A laboratory study of dilatant hardening:

A mechanism for slow shear of granular materials

by

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This is to certify that the Master’s thesis of

Peter L. Moore

has met the thesis requirements of Iowa State University

Signatures have been redacted for privacy
"I can’t pick my cereal because I’m spinning”

-Arius Liuzzi, age 3, in awe of the universe
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>ABSTRACT</td>
<td>v</td>
</tr>
<tr>
<td>INTRODUCTION</td>
<td>1</td>
</tr>
<tr>
<td>BACKGROUND</td>
<td>4</td>
</tr>
<tr>
<td>&amp; Dilatant Hardening</td>
<td>4</td>
</tr>
<tr>
<td>&amp; Literature Review</td>
<td>5</td>
</tr>
<tr>
<td>METHODS</td>
<td>8</td>
</tr>
<tr>
<td>&amp; The Ring-Shear Device</td>
<td>8</td>
</tr>
<tr>
<td>&amp; Test Specimens</td>
<td>16</td>
</tr>
<tr>
<td>&amp; Experimental Procedure</td>
<td>19</td>
</tr>
<tr>
<td>RESULTS</td>
<td>21</td>
</tr>
<tr>
<td>DISCUSSION</td>
<td>28</td>
</tr>
<tr>
<td>&amp; Influence of Device Compliance on Results</td>
<td>28</td>
</tr>
<tr>
<td>&amp; Influence of Intrinsic Material Properties</td>
<td>30</td>
</tr>
<tr>
<td>&amp; Influence of Pore Pressure</td>
<td>31</td>
</tr>
<tr>
<td>&amp; Influence of Episodic Dilatant Hardening on Shearing Speed</td>
<td>38</td>
</tr>
<tr>
<td>IMPLICATIONS</td>
<td>42</td>
</tr>
<tr>
<td>&amp; Hillslope Processes</td>
<td>42</td>
</tr>
<tr>
<td>&amp; Glacier-Bed Mechanics</td>
<td>43</td>
</tr>
<tr>
<td>&amp; Fault Gouge Mechanics</td>
<td>44</td>
</tr>
<tr>
<td>CONCLUSIONS</td>
<td>46</td>
</tr>
<tr>
<td>APPENDIX A. TEST DATA</td>
<td>47</td>
</tr>
<tr>
<td>APPENDIX B. DEVICE STIFFNESS</td>
<td>66</td>
</tr>
<tr>
<td>REFERENCES</td>
<td>69</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>78</td>
</tr>
</tbody>
</table>
ABSTRACT

Data from stress-controlled ring-shear experiments indicate that shear deformation of saturated granular materials may be repeatedly stabilized by dilatant hardening. For initially dense, saturated sediments, deformation caused by an externally-controlled decrease in effective stress is suppressed by the drawdown of pore-water pressure that results from shear-zone dilation. Halting of deformation and dilation allows re-equilibration of pore pressure and results in renewed shear. This process is intrinsic to granular materials that dilate during shear displacement and is capable of self-sustaining cyclicity. These findings have important implications for processes such as slow landsliding, aseismic fault slip, and glacier-bed deformation.
INTRODUCTION

Granular materials deform slowly in a great number of environments at and near the earth's surface. Glaciers and ice sheets often move in part by deforming their underlying till beds. In the past, such subglacial till deformation may have given rise to ice-sheet instability, influencing past sea level and global climate change (Clark, 1994; Clark, et al, 1999). Subglacial till deformation has been implicated in possible future rapid collapse of the West Antarctic Ice Sheet, which would raise global sea level by as much as 5 meters (MacAyeal, 1992). Similarly, shearing of gouge in active faults has been shown to influence the frequency and magnitude of earthquakes (Byerlee and Summers, 1976). It has been suggested that the development of thick zones of gouge within faults may promote stable aseismic fault creep and suppress the tendency for high-magnitude earthquakes (Marone, 1998). More familiar is the shearing of hillslope materials in landslides, which can creep slowly for many years creating an engineering nuisance, or unexpectedly accelerate resulting in property damage and even loss of life (Turner and Schuster, 1996).

Sustained, slow deformation has been measured in till beneath modern glaciers (Boulton and Hindmarsh, 1987; Blake, et al, 1992; Iverson, et al, 1995; Iverson, et al, 2001; Kavanaugh and Clarke, 2001), colluvial material on hillslopes (Iverson and Major, 1987; Stump, et al, 1993), and gouge in active fault zones (King, et al, 1973; Moore and Byerlee, 1992; Linde, et al, 1996). Despite these measurements, an acceptable mechanical explanation for slow deformation has remained elusive. Although the length scales and rates of deformation in these environments may vary, each is characterized by near stress equilibrium—that is, driving stresses are largely balanced by resisting stresses. As a result,
accelerations of grains are small and brief and deformation proceeds in a slow, sometimes unsteady, but stable manner.

In each of these environments, water-saturated granular materials deform slowly in response to relatively constant driving stresses. Yield of granular materials is governed fundamentally by the Coulomb-Terzaghi criterion which states that the shear strength $\tau$ is a linear function of the effective normal stress,

$$\tau = (\sigma - p) \tan \phi + c,$$

where $\sigma$ is the total normal stress, $p$ is the pore-water pressure, $\phi$ is the angle of internal friction, and $c$ is the cohesion intercept. This relation has been demonstrated repeatedly in a variety of laboratories (Hungr and Morgenstern, 1984; Blanpied et al, 1987; Tika, et al, 1996; Iverson, et al, 1998; Tulaczyk, et al, 2000). The Coulomb-Terzaghi equation, however, is simply a yield criterion and does not prescribe the manner in which deformation will progress after yield occurs. As a result, modelers occasionally postulate viscous components (Savage and Chleborad, 1982; Boulton and Hindmarsh, 1987), appeal to thermodynamic analogs (rate-process creep of Mitchell et al, 1968; Feda, 1989), or call on slightly rate-dependent friction (e.g., Dieterich, 1981; Marone et al, 1990) in order to explain post-yield strain rates. Making such assumptions is not only unsatisfying academically but can lead to inaccurate hazard assessment and misleading conclusions.

Instead of fitting phenomenological models to observed behavior of slowly shearing granular materials, some researchers have sought mechanical explanations through laboratory and field experiments with some success (e.g., Marone et al, 1990; Iverson et al., 2000). One explanation invokes the observed interaction between a dense, saturated granular material and its pore water. This process, called dilatant hardening, causes granular materials that
have begun shearing to temporarily strengthen. This strengthening results from the decrease in pore pressure that accompanies dilatant shear of the dense material. If the strengthening is sufficient to cause shearing to stop or slow down sufficiently, pore pressure diffusively climbs back toward its pre-shearing value, possibly giving rise to renewed shear. When dilatant hardening occurs repeatedly in a material, the resulting record of displacement with time resembles the slow deformation associated with viscous fluids.

The most attractive aspect of dilatant hardening is that it is simply an extension of the Coulomb-Terzaghi criterion. Consequently, it requires no arbitrary postulates or parameters that cannot be measured. It results from the interdependencies between pore pressure, pore dilation, shear strength, and shear rate, all of which can be measured in the laboratory. The experiments conducted for this thesis were designed to make these measurements in order to evaluate the efficacy of dilatant hardening as a mechanism for slow shear of granular materials and to determine what properties and processes govern its occurrence.
BACKGROUND

_Dilatant Hardening_

When external influences (e.g., a gradual increase in tectonic driving stress on a fault or a transient pore-water pressure rise in a hillslope or beneath a glacier) result in the satisfaction of the failure criterion (Equation 1), deformation commences. Granular materials may weaken slightly after an initial peak strength (Lambe and Whitman, 1969), and if the applied shear stress remains relatively constant, a force imbalance may arise in which the applied shear stress is greater than the shear strength. Newton’s second law requires that a net force induce acceleration of the mass until the force imbalance disappears. Such acceleration might result in a catastrophic landslide on a hillslope, a brief surge of a soft-bedded glacier, or the seismic rupture of a fault. However, in order to explain sustained, slow shear, some mechanism for maintaining stress equilibrium in the moments after yield is required.

Shear of granular materials tends to localize in discrete zones (e.g., Mandl et al., 1977), the thicknesses of which scale with the mean grain size of the material (e.g., Oda and Kazama, 1998). If the material is overconsolidated, it must dilate in the shear zone to accommodate shear displacement. In a saturated material, dilation in the shear zone prompts water pressure diffusion into the expanding pore space. As the rate of dilation increases, the rate of pore-pressure diffusion must also increase by steepening of the hydraulic gradient toward the shear zone. The result is reduced pore pressure where dilation takes place and an increase in effective stress and shear strength. If this strengthening is sufficient, it slows or terminates deformation, which in turn allows pore pressure to increase toward its hydrostatic value through continued diffusion. This increase in pore pressure lowers the effective stress
again, and thus failure may initiate anew and be suppressed in the same way. Resulting
quasi-periodic slow shear is herein referred to as episodic dilatant hardening.

**Literature Review**

The tendency for granular materials to dilate when sheared was first recognized late
in the nineteenth century by Osborne Reynolds (1885, 1886). One of the implications
Reynolds cited for his discovery was that if the flow of interstitial fluid was restricted in the
dilating granular material, it abruptly strengthened. Since then, dilatant hardening has been
the subject of extensive modeling and speculation in a variety of earth science and
engineering fields, and has been applied to rocks as well as granular materials. Many
geophysicists have suggested that dilatant hardening could play a role in various aspects of
the earthquake cycle such as regulating the rate of crack growth and progressive failure,
halting fault rupture at extensional fault bends, and controlling the frequency and magnitude
of aftershocks (e.g., Frank, 1965; Rice, 1975; Rudnicki, 1984; Sibson, 1985; Rudnicki and
Chen, 1988; Segall and Rice, 1995; Matthäi and Fischer, 1996). Engineers and geologists
have more recently suggested its operation in slowly-moving landslides (Humphrey and
Leonards, 1986; Van Genuchten and De Rijke, 1989; Harp *et al*, 1990) and subglacial
deforming sediments (Clarke, 1987; Iverson *et al*, 1998, Moore and Iverson, 2002), invoking
dilatant hardening as a means of maintaining slow shear strain rates and distributing strain
vertically in the deforming medium. However, despite its frequent appearance in literature,
there are only a handful of experimental studies which elucidate the factors that affect the
occurrence, efficacy, and sustainability of dilatant hardening. As a result, its incorporation
into mainstream research and scientific interpretation has been hindered. Overall, it has been
difficult to isolate the interdependence of shear strength and rate of deformation in an experimental setting. Most experimental devices in soil and rock mechanics laboratories are intended to yield strength parameters for engineering design, and do not allow the response of deformation rate to other factors to be evaluated, particularly at the large strains that typify many geological processes. Similarly, most field experiments cannot isolate the important variables and constrain the appropriate boundary conditions.

There are, however, a few key experimental studies that have provided insight into the mechanics of dilatant hardening. Brace and Martin (1968; and comment by Ladanyi, 1970) demonstrated that saturated, low permeability rocks fractured in triaxial experiments strengthened as a result of dilatant hardening. They also recognized that strengthening associated with dilatant hardening increased when either strain rate or pore-fluid viscosity was increased, demonstrating the strong coupling between the rate of dilation in the fracture and the rate of pore-pressure diffusion.

Lockner and Byerlee (1994) showed that a similar process occurred during shear of consolidated, saturated fault gouge. They deformed a thin layer of synthetic gouge in both "drained" and "undrained" tests with a triaxial apparatus and found significant strengthening associated with dilation in the undrained case (when the only water available to permit dilation in the gouge layer had to diffuse through the surrounding low-permeability granite). The resulting rate-effect due to dilatant hardening was determined to be "overwhelmingly" larger than any intrinsic strengthening previously documented in friction tests on fault gouge.

Recently, flume experiments by Iverson and others (2000) documented the effects of initial porosity on pore-pressure response and rate of shearing in sandy hillslope materials. The soils were placed on a slope, compacted to different initial densities, and artificially
irrigated until failure began. Experiments with initially dense material dilated at the onset of failure, resulting in decreased pore pressure and slow deformation. This is the first case in which repeated episodes of dilatant hardening were observed with measurements of both episodic displacement and pore pressure.
METHODS

The Ring-Shear Device

A series of laboratory experiments were conducted with a large ring-shear device configured to shear granular materials at prescribed shear stresses. Use of this device, when configured for shear-rate-controlled testing, has yielded results concerning the shear strength of till and its insensitivity to strain rate (Iverson et al, 1998), the effects of clay content on the shear strength of tills (Iverson et al, 1997), and the development of clast fabrics in sheared tills and fault gouge (Hooyer and Iverson, 2000). Our stress-controlled configuration of the ring-shear device is unique, although ring-shear devices with similar configurations have been used for different applications (Ter-Stepanian, 1975; Bons, et al, 1996).

With this device (Figures 1-3), a large annular specimen (0.115 m wide, 0.065-0.075 m deep, and 0.117 m³ volume) of water-saturated sediment was contained in a sample chamber and subjected to external stresses through the application of dead-weight loads (Figure 3). A vertical stress, normal to the direction of shearing displacement, was applied to the end of a lever arm and distributed over the top surface of the sediment by a normal-load plate with a toothed surface. Dead weights suspended from a series of chains and sprockets on an adjacent tower provided a torque on the axis of the device and thus applied a shear stress to the base of the specimen. If the sediment was sufficiently weak, this torque caused the base to rotate counterclockwise, shearing the sediment between the rotating base and the normal-load plate. The normal-load plate was prevented from rotating by a pair of armatures extending from it and pressing on stationary load-cells. These load-cells (Sensotec model 41/572-05-03), therefore, measured the shear force supported by the sediment, from which the mobilized shear strength of the sediment could be calculated.
Figure 1: The ring-shear device in stress-controlled configuration.
Figure 2: The sample chamber of the ring-shear device showing the toothed lower platen, the interface between upper and lower walls where sliding occurs, and the water reservoir between the inner walls and the steel collar protecting the main bearing.
Figure 3: Line drawings of the ring-shear device in stress-controlled configuration. 

a) To-scale cross-sectional view of major components of the device.  
b) Perspective cross-sectional view of the sample chamber (black), the confining walls, and the upper and lower platens (teeth not shown). Typical cross-sectional extent of the shear zone is indicated by the gray lens in the specimen cross-section. Only the shaded components rotate in both drawings.
Bishop and others (1971) discussed the implications of different assumptions regarding how normal stress is distributed in a ring-shear device, and concluded that it is justifiable to assume uniform normal-load distribution over the surface of the specimen. Therefore, the applied normal stress on the specimen was calculated as follows:

\[
\sigma_n = \frac{m_n g + m_L g \left( \frac{L_2}{L_1} \right) + (m_w + m_h) g \left( \frac{L_3}{L_1} \right)}{\pi \left( r_o^2 - r_i^2 \right)},
\]

(2)

where \( m_n, m_L, m_w, \) and \( m_h \) are the masses of the normal-load plate, lever arm, weights, and hanger, respectively, \( g \) is the acceleration due to gravity, \( L_1, L_2 \) and \( L_3 \) are lengths of the lever arm as shown in Figure 4, and \( r_o \) and \( r_i \) are the radii of the sample chamber from the axis of the device to the outer wall and inner wall, respectively.

The applied shear stress was calculated with the following equation:

\[
\tau = \frac{2 g (m_w + m_h) \left( r_i r_3 \right)}{\pi \left( r_o^2 - r_i^2 \right)}
\]

(3)

where the \( r_1 \) and \( r_2 \) are sprocket radii as designated in Figure 4. The numerator of this term is the applied shear force multiplied by the mechanical advantage, and the denominator is the surface area of the specimen.

The stresses exerted on the inner and outer confining walls of the sample chamber can be measured separately with tension/compression and torque load cells (Sensotec models 41/572-05-03 and QFFH-9/2563-01, respectively) and used to estimate the total stresses within the specimen. In all but the last three experiments, however, the upper and lower walls were pressed together to prevent sediment loss through the wall interface, which prevented accurate measurement of wall stresses. This was justified since earlier
Figure 4: Schematic mechanical diagrams of the shear and normal stress systems in the ring-shear device.
experiments with this device had indicated that considering wall stresses results in only minor (17%) adjustment of total stresses (Iverson et al., 1999).

The change in thickness of a test specimen during shear was measured continuously with Linear Variable Displacement Transformers (LVDTs; Sensotec model 060-3611-02 and S2C-200), mounted at three equally-spaced positions around the perimeter of the device. Each LVDT pressed on a tab extending from the normal-load plate, which was free to move vertically. These LVDTs measured the vertical expansion of the sediment within the sample chamber.

Before each experiment, three vertical columns of twelve wooden beads (4-5 mm diameter) were positioned across the specimen. Afterward, each bead was manually excavated, and its displacement with respect to its initial position was measured. These measurements yielded an approximate shear-zone thickness in a cross-section of the specimen. Assuming that all non-elastic thickness changes measured with the LVDTs occurred in the shear zone, and recognizing that only vertical expansion or contraction was possible, the LVDT record could then be converted into a record of shear-zone porosity change,

$$\Delta n = \frac{\Delta h}{H_z + \Delta h},$$

where $\Delta n$ is the change in shear-zone porosity, $\Delta h$ is the thickness change measured by the LVDTs, and $H_z$ is the total thickness of the shear zone.

In addition to bead excavation, 8-12 samples of sheared sediment were collected to determine the differences in porosity with depth in the sample after each experiment.

Porosity of an excavated sample was found using the relation:
where $V_T$ is the total volume of the excavated sediment sample (measured by exactly filling the plastic-lined excavation hole with water, finding the mass of the water and converting mass to volume assuming a water density of $1.0 \text{ g/cm}^3$), $W_s$ is the weight of the oven-dry soil, $\gamma_w$ is the unit weight of water and $G_s$ is the specific gravity of soil solids. The porosity contrast between samples from the shear-zone (as demarcated by bead-excavation) and those that were relatively unsheared was inferred to be due to dilation with shear. This difference was equivalent to the porosity change as calculated with equation 4.

A fourth LVDT (Sensotec model 060-3611-02) was mounted horizontally on a secured ringstand adjacent to the rotating base of the device. The LVDT pressed on a C-clamp fixed to the rotating base and measured the displacement of the clamp, from which the horizontal shear displacement at the centerline of the specimen was calculated.

Displacement of the weights on the shear-stress tower was also measured in most experiments. An extensometer (UniMeasure model HX-PA-60) was mounted to the top of the tower frame and its cable fixed to the weight hanger. Because of the gear ratio between the shear stress tower and the specimen, the displacement of these weights was ten times the shear displacement at the centerline of the specimen.

Each of the toothed platens that gripped the top and bottom of the specimen had 768 holes (0.5 mm diameter) that communicated water between the sample chamber and a water reservoir maintained at atmospheric pressure. The effective hydraulic conductivity of these platens is $7 \times 10^{-5}$ meter per second. This is several orders of magnitude larger than the most
permeable of the sediments used (Table 1), implying that the platens did not limit pore-pressure diffusion into or out of the specimen.

Miniature pore-water pressure sensors (Honeywell Microswitch 26PCCFA6D differential sensors) were armored with hardened epoxy and dry-vented through PTFE catheter tubing that ran from the sensor through the normal-load plate to the atmosphere. Each finished sensor (Figure 5) was buried in the upper half of the specimen as the normal-load plate was set in place. Sensors were wired with smooth, flexible cable so that they could travel passively with the deforming sediment. In each experiment, two or three pore-pressure sensors were used, but for various reasons, one or more often malfunctioned resulting in variable quality of pore-pressure records. Such malfunctions usually resulted from breakage of the seal around the catheter tube, allowing water and debris to enter the “dry” port of the sensor resulting in erroneous measurements. In two cases, a kinked or pinched cable caused a short-circuit and resulted in sensor failure. When a pore-pressure sensor was found to be malfunctioning, it was disconnected from the data acquisition system to prevent it from influencing other sensors. As a result, reliable pore pressure data is not available from all experiments.

Test Specimens

A total of seventeen experiments were completed using the stress-control configuration (Table 2). Twelve of these were tests on overconsolidated, water-saturated tills, two were the same except normally-consolidated and the remaining three were conducted on overconsolidated dry till and sand. Three different tills with contrasting textures were used (Table 1). One till was taken from the forefield of Storglaciären, a small
Figure 5: Diagrams of pore-pressure sensor preparation. A) Center and profile views of sensors before preparation. B) Cable was soldered to leads, PTFE catheter tubing connected to the dry port, and sensors were potted in epoxy-filled plastic tubing. Monofilament porous fabric was glued over wet port to protect diaphragm from air-bubbles and debris. C) Finished pore-pressure sensor.
Table 1: Physical properties of the tills.

<table>
<thead>
<tr>
<th>Material</th>
<th>Grain size %</th>
<th>Hydraulic Conductivity$^1$ (m/s)</th>
<th>Hydraulic Diffusivity$^2$ (m$^2$/s)</th>
<th>Specific Gravity$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storglaciären Till</td>
<td>75 21 4</td>
<td>$3.6 \times 10^{-8}$</td>
<td>$2.6 \times 10^{-6}$</td>
<td>3.02</td>
</tr>
<tr>
<td>Des Moines Lobe Till</td>
<td>52 32 16</td>
<td>$2.5 \times 10^{-10}$</td>
<td>$6.0 \times 10^{-9}$</td>
<td>3.05</td>
</tr>
<tr>
<td>Engabreen Till</td>
<td>80 14 6</td>
<td>$2.1 \times 10^{-8}$</td>
<td>$8.1 \times 10^{-7}$</td>
<td>2.60</td>
</tr>
</tbody>
</table>

$^1$ Hydraulic conductivity calculated from diffusivity and compressibility assuming Poisson’s ratio ~0.4.

$^2$ Hydraulic diffusivity determined in consolidometer tests (ASTM-D2435-90).

$^3$ Specific gravity of till solids was found using hydrometer method (ASTM-D854-83).

Polythermal glacier in the Kebnekaise massif of northern Sweden. The Storglaciären till is silty, contains no clay-minerals, and is composed of primarily comminuted metamorphic rock particles. A second till came from a Little Ice Age moraine of Engabreen, an outlet glacier of the Svartisen ice cap in Norway. The Engabreen till is sandy and silty, contains some soil organic material, and is also composed of comminuted metamorphic rock particles. The third till was retrieved from the Whatoff gravel pit in Ames, Iowa, and is the lower (“Alden”) member of the Dows formation, a Late Wisconsinan basal till from the Des Moines Lobe of the Laurentide ice sheet. It has a significant fraction of clay minerals (15%, primarily smectites) and contains both crystalline and carbonate rock particles. The Storglaciären till has been used in ring-shear experiments in the past with positive results (e.g., Hooyer and Iverson, 2000).

Although the granular materials used in these experiments were tills, the properties of tills are usually similar to those of fault gouge, owing to their common origin by wear and
Table 2: Summary of stress-controlled ring-shear experiments.

<table>
<thead>
<tr>
<th>Test</th>
<th>Material</th>
<th>Test Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Storglaciären Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>2</td>
<td>Storglaciären Till</td>
<td>Saturated, Normally Consolidated</td>
</tr>
<tr>
<td>3</td>
<td>Storglaciären Till</td>
<td>Saturated, Normally Consolidated</td>
</tr>
<tr>
<td>4</td>
<td>Storglaciären Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>5</td>
<td>Storglaciären Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>6</td>
<td>Des Moines Lobe Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>7</td>
<td>Des Moines Lobe Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>8</td>
<td>Des Moines Lobe Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>9</td>
<td>Des Moines Lobe Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>10</td>
<td>Engabreen Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>11</td>
<td>Engabreen Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>12</td>
<td>Play Sand</td>
<td>Dry, Overconsolidated</td>
</tr>
<tr>
<td>13</td>
<td>Engabreen Till</td>
<td>Saturated, Overconsolidated, Cyclic Load</td>
</tr>
<tr>
<td>14</td>
<td>Storglaciären Till</td>
<td>Dry, Overconsolidated</td>
</tr>
<tr>
<td>15</td>
<td>Storglaciären Till</td>
<td>Dry, Overconsolidated</td>
</tr>
<tr>
<td>16</td>
<td>Des Moines Lobe Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
<tr>
<td>17</td>
<td>Engabreen Till</td>
<td>Saturated, Overconsolidated</td>
</tr>
</tbody>
</table>

crushing of rock. Hillslope materials vary significantly in their physical properties depending upon their location and origin, but their range of properties overlap those of till and gouge. Results of these experiments, therefore, bear on the deformation of both fault gouge and hillslope materials, as well as till.

**Experimental Procedure**

Each till required some preparation before it could be used in an experiment. Observing the guideline that the largest clast's diameter should be no larger than one tenth of
the smallest dimension of the sample chamber (Head, 1989), clasts larger than 6 mm in diameter were manually removed. The remainder was then saturated with a mixture of de-aired water and powdered sodium hexametaphosphate (Calgon bath powder, used as a deflocculant). Specimens were mixed initially by hand and then mechanically mixed using a paint-mixing bit attached to a power drill. The proportions of till and water were adjusted to yield a slurry, since in this form the sediment was clearly water-saturated and filled the sample chamber of the ring-shear device with minimal air bubbles. The till slurry was then scooped in two or three lifts into the sample chamber, de-aired manually (by vibration), and leveled at the top. After preparing bead columns and inserting pore-pressure sensors, the normal-load plate was then placed over the till and allowed to settle. De-aired water was added to the internal reservoir and allowed to permeate the till through the upper and lower platens.

The consolidation load was gradually applied over a period of two or three days until the desired consolidation stress was attained (250 kPa in tests on overconsolidated specimens). When the LVDTs showed that consolidation was largely complete, the normal stress was reduced to a smaller value, 70-95 kPa, and the soil was allowed to rebound for 1-2 days. A shear stress, too small to cause permanent deformation at the reduced normal stress, was then applied and data acquisition commenced. The normal stress was gradually reduced in 1.2-2.5 kPa increments in order to eventually cause permanent shear deformation of the specimen. When a normal-stress reduction resulted in sustained deformation, no further changes in applied stresses were made.
RESULTS

Because of the large volume of data gathered in these experiments, a complete summary of data from each of the experiments is reserved for Appendix 1. Only selected data are shown here to illustrate the behavior of each till. It should also be noted that each experiment consisted of several days of incremental normal-stress reductions before sustained deformation of the sample began. In the interest of brevity and to better highlight the important aspects of the data, only the final stage of each selected experiment is displayed herein.

Figure 6 shows the final two hours of an experiment on the Storglaciären till (Experiment 5). After several days of gradually weakening the specimen, it began to deform irreversibly after approximately 8321 minutes. This was evident because all previous increments of normal-stress reduction resulted in only brief elastic deformations that ended abruptly, whereas the normal-stress reduction that caused permanent deformation was initially rapid, with slower but continued shear thereafter. After this event, no further changes were made to the applied stresses.

This data series illustrates episodic dilatant hardening (Figure 6). When shear deformation of the specimen began, dilation-induced pore-pressure decline caused strengthening and suppression of deformation. Subsequent equilibration of pore pressure triggered renewed shear but deformation was again arrested by dilatant hardening. This complete sequence of events occurred ten times during the period shown in Figure 6 before the specimen failed catastrophically at 8421 minutes. Figure 7 shows an enlarged portion of this time series, highlighting the fourth and fifth episodes. Diffusive pore-pressure rise weakened the specimen causing it to begin to shear. As shear deformation and concomitant
Figure 6: Data from the final 125 minutes of Experiment 5, overconsolidated Storglaciaren till. The arrow indicates a 1.2 kPa reduction in applied normal stress. No further changes were made to applied stresses.
Figure 7: Enlargement of the fourth episode from Figure 6. Periods of dilatant hardening are highlighted in yellow.
shear-zone dilation began to accelerate, pore pressures decreased abruptly causing the shear strength to rise significantly, slowing and sometimes halting shear. Suppressed shear and dilation allowed pore pressure to diffusively recover, again weakening the specimen sufficiently to permit shear deformation.

Similar results were observed in other stress-controlled ring-shear experiments using the two other tills. Figure 8 shows the final three days of data from an experiment with the Des Moines Lobe till (Experiment 9). Twice within this data series, normal-stress reductions were necessary to continue deformation, but all other displacement events were not provoked by changes in externally-applied stresses. The episodic nature of the displacement and the strengthening associated with each shear event suggest that episodic dilatant hardening regulated failure of this specimen. Unfortunately, pore-pressure data do not corroborate that pore-pressure fluctuations were responsible for the observed strengthening. As described in the Discussion section, the lack of a distinct pore-pressure response likely reflects isolation of pore-pressure sensors from the narrow shear zone that developed in the Des Moines Lobe till.

Figure 9 shows data from the final stages of a test on the Engabreen till (Experiment 10). In this case, all pore-pressure sensors failed to return reliable data as a result of a short-circuit caused by a pinched cable. However, like the Des Moines Lobe till in Figure 8, the shear-strength fluctuations indicate that strengthening resulted from incipient deformation. Additionally, similar results were obtained in Experiment 11 (see Appendix A) on the Engabreen till with corroborative pore-pressure fluctuations. Experiment 10 is shown here because several episodes occurred without external perturbations as indicated in the figure
Figure 8: Data from Experiment 9 on the Des Moines Lobe till. The apparent lack of pore-pressure response is likely the result of the very low hydraulic diffusivity of the Des Moines Lobe till causing pore pressure changes to be focused in a narrow region around the shear zone, which was not sampled by the pore-pressure sensor. Δ indicates a reduction in applied normal stress. All other episodes were unprovoked. This series of slow shear episodes was followed by abrupt catastrophic failure at just under 15000 minutes.
Figure 9: Data from Experiment 10 on Engabreen till. No pore-pressure sensors returned reliable data, but sharp shear strength increases followed by gradual decay are indicative of pore-pressure changes. Δ indicates a reduction in applied normal stress, except in the last case when wall stresses were manipulated. Between 10320 and 10540 minutes, the LVDT measuring shearing displacement lost contact with the clamp fixed to the rotating base, resulting in no measurement of displacement during that time. As a result, the cumulative displacement after 10320 is somewhat larger than indicated (approximated by the dashed line).
Like the Storglaciären and Des Moines Lobe till time series, episodic shear due to dilatant hardening was eventually terminated by catastrophic failure, although it is not shown in Figure 9.

Bead displacements from these three experiments indicate that most shear deformation occurred in a discrete plane near the interface between the upper and lower walls of the sample chamber, which is consistent with past observations in the ring-shear device (e.g., Iverson, N. R. et al, 1997). In addition, post-experimental measurements of bulk density show a significant difference between the porosity of the shear zone and the relatively unsheared till above and below (Table 3). This pattern confirms that dilation results from shear.

**Table 3:** Properties of sheared tills from Experiments 5, 9 and 10.

<table>
<thead>
<tr>
<th>Material</th>
<th>Test #</th>
<th>Shear-zone thickness (mm)</th>
<th>Shear-zone porosity</th>
<th>Adjacent porosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storglaciären till</td>
<td>5</td>
<td>26</td>
<td>0.27</td>
<td>0.22</td>
</tr>
<tr>
<td>Des Moines Lobe till</td>
<td>9</td>
<td>18</td>
<td>0.359</td>
<td>0.310</td>
</tr>
<tr>
<td>Engabreen till</td>
<td>10</td>
<td>20</td>
<td>0.360</td>
<td>0.311</td>
</tr>
</tbody>
</table>
DISCUSSION

The quasi-periodic pattern of displacement and dilation, pore-pressure decline and strengthening that occurs in all of the data series shown above are attributed to episodic dilatant hardening. However, similar patterns of displacement have been observed in deformation testing on various materials in the absence of dilatant hardening. Under some circumstances, sliding between frictional surfaces and within granular materials exhibits quasi-periodic oscillations in displacement rate. This “unstable” behavior is often the result of the elastic compliance of the medium surrounding the sliding surface, which in a laboratory setting is the loading device (e.g., Cook, 1981). Before proceeding with a discussion of dilatant hardening, it is necessary to rule out the possibility that the slow episodic nature of shear was governed by the compliance of the ring-shear device.

Influence of Device Compliance on Results

The compliance of the testing apparatus can become an important issue when the stiffness of the test specimen is larger than the stiffness of the loading mechanism. The most important instability that results from the relative softness of the loading device is so-called “stick-slip sliding” (Rabinowicz, 1965; Cook, 1981; Scholz, 1990). In most situations involving frictional deformation, the static friction on a sliding surface is larger than the dynamic or sliding friction, implying that once slip begins, a lower shear stress is required to continue sliding. If the decrease in frictional strength of a specimen occurs over less displacement than that required for relaxation of the loading mechanism’s “spring” (the
rate of dissipation of elastic energy stored in the loading mechanism), a brief departure from stress equilibrium occurs and sliding accelerates (Figure 10). When the spring relaxes sufficiently to restore stress equilibrium, sliding decelerates to a stop, and the specimen "heals" once again to its at-rest frictional strength. Subsequently, stress builds in the loading mechanism and some elastic energy is stored to begin another cycle.

FIGURE 10: Schematic illustration of the stick-slip instability that can affect experimental measurements in "soft" machines. The curved line represents the change in frictional strength, $F_s$ (on a surface where relative displacement occurs) with displacement, $u_s$. Frictional strength declines after its peak at Point A until, at Point B, the slope of this curve exceeds $-k_s$, the stiffness of the device. This results in an imbalance of driving and resisting forces, causing acceleration of displacement. This imbalance continues until, at Point C, the relaxation of the device's "spring" has allowed the applied force to once again equal the resisting force at which point displacement begins to decelerate. At Point D, the area beneath the stiffness line and the frictional strength curve between Points B and C is equal to the same area between Points C and D, at which point displacement stops.

Soft devices commonly cause the applied stress to transiently exceed the post-failure strength of the specimen, resulting in a failure burst and accelerated slip until the "spring" in the loading system relaxes sufficiently to restore stress equilibrium. Precise determination of the stiffness of the ring-shear device is challenging due to the complexity of the device and
its interaction with the specimen (though an estimate is made in Appendix B). However, if
the stick-slip instability were to operate in the ring-shear device upon failure of a specimen,
accelerating displacement would be accompanied by a decrease in the measured shear
strength. The associated reduction in shear force would necessarily be more negative than
\(-k_m u_s\), since the criteria for the stick-slip instability is

\[
\left| \frac{\partial F_s}{\partial u_s} \right| > k_m ,
\]  

(7)

where \( F \) is the shear force (Figure 10) (Scholz, 1990). Instead, in each ring-shear
experiment, as slip accelerates shear strength increases. Thus, the stiffness of the ring-shear
device does not appear to play a clear role in the episodic shear observed in these experiments.

Instead, it appears that the device softness permits these laboratory experiments to
more accurately mimic common stress-strain interactions in nature. Storage of elastic energy
in the media bounding a shear zone is well-documented in fault mechanics (Scholz, 1990)
and glacier-bed mechanics (Bahr and Rundle, 1996; Fischer and Clarke, 1997; Iverson et al,
1999) and may play a role in hillslope failure when shear stress is a significant percentage of
the shear modulus of the hillslope material (e.g., Gomberg et al, 1995).

**Influence of Intrinsic Material Properties**

In addition to the experiments on saturated, overconsolidated till, a number of other
tests were conducted with the ring-shear device to determine whether processes other than
dilatant hardening were controlling deformation. A pair of experiments were conducted on
normally consolidated Storglaciären till. In these experiments (Experiments 2 and 3 in
Appendix A), all preparations and boundary conditions were the same as tests on overconsolidated till except that no preconsolidation load was applied. In both tests, complete catastrophic failure occurred immediately or shortly after the specimens were subjected to a shear stress. This was the expected outcome if the slow shear of overconsolidated specimens was dependent upon shear-induced dilation.

Another pair of experiments was conducted using overconsolidated Storglaciären till, but without pore water (Experiments 14 and 15 in Appendix A). The till was oven-dried, aggregates were crushed manually, and the till was otherwise tested in the same manner as other tests on overconsolidated specimens. Failure in these experiments was also abrupt once irreversible deformation began, and was not marked by episodic shear. These results indicate that the episodic slow shear observed in the saturated, overconsolidated tests was not due to an intrinsic property of the till or the coupling between the till and the device.

The results of these auxiliary tests, together with the insignificance of device effects, support our hypothesis that the slow, episodic shear that was observed in experiments like Experiments 5, 9, and 11 (Figures 6, 8, and 9, respectively) were unrelated to intrinsic properties of the materials tested or the testing apparatus. Clearly, the slow deformation is regulated by the coupling between a saturated, dilating granular material and its pore fluid.

**Influence of Pore Pressure**

If abrupt decline in pore pressure resulting from dilatant shear is responsible for strengthening, the magnitude of an increase in shear strength should be proportional to the product of the effective stress increase and the tangent of the angle of internal friction. This can be illustrated using data from the portion of Experiment 5 shown in Figure 6. It is well
understood that the friction angle of a shear zone decreases slightly as it dilates toward its critical state porosity (e.g., Rowe, 1962), and at critical state there is a unique residual strength for a given effective stress. The angle of internal friction of the Storglaciären till at critical state has been measured with the ring-shear device in the past (26.3° ± 0.9°, Iverson et al., 1998). If a linear relationship is assigned between the porosity change and friction angle, the shear strength of the Storglaciären till can be calculated during the final two hours of Experiment 5 by multiplying this simulated friction term by the known effective stress. This results in the Coulomb-Terzaghi equation modified to include a porosity-dependent friction angle,

\[ \tau = (\sigma - p)\{\tan(\phi_e + (\xi(n_c - n))}\}, \quad (8) \]

where \( \phi_e \) is the shear strength at critical state, \( n_c \) is the critical-state porosity, \( n \) is the instantaneous measured porosity, and \( \xi \) is the coefficient that relates change in porosity to change in friction angle. To apply this equation to the measurements of Experiment 5, the critical-state friction angle is taken to be the value measured at the moment catastrophic failure began (24.5°), at which time the porosity difference \( n_c - n \), is zero. A comparison between the shear strength curve generated with this equation and the measured shear strength is shown in Figure 11. The timing and magnitude of each strengthening event is reasonably well predicted by Equation 8, indicating that the pore-pressure fluctuations are indeed responsible for the measured strengthening and suppression of shear.

The absence of pore-pressure response from Experiment 9, despite proper functioning of one pore-pressure sensor, is not surprising. The sharp peaks in shear strength accompanying episodic displacement followed by exponential shear-strength decay indicate that, like in Experiment 5, dilatant hardening regulated shear of the specimen. However,
Figure 11: Comparison between shear strength calculated with Equation 12 and the measured shear strength from Experiment 5.
because the Des Moines Lobe till has such a low hydraulic diffusivity, pore-pressure fluctuations were felt only very near the shear zone.

To illustrate this spatial damping of pore pressure, consider a simple heat-flow analogy. Suppose an infinite solid is at a uniform temperature everywhere. At $t = 0$, a heat source is turned on at $z = 0$ (a plane in the solid) for an instant, and then off again. As time proceeds, this pulse of heat spreads into the solid, and the temperature at $z = 0$ slowly decays as governed by the process of heat conduction. The analogous pore-pressure diffusion scenario is similar to the sequence of events that is observed in a single dilatant hardening episode: shear failure causes a nearly instantaneous drawdown of pore pressure in the shear zone, causing shear to stop and allowing the pore pressure to gradually diffuse into the shear zone. The solution to this diffusion problem,

$$u_n = \frac{1}{\sqrt{t}} \exp \left( - \frac{z^2}{4Dt} \right),$$

where $u_n$ is the non-hydrostatic pore pressure, $t$ is time, and $D$ is the hydraulic diffusivity of the till, can be used to predict the spatial distribution of pore pressure after an initial perturbation in the shear-zone, $z = 0$ (Eckert and Drake, 1972, Equation 4-54). Figure 12 shows the decay of non-hydrostatic pore pressure after an instantaneous decline measured at three distances from the shear zone in the Des Moines Lobe till ($D = 6.0 \times 10^{-9}$ m$^2$/s). Clearly, even if the pore-pressure sensor was only 1 mm from the shear zone, it would only have measured 5% of the magnitude of any pore pressure fluctuation, and that signal would have been delayed by two minutes. Because the location of the sensing port of a pore-pressure sensor could not be controlled on a millimeter-scale, it is likely that sensors were more than 1 mm from the shear zone, and therefore not surprising that pore-pressure
FIGURE 12: Calculated pore pressure at three locations with respect to the shear zone in the Des Moines Lobe till \( (D = 6 \times 10^9 \text{ m}^2/\text{s}) \). A unit decline in pore pressure at time = 1 sec. is damped and delayed significantly when measured a small distance from the location of the perturbation. As a result, a pore-pressure sensor located more than 1 mm from the shear zone measures less than five percent of the magnitude of the pore pressure decline.
fluctuations like those observed in Experiment 5 were not observed in Experiment 9, particularly since the shear zone in the Des Moines Lobe till was typically thinner than in other tills (Table 3).

For a material to be subject to dilatant hardening, it must be initially overconsolidated and its hydraulic diffusivity must be sufficiently low to preclude rapid equilibration of pore pressures when perturbed. The first of these two conditions should be true of many natural stress paths to failure. Since the porosity of a natural granular material is commonly a function of the greatest effective normal stress it has experienced since being disturbed or sheared, a material brought to failure by a decrease in effective stress should dilate when sheared. Moreover, materials that are at a porosity greater than their critical-state porosity do not shear slowly when brought to failure under steady stresses – they fail catastrophically as incipient motion causes them to consolidate, elevating pore pressure and reducing intergranular friction (Lambe and Whitman, 1969, p. 443; Iverson, et al, 2000).

Additionally, many experimenters have shown that dilation accompanies increases in shear-strain rate in fault gouge (e.g., Morrow and Byerlee, 1989). The second condition, associated with hydraulic diffusivity, is somewhat more complex. Dilatant hardening results only when the rate of pore-pressure diffusion cannot keep pace with the rate of dilation, and this second condition can be investigated approximately by considering two characteristic timescales. The timescale for pore pressure diffusion, $t_d$, is $L^2/D$, where $L$ is the length of the shortest drainage path and $D$ is the hydraulic diffusivity of the material (e.g., Iverson and LaHusen, 1989). Vertical displacement resulting from dilation is related to shearing displacement by a dimensionless dilatancy parameter $\psi$ which varies for different materials and for a single material at different initial porosities. If we define dilation as positive vertical displacement
and compaction as negative, \( \psi = 0 \) corresponds to deformation at the critical-state porosity.

The timescale for acceleration of dilation is simply the dilatancy parameter multiplied by the timescale for acceleration of shear displacement, \( t_a \), which is equal to \( \psi (L/a)^{1/2} \) (cf. Iverson, et al., 2000) where \( a \) is acceleration. Acceleration is somewhat difficult to estimate. Aside from earthquake accelerations, the upper bound of \( a \) will be the acceleration due to gravity, but in reality the acceleration could be much smaller than that. For this analysis, we use the maximum acceleration (second derivative of the displacement record) at the instant shearing deformation is measured for each till. If the ratio between the timescale for diffusion and the timescale for acceleration of dilation, \( S = t_d/t_a \), is much larger than 1, the material should exhibit dilatant-hardening behavior when sheared under steady applied total stresses. If \( S \ll 1 \), dilatant hardening will not occur because sufficiently rapid pore-pressure diffusion can take place to maintain near-hydrostatic pore pressure. In the case of the three tills used in the ring-shear experiments, \( S \) ranges from 4-1000 (Table 4), indicating that each till will be susceptible to dilatant hardening.

**Table 4:** Characteristic timescales for diffusion and dilation. Accelerations used for calculation of the dilation timescale were the largest instantaneous accelerations measured. The characteristic length in both timescales was the measured thickness of the shear zone.

<table>
<thead>
<tr>
<th>Material</th>
<th>Timescale for dilation</th>
<th>Timescale for diffusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Storglaciären till</td>
<td>15 sec.</td>
<td>200 sec.</td>
</tr>
<tr>
<td>Des Moines Lobe till</td>
<td>17 sec.</td>
<td>17,000 sec.</td>
</tr>
<tr>
<td>Engabreen Till</td>
<td>30 sec.</td>
<td>120 sec.</td>
</tr>
</tbody>
</table>
Another important consideration is the sustainability of dilatant hardening cyclicity. Clearly it is impossible for a shearing granular material to dilate indefinitely – there is a "critical state" at which no further pore expansion can occur, as illustrated by the catastrophic failure in each of these ring-shear experiments (e.g., Figures 6, 8). The shear strain necessary to reach this critical state is a function of the initial density of the material. However, in typical geological environments, external effective-stress forcing is unsteady due to diurnal, seasonal, or multiannual cycles of moisture flux and associated pore-water pressure. If the critical state is not reached while ambient effective stresses are low, the material may reconsolidate or "heal" when ambient effective stresses return to higher values (Sleep, 1995). In this way, dilatant hardening may represent a sustainable process.

**Influence of Episodic Dilatant Hardening on Shearing Speed**

In slow shear regulated by episodic dilatant hardening, the time-averaged shearing speed depends on both the displacement that occurs in each episode and the episode recurrence interval. To learn more about these dependencies, two parameters can be defined to characterize individual episodes from the laboratory data. An episode is defined here from the moment that rapid shearing starts until a subsequent rapid shearing event begins. Let the average shearing speed of an episode \( \bar{v} \) be the total shearing displacement during that episode, \( x \), divided by the episode duration, \( t_e \):

\[
\bar{v} = \frac{x}{t_e}.
\]  

(10)

The average dilatancy, \( \bar{\zeta} \), is defined as the thickness change, \( \zeta \) (vertical displacement), divided by the shearing displacement:
Figure 13 shows the average episode shearing speed as a function of average dilatancy for episodes compiled from tests on the three overconsolidated tills. To ensure that the episode durations were complete and that only shear-induced dilation (as opposed to elastic expansion) was included in measurements of thickness change, only episodes that were not artificially terminated by an external change in applied stress were considered.

One significant pattern revealed by Figure 13 is that, for a given average dilatancy, the average shearing speed is lower for less diffusive materials. The simplest explanation for this pattern is that for a given drawdown in pore pressure, in less diffusive sediment it takes
longer for diffusion to raise pore pressure sufficiently for renewed deformation. Thus, in effect, a lower hydraulic diffusivity leads to longer episode recurrence intervals at a given state of dilatancy.

A second significant pattern illustrated by Figure 13 is the increase in average shearing speed as dilatancy decreases. Thus, as a material approaches critical-state porosity in its shear zone, it shears faster. This effect may result from two different factors. As a shear zone approaches its critical-state porosity, more shearing displacement is needed to cause a given amount of dilation, and thus, more displacement may occur at the onset of an episode before shearing is halted by dilatant hardening. An alternative explanation is that dilation in the shear zone causes a significant increase in its porosity, resulting in locally enhanced diffusivity, pore pressures closer to hydrostatic, and less strengthening. However, if pore pressure diffusion is limited principally by the unsheared low-diffusivity material above and below the shear-zone, this may be a small effect. In reality, both of these mechanisms may contribute to varying extents to creating this pattern.

These two observations lead to the conclusion that, if episodic dilatant hardening alone regulates slow shear, the time-averaged shearing speed should be smaller if the material has a lower hydraulic diffusivity and/or a higher dilatancy. More generally, denser and less diffusive materials should shear more slowly. As indicated by Figure 13, this relation can be more easily visualized with an exponential relationship,

\[ \bar{v} = Be^{(-mw)} \]  

(12)

where \( m = f(D) \), and \( B \) is a constant that represents the highest shearing speed that can be reached during catastrophic failure. The effect of a large hydraulic diffusivity would be similar to the effect of dilatancy approaching zero, so \( m \) is related to \( S \), the timescale ratio
described previously. If $S < 1$ or dilatancy approaches zero, the average shear rate should approach $B$. However for $S > 1$ and significant dilatancy, episodic dilatant hardening will occur and the average shearing velocity will be very small compared to $B$.

Given these generalizations, it may be possible to outline a critical external forcing timescale that would help to determine whether catastrophic failure or reconsolidation would be likely to occur first. In the case of a process which is forced by a periodic water-pressure oscillation (e.g., some hillslopes and most temperate, soft-bedded glaciers), the timescale, $t_f$, for positive forcing stretches from the time that hydrostatic pore pressure rises past the threshold for failure to the time at which hydrostatic pore pressure again drops below the failure threshold. The period of time that episodic dilatant hardening can be sustained before catastrophic failure occurs, $t_c$, is

$$t_c = \frac{d_c}{\nu},$$

where $d_c$ is the amount of shearing displacement necessary for dilatancy to reach zero under a given effective stress. Since $d_c$ varies significantly with grain-size distribution and shear-zone thickness, consider a representative range of displacements from 3 mm to 400 mm (cf., Lupini et al, 1981). Using shear velocities for dense, low-diffusivity materials of 1 mm/day and for less dense, more diffusive materials of 500 mm/day, the timescales for reaching catastrophic failure range from 8.6 minutes to more than a year. Thus, if this timescale is longer than the timescale either for $S$ to become less than 1 or for the timescale for dilatancy to reach zero, catastrophic failure will eventually ensue. However, if $t_f$ is shorter, the material might reconsolidate before catastrophic failure occurs and permit sustainable slow displacement when a subsequent pore-pressure increase initiates shear.
IMPLICATIONS

It is clear from the preceding discussion that if a material undergoes sustained displacement regulated by dilatant hardening, the resulting time-averaged shearing speed should be strongly influenced by the porosity and hydraulic properties of the deforming medium and the length of the shortest drainage path. There is no indication from these data that shearing speed depends on applied shear stress, and hence there is no evidence of viscous or viscoplastic behavior. These conclusions have significant implications for interpretations of creep behavior in near-surface environments, as well as modeling earthquake, landslide, and glacier processes.

Hillslope Processes

Geologists and engineers have long understood the mechanisms by which slopes become unstable due to the interaction between soil and pore water, but only much more recently has it been acknowledged that in some situations, pore-water provides transient support for unstable hillslopes (Vaughan and Walbancke, 1973; Humphrey and Leonards, 1985). Some studies of instrumented hillslopes have recognized a pattern of increasing displacement rate accompanied by sharp decreases in pore pressure prior to catastrophic failure (e.g., Harp, et al, 1990). These events are well explained within the context of dilatant hardening. If a hillslope is subjected to a sustained period of sufficiently high pore pressure to induce motion, episodic dilatant hardening can give rise to episodes with gradually increasing frequency and velocity. If external forcing (e.g., a decrease in water flux and pore pressure) does not restabilize the slope before the critical state is reached in the
shear zone, catastrophic failure ensues. This mechanism of transition from slow, meta-stable creep to unstable failure may explain how destructive debris flows mobilize unexpectedly from landslides (cf. Iverson, R. M., et al., 1997). Debris-flow hazard mitigation could be improved if the critical timescales or length-scales for stability of dilatant-hardening could be determined for creeping landslides.

Glacier-Bed Mechanics

Large-scale glaciological models that include glacier-bed deformation continue to use viscoplastic till rheologies (Ng, 2000; Thorsteinsson and Raymond, 2000), despite rapidly-accumulating evidence against such behavior in tills (Kamb, 1991; Iverson et al., 1998; Tulaczyk et al., 2000; Kavanaugh and Clarke, 2001). However, in the absence of a viscous component to till deformation, it is difficult to model low strain-rates that must accompany deformation at glacial speeds. Dilatant hardening is a mechanism that can help to rationalize this time-dependent slow deformation within the context of Coulomb-plasticity. If dilatant hardening periodically affects till deformation, rates of deformation could be predicted using the relationships between shearing velocity, diffusivity, drainage length and dilatancy discussed above. This could lead to a better understanding of glacier dynamics in the presence of a deformable till substrate. Glacier driving stress need not correspond to shear rates in subglacial tills if episodic dilatant hardening regulates shear, and, by extension, changes in driving stress do not necessarily correspond to changes in till transport rates by bed deformation.
Fault Gouge Mechanics

Below a critical depth in the crust, slow shearing of fault-zone materials is accommodated by ductile processes (Marone and Scholz, 1988; Scholz, 1990). However, nearer to the surface these processes do not operate, and frictional deformation dominates relative motion on faults. Although earthquakes result from rapid frictional displacement along fault surfaces, there are many places where slip occurs on faults stably without significant seismicity. These are segments of faults that creep gradually and produce only minute, often periodic, slip events (e.g., Nadeau et al, 1995; Miller, et al, 2002). As a result, these fault segments release strain aseismically and reduce the local risk of destructive earthquakes. Aseismic, episodic creep on such fault segments may be facilitated by quasi-stable shear of gouge regulated by dilatant hardening.

Seismologists have occasionally attributed patterns in the magnitude and periodicity of aftershocks to dilatant hardening (e.g., Nur and Booker, 1973), though not necessarily restricting the effect to fault-zone materials. They reason that exponentially-decreasing magnitude and frequency of aftershocks following an earthquake may be the signature of hydraulic processes controlling slip on the fault plane. If only episodic dilatant hardening within granular fault-zone materials is allowed, the results presented here suggest that not decreasing, but rather increasing frequency and magnitude should typify aftershock slippage.

Many seismologists have noted the tendency for water wells adjacent to faults to undergo unusual water-level fluctuations immediately before and during nearby earthquakes (Johnson et al, 1973; Kovach et al, 1975). Use of these water-well measurements in predicting imminent earthquakes, though promising, has been difficult because of the spatial and temporal complexity of precursory water-pressure changes. However, if water-pressure
fluctuations in fault-proximal wells are interpreted in the context of episodic dilatant hardening, they could still prove useful in fault monitoring and earthquake prediction.
CONCLUSIONS

Episodic dilatant hardening has been measured in several stress-controlled ring-shear experiments on tills of varying textures. The results of this laboratory work highlight the importance of considering the coupling between rates of shear and pore pressures in mechanical models of deforming granular materials. The data indicate that episodic dilatant hardening is a viable mechanism for suppressing incipient Coulomb-plastic failure of granular materials and that it can impart transient stability. The efficacy of dilatant hardening depends on the initial porosity of the granular material with respect to its critical-state value and its hydraulic diffusivity. These two material properties can be measured independently in the laboratory and the field, and thus, in principle, can allow prediction of deformation rates if the appropriate boundary conditions are known.
APPENDIX A: EXPERIMENTAL RESULTS
<table>
<thead>
<tr>
<th>TEST #</th>
<th>MATERIAL</th>
<th>TEST TYPE</th>
<th>OC Load</th>
<th>Start Normal Stress</th>
<th>Final Normal Stress</th>
<th>Final Shear Stress</th>
<th>Strength</th>
<th>Final Shear Strength</th>
<th>Total Displacement</th>
<th>Total Dilation</th>
<th>Shear Zone Thickness</th>
<th>Zone Porosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Storglaciaren Till</td>
<td>S,OC</td>
<td>250</td>
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<td>24.8</td>
<td>11.7</td>
<td>10.6</td>
<td>39.7</td>
<td>1.29</td>
<td>14</td>
<td>0.457</td>
</tr>
<tr>
<td>2</td>
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<td>18.8</td>
<td>18.8</td>
<td>17.8</td>
<td>17.8</td>
<td>4.2</td>
<td>4.2</td>
<td>140</td>
<td>-0.93</td>
<td>2</td>
<td>NA</td>
</tr>
<tr>
<td>3</td>
<td>Storglaciaren Till</td>
<td>S,NC</td>
<td>-</td>
<td>18.8</td>
<td>7.2</td>
<td>9</td>
<td>2</td>
<td>3.2</td>
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**Table A1**: Summary of results from stress-controlled ring shear experiments.
Experiment 1

**Figure A1**: Experiment 1 with overconsolidated Storglaciaren till.
Experiment 2

Figure A2: Experiment 2 on normally consolidated Storglaciaren till.

Notes:

1) Time scale is in seconds.

2) Specimen failed immediately upon application of shear stress.

3) Horizontal displacement continued after 34 seconds, although data from the horizontal LVDT only recorded the first 5 mm.
Experiment 3

Figure A3: Experiment 3 on normally-consolidated Storglaciaren till.

Notes:

1) Time scale is in seconds.

2) Initial applied shear stress was very low. Shear stress was then increased in three small increments until significant motion occurred.

3) Complete, catastrophic failure occurred after the shear-stress increase at 405 seconds.
Experiment 4

Figure A4: Experiment 4 on overconsolidated Storglaciaren till.

Notes:

1) All shearing displacement occurred in upper 31 mm of the specimen.
Experiment 5

Figure A5: Experiment 5 on overconsolidated Storglaciaren till.
Experiment 6

Figure A6: Experiment 6 on overconsolidated Des Moines Lobe till.
Experiment 7

Figure A7: None.

Notes:

1) Experiment was aborted due to significant loss of specimen through inner wall interface.

2) Sample chamber was refilled and reconsolidated and begun as Experiment 8.
Experiment 8

**Figure A8:** Experiment 8 on overconsolidated Des Moines Lobe till.

**Notes:**

1) Restarted after aborting Experiment 8.

2) Small fluctuations in pore pressure (~10 minute period) are an artefact of ambient pressure or temperature changes in the laboratory due to cycling of air-conditioner.
Experiment 9

Figure A9: Experiment 9 on overconsolidated Des Moines Lobe till.

Notes:

1) 2 of 3 pore-pressure sensors failed. The third sensor did not document pore-pressure changes for reasons outlined in the Discussion section.
Experiment 10

Figure A10: Experiment 10 on overconsolidated Engabreen till.
Experiment 11

Figure A11: Experiment 11 on overconsolidated Engabreen till.
Experiment 12

Figure A12: Experiment 12 on dry, overconsolidated play sand.

Notes:

1) All shear deformation and dilation except for the event at 210 minutes and final failure at 288 minutes was the elastic response to normal-stress reductions. Failure commenced initially at 210 minutes but was apparently halted by the slight increase in normal stress and/or decrease in shear stress supported by the specimen resulting from wall effects.
Experiment 13

Figure A13: Experiment 13 on overconsolidated Engabreen till.

Notes:

1) After initial reductions of normal stress as in other experiments, the applied normal stress was cycled 2.5 times over a 12 kPa range using a water reservoir attached to the lever arm. Normal stress was gradually increased by allowing water to drain under constant head into the reservoir, and gradually decreased by draining water under falling head out of the reservoir.

2) The results shown do not include the final, catastrophic failure. The results in Figure A13 demonstrate that rising effective stress on a till very close to its yield strength can allow the material to compact, supporting the hypothesis that reconsolidation may provide a mechanism for sustainability of dilatant-hardening regulated creep.
**Experiment 14**

![Graph showing Shear Strength, Horizontal Displacement, Thickness Change, and Applied Normal Stress over time.](image)

**Figure A14:** Experiment 14 on dry, overconsolidated Storglaciaren till.

**Notes:**

1) Experiments 14 and 15 were performed on the same specimen, with a period of reconsolidation between tests. Experiment 14 was performed with the upper and lower walls pressed together like all other previous tests, whereas Experiment 15 was performed with the upper walls pulled away from the lower walls. These experiments were expected to demonstrate the lack of an intrinsic strengthening mechanism, and also to isolate what affect the wall-interface friction might have on the behaviour of a specimen in the absence of dilatant hardening.

2) All deformation before 421 minutes is the elastic response to reductions in applied normal stress. Catastrophic failure ensued immediately after the reduction at 421 minutes.
Experiment 15

Figure A15: Experiment 15 on dry, overconsolidated Storglaciaren till.

Notes:
See notes for Experiment 14.
**Experiment 16**

![Graph showing horizontal displacement, thickness change, and shear strength over time.](image)

**Figure A16**: Experiment 16 on overconsolidated Des Moines Lobe till.

**Notes:**

1) This experiment was performed with tension on the upper walls (a "wall-gap test").

2) None of the pore-pressure sensors used in this experiment recorded reliable data.
Experiment 17

Figure A17: Experiment 17 on overconsolidated Engabreen till.

Notes:

1) This experiment was performed with a wall gap.

2) Catastrophic failure took place at 22908 minutes after a period of relatively rapid creep in short episodes that followed a increase in wall tension (decrease in wall load). Because these episodes may have been provoked by this wall-load manipulation, they are not used in the analysis for Figure 13.
APPENDIX B: DEVICE STIFFNESS

The stiffness of the shear-stress system in the ring-shear device can be estimated by envisioning a simpler analog. A model for the uniaxial extension of a metal rod considers the lengthening of the rod, \( u_s \), and the displacement of the loading system, \( u_w \). The displacement of the loading system results from the combination of the lengthening of the specimen and elastic deformation of the components of the device, such that:

\[
 u_w = u_s + \frac{\tau A_0}{k_d},
\]

where \( \tau \) is the applied stress, \( A_0 \) is the cross-sectional area, and \( k_d \) is the stiffness of the device (Glazov, et al., 1982, Eq. 1). Rearranging Equation B1,

\[
 k_d = \frac{\tau A_0}{u_w - u_s}. 
\]

While the stress-controlled ring-shear device has many more components than that of the simple example above and does not have the same uniaxial geometry, the bulk stiffness of the shear-stress system can be estimated if Equation B2 is adapted to account for the gear ratio of the ring-shear device. As illustrated in Figure B1, if the ring-shear device were perfectly stiff, the displacement of the weights, \( u_w \), would be ten times the shearing displacement at the centerline of the specimen, \( u_s \). Thus,

\[
 k_d = \frac{\tau_s A_0}{0.1u_w - u_s},
\]

where \( \tau_s \) is the applied shear stress and \( A_0 \) is the surface area of the specimen. To remain consistent with the example given by Glazov and others (1982), displacements of the weights and rotating base measured during storage of elastic energy, rather than release (i.e., periods
Figure B1: Role of elastic compliance of the ring-shear device in the shear of a specimen. A) Forces and displacements in the stress-controlled ring-shear device. B) Mechanical analog: spring and slider-block with gravitational driving force. Granular material, in stippled pattern, shears on both sides of the slider block when the driving force exceeds the resisting force, $\mu F_N$, where $\mu$ is a coefficient of friction.
when little shear of the specimen occurred but there was appreciable displacement of the weights) were used to calculate $k_d$. The mean of four such measurements yields an approximate device stiffness of 4500 kN/m ($\pm$ 1390 kN/m).

For comparison, the shear moduli of the three tills can be calculated using deformation data from the rebound phase of oedometer tests and the relationship

$$G = J \left( \frac{1-2\nu}{2-2\nu} \right),$$

where $G$ is the shear modulus, $J$ is the confined modulus measured in the oedometer test, and $\nu$ is Poisson’s ratio (Lambe and Whitman, 1969, Eq. 12.4 and 12.8). Young’s modulus can be converted to a stiffness using

$$k = \frac{A_o E}{l},$$

where $l$ is the “length” of the specimen (Jaeger and Cook, 1979). Substituting $G$ for $E$ in Equation B5 and taking $l$ to be the average thickness of the specimen in the ring-shear device, the stiffness of a sample in shear is

$$k_s = \frac{A_o}{l} \left[ J \left( \frac{1-2\nu}{2-2\nu} \right) \right].$$

The sample stiffnesses calculated using Equation B6 (3,000-25,000 kN/m) are slightly larger than the stiffness calculated for the device except when Poisson’s ratio is very small ($<0.2$). Unfortunately, Poisson’s ratio for these materials is not well known, but sandy materials near shear failure typically have $\nu$ greater than 0.2 (Lambe and Whitman, 1969, p. 160). Therefore, it appears that the stress-controlled configuration of the ring-shear apparatus is slightly “soft” compared to the tills used in these experiments, and may be theoretically susceptible to instabilities like stick-slip oscillations.
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