. As we ran the simulations in 2D, the force values presented below are per unit width.

#### 3.1. Force-displacement records and maximum indentor force

We observed that the freeze bond strength, friction coefficient, and ridge geometry all affected the force-displacement \((F – y_I)\) records and load levels. In the following section we first present the effect of ridge geometry on the \(F – y_I\) records and load levels. Then, we describe the effect of freeze bond strength on the \(F – y_I\) records and load levels.

##### 3.1.1. Force records and ridge geometries

The typical features observed in the simulations can be described using the force-displacement \((F – y_I)\) records from the simulations performed on Ridges 1 and 3 shown in Fig. 6a and b. The \(F – y_I\) records
in the figures are from the simulations which were performed with a freeze bond shear and tensile strength of $\tau_{cr} = 50$ kPa and $\sigma_{cr} = 10$ kPa, respectively, and with friction coefficients of $\mu = 0.05$ and 0.3. For more detail, Fig. 7a and b show close-ups of both the $F - y_1$ records in Fig. 6a and b from the beginning of the indentor penetration.

As Figs. 6 and 7 illustrate, $F$ initially increased with a high rate, $\partial F / \partial y$, due to rubble buoyancy and system stiffness. The friction coefficient $\mu$ had virtually no effect on the rate, $\partial F / \partial y$, during the initial increase in $F$, as shown by Fig. 7. Following the steep increase in $F$, the simulations yielded a cohesive peak load $F^m$, which corresponded to the onset of the freeze bond failures within the rubble. This peak load, $F^m$, was also the maximum load in the simulations.

A comparison of Fig. 7a and b already suggests that the value of $F^m$ was significantly affected by the ridge geometry (the different block arrangements generated during the first simulation phase, described in Section 2.4), whereas with these bond strength parameters ($\tau_{cr} = 50$ kPa and $\sigma_{cr} = 10$) the friction coefficient $\mu$ did not have a major effect on the load. This can clearly be seen from the $F^m$ values from the simulations for Ridges 1–4 with the values $\tau_{cr} = 50$ kPa and $\sigma_{cr} = 10$ kPa.

The data in the close-ups covers the initial increase and the peak load, $F^m$, given in Fig. 8. The data in Fig. 8 clearly shows that the difference in $F^m$ yielded by the different ridges with constant $\mu$ was up to ~50%. On the other hand, the difference in the $F^m$ due to $\mu$ was only 2–7% depending on the ridge.

After the peak load $F^m$, with increasing indentor penetration $y_1$, $F$ decreased steeply as Fig. 6a and b show. During the steep decrease, the freeze bonds failed through the rubble thickness, that is, the decrease in $F$ was due to a global failure of the rubble freeze bonds. In this global failure, the majority of the failing bonds lay on distinct zones around the indentor perimeter. A failure plane could be defined through these zones, as described in the next section. The steep decrease in $F$ lasted up to $y_1 \approx 20$ mm of indentor penetration.

As the $F - y_1$ records in Fig. 6a and b further illustrate, the post-peak load after the steep decrease in $F$ from $F^m$ was depended not only on the ridge geometry, but also on the friction coefficient $\mu$. With the friction coefficient $\mu = 0.05$, the load typically decreased up to an indentor penetration of $y_1 = 50...150$ mm, whereas with the friction coefficient $\mu = 0.3$ the decrease was either very slow (Ridge 1 in Fig. 6a) or virtually nonexistent (Ridge 3 in Fig. 6b). The value of $F^m$ with the value $\mu = 0.05$ ($\mu = 0.3$) for each ridge was between...
strengths on top of the rubble were $\tau_{cr} = 50$ kPa and $\alpha_{cr} = 10$ kPa, respectively, for all ridges.

2.0 and 3.2 (1.9 and 2.6) times higher, than the mean load for the penetration interval $y_1 = 150...300$ mm for simulations with the bond strength values $\alpha_{cr} = 10$ kPa and $\tau_{cr} = 50$ kPa.

### 3.1.2. Force records and freeze bond strength

The freeze bond shear strength, $\tau_{cr}$, affected the indentor force-displacement $(F - y_1)$ records most clearly during the cohesive peak at the beginning of the indentor penetration. The effect of $\tau_{cr}$ on the cohesive peak is illustrated by Fig. 9, which shows the $F - y_1$ records from the beginning of the simulated indentor penetration on Ridge 1 with $\tau_{cr}$ equal to 5, 12.5, 25 and 50 kPa. Besides the beginning, the $F - y_1$ records with all $\tau_{cr}$ showed features similar to those described above.

The peak load $F_{m}$ values increased with $\tau_{cr}$, as shown in Fig. 10. As the figure illustrates, the rate $\partial F_{m}/\partial \tau_{cr}$ was not constant, but in general it was somewhat higher with low rather than with high $\tau_{cr}$ values. The change in the rate $\partial F_{m}/\partial \tau_{cr}$ when using different $\tau_{cr}$ values was explained by the relation of the rubble deformation patterns and $\tau_{cr}$ as shown in the next section.

Furthermore, Fig. 10 suggests, that an increase in $\mu$ increased the $F_{m}$ more in the case of weakly bonded ridges, rather than strongly bonded ridges. This is clearly shown by the ratios of the $F_{m}$ values from the simulations with $\mu$ being equal to 0.3 and 0.05 for Ridges 1–4 given in Fig. 11. As the figure shows, the effect of $\mu$ on the $F_{m}$ values depended on the geometry of the ridge, but decreased as the $\tau_{cr}$ was increased. In the case of loosely bonded ridges, $\mu$ increased the maximum load by up to 28%. On the other hand, with the highest value of $\tau_{cr}$ used here, the effect of $\mu$ on the value of $F_{m}$ was negligible (~2.5%).

### 3.2. Failure process and rubble deformation

The failure process and deformation patterns were related to the maximum force, $F_{m}$, and the post peak indentor load, $F$. In general, the ridges yielding the highest indentor load values had the highest amount of rubble mass displaced due to initial global failure. Furthermore, the post-peak indentor load records depended on the rubble mass displacing, not only directly under the indentor but also outside the perimeter of the indentor. The analysis in this section is divided into two parts: first we study $F_{m}$ using the initial failure planes within the rubble, after which we analyze the post-peak load levels using the deformation patterns.

#### 3.2.1. Failure planes and maximum load

The effects of ridge geometry and freeze bond shear strength on the maximum load, $F_{m}$, were related to differences in the initial failure planes within the rubble. The initial failure planes affected the amount of rubble mass supported by the indentor and the load component due to rubble buoyancy. We assessed the initial rubble failure using angle $\alpha$ of the failure plane, defined in Fig. 12.

To define $\alpha$, we collected the positions of the failed freeze bond points at some instant from the simulation data (Fig. 12). Then, we used the least squares method to determine two linear fits: One for the failures with $x < w_1/2$ (with $w_1$ being the indentor width) and another for failures with $x > w_1/2$. The values for $\alpha$ for $x < w_1/2$ and $x > w_1/2$ were then obtained from the angles of the linear fits in relation to the vertical with the positive direction being away from the center line of the indentor (see Fig. 12). It should be noted, that the
failure planes, as they are defined here, were not constrained to align with the indentor platen corners. Angle $\alpha$ described the initial rubble failure fairly well, but it should not be used for large indentor displacements when the zones for failed freeze bonds became less distinct.

The $\alpha$ values showed a similar increase with freeze bond shear strength $\tau_{cr}$ as the peak load, $F^m$, in Fig. 10: $\alpha$ increased with $\tau_{cr}$ up to 50 kPa; after that, it remained fairly constant. This increase in $\alpha$ is shown in Fig. 13, which gives the mean values of $\alpha$ with their standard deviations as a function of $\tau_{cr}$. The data from all of the simulations with the indentor displacement interval $y_i = 1 \ldots 20$ mm are included in the figure. The displacement interval was chosen so, that all of the simulations had advanced to a point after the steep decrease in $F$ following the initial cohesive peak. As the standard deviations in the figure indicate, $\alpha$ showed relatively wide scatter. The friction coefficient $\mu$ had a negligible effect on $\alpha$; for example, simulations with $\tau_{cr} = 50$ kPa yielded mean $\alpha$ values of 43 $\pm$ 8° and 44 $\pm$ 7° with $\mu = 0.05$ and $\mu = 0.3$, respectively.

To study the initial rubble failure patterns, the linear fits used to determine $\alpha$ were further used to define the area, $A_p$, of the plug that formed during the initial rubble failure (see Fig. 12). The $A_p$ is limited by the linear fits used to define the angle $\alpha$ and the lines passing the uppermost and lowermost failed bond on each side of the indentor. Fig. 14 gives the mean $A_p$ values for an indentor penetration interval $y_i = 1 \ldots 20$ mm for each ridge (Fig. 14a) and the mean for all of the ridges (Fig. 14b) with different friction coefficient $\mu$ values. As $\tau_{cr}$ increases further. The change in the rate $\partial A_p/\partial \tau_{cr}$ with low $\tau_{cr}$ values is likely related to the change in the rate $\partial F^m/\partial \tau_{cr}$, which occurs around the same freeze bond strength values (see Section 3.1.2 and Fig. 10). Hence, high $\partial F^m/\partial \tau_{cr}$ is due to changes in both, the rubblle strength and the initial failure patterns, whereas with a high $\tau_{cr}$, the $\partial F^m/\partial \tau_{cr}$ is only due to an increase in the rubble strength.

The initial plug areas, $A_p$, were further used to derive the buoyant load, $F_b$, resulting the plug in order to compare the buoyant force and load levels. From the $A_p$ values, and the porosities $\eta$ of each ridge, we calculated the $F_b$ using the following equation:

$$F_b(A_p, \eta) = (1-\eta) (\rho_w-\rho_i) g A_p,$$

where $\rho_w$ and $\rho_i$ are the mass densities of water and ice, respectively, and $g$ is the gravitational acceleration.

The values for $F_b(A_p, \eta)$ are shown in Fig. 15 at an indentor penetration $y_i = 20$ mm, together with $F^m$ and the load $F$ at $y_i = 20$ mm, as function of $A_p$ for Ridges 1–4 with $\tau_{cr} = 50$ kPa. Hence, the figure...
illustrates the effect of the ridge geometry on the \( F^\text{m} \) and the difference in the buoyant load component included in \( F^\text{m} \) between ridges.

The three main observations illustrated by Fig. 15 are as follows: (1) the \( F^\text{m} \) is very likely influenced by the buoyant load \( F_b(A_p,y) \) of the plug, (2) the \( F_b(A_p,y) \) is different for different ridges, and (3) after the cohesive peak, the load is due to buoyancy of the rubble plug separating from the rest of the rubble. These observations are shown by the fact that the \( F^\text{m} \) and \( F(y_1=20 \text{ mm}) \) values in the figure increase with \( A_p \), with only a slight difference between the rates \( \partial F^\text{m}/\partial A_p \) and \( \partial F(y_1=20 \text{ mm})/\partial A_p \). Hence, \( F^\text{m} \) and \( F(y_1=20 \text{ mm}) \) are likely both related to \( A_p \) and thus related on the buoyant force \( F_b(A_p,y) \) due to the plug.

Since the data from all four ridges is included in Fig. 15, the difference in the \( F^\text{m} \) due to ridge geometry is likely related to the difference in the buoyant load due to the rubble plug. From Fig. 15 it should be noticed, that \( F_b(A_p,y) \) and \( F^\text{m} \) show similar behavior with \( A_p \) and that the \( F_b(A_p,y) \) values are ~50% of the \( F^\text{m} \) values. These observations indicate, that \( F^\text{m} \) values likely include a considerable buoyant component, which should be taken into account when interpreting the \( F-y_1 \) records, as will be discussed in Section 4.1.1.

3.2.2. Deformation patterns and post-peak load

The post-peak indentor load was related to the rubble deformation patterns. The deformation patterns depended on the ridge geometry as illustrated in Fig. 16. The figure shows the vertical displacement field \( u_x \) of the rubble at an indentor penetration of \( y_1=5 \text{ mm} \).

The deformation patterns of Ridges 1 and 3, which are shown in Fig. 16 are clearly different: The area of the rubble mass moving downwards is significantly larger on Ridge 1 than on Ridge 3. Correspondingly, the indentor load for Ridge 1 was ~50% higher than for Ridge 3 around \( y_1=5 \text{ mm} \) (see Figs. 6–8). For the following discussion, we should note, that the area of rubble having the same displacement as the indentor \( (y_1=5 \text{ mm}, \text{indicated by darkest blue in the figure}) \), was approximately equal for Ridges 1 and 3.

In order to show the relationship between the observations of the previous paragraph and the post peak indentor load, we defined the areas of the displaced rubble as illustrated by Fig. 17. At various indentor displacements, \( y_1 \), we used downward rubble displacement fields, \( u_x \), illustrated by the colors in Fig. 16 to define areas \( A_p \) of the rubble displaced by more than some ratio \( K = \{0, 1\} \) of \( y_1 \). Hence, \( A_p=\Sigma(A_p(K,y)) \) and, for example, the area \( A_p(K_0, y_1=100 \text{ mm}) \) includes rubble with \( u_x > 0.5 \times 100 \text{ mm} = 50 \text{ mm} \). Similarly to the previous section, the areas, \( A_p(K,y) \), were also used to derive a buoyant load

\[
F_d(K,y_1) = (1-\lambda)/\rho_u \gamma_l |A_p(K,y_1)|
\]

which can be compared with the post-peak indentor load. As an example, the load \( F_d(K_0, y_1=100 \text{ mm}) \) is an estimate for buoyant load of rubble with \( u_x \geq 50 \text{ mm} \).

An example of the results of the analysis described above in Fig. 18 demonstrates the effect of rubble deformation patterns on the post-peak indentor load, and it shows that a mass larger than just the rubble directly under the indentor likely contributed to the buoyant load component included in \( F \). Fig. 18 shows the \( F(y_1) \), \( F_d(K=0.45, y_1) \), \( F_d(K=0.7, y_1) \), and \( F_d(K=1, y_1) \).
and \( F_0(K = 0.9, y_1) \) graphs from a simulation done on Ridge 1 plotted against the indentor displacements.

The three plots in Fig. 18 depict the change in the rubble deformation field with displacement, which can also be seen by comparing Figs. 16a and 19: The rubble plug that started to move after the initial failure (Fig. 16a) dissolved due to freeze bond failures during the indentor penetration, which led to a smaller amount of rubble moving downwards later in the simulation (Fig. 19). Corresponding to this change in rubble deformation field, \( F \) and \( F_0(K = 0.45, y_1) \) in Fig. 18 both decreased with increasing \( y_1 \). Furthermore Figs. 16a and 19 show, that the amount of rubble that moved the same amount as the indentor (dark blue area in the figures) remained approximately constant, which explains the smaller decrease in \( F_0(K = 0.9, y_1) \) in Fig. 18. It should be noted, that the value \( F_0(K = 0.45, y_1) \) in Fig. 18 cannot be the exact buoyant component of \( F \) because at certain instants \( F_0(K = 0.45, y_1) > F \); rather, it is merely an estimate of the buoyant component and how it changes.

A similar relationship between the deformation fields and the post-peak load levels applied for all simulations. To show this, we derived data sets with values of \( A_d(K,y_1) \) with \( K = [0.1,0.45,0.9] \) and \( y_1 = [20,30,40, ..., 150] \) mm and corresponding \( F \) values for each simulation. One such set would include the \( F_0(K,y_1) \) and \( F \) data given by the markers in Fig. 18. The \( F - A_d(K,y_1) \) sets from all of the simulations for each \( K = [0.1,0.45,0.9] \) are shown in Fig. 20a and b, together with their correlation coefficients \( r \) and linear fits. Fig. 20a gives \( F - A_d \) data from the simulations done on Ridges 1–4 with freeze bond shear and tensile strengths of \( \tau_v = 50 \) kPa and \( \alpha_w = 10 \) kPa, respectively, and Fig. 20b includes the data from all of the simulations. In both figures the data from simulations with the values \( \mu = 0.05 \) and \( \mu = 0.3 \) are included. In addition, the figures show a line for buoyant force \( F = (1 - \eta)(\rho_w - \rho_A)gA \), where \( \eta \) is the mean porosity of Ridges 1–4.

Figs. 18 and 20 illustrate three important findings related to the interpretation of punch through test \( F - y_1 \) records on the post-peak regime: (1) The assumption, that only the rubble directly under the

---

**Fig. 16.** Rubble deformation patterns for Ridges (a) 1 and (b) 3 after an indentor penetration of \( y_1 = 5 \) mm. The colors indicate vertical displacement \( u_y \). In both simulations, \( \tau_v = 50 \) kPa, \( \alpha_w = 10 \) kPa, and \( \mu = 0.05 \). The figures show part \(-10 \text{ m} \leq x \leq 10 \text{ m} \) of the domain. The indentor perimeter in the x-direction is indicated by thick white dashed lines.

**Fig. 17.** Definition of \( A_d(K,y_1) \) for \( K \) values of 0.5 and 0.9: As shown in the illustration, for example the \( A_d(K = 0.5, y_1) \) at indentor displacement \( y_1^* \) includes the rubble with downward displacements of \( u_y > 0.5y_1^* \).
4. Discussion

4.1. Rubble shear strength and material modeling

In the following section, we discuss the interpretation of the punch through experiment results based on the results presented in the previous section. First we discuss the derivation of the rubble shear strength, \( \tau \), and consider the effect of incorrect assessment of the buoyant load component. We demonstrate that an incorrect estimate for the buoyant indenter load component has a major effect on \( \tau \) values derived for rubble after punch through experiment results. Then, we discuss how the rubble friction angle is estimated, and we argue that the punch through experiments may not be a suitable method for deriving a friction angle for ice rubble. This is due to the fact that the tensile freeze bond failures dominate the rubble failure process.

4.1.1. Shear strength of the rubble

To derive \( \tau \) for the rubble, it is necessary to make assumptions about the rubble failure process and failure patterns. Usually as a first assumption, the failure is assumed to occur on the vertical planes that are aligned with the indenter perimeter and reach through the rubble (Croasdale et al., 2001; Heinonen, 2004; Serré, 2011). In 3D with a round indenter, this assumption leads to a so-called cylindrical failure. For brevity, the term cylindrical failure is subsequently applied to a 2D failure with failure planes that are aligned with the indenter corners and have angle \( \alpha = 0 \) in relation to the vertical (see Fig. 12).

The rubble shear strength, \( \tau \), is often derived using the maximum indenter load, \( P^m \), and is assumed to be given by the following equation (Azarnnejad and Brown, 2001; Leppäranta and Hakala, 1992):

\[
\tau = \frac{F^m - F_b}{A_b} \tag{15}
\]

where \( F_b \) is the buoyant component of the indenter load and \( A_b \) is the area of the shear planes within the rubble. For the following analysis, we chose the \( A_b \) after cylindrical failure in 2D, thus \( A_b = 2h \) (with \( h \) being the rubble thickness).

In earlier studies, different estimates for the buoyant load component, \( P^b \), in Eq. (15) has been used. Here, we consider three options: (1) Only taking account the maximum load \( P^m \) (\( P^b = 0 \)), as in, for example Heinonen (2004), Croasdale et al. (2001) and Serré (2011); (2) assuming that the buoyant load is a result of a cylindrical plug \( F_b \approx 11 \text{kPa} \) for all simulations here), as in, for example Leppäranta and Hakala (1992), and (3) using the formula \( F^b = F_b(A_p) \) from Eq. (13), assuming that the buoyant load is a result of a plug forming in the initial failure (see Fig. 12). As in the case of Fig. 15, \( F_b(A_p) \) at the instant of the indenter displacement \( y_1 = 20 \text{ mm} \) is used in the following.
The values of $\tau$ when using assumptions (1)-(3) for $P^b$ are given in Table 2 for freeze bond shear strengths $\tau_{cb} = 5, 50, 100$ kPa. The symbols $\tau^{(1)}$, $\tau^{(2)}$ and $\tau^{(3)}$ in the table refer to the au values derived when using assumptions (1)-(3) in previous paragraph for $P^b$, respectively. The data in the table is from the simulations with both friction coefficients $\mu_{cb}$ and the linear fits of the data sets for each $K$. Data points from the simulations with both friction coefficients $\mu = 0.05$ and 0.3 are included in the figure.

<table>
<thead>
<tr>
<th>$\tau_{cb}$ [kPa]</th>
<th>$5$</th>
<th>$50$</th>
<th>$100$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\tau$ values [kPa]</td>
<td>$\tau^{(1)}$</td>
<td>$\tau^{(2)}$</td>
<td>$\tau^{(3)}$</td>
</tr>
<tr>
<td>$\tau^{(1)}$</td>
<td>$3.6 \pm 0.8$</td>
<td>$5.8 \pm 0.9$</td>
<td>$7.0 \pm 1.3$</td>
</tr>
<tr>
<td>$\tau^{(2)}$</td>
<td>$2.3 \pm 0.8$</td>
<td>$4.5 \pm 0.9$</td>
<td>$5.7 \pm 1.3$</td>
</tr>
<tr>
<td>$\tau^{(3)}$</td>
<td>$2.0 \pm 0.6$</td>
<td>$3.0 \pm 0.6$</td>
<td>$4.2 \pm 1.0$</td>
</tr>
<tr>
<td>$\tau$ ratios $[-]$</td>
<td>$\tau^{(1)}/\tau^{(3)}$</td>
<td>$\tau^{(2)}/\tau^{(3)}$</td>
<td>$\tau^{(3)}/\tau^{(3)}$</td>
</tr>
<tr>
<td>$\tau^{(1)}/\tau^{(3)}$</td>
<td>$1.79$</td>
<td>$1.96$</td>
<td>$1.68$</td>
</tr>
<tr>
<td>$\tau^{(2)}/\tau^{(3)}$</td>
<td>$1.14$</td>
<td>$1.52$</td>
<td>$1.36$</td>
</tr>
</tbody>
</table>

Fig. 14: The area $A_0$ of the initial plug increases with $\tau_{cb}$ and is considerably higher than the area of a cylindrical plug (16 m²). With a $\tau_{cb} > 50$ kPa, the increase in $\tau$ is driven only by the increase in the rubble strength ($\sigma_{cb}$ stays virtually constant with $\tau_{cb}$ for $\tau_{cb} > 50$ kPa as Fig. 14 shows). As a result, $\tau$ estimates which do not properly take account the buoyant load component, improve somewhat. Still, the difference in the $\tau$ is up to 68% depending on the estimate.

4.1.2. Derivation of the material parameters

The simulations suggest that the punch through experiments may not be a suitable method for deriving the rubble internal friction angle. This observation is explained by the failure modes of the freeze bonds: The ratio of bond points failing due to tensile failure instead of shear failure under compression increased when the freeze bond shear strength, $\tau_{cb}$, was increased. In tensile failure, friction does not affect the cohesive failure process of a bond because the bonded blocks are not in contact with one another.

In other words, the frictional resistance is not mobilized at the instant of the peak load in a punch through experiment, which has also been formerly suggested by, for example, Crossdale in Azarniejad and Brown (2001) and Lifener and Bonnemaire (2005). It should be noted here, that the frictional resistance is due to contact friction, which is evidently related to the internal friction angle of the rubble mass, that is, a change in friction coefficient or rubble friction angle would be expected to cause similar change in the measured forces.

The failure modes of the bond points were studied here similar to Ortiz and Pandolfo (1998). For each freeze bond failure, the normal ($\delta_n$) and tangential ($\delta_t$) components of cohesive crack opening displacement (see Section 2) were monitored. If $\delta_t$ $\geq$ $\delta_n$ at the end of the cohesive crack growth process ($\delta_t = \delta_n$ in Fig. 4), then the failure was tensile, otherwise, the failure was due to shear. The mean ratios of the bond point tensile failures to the failures due to shear, together with the ratios of rubble shear strength of $\tau$, with different friction coefficients $\mu$, are collected to Table 3.

As Table 3 shows, already with a value $\tau_{cb} = 5$ kPa close to half of the freeze bond failures occurred in tension (38%). Anyhow, the $\tau$ values with $\tau_{cb} = 5$ kPa increase with $\mu$ as most of the freeze bonds still fail due to shear. As the rubble gets stronger, the relative number of tensile failures increases, leading to virtually equal $\tau$ values irrespective of $\mu$. With a value $\tau_{cb} = 100$ kPa all shear strength measures $\tau^{(1)}$ - $\tau^{(3)}$ indicate a virtually equal rubble shear strength with both $\mu$ and the tensile freeze bond failures clearly dominate.

The above results suggest that in addition to the fair amount of experimental work on freeze bond shear strength under compression already conducted (Ettema and Schaefer, 1986; Repetto-Llamazares et al., 2011a, 2011b; Shafrova and Høyland, 2008), the freeze bond tensile strengths should be studied in order to improve the interpretation of punch through experiment results. So far, the experimental work on freeze bond tensile strengths appears to be limited to the
Table 3
Ratios of rubble shear strength, τ, resulting from the simulations done with different friction coefficients, μ, τ(μ=0.05), and τ(μ=0.3), and the ratio of the number of tensile bond failures (n_t) to the number of bond failures due to shear (n_s). Shear strength measures τ1(μ) and τ2(μ) refer to the τ values with different estimates for the buoyant load component, P, in Eq. (15): P = 0 was used for τ1(μ), P = 11 kN, assuming cylindrical failure, for τ2(μ), and P = P^cr(x, y) from Eq. (13) while taking account the buoyant load of the plug forming in the initial failure, for τ3(μ). The table includes data from the simulations done on Ridges 1-4 with a freeze bond tensile strength 10 kPa.

<table>
<thead>
<tr>
<th>Behavior</th>
<th>τ measure</th>
<th>5</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>τ(μ=0.05)</td>
<td>τ(μ=0.05)</td>
<td>1.17 ± 0.09</td>
<td>1.06 ± 0.03</td>
<td>1.02 ± 0.01</td>
</tr>
<tr>
<td>τ(μ=0.3)</td>
<td>τ(μ=0.3)</td>
<td>1.29 ± 0.12</td>
<td>1.07 ± 0.04</td>
<td>1.02 ± 0.01</td>
</tr>
<tr>
<td>n_t/n_s</td>
<td>n_t/n_s</td>
<td>0.38 ± 0.14</td>
<td>1.44 ± 0.61</td>
<td>2.54 ± 0.91</td>
</tr>
</tbody>
</table>

so-called pull out tests briefly reported in Timco et al. (2000) and Croasdale et al. (2001).

4.2. Comparison to experiments and continuum models

The results from the simulations were in agreement with earlier experimental work, as shown below. Earlier modeling efforts and the results from this study are also compared below. This comparison includes two principal observations: (1) The continuum models used previously differ from the model used here, as they do not predict tensile failures within the rubble, and thus, show peak load values that are highly dependent on the rubble friction, and (2) based on the results presented here, the physical phenomena behind decreasing post-peak indentor force, which is generally accounted on the material softening in the continuum models, is related to the change in the rubble geometry.

4.2.1. Comparison to experiments

The indentor force-displacement (F−y) records in Fig. 6 show features similar to those from full-scale experiments done by Heinonen (2004): A peak load F^m occurred early in the experiments and was followed by a drop in F values. Here, the initial drop due to global plug failure was steeper than in their experiments, but Heinonen (2004) reports that in their experiments, the post-peak load decrease does not correctly present the rubble resistance due to a transient rapid motion of the indentor plate after the peak load.

The peak load, F^m, and global plug failure occurred here at a very early stage of the indentor penetration (−1 mm), as shown by Fig. 7. The indentor penetration at F^m was smaller than usually observed in the field, even if the indentor displacements at F^m have also been low in the field. In the full-scale experiments performed and analysed in Heinonen (2004), F^m occurred at an indentor displacement of y_1 = 8.8…18.1 mm (with a mean of 13.1 mm). Also Croasdale et al. (2001) report that the F^m and global plug failure occurred with a very small indentor penetration, but they unfortunately do not give exact indentor penetration values.

The failure patterns from the simulations can be compared to those observed in the laboratory, where detailed data on the failure patterns is available. The failure patterns are in agreement with the findings of Lernee and Brown (2002), who describe slow, small-scale punch through experiments showing a trapezoidal plug of rubble moving with the initial failure that occurred at the edges of the plug. Jensen et al. (2001) describe the failure in their experiments also having a trapezoidal plug, with the plug edges that were approximately 40° in relation to the vertical. This observation is in agreement with the failure patterns observed here, in which the angle of the plug edges was in the range of the α values in Fig. 13.

The shear strength values, τ, given in Table 2 are well within the range of those found in the literature. Croasdale et al. (2001) and Heinonen (2004) reported values τ = 6-12.8 kPa and 1.3-18 kPa, respectively, both not taking account the buoyant load component (F_b = 0 in Eq. (15), hence the results should be compared to τ1(μ) in Table 2), and Leppäranta and Hakala (1992) τ = 1.7…4 kPa assuming the buoyant load component being due to cylindrical plug (comparable to τ2(μ) in the table).

4.2.2. Comparison to earlier models

The fact that the friction coefficient had a fairly small effect on the peak load F^m was unexpected based on earlier work on punch through experiments using continuum models: In the model by Heinonen (2004) the maximum load increased with an increase in the cohesion and with an increase in the friction angle of the continuum presenting the rubble. Also, the continuum model by Serré (2011) showed a similar dependency between F^m and the friction angle for simulations without cohesion (no study on the combined effect of friction and cohesion was performed in his work).

According to Heinonen (2004), this result was due to compressive pressure increasing within the rubble due to indentor penetration. This leads to an increase in frictional resistance of the rubble on the shear planes, which then hinders rubble displacements. Furthermore, this effect becomes more prominent in his simulations with an increase in the rubble cohesion, while the model does not show tensile stresses within the pile. This tendency is very different from the observations made in crepheli; in those simulations tensile freeze bond failures clearly occur (see Table 3) and the simulations show vanishingly small effect of friction on F^m with high freeze bond strengths (see Fig. 10).

On the other hand, when compared to the so-called pseudo-discrete continuum model in Liferov (2005), the observations here met our expectations. Liferov (2005) reports that the effect of the friction angle on the initial failure and strength of the rubble in virtual shear box experiments was negligible. It should also be noted, that his virtual shear box experiments showed tensile freeze bond failures dominating the rubble failure process under low compressive pressures and further showed a combination of shear and tensile freeze bond failures with higher compression. These findings support the observations on the importance of the tensile freeze bond failures in Section 2.

The post-peak indentor load displacement records were similar to those in the model provided by Heinonen (2004), excluding the initial steep decrease after the maximum load, F^m, which occurred here. After this steep decrease, both models show similar behavior, as the post-peak F decreased with an increase in the indentor displacement. In Heinonen (2004), the rate ∂F^m/∂y of the decrease was affected by the internal friction of the material, with a higher internal friction causing lower rate of decrease. Increasing the friction coefficient here caused a similar change in the post-peak ∂F^m/∂y, as Fig. 6 shows.

Based on the results here, our interpretation of the post-peak rubble behavior differs from that offered by the continuum models. Heinonen (2004) accounts for the decrease in the post-peak F as being due to the rubble material softening, with the details of the exact mechanism and physical phenomena being rendered out by the continuum description of the rubble. The results here clearly indicate (see Section 2 and Fig. 20 in particular) that the change in the post-peak F is due to a change in the rubble deformation patterns, that is, the post peak load is decreasing due to a change in the ridge geometry during a punch through experiment.

5. Conclusions

In this paper we introduced and described in detail a technique for modeling partly consolidated ice rubble using a two-dimensional combined finite-discrete element method. In the technique, each ice block within the rubble, the contact forces between the blocks, the block deformation, and the rubble freeze bonds are modelled. The technique for modeling freeze bonds was based on initially rigid cohesive elements.
The maximum load in the experiments is dependent upon the initial failure pattern of the rubble, which, on the other hand, is dependent upon the geometry and strength of the ridge (see Sections 3.1–3.2.1). The post-peak-load levels show a clear dependency on the rubble volume being displaced by the indenter plate during the experiment, that is, the post-peak load is a function of the buoyant load of the rubble displacing during the experiment (see Section 3.2.2).

Accurate assessment of the buoyant indenter load component can lead to inaccuracies when in deriving the rubble shear strength: The buoyant load component a result not only of the rubble directly under the indenter and if not correctly taken account, could lead to severe over estimation of ridge keel shear strength (see Sections 3.2.1 and 4.1.1). The decrease in the indenter load on the post-peak regime, usually accounted for material softening, is due to a change in the rubble mass supported by the indenter (see Figs. 18–20).

Punch through experiments may not be a proper method for deriving the rubble friction angle for a strongly bonded ridge keel: The rubble failure process mainly involves tensile failures of the freeze bonds and the friction does not affect the peak load or the shear strength (see Section 4.1.2).

Future work should include a more detailed investigation of the reasons for the effect of the ridge geometry. This work should involve various rubble depths, block geometries and indenter widths so as to include the potential scale effects. More detailed study on the effect of the freeze bond tensile strength should also be performed to investigate whether or not the above findings apply to all freeze bond tensile to shear strength ratios. In addition, punch through experiments should be modeled in parallel to modeling of shear box experiments.

The model should also be used in parallel with continuum models, since large-scale problems are still more efficiently solved using the continuum models. On the other hand, continuum models can benefit from an understanding on the smaller scale phenomena provided by discrete models. In other words, advantage of the complementary roles of discontinuum and continuum modeling should be exploited in modeling.

Related to the experiments, work on the modeling of punch through experiments should include detailed identification of the relationships between quantities measurable during the field experiments and the material parameters. This work would make it easier to interpret the results even when only a limited number of full-scale measurements (often in very challenging environment) can be conducted.

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References


