Degradation of Rock and Shotcrete Due to Ice Pressure and Frost Shattering

A REVIEW

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PREFACE

This literature review is part of a research project initiated by Banverket. The purpose of this report is to gather experience and information about how ice is formed and how ice pressure influences fault zones, cracks and the interface between rock and shotcrete. The financial support for this project is being provided by Banverket HK in Borlänge.

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SUMMARY
In the past few years Banverket has observed an increase in the number of incidences involving fall outs of shotcrete and rock in railway tunnels. Several research projects were therefore initiated by Banverket. This project “Degradation of rock and shotcrete due to ice pressure and frost shattering” is one of them. The purpose of this report is to gather experience and information about how ice is formed and how ice pressure influences fault zones, cracks and the interface between rock and shotcrete.

When water freezes, a 9% volumetric expansion occurs according to the phase transition into ice. This can exert a pressure on the adjacent material. If there is water in small openings such as pores or cracks that cannot allow for a 9% volume expansion, breakage of the adjacent material will occur. The volumetric expansion of water/ice can only be prevented by pressures of 207 MPa, which is the pressure melting point of ice. As a comparison, consider a rock with a tensile strength in the order of 10 MPa. This rock material cannot prevent the ice from forming. Therefore, in a saturated rock there will always be a breakage if the water is freezing in a confined space.

If the pressure of ice exceeds the tensile strength of the adjacent material, the material will be damaged, and the degree of damage is besides other factors dependent on the degree of saturation of the rock. A partially saturated rock can resist breakage despite its low strength because the expansion of ice and distribution of pore water can occur in pores that were initially filled with air. A fully saturated rock however yields to frost action regardless of its strength, because it doesn’t have any free space, which is needed for the expansion.

But the 9% volumetric expansion is not the only cause of frost shattering. Research shows that the frost action in rocks is the same as in soils when the rock has access to water during freezing. In soil, water is drawn towards the frozen fringe and causes ice lenses to grow. In a similar manner water tends to migrate in rock and causes growth of ice bodies inside pores or cracks. The water migration takes place due to the fact that a thin water film of adsorbed water occurs at the surface of mineral particles and it is in this water film that the water has the opportunity to migrate towards the frozen zones. Experimental work has shown that considerable amount of adsorbed water remains unfrozen at subfreezing temperatures not only in soils, but also in rocks. This encourages the migration. With continuous decrease of temperature, the adsorbed water in the water film starts to freeze and the part of unfrozen water is reduced. Thus with decreasing temperature the water film, which separates the ice from the solid particles become thinner. This reduces the permeability of the material and inhibits the water migration towards the frozen fringe. With further decreasing of the temperature the migration can stop and so also the growth of ice bodies.
The water migration and thereby the ice growth is not only dependent upon access to water and freezing temperatures, but also on the freezing rate and duration. If the rock is exposed to a rapid freezing rate, the thickness of the water film is quickly reduced and the water migration becomes inhibited, which delimits the frost damages of rock and shotcrete. In contrast, slow freezing rate permits water migration to occur for a longer period, which can result in greater frost damage of rock and shotcrete.

Keywords: Ice pressure, rock and shotcrete degradation, water migration, adhesion, frost shattering.
SAMMANFATTNING

På senare år har Banverket märkt en ökning av inrapporterade nedfall av berg och sprutbetong i sina järnvägstunnlar. I och med detta så startades en rad forskningsprojekt kring problemen med vattenläckage och isbildning i tunnlar. Detta projekt ”Nedbrytning av berg och sprutbetong på grund av istryck och frostsprängning” är ett av dessa. Syftet med denna rapport är att samla erfarenhet och information om hur is bildas samt hur istryck påverkar krosszoner, sprickor och skiktet mellan berg och sprutbetong.

När vatten fryser till is sker en 9 % volymsutvidgning och denna expansion kan orsaka att ett tryck uppstår mot det omgivande materialet. Om vattnet befinner sig i ett litet innestängt område, exempelvis i en por eller i en sluten spricka som inte tillåter att en 9 % volymsutvidgning sker, kommer ett brott att uppstå i det omgivande materialet. Volymsutvidgningen som sker vid isbildning kan bara förhindras genom att trycksätta vattnet/isen med ett tryck från det omgivande materialet på 207 MPa, vilket motsvarar isens trycksmältpunkt. Som en jämförelse är draghållfastheten i berg ca 10 MPa vilket leder till att brott alltid kommer att uppstå i vattenmättat berg om vatten fryser i ett innestängt område.

Det omgivande materialet kommer att utsättas för brott om trycket från isen överstiger materialets draghållfasthet, men storleken på skadan är bland annat beroende av materialets vattenmättnad. Ett delvis vattenmättat material kan klara sig från brott, trots att dess draghållfasthet är låg, genom att expansionen av isen och omfördelning av porvatten kan ske i de porer som från början var fyllda med luft. Ett helt vattenmättat material ger istället efter för frostsprängningen oberoende av sin draghållfasthet, på grund av att materialet inte har något fritt utrymme som kan ta upp expansionen.

Men det är inte bara den 9 % volymsutvidgningen som orsakar frostsprängning. Forskning visar att om berg har tillgång till fritt vatten under nedkylningen, sker en process som liknar tjällyftning i jord. I jord vandrar vatten fram mot frysfronten och där bildas islinser. På ett liknande sätt verkar vatten vandra i berg och orsaka att iskroppar växer i porer och i sprickor. Vattenandringen sker på grund av det faktum att det finns en tunn vattenfilm av adsorberat vatten längs ytorna av mineraalkornen och i denna vattenfilm finns möjlighet för vatten att vandra mot frysfronten. Experimentellt arbete har visat att en betydande del av det adsorberade vattnet förblir ofruset vid negativa temperaturer, inte bara i jord utan även i berg och detta gynnar vattenandringen. Men när temperaturen sjunker mer och mer börjar även det adsorberade vattnet i vattenfilmen att fryska och andelen ofruset vatten minskar. Så med sjunkande temperatur blir vattenfilmen, som separerar isen och mineraalkornen, tunnare och tunnare. Detta reducerar vattenandringen fram mot frysfronten. Om temperaturen fortsätter att sjunka, kan vattenandringen avta helt och så även istillväxten.
Vattenvandringen och istillväxten är inte bara beroende av tillgången till vatten och frystemperatur, utan även av fryshastighet och varaktighet av köldgrader. Om berget utsätts för snabb nedfrysning (hög fryshastighet) minskar vattenfilmens tjocklek fort och vattenvandringen förhindras, vilket begränsar frostsprängning av berget. Om istället berget kyls ned långsamt (läg fryshastighet), tillåts vattenvandring att ske under en längre period, vilket kan resultera i större frostsprängning av berget.

Nyckelord: Istryck, nedbrytning av berg och sprutbetong, vattenvandring, vidhäftning, frostsprängning.
LIST OF SYMBOLS AND ABBREVIATIONS

$\alpha$  = thermal expansion or contraction
$\varepsilon_{L,\text{max}}$  = maximum freezing strain
$\kappa$  = thermal diffusivity
$\lambda$  = thermal conductivity
$\rho$  = density
$B$  = slot width
$c$  = crack radius
$c_0$  = initial crack radius
$c_p$  = specific heat
$F$  = frost index
$H$  = slot depth
$K_c$  = fracture toughness
$K_I$  = stress-intensity factor
$l$  = slot length
$L_0$  = length at temperature $T_0$
$L_T$  = length at temperature $T$
$M$  = molecular weight of water
$p_i$  = internal ice pressure in crack
$Q$  = thawing thermal capacity of water
$S_r$  = initial degree of saturation
$T_{\alpha 0}$  = temperature at the start
$T_0$  = freezing point of bulk water
$T_\alpha$  = temperature after alteration
$T$  = freezing point of pore water
$T_H$  = upper temperature limit
$T_L$  = lower temperature limit
$V_p$  = longitudinal wave velocity
$w$  = crack width at point of widest opening
$Y_f$  = surface tension of water
$Y_k$  = pore radius

closed system  = no access to water during the freezing period
frozen fringe  = layer between frozen and unfrozen rock or soil
microgelivation = degradation of material in small scale, which involves granular
disintegration or flaking
macrogelivation = degradation of material in a greater scale than microgelivation, which involves opening or wedging of pre-existing macrofractures or joints
open system = access to water during the freezing period
permeability = hydraulic conductivity
P-wave velocity = longitudinal wave velocity
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1 INTRODUCTION

Each winter some of Banverket’s railway tunnels are faced with problems related to leakage of water and freezing temperatures. The water leakage causes ice formation along the tunnel contour in the shape of icicles and ice pillars, which can cause damage at the tunnel construction, hand rail, cable rack, installations and drainage. These problems require a large maintenance effort during winter. The problem with ice raises more concern when it occurs in a crack near the tunnel contour or in the interface between the rock and shotcrete since this can cause fall outs of rock or shotcrete.

The purpose of this report is to gather experience and information about how ice is formed and how ice pressure influences fault zones, cracks and the interface between rock and shotcrete.

1.1 Background

Ice formation has always been a major problem in railway tunnels in Sweden. In the past few years Banverket has observed an increase in the number of incidents involving fall outs of shotcrete and rock in old as well as newer railway tunnels. The problem with ice is a result of failure in the effort to prevent water reaching the cold tunnel air. With successful prevention of water leakage there will be no problems with ice formation in the tunnels. Creating a completely dry tunnel without using an impermeable tunnel construction such as lining is difficult. In Sweden the tradition is to use grouting to prevent water leakage instead of the more expensive alternative – an impermeable tunnel construction.

The leakage to the tunnel is dependent on factors such as the rock mass properties, tunnel position below ground surface, ground water level, etc.

1.1.1 Leakage

Rocks are classified into three different groups based upon the process of formation, i.e. igneous, sedimentary and metamorphic rocks. Igneous and metamorphic rocks (crystalline rocks) are the most common rocks in Sweden (Loberg 1993). Crystalline rocks are often
dense and water occurrences are concentrated to crack and fissure systems (Fairhurst et al., 1993). Micro cracks occur between the mineral particles, but on the whole the porosity and permeability is low (Gustafson, 1986). Since the fissures often do not form a fully connected system, it is misleading to talk about a ground water level in crystalline bedrock. Each fissure system is a separate aquifer with a specific ground water level; see Figure 1.1 (Miskovsky, 2003). Sedimentary rocks can be porous and have high permeability and the groundwater occurs in the pores and is uniformly distributed in the rock mass (Fairhurst et al., 1993).

![Figure 1.1 Ground water level in crystalline bedrock (Miskovsky, 2003)](image)

When a tunnel is excavated, the characteristics of the rock mass closest to the tunnel can change (Pusch, 1989). The excavation of a tunnel leads to changes in the stress field in the rock mass and the geometry of the aperture between the crack surfaces can change due to increase or decrease of the normal stress over the crack. The increase of compressive stress can cause closure of a crack, while others cracks can open up due to a decrease of compressive stress or shear stress (Hakami, 1988). By shooting and blasting of the tunnel the cracks can widen and cause an increase of the leakage to the tunnel. Nowadays, to avoid these problems most tunnels are performed by smooth blasting. The leakage is also influenced by the topography and the distance to the groundwater level. If a tunnel is located below the groundwater level, the tunnel always has access to water while the leakage to a tunnel near or above the groundwater level is dependent on for example precipitation and frost in the ground (e.g. Andrén, 1995).

From experience the crack frequency along a tunnel in Sweden is roughly 1-3 cracks per meter with an aperture of 0.1-1 mm. 75 % of the leakage of water are estimated to originate from only a few larger cracks while the remaining 25 % of the leakage originates from a lot of small cracks (Vägverket, 1994).
1.1.2 Prevent water leakage

In order to decrease the problems with ice in the tunnels, the water leakage into the tunnel must be prevented. The permissible amount of water leakage into the tunnel per day varies with each specific tunnel object. Leakage into the tunnel must not be allowed to influence the surroundings of the tunnel, such as lowering the groundwater level.

Grouting is the most common method used in Sweden to prevent water leakage. If grouting is not sufficient other methods must be employed to deal with the water leakage. These include diversion of water from the tunnel using lining, insulated drainage or geotextile and using impermeable constructions to prevent water from reaching the tunnel. If the leakage causes lowering of the groundwater level in areas where it is prohibited, infiltration can be used. However, diversion and infiltration increase the operative expenses. Life cycle costs therefore have to be analysed when choosing a method in order to find the most suitable solution for each specific object (Banverket, 2004).

Unlike other European countries that use impermeable tunnel constructions to prevent leakage of water, grouting is the most commonly used method in the Nordic countries. Impermeable tunnel construction is much more expensive compared to grouting but on the other hand any problems with ice formation can be completely eliminated with the use of impermeable tunnel construction.

1.1.3 Rules and regulations

When ice forms in a tunnel it can generate an ice pressure or an ice load at tunnel constructions and installations. According to Vägverket’s rules and regulations “Tunnel 2004” it is required that tunnel constructions and installations must be designed for an ice load when risk of freezing is present. The value of the ice load is 3 kN/m² assuming it to be a free load that acts perpendicular to the construction. This value includes both ice pressure and drop load from ice (Vägverket, 2004). The value originates, according to Vägverket, from the requirements in Håndbok 163 published by Statens vegvesen in Norway (Statens vegvesen, 1995). Jernbaneverket in Norway uses a “general payload” of 3 kN/m² purposely to increase the constructions capacity to handle ice load, drop load and special conditions due to pressure and suction loads from traffic (Jernbaneverket, 2004).

Choosing a value for the ice load can be wearisome since the magnitudes of the ice load and ice pressure are dependant on factors such as access to water, the rigidity of the adjacent material and temperature conditions. Banverket has chosen not to design the tunnel constructions for ice load, but has requirements that a tunnel should be designed in such way
that damage due to freezing is avoided (Banverket 2004).

1.2 Problem statement

The cause of rock and shotcrete fall outs is not clear. One scenario is that the material undergoes degradation due to weathering processes such as frost shattering resulting in fall outs while another is that the shotcrete falls down due to poor adhesion between the rock and shotcrete.

The fall outs often occur in tunnel sections where problems with leakage of water exist. Therefore a likely scenario is that the water is subsequently freezing and expanding in cracks near the tunnel contour or in the interface between the rock and shotcrete. This process can produce a large pressure which can cause pieces of rock to break lose from the tunnel wall/roof as well as the shotcrete to crack. Cracking will then lower the load-bearing capacity of the shotcrete and can in the worse case cause fall outs.

Ice pressure is a complex problem and at present knowledge with regard to the magnitude and the effect of ice pressure between the rock and shotcrete is insufficient. In order to have a better understanding of the processes of ice growth further research is needed.

1.3 Objective

This report will attempt to provide an understanding of the factors and processes that control the growth of ice, the development of ice pressure and frost shattering of the rock and shotcrete.

The factors governing the growth of ice in tunnels include the freezing rate, the duration of negative temperatures, the temperature variation (see chapter 4), the rock mass properties, the rigidity of the adjacent material (see chapter 3) and access to water (see section 4.2). All these factors have an influence on the processes in progress such as the manner in which the ice freezes and how the ice pressure develops.

The report’s focus will be on the problem of the degradation of the rock and shotcrete due to ice pressure and frost shattering. Degradation can occur due to the material’s weathering processes or to the poor adhesion between the rock and the shotcrete. Poor adhesion on its own does not pose a degradation problem but voids can form as a result of the poor adhesion in the interface between the rock and shotcrete which when filled with water can lead to the development of ice pressure at freezing temperatures. Furthermore the ice exerts pressure at the interface causing cracking and degradation of the shotcrete.
2 ROCK AND SHOTCRETE DEGRADATION

The fall out of rock and shotcrete can be due to the weathering processes of the material or the fact that there may be poor adhesion between the rock and shotcrete.

2.1 Weathering

Weathering is the decomposition of geological materials through mechanical or chemical processes. Materials exposed at the earth’s surface are constantly being altered by water, air, the changing temperature and other environmental factors. Mechanical weathering includes processes that physically break down material into smaller pieces without changing the material’s chemical composition, i.e. the minerals are unchanged. Chemical weathering is the decomposition of material from exposure to water or atmospheric gases, where some of the original minerals are chemically changed into different minerals (Plummer and McGeary, 1996).

2.1.1 Mechanical weathering

The most destructive processes that cause rocks to disintegrate are frost action, abrasion and pressure release.

Frost action is a collective term used to describe a number of distinct processes, which result mainly from alternate freezing and thawing in soil, rock and other materials (French, 1996). During the water-ice phase transition, a 9 % volumetric expansion of the water occurs and the expansion pries rock apart. Frost action is most destructive in regions with frequent freezing and thawing, where partial thawing during the day adds new water into a crack. When the new water freezes during the night, more ice is formed and the expansion causes the crack to widen even more. This is called frost wedging (Plummer and McGeary, 1996). In Sweden frost wedging or frost shattering is often described to be the most destructive process for rock weathering (Svensson, 2004).

The two other processes are abrasion and pressure release. Abrasion is when pieces of rock material are grinded away from the rock surface by friction and impact during transportation.
Pressure release occurs when a rock, which was originally formed under great pressure, is gradually exposed by tectonic uplift and erosion of the overlying layers of soil and rock. This causes cracks parallel to the surface to develop (Plummer and McGeary, 1996). In Sweden these types of cracks appear in granite (Svensson, 2004).

2.1.2 Chemical weathering

All mechanical activities, which lead to the widening of a crack, help to speed up the chemical weathering by enlarging passageways for water and air. No matter what process of mechanical weathering it is, the rock disintegrates into smaller fragments and the total surface area increases (Figure 2.1). Therefore more extensive chemical weathering can take place (Plummer and McGeary, 1996).

![Figure 2.1 Increase of the surface area as a rock breaks up into smaller pieces (Plummer and McGeary, 1996)](image)

At the earth’s surface, minerals change gradually until equilibrium is reached between the mineral and the surrounding conditions. Elements within the exposed minerals often react with oxygen which is abundant in the atmosphere. A common weathering product is iron oxide, which is formed from iron in the ferromagnesian group – pyroxenes, amphiboles, biotite and olivine (Plummer and McGeary, 1996).

Acids which give off hydrogen ions (H+) are the most active agents of chemical weathering. Since a hydrogen ion has a positive charge it can substitute for other positive ions, such as Ca\(^{2+}\), Na\(^+\) or K\(^+\) (Plummer and McGeary, 1996). With this substitution the chemical composition of the mineral is changed and its atomic structure is disturbed. One by one each atom is removed from the original material making the remaining material more porous and decomposed (Svensson, 2004).
2.2 Adhesion

Poor adhesion can occur between rock and shotcrete and this can cause fall out of shotcrete. But it is not obvious whether the fall outs are a result of poor adhesion which appears directly when applying the shotcrete or if poor adhesion is an effect of degradation.

The problems arising from poor adhesion between rock and shotcrete can be due to the following:

- excess leakage from the rock during shotcreting
- bad adhesive strength due to poor rock conditions
- uneven rock contour – which makes it difficult to get good contact with the rock surface.

Hahn (1983) accounted for two tests that were conducted to observe how the adhesion of shotcrete was influenced by the moisture of the rock surface. Hsu and Slate (1964) showed that the difference between the adhesions on the dry and wet ballast was only 3 %, which is negligible. Their tests were conducted on ballast that had been (i) in a water bath for 24 hours and (ii) heated in a heating oven. Karlsson (1980) showed by conducting field tests that the adhesion of shotcrete is not affected whether the rock surface is dry or wet when shotcrete is applied.

But in areas with complex water conditions, for example where there are relatively open joints and fissures, the shotcrete tend to cause a stability problem instead of solving it. It is important that a good adhesion is obtained between shotcrete and rock which otherwise when exposed to frost can have ice pressure developing in the interface between them causing cracking of the shotcrete. The load-bearing capacity is lowered as a consequence, and fall outs of fragments of shotcrete can occur (Selmer-Olsen and Broch, 1976).

Another problem which hinders achieving good adhesive strength is the fact that the rock surface needs a thorough cleaning, prior to shotcreting since the shotcrete lining is entirely dependent on an absolute adhesion between the rock and the shotcrete (Selmer-Olsen and Broch, 1976). Malmgren (2001) has shown that when the rock surface was cleaned by water-jet scaling (water pressure of 22 MPa) instead of using normal treatment (water pressure of 0.7 MPa), the adhesive strength increased from 0.21 MPa to 0.61 MPa.

Hahn (1983) conducted tensile tests of the adhesive strength of shotcrete and observed that breakage occurred (i) in the rock material, (ii) at the interface between rock and shotcrete
(adhesion break) and (iii) in the shotcrete. Where the breakage appears depends partly on the material and partly on the location of the highest stress concentration. The most frequent breakage was the adhesion breakage with the exceptions of porous sandstone and limestone where the breakages occurred in the rock and in the shotcrete respectively. The results showed that the adhesive strength was dependent on the roughness of the rock surface. As shown in Figure 2.2, a rough surface gives a higher value of the adhesive strength than a smooth surface. Hahn’s result also showed that the type of feldspar in the granite samples had an influence on the adhesive strength. It was proven in his tests that both the mineral composition and the roughness of the rock surface had influences on the measured adhesive strength. He concluded that the mineral composition had more effect than the roughness.

Figure 2.2  Adhesive strength for different rock types and rock surfaces (Hahn, 1983 – from Malmgren, 2001)
2.3 Case studies of fall outs

There has been an increase in the number of incidents of fall outs of rock and shotcrete in Banverket’s tunnels in recent years. So far there have been no serious casualties or damages to trains related to the above. Some installation have however been damaged (Andrén, in press 2007). In this section some cases of fall outs of rock and shotcrete are gathered. Several of these fall outs have occurred even though inspection of the tunnels have been performed as prescribed in Banverket’s regulations. This might show that the problem with ice is more complicated than previously assumed and furthermore that the regulations might need to be revised.

According to Banverket’s regulations, safety and maintenance inspection of the tunnels must be performed. Safety inspection should be performed twice a year and in addition to that, maintenance inspection must be conducted. The regularity of the maintenance inspection should be adjusted to the needs for each individual tunnel.

The safety inspection (Banverket, 2005a) includes:

- checking to see if any fall out of rock had occurred
- checking to see if there is any risk of fall out of rocks
- checking that damage, cracks or other signs of movements don’t occur in the shotcrete.

The maintenance inspection (Banverket, 2005b) includes:

- checking to see if there is any need for rock mechanical measuring or scrapping.
- checking damage on reinforcement due to degradation processes like frost shattering, rust shattering, leaching, corrosion, depositing, etc.

2.3.1 Rock fall outs

Although inspections are being preformed as prescribed, fall outs of rock can appear in the tunnels. The following section presents some of the reported fall outs.

The Bergträsk tunnel

A fall out of rock was reported from the Bergträsk tunnel in Älvsbyn in November 2005. Two blocks with a diameter of 1 m each have fallen off the tunnel wall. The rock surface above the
fall outs was fractured and this section was reinforced with rock bolts. The latest scrapping of the tunnel was executed in 2002 (Banverket BRN, 2005). The cause of the fall outs was not clear, but since the ground freezing period had started, frost action may have been one of the reasons for movement of the blocks.

**The Aspen tunnel**

In December 2003 a train-driver reported a fall out in the Aspen tunnel, between Lerum and Jonsered. A rock block (1×0.5 m) had fallen off the roof. Upon inspection of the tunnel roof it was noted that the rock surface consisted of open oxidized cracks. The cause of the fall out was presumed to be vibration from the train traffic. Incidentally this fall out also happened during the winter period and again the frost action may have been one of the reasons for movement of the block (Bergab, 2003b).

**The Herrljunga tunnel**

In January 2006 a fall out of six smaller rock blocks (diameter from 0.2 to 0.4 m) were reported from the Herrljunga tunnel near Uddevalla. The blocks had disintegrated crack surfaces and some of the crack surfaces were also covered by thin layers of ice. The cause of the fall outs was, according to the inspectors, presumed to be frost shattering of the rock (SwedPower, 2006).

### 2.3.2 Shotcrete fall outs

The shotcrete usually falls out as sheets some of which are usually covered with ice on the interface side, i.e. the side that has been attached to the tunnel roof/wall. Furthermore there is often an ice cover on the exposed rock surface. This implies that water has accumulated in the interface between rock and shotcrete. According to the reports summarized below, the fall outs had occurred despite inspections being performed within some years. At the time of inspection the shotcrete showed no indication of faults. The question is whether the shotcrete could degrade considerably such that it can fall out in just a year or two, or if any defects were missed during the inspection.

**The Gårda tunnel**

The Gårda tunnel was built in the late 1960s and a complementary station was built in the beginning of 1990. In January 2003 a fall out of shotcrete and a thin sheet of rock with an area of 1 m² occurred in the tunnel. An additional 2-3 m² of loose material was removed when scrapping the damage area. Upon inspecting the whole tunnel more areas with reduced
surface stability were detected. The inspectors found that, the shotcrete had degraded and that it is was a continuing and accelerating process (Bergab, 2003a).

**The Nuolja tunnel**

The Nuolja tunnel was built in 1990 and in January 2003 an area of 2 m² of shotcrete had fallen on to the rail and the overhead wire was slightly damaged. In 1995 there was another fall out of 1 m² shotcrete in the same area. The fall out in 2003 had an ice layer on the interface side and in the tunnel roof a layer of ice was noted in the interface of shotcrete and rock. The tunnel was scraped in July 2001 and according to that report the tunnel was in a good condition at the time and the shotcrete was intact in the whole tunnel (Banverket BRN, 2003a).

**The Bergträsk tunnel**

The Bergträsk tunnel was built in 1982 and a fall out of shotcrete with an area of 1-2 m² was detected during a routine check in September 2003. The tunnel was inspected in the spring of 2002 and scrapping was executed the same year. The section where the fall out took place had several spots of water leakage and according to Banverket’s report (Banverket BRN, 2003b) the fall out was caused by frost shattering.
3 THERMAL AND THERMO MECHANICAL PROPERTIES

To understand the processes of rock and shotcrete degradation caused by frost shattering, some knowledge about the materials thermal and thermo mechanical properties are required.

3.1 Introduction

The volumetric expansion that occurs when water transforms into ice, will occur even if the adjacent material is very rigid. To prevent ice formation and the resulting volumetric expansion, the adjacent material needs to exert a pressure of 13.7 MPa for each degree of decrease in temperature (from Fridh, 2005). This value exceeds the tensile strength of most rock material (tensile strength of hard intact rocks is in the order of 10 MPa (e.g. Matsuoka, 1990b)) and also exceeds the tensile strength of shotcrete, 4 MPa (Brandshaug, 2004). Because the tensile strength of the adjacent material is lower than the pressure needed to prevent the ice formation occurring, the material yields to the pressure and thus cracks will appear.

In the process of excavating a tunnel in the cold region the original stable thermodynamic condition in the rock is destroyed and replaced by a new thermodynamic system. During winter the rock walls and the roof in a tunnel are exposed to negative temperatures and with the temperature drop, water in fissures and pores freezes and volumetric expansion of the water/ice occurs. This expansion is restrained by the tunnel lining and the surrounding rock/soil resulting in pressure on the lining. This ice pressure acting on the tunnel lining may result in cracking and flaking and can become a serious threat to the lining stability (Lai, 2000).

3.2 Properties of water and ice

Ice is an important factor that influences several of the rock properties at low temperatures. However there is a great difference between rock and ice. The deformation of ice is time dependent and can be both elastic and viscoplastic. Dahlström (1992) compiled results of several experiments of the properties of ice, which show that the magnitudes of Young's modulus, Poisson’s ratio and compressive and tensile strengths increase with decreasing
temperature. For example the compressive strength increases from 4 MPa at 0 °C to 10 MPa at -20 °C.

The conventional view has been that frost shattering is the result of the 9 % volumetric expansion, which occurs during the water-ice phase transition. When freezing from 0 °C down to -22 °C the expansion of ice is 13.5 %. Theoretically, the pressure at the freezing point increases almost linearly from zero at 0 °C to a maximum of 207 MPa (the ice pressure melting point) at -22 °C, see also section 4.4 (Tharp, 1987). At temperatures below this, the pressure decreases because the ice begins to contract (French, 1996). The thermal expansion of ice, \( \alpha \), is not constant when the temperature is altered. Glamheden (2001) summarized some experiments of the thermal expansion of ice and as shown in Figure 3.1, \( \alpha \) decreases with decreasing temperature. From 0 °C to about -70 °C the thermal expansion is 46.8 \( \cdot 10^{-6}/°C \). Below -70 °C and down to -120 °C the thermal expansion decreases to 32.5 \( \cdot 10^{-6}/°C \) and below -120 °C the thermal expansion is 20.2 \( \cdot 10^{-6}/°C \).

\[ \text{Figure 3.1} \quad \text{Thermal expansion of ice (Powell, 1958 – from Glamheden, 2001)} \]

### 3.2.1 Latent heat

To get water to transform into ice, heat must be removed from the water without any change in temperature. This is called latent heat, \( L \), and the magnitude is 334 kJ/kg. In the same way, heat is needed to melt ice into water at 0 °C. The thermal flow occurs in direction towards the colder media and it is proportional to temperature difference between two points. Therefore the rate of the ice growth is fast in the beginning of freezing and then decrease as a result of the isolating effect of the ice (Fransson, 1995).
While the amount of heat which is either being released or consumed at the phase transition is considerable, the latent heat has a great influence in the frost action of soil and rock. The frost depth in a material with low water content is much greater than in a material with higher water content. In the material with high water content, there is a large amount of heat that has to been transported away to transform the material into frozen conditions in comparison with material with low amount of water (Knutsson, 1981).

### 3.3 Thermal properties

The thermal properties of a material change due to temperature and are dependent upon the water content in the material. This is due to the fact that water and ice have very different properties.

#### 3.3.1 Thermal conductivity

The thermal conductivity, \( \lambda \) (W/m·K), of a material can be described as the material’s ability to conduct heat and it varies with temperature, porosity, water content, etc.

High porosity gives normally a low thermal conductivity. If a porous material contains water, the conductivity changes when the temperature decreases and the water transforms into ice. This is because the thermal conductivity of ice is 2.25 W/m·K and the thermal conductivity of water at ±0 °C is 0.56 W/m·K (Knutsson, 1981).

The thermal conductivity is dependent on the mineral composition of the rock or soil. Some values of the thermal conductivity of the most frequently occurring rock minerals in gneiss and granite are shown in Table 3.1.

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Thermal conductivity (W/m·K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Quartz</td>
<td>7.7</td>
</tr>
<tr>
<td>Alkali feldspar</td>
<td>2.5</td>
</tr>
<tr>
<td>Plagioclase</td>
<td>1.9</td>
</tr>
<tr>
<td>Biotite</td>
<td>2.0</td>
</tr>
<tr>
<td>Muscovite</td>
<td>2.3</td>
</tr>
</tbody>
</table>

The Swedish bedrock is dominated by gneiss and granite, which are highly quartzose rock types. An average value of the thermal conductivity of these rocks is 3.5 W/m·K. The value
for basic rock is lower than for highly quartzose rock types, while quartz itself has a high value (Dahlström, 1992). An average value of the thermal conductivity of moraine is 1.9 W/m-K (Knutsson, 1999). The typical thermal conductivity of shotcrete may vary depending on material content, but an average value is 1.4 W/m-K (Schwarz, 2004).

### 3.3.2 Specific heat

Specific heat, $c_p$ (J/kg·K), is the amount of heat which is needed to raise the temperature 1 degree K for 1 kg of the material. The specific heat is temperature dependent and decreases with decreasing temperature.

In a composite material such as rock, the overall specific heat consist of the specific heat of the individually components. The specific heat of water is 4200 J/kg·K and of ice 2040 J/kg·K at ± 0 °C. The specific heat of granite varies in the literature but an average value is 730 J/kg·K (Dahlström, 1992).

### 3.3.3 Thermal diffusivity

The thermal diffusivity, $\kappa$ (m²/s), determines the rate of temperature change through the material. It is related to the thermal conductivity and the heat capacity according to:

$$\kappa = \frac{\lambda}{c_p \cdot \rho}$$

where;

- $\lambda$ = thermal conductivity (W/m·K)
- $c_p$ = specific heat (J/kg·K)
- $\rho$ = density (kg/m³)

The specific heat decreases with decreasing temperature and thermal conductivity increases with decreasing temperature. Dahlström (1992) pointed out that the thermal diffusivity of granitic and gneissic rocks increase with decreasing temperature. Concrete behaves in a similar way as rock material.

### 3.4 Thermo mechanical properties

#### 3.4.1 Strength

weathering and frost durability tests, which were performed on the same group of sedimentary rocks. The results indicated that most of the rocks deteriorated in a similar way regardless of the environmental conditions, which is a strong indication that the properties of the rock material supersede the influence of environmental conditions. The results of the tests suggested that rock strength provides the basic resistance to mechanical weathering of sedimentary rocks, but the presence of pre-existing flaws overrides this resistance.

Tensile strength of hard, intact rocks may be in the order of 10 MPa. However, the strength decreases significantly in jointed bedrock of the same lithology (e.g. Matsuoka, 1990b). The mechanical strength of rock is dependent on porosity. The intrinsic low mechanical strength of weak rocks usually equates with high porosity (Winkler, 1994) and hence greater frost susceptibility (McGreevy, 1982).

**Uniaxial compressive strength**

The influence by temperature on uniaxial compressive strength is dependent on rock mass, texture, mineral composition, porosity and water content. The porosity and water content have at normal temperatures a reducing effect on the compressive strength, but at freezing temperatures the properties behave differently. The uniaxial compressive strength increases with decreasing temperature down to -120 °C and is then relative constant. For rock with high porosity, like sandstone and limestone, the increase in strength is greater than for rocks with lower porosity, like granite. The effect of porosity and water content has a great influence, and cause a considerable increase in the uniaxial compressive strength when the water in the pores freezes (Dahlström, 1992).

**Tensile strength**

The tensile strength of rock is in the order of 1/8 - 1/10 of the uniaxial compressive strength. The behaviour of tensile strength resembles compressive strength when freezing, and the increase of the strength is in the same magnitude (Dahlström, 1992).

**3.4.2 Young’s modulus**

The porosity and water content is of considerable significance for Young’s modulus when the temperature decreases. Dahlström (1992) summarized some tests which showed that Young’s modulus stays relatively constant for air-dry rock when the temperature decreases. But for saturated rock, the Young’s modulus increases with decreasing temperature. For saturated granite the Young’s modulus increases down to -120 °C because absorbed water continues to solidify down to these temperatures.
### 3.4.3 Poisson’s ratio

Poisson’s ratio doesn’t show the same behaviour as the parameters above when the temperature is altered. Different freezing tests show that the value of Poisson’s ratio varies irregularly for saturated granite samples, whereas the value is constant for dry granite (Dahlström, 1992).

### 3.4.4 Thermal expansion and contraction

When the temperature is altered in a body, a change in length and volume will occur (Glamheden, 2001). The definitions for thermal linear expansion or contraction, $\alpha$, is:

$$
\alpha = \frac{L_T - L_0}{L_0(T_\alpha - T_{\alpha0})}
$$

where;

$L_0$ = length at temperature $T_{\alpha0}$

$L_T$ = length at temperature $T_\alpha$

$T_{\alpha0}$ = temperature at the start

$T_\alpha$ = temperature after alteration

Glamheden (2001) put together experiments that show temperature dependency of the thermal expansion of granite. Investigations had been undertaken by Khan et al. (1967), Mellor (1970a), Inada and Yagi (1980), Kuriyagawa et al. (1980), Ehara et al. (1985), Ishizuka et al. (1985), Aoki et al. (1989) and Dahlström (1992) and the results are gathered in Figure 3.2 and Figure 3.3. The thermal expansion versus temperature is shown for air-dry granite samples and water saturated granite samples in Figure 3.2 and Figure 3.3 respectively. The dotted line represents a mean coefficient of linear thermal expansion.

In Figure 3.2 the thermal expansion for air-dry sample seems to be linear and independent of the temperature according to Mellor, Kuriyagawa and Ishizuka. The linear coefficient of thermal expansion in the range +20 °C to -120 °C is about of $\alpha = 4.3 \cdot 10^{-6}/°C$. The result from Inada diverges from the other and shows that the rate of thermal expansion decreases with decreasing temperature.
Figure 3.2 Thermal expansion of air-dry granite samples (Glamheden, 2001)

Figure 3.3 for water saturated samples, shows a different situation. The test samples show irregular expansion in the interval 0 to -10°C and the rate of expansion decreases with decreasing temperature. The results presented by Mellor, Ebara and Ishizuka, gives a mean coefficient of linear thermal expansion of $\alpha = 2.8 \cdot 10^{-6}/^\circ C$ in the range +20 °C to 0 °C. In the range -5 °C to -100 °C the thermal expansion is $\alpha = 5.3 \cdot 10^{-6}/^\circ C$ and in the range -100 °C to -180 °C the thermal expansion is $\alpha = 2.7 \cdot 10^{-6}/^\circ C$ (Glamheden, 2001).

Figure 3.3 Thermal expansion of saturated granite samples (Glamheden, 2001)
Dahlström (1992) pointed out the difference in behaviour concerning thermal expansion between rock and soil with an experiment made by Boulanger and Luyten (1983). The difference between clay, gneiss and limestone are shown in Figure 3.4. Clay has higher water content than rock and demonstrates expansion down to a temperature of -50 °C, when a small contraction starts. Limestone starts to expand, and then contract, while gneiss only shows contraction.

![Figure 3.4 Schematic contraction and/or expansion of clay, limestone and gneiss with decreasing temperature (After Boulanger and Luyten, 1983 – from Dahlström, 1992)](image)

In cracks, the joint filling can consists of clay minerals. The clay can be problematic during freezing, because clay expands more than the rock contracts (see Figure 3.4). This can cause a pressure to develop at the adjacent rock surfaces. This problem can also be encountered in fault zones, where the rock material is often weathered and has altered into clay minerals.

The thermal contraction of ice is $48 \cdot 10^{-6}/°C$ (see Figure 3.1) compared to the thermal contraction of rock which is about $5.3 \cdot 10^{-6}/°C$ (see Figure 3.3). Therefore, the ice contracts more than the rock when the temperature decreases. This has the result that when the ice has expanded as much as possible (13.5 % at -22 °C, see section 3.2) it starts to contract and therefore the pressure at the rock surfaces decreases.
4 FROST PHENOMENA

The factors that govern freezing include the temperature, the material’s thermal properties, the freezing rate and the duration of the temperature and the most important factor that affects the magnitude of the ice pressure is the access to water during freezing.

A tunnel exposed to negative temperatures usually has no problems in sections where the rock material is intact and of good quality. Problems occur in sections with bad rock conditions and water leakage. The frost action in crushed or heavily weathered rock resembles the frost action in soils.

It is generally accepted that when water freezes in a rock crack, the expansion of the ice creates stresses, which tend to propagate the crack (Davidson and Nye, 1985). However, the physical details are not well understood and in spite of extensive literature on the subject in general, there appears to be no quantitative theoretical analysis of the basic process.

4.1 Frost action in soil and rock

The phase transition of water is fundamental to the understanding of frozen and freezing soils and rocks. Frost weathering refers to the combination of mechanical-chemical processes, which causes the in situ breakdown of rock in cold-climate conditions. Many studies over the years have questioned the basic fundamentals of cold-climate weathering. Both field studies and laboratory simulations and modelling have been undertaken with the aim of increasing the availability of data on the rock temperatures, the rock moisture contents, the rate of rock disintegration, the role of ice segregation as a weathering mechanism and the influence of freeze-thaw cycles (French, 1996).

The weathering of rock as a result of freezing is of primary interest for this report. Among mechanical weathering processes in rock, frost action is one of the dominant factors in cold regions (French, 1996). To understand the problem relating to frost action in rocks, it is relevant to look at the well-established geotechnical principles regarding frost action in soil.

Frost action in soil is caused by two processes. These are (i) freezing of in situ pore water and
(ii) the growth of ice lenses, due to water migration to the frozen fringe from underlying unfrozen layers (see Figure 4.1). The freezing of pore water alone, contributes a minor part of the total frost heaving. In coarse-grained soils there is almost no frost heaving at all, because the pore water are being pressed out of the soil by the ice formation. The dominating part of the frost heaving is the growth of ice lenses. The soils ability to grow ice lenses depends on the grain-size distribution, permeability, the specific surface, the mineral content and capillarity etc. Ice lenses are generally formed perpendicular to the direction of the thermal flow with the thickness of the lenses being governed primarily by the access to water. In soils with low permeability, the water cannot migrate to the frozen fringe in the rate needed for frost heaving. Therefore frost heaving is limited in this kind of soil (Knutsson, 1981).

![Diagram of frost heaving](image)

Figure 4.1 Frozen fringe – the zone between the frozen and the unfrozen layers (modified from Walder and Hallet, 1985)

For a long time the 9 % volumetric expansion of water was thought to be the primary cause of frost shattering. An alternative mechanism was first presented by Everett (1961). He suggested that capillary suction caused water migration towards the freezing front. Then the ice pressure led to the shattering of the rock. Water migration in freezing porous rock was observed in laboratory tests performed by Fukuda and Matsuoka (1982) and Fukuda (1983). The experiments showed that water migration could be responsible for frost shattering. Matsuoka’s laboratory tests (1990a) of the influence of access to water during freezing, led to a hypothesis that a combination of the two processes, volumetric expansion and water migration, controlled frost shattering, which also was suggested by Tharp (1987).

The leakage to the tunnel can be influenced by the frost action in rock. For instance, frost action can change the permeability of the clay filled cracks in the rock mass. When soils consisting of clay freezes, a restructuring of the clay particles occurs – freeze consolidation, see Figure 4.2. During each freezing period the soil becomes more and more consolidated
causing settlements and increase in the permeability of the soil (Chamberlain and Gow, 1978). In the same manner the clay particles in a weathered crack can consolidate, when the clay is initially exposed to the negative temperatures of the cold air in the tunnel. This can cause problems in newly excavated tunnels. A crack that appeared to be impermeable at the time the tunnel was excavated can start to leak after one freezing period due to restructuring of the clay particles (Andrén, in press 2007). In contrast to the information above, other cracks can stop leaking, as a result of natural clogging by the precipitation of calcium oxide and by precipitations containing iron (Statens vegvesen, 2004).

![Restructuring of clay particles when freezing](image)

**Figure 4.2** Restructuring of clay particles when freezing (after Chamberlain and Gow, 1978 – from Johansson, 2005)

### 4.2 Access to water

The ice formation and ice pressure, which can occur in a crack, are influenced by the access to water. If a crack has access to water during freezing the thickness of the ice layer can increase due to water migration. But if the crack doesn’t have any access to water, there will only be an expansion of the existing water in the crack at the time of freezing.

Two moisture parameters govern frost shattering; (i) the degree of saturation before freezing and (ii) the amount of water migration during freezing (Matsuoka, 1990a).

#### 4.2.1 Water migration

Frost heaving in soils occurs due to the migration of water from unfrozen parts of the soil to the frozen parts with ice concentrating generally as ice lenses perpendicular to the direction of the thermal flow. The growth of ice lenses is the dominating part of frost heaving. In rock the water can migrate in a similar manner inside a crack. The water migrates in the water film at the interface between ice and rock allowing expansion of the ice layer, which exerts pressure on the rock surfaces through the water film. This can cause frost shattering (Tharp, 1987).
Ice crystallization in a soil normally starts in the centre of the larger pores, where the energy level of the water is highest. The crystals grow until a thermodynamic state of equilibrium is reached between the growing crystals and the adsorbed water at the surface of the mineral particles. Different mineral particle surfaces have different adsorption properties (French, 1996). This implies that at a given negative temperature there are ice crystals as well as unfrozen water in the soil, both adsorbed and free water (water that has not been absorbed at mineral particles). Adsorbed water has a lower energy level compared to free water, and the adsorbed water demands lower temperatures to freeze. When the temperature decreases and all free water has been frozen, water with lower energy level starts to freeze. The part with unfrozen water is reduced (see Figure 4.3), causing the water film that separates the ice from the solid particles to become thinner (Knutsson, 1999).

As the temperature continues to decrease, more of the absorbed water freezes and the energy level in the unfrozen water decreases. In a volume of soil or rock there will be sections of different energy levels in the unfrozen water due to the temperature variation. Water always endeavours to achieve lowest energy possible and this causes unfrozen water to migrate from warmer to colder zones, because the energy level is lower there (Knutsson, 1999).

![Figure 4.3](modified after Knutsson, 1981)

Experimental work has shown that a considerable amount of water remains unfrozen at subfreezing temperatures not only in soil, but also in rock; and that unfrozen water tends to
migrate towards freezing centre in rock as well as in soil (Walder and Hallet, 1985). Walder and Hallet (1985) developed a theoretical model for the growth of ice within cracks (see section 5.4) with the assumption that progressive crack growth results from water migrating to ice bodies in cracks, much as water migrates to ice lenses in freezing soil. According to Tharp (1987) the water migration creates an expansion of the ice layer, which can produce a pressure at the crack surfaces.

Matsuoka (1990a) used 47 rock samples for freeze-thaw experiments to demonstrate that water migration plays an important role for frost shattering of rock (see also section 5.1.3). The samples were exposed to both an open system\(^1\) and a closed system\(^2\). The water migration in the open system led to an increase in ice volume, which caused large damage of the rock. In the closed system the resultant damage was rather small, because only pre-existing water became ice. The deterioration of the specimen was detected through a reduction in longitudinal wave velocity; named P-wave velocity or \(V_p\). The longitudinal wave velocity is commonly used in laboratory tests to prove deterioration of rock. If the longitudinal wave velocity reduces in the rock sample, it is exposed for deterioration.

Figure 4.4 shows reduction of \(V_p\) of tuff (a volcanic sedimentary rock) and sandstone specimens during 50 freeze-thaw cycles. The filled dots are the results for reduction of \(V_p\) during freezing in a closed system, and the unfilled dots represent an open system. By comparing the reduction in \(V_p\) for the same specimen but with different access to water, Matsuoka showed that an open system is much more susceptible to breakdown by frost action than a closed system. He concluded that water migration caused by adsorptive suction participates in the frost shattering of rock, as well as the 9 % volumetric expansion (Matsuoka, 1990a).

---

\(^1\) open system – access to water during the freezing period

\(^2\) closed system – no access to water during the freezing period
One hypothesis about water distribution in porous material is that when a porous material is immersed in water, it is unlikely for pore water to be uniformly disturbed over the specimen. This non-uniformity would become greater due to the migration of pore water during freezing. This hypothesis was checked by Chen et al. (2004). Chen et al. tested the degree of saturation at the surface layer and at the centre of two specimens with one being rapidly frozen by liquid nitrogen (Figure 4.5) and the other frozen in the low-temperature chamber (Figure 4.6). Figure 4.5 shows the specimen which was rapidly frozen and the degree of saturation was randomly distributed over the specimen with a range from 69% to 74%. This result indicates that sudden freezing prevents movement of pore water during freezing.
The specimen that was slowly frozen shows a different behaviour. Figure 4.6 shows that the degree of saturation in the surface layer was significantly higher than in the centre.

The degree of saturation in the surface layer was higher than in the centre of the specimen. The difference in distribution of pore water in the frozen specimens showed that migration of pore water occurred during freezing and water tends to move to the colder surface (Chen et al., 2004).
Most rocks contain water in pores of various sizes. Mellor (1970, 1971) had shown that pore water in rock freezes progressively as the temperature is lowered and that some of the pore water is almost unfreezable at natural temperatures (Davidson and Nye, 1985). The freezing point of pore water decreases with pore size. The relationship between the pore radius and the freezing point of pore water was established in a thermal-mechanical equation (Setzer, 1997):

\[
\ln\left(\frac{T}{T_0}\right) = -\frac{2 \cdot Y_f \cdot M}{\rho \cdot Q \cdot Y_k}
\]

where;

- \(Y_f\) = surface tension of water (75.64 dyn/cm)
- \(M\) = molecular weight of water (18.028 g/mol)
- \(\rho\) = density of water (0.9998 g/cm\(^3\) at 0 °C)
- \(Q\) = thawing thermal capacity of water (6.01 KJ/mol)
- \(Y_k\) = pore radius (nm)
- \(T_0\) = freezing point of bulk water (in K)
- \(T\) = freezing point of pore water (in K)

Therefore, the relationship between the freezing point of pore water, the freezing point of bulk water and the pore radius is given by (plotted in Figure 4.7);

\[
T = T_0 \cdot \text{EXP}\left(-\frac{0.4545}{Y_k}\right) - T_0
\]

![Figure 4.7](image.png)  
**Figure 4.7** Relationship between freezing point of pore water and pore radius (Chen et al., 2004)
4.2.2 Saturation and moisture content

To understand the mechanisms of rock breakdown, the influence of water saturation needs to be investigated. During freezing, water in a porous medium such as rock and concrete tends either to form ice or migrate, leading to redistribution of pore water. Chen et al. (2004) conducted freeze-thaw tests with rock specimens prepared from welded tuff to see the deterioration of the rock in relation to saturation (see also section 5.1.4). Specimens were examined by the changes in the uniaxial compressive strength, P-wave velocity and porosity. The experimental results of the freeze-thaw tests showed that when the initial degree of saturation was maintained below 60 %, the above properties did not change, but when the initial degree of saturation exceeded 70 %, the rock was significantly damaged. Therefore, the critical degree of saturation for this particular rock was about 70 %. It is to be noted that the critical degree of saturation changes depending on the rock material.

The degree of saturation may be important for ice lens growth in concrete since increases in the degree of saturation causes water absorption in air pores. When these pores are filled with water instead of air the pressure at the pore walls increase due to the ice growth, because there is no free space for the expansion. This can cause damage to the concrete construction (Fridh, 2005).

McGreevy and Whale (1985) found for intact rock that the enhanced frost damage occurred at cracked zones compared to damage in the intact material. This occurred due to the concentration of moisture in the cracks in proportion to the moisture content of the intact material. This moisture is essential in the freeze-thaw process, but may also play a role in deterioration by swelling of clay minerals (McGreevy, 1982). Swedish rocks often have a high content of mica and mica has a higher tendency to absorb moisture than other minerals.

The low mechanical strength of weak rocks usually coincides with high porosity (Winkler, 1994) and hence greater frost susceptibility (McGreevy, 1982). Since the absorption of moisture, necessary for freeze-thaw weathering, depends upon the rock’s microstructure it is likely that deterioration in these rocks will be more closely associated with void-dependent properties and microcracks than with macroflaws (Nicholson and Nicholson, 2000).

Matsuoka reported 1990-1991 from some field observations in the Japanese Alps, Svalbard and Antarctica of the factors influencing cold-climate rock disintegration. The conclusion was that the annual frost shattering rate is not only a function of freeze-thaw frequency per year, but also of the degree of water saturation and the bedrock’s tensile strength. The most effective environmental factor controlling the shattering rate was the moisture content (Matsuoka, 1990b and 1991). If the water migration is prevented during freezing and the
condition favoured by short-term, rapid freezing or the lack of an external water source, it is the initial degree of saturation that constrains expansion of rock (Matsuoka, 1990a). Such a situation occurs for rocks lying distantly from open water (e.g. stream, lake or sea) or far above the subsurface water table (Matsuoka, 2001b).

Konishchev and Rogov (1993) found out and reported from an experiment that the speed of crack growth in water saturated samples far exceeds that of dry samples when subjected to numerous freeze-thaw cycles. Their data for different rock types gave some indication of the approximate maximum speed of frost weathering. The average thickness of the disintegration layer for saturated rocks during one freeze-thaw cycle range from a high of 3.5 mm in marl to a low of 30 to $5 \times 10^{-5}$ mm in sandstone and porphyry. For dry samples of these rocks the disintegration layer was $600 \times 10^{-5}$ mm for marl and 6 to $11 \times 10^{-5}$ mm in sandstone and porphyry. Konishchev and Rogov also reported that in Ukraine, the speed of frost weathering of water-saturated limestone on buildings in Simpheropol City has been 1-10 mm/year, while on dry or only locally saturated limestone, rates were lower, between $10^1$ and $10^2$ mm/year.

4.3 Freezing

Four variables are usually evaluated from field data and can be used to describe the freezing process. They are; the freezing intensity, the freezing rate, the duration and the number of freeze-thaw cycles.

4.3.1 Freezing intensity

The freezing intensity is represented by the minimum subzero temperature that the rock surface experiences during a freeze-thaw cycle. The freezing intensity gives a basis for counting effective freeze-thaw cycles occurring in the bedrock (Matsuoka, 2001b).

4.3.2 Freezing rate

The freezing rate (decrease in temperature per unit of time, °C/h) mainly controls the magnitude of the expansion of rocks and its effect is variable in combination with moisture content (Matsuoka, 2001b). The freezing rate has an influence on water accessibility and therefore an influence on the rate of ice growth and the magnitude of the ice pressure developed.

Frost damage can occur in initially unsaturated rocks when slow freezing drives water migration from surrounding rock or an external moisture source. But when a rock undergoes
rapid freezing, frost damage can occur only when the degree of saturation is high (>80 %) or if water can migrate from a nearby moisture source (Matsuoka, 2001b). If the freezing rate is rapid enough it can minimize the water uptake, thus preventing frost damages. In contrast, long-term slow freezing which permits water migration can result in frost damages (Matsuoka, 2001a).

Walder and Hallet (1985) developed a theoretical model (see section 5.4) for the breakdown of rock by the growth of ice within cracks. They found that in open systems, crack-growth rates during continuous cooling generally were greatest at slow freezing rate, less than 0.1-0.5 °C/h. At more rapid freezing rate, the influx of water to growing cracks was significantly inhibited. For example, for cooling from -1 °C to -25 °C, a crack with 5 mm radius in granite grows nearly 2 mm if cooled at 0.025 °C/h, but only 0.4 mm if cooled at 0.1 °C/h.

For shotcrete, one of the main destructive mechanisms is hydraulic pressure. The pressure occurs when water become confined due to rapid freezing, so in this case rapid freezing is more harmful than slow freezing. But on the other hand, the other main destruction mechanism of shotcrete is ice lens growth, which is worse when the freezing rate is slow (Fridh, 2005).

**Field observations**

Observations in the Swedish railway tunnels have shown that water can continue to leak for a long time if the freezing rate is slow. This can cause ice formations like icicles and ice pillars to form in the tunnels, which can be a great problem. The ice pillars at the tunnel walls can grow so large that they intrude on the clearance gauge. The ice layers can also spread out over the tunnel floor and on the rails, which can cause derailment. Another problem in railway tunnels are that icicles can grow so long that they reach the overhead contact line, which can cause a short-circuit. The icicles are also a danger to the train traffic. They can fall down when a train passes the tunnel and can easily break through a window. The growth of icicles is dependent on access to water. It is known that icicles usually stop growing when the temperature drops rapidly – rapid freezing rate. This is because the crack, which provides the icicle with water, freezes (Andrén, in press 2007).

### 4.3.3 Duration and frost index

The duration and intensity of the temperature drop below 0 °C will affect the rate and amount of freezing of the soil and rock (French, 1996). The duration of freezing influences both the frost depth (Matsuoka, 2001b) and the internal damages for both rocks and shotcrete. The internal damages occur because a longer duration of constant low temperature gives the
mechanism of water migration more time to effectively produce an ice pressure (Fridh, 2005).

The temperature and the duration vary between the climate zones in our country which leads to different frost depth in respective zones. By means of climate data, every area in Sweden has a specific “frost index” based upon the freezing degree days. The calculation of the thawing and freezing degree days for a specific area is based upon the cumulative total air temperatures above or below zero for any one year. The mean daily air temperatures provide this data. A question commonly asked is whether the effect of the temperature remaining just below 0 °C for a certain length of time is the same as that of a more extreme drop but of a much shorter duration. This question shows that frost action is a complicated process, which is still not fully understood (French, 1996).

Field observations

Observations in Swedish railway tunnels have shown that the duration and alteration of the freezing periods have a great influence on the amount and location of leakage spots in a tunnel. If the freezing period has a long duration, some of the leakage spots become frozen. The water freezes and icicles are created, which act like a plug for the leakage spot. If the leakage instead is exposed to short periods of freezing and thawing, the crack will never become frozen and the water will continue to leak with growing icicles as a consequence. This phenomenon is most common in the middle and south parts of Sweden, where the temperature often fluctuate around 0 °C during the winter. In the north of Sweden there is a colder climate and the spots of leakage close to the tunnel entrance almost always become frozen. The problem with growing icicles in these sections just appears in the autumn and spring, not during the winter. But the problem can occur further in along the tunnels, where the air temperature is higher due to heat transport from the rock. Another problem is that the leakage water transports heat content from inner parts of the rock mass to the cold tunnel wall. The heat content of the water keeps the rock unfrozen in spite of negative temperatures in the tunnel. Therefore, the leakage spot will continue to leak (Andrén, in press 2007).

4.3.4 Freeze-thaw cycles

Discussions regarding weathering in cold environments generally centre on mechanical processes, like frost wedging and rock shattering and particularly on the freeze-thaw mechanism. Despite the almost ubiquitous assumption of freeze-thaw weathering, clear proof of interstitial rock water actually freezing and thawing is lacking. Many studies have used the crossing of 0 °C as the basis for determining the number of freeze-thaw cycles. However, Hall (2004) proved that freezing of rock water did actually take place, but he found that the
temperature at which freezing occurred, varied considerably through the year (see also section 5.2.2).

In his laboratory tests 1990 (see also section 5.1.3) Matsuoka used 47 saturated rock samples for freeze-thaw experiments. The deterioration of the specimen was detected through a reduction in longitudinal wave velocity; $V_p$. Figure 4.8 shows reduction of longitudinal wave velocity of sandstone specimens during 300 freeze-thaw cycles. The letters in the figure show different kinds of sandstone, a) has a porosity of 0.26% and g) has a porosity of 28%. The test proved that porous rocks are more sensitive to freeze-thaw cycles than rocks with lower porosity (Matsuoka, 1990a).

![Figure 4.8 Reduction of longitudinal wave velocity $V_p$ of sandstone specimens during 300 freeze-thaw cycles (Matsuoka, 1990a)](image)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Porosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>0.26</td>
</tr>
<tr>
<td>b</td>
<td>3.55</td>
</tr>
<tr>
<td>c</td>
<td>12.0</td>
</tr>
<tr>
<td>d</td>
<td>8.43</td>
</tr>
<tr>
<td>e</td>
<td>13.4</td>
</tr>
<tr>
<td>f</td>
<td>5.77</td>
</tr>
<tr>
<td>g</td>
<td>28.0</td>
</tr>
<tr>
<td>h</td>
<td>32.2</td>
</tr>
<tr>
<td>i</td>
<td>32.9</td>
</tr>
</tbody>
</table>

4.3.5 Temperature range

Matsuoka wrote an article in 2001 (Matsuoka, 2001b) which summarizes temperature ranges for effective frost weathering (see Table 4.1). Some care should be given to compare the listed values, since the methodology is different between researchers. A comment to the table is that laboratory data is not available for rocks with low porosity, while they rarely fail even after hundreds of freeze-thaw cycles in the laboratory, unless the samples contain pre-existing flaws (e.g. Lautridou and Ozouf, 1982; Matsuoka, 1990a).

According to Table 4.1 high porosity rocks begin to crack at 0 to -1 °C and terminate at about -5 °C. In medium porosity rocks the cracking temperature is between -3 °C and -6 °C (Hallet et al., 1991) and in low porosity rocks the cracking starts below -4 °C. Note that for low porosity rocks the temperature range for effective crack growth (theoretically suggested by Walder and Hallet, 1985) is applicable only to intact rocks and has to be validated by experiments. The experiments in Table 4.1 show that cracking progresses most rapidly just
below the upper temperature limit $T_H$, and decelerate toward the lower temperature limit $T_L$ (Matsuoka, 2001b).

Table 4.1 Temperature range for effective frost weathering, defined by the upper limit ($T_H$) and lower limit ($T_L$) (Matsuoka, 2001b)

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Porosity (%)</th>
<th>$T_H$ ($^\circ$C)</th>
<th>$T_L$ ($^\circ$C)</th>
<th>Indicator</th>
<th>Moisture supply</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low porosity rocks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite and marble</td>
<td>1–3</td>
<td>-4</td>
<td>-15</td>
<td>Theoretical</td>
<td>Open</td>
<td>Walker and Hallet (1985)</td>
</tr>
<tr>
<td>Medium porosity rocks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Berea sandstone</td>
<td>20</td>
<td>-3</td>
<td>-6</td>
<td>AE generation</td>
<td>Open</td>
<td>Hallet et al. (1991)</td>
</tr>
<tr>
<td>Berea sandstone</td>
<td>20</td>
<td>-0.2</td>
<td>-8</td>
<td>Length change</td>
<td>Closed</td>
<td>Mellor (1970)</td>
</tr>
<tr>
<td>Indiana limestone</td>
<td>14</td>
<td>-0.5</td>
<td>-8</td>
<td>Length change</td>
<td>Closed</td>
<td>Mellor (1970)</td>
</tr>
<tr>
<td>German sandstones</td>
<td>4–19</td>
<td>0</td>
<td>-5</td>
<td>Length change</td>
<td>Closed</td>
<td>Weiss (1992)</td>
</tr>
<tr>
<td>High porosity rocks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tuff, shale and andesite</td>
<td>23–46</td>
<td>0</td>
<td>-5</td>
<td>Length change</td>
<td>Open</td>
<td>Matsuoka (1990a)</td>
</tr>
<tr>
<td>Ohywa tuff</td>
<td>38</td>
<td>-1.4</td>
<td>-5</td>
<td>Ice lens formation</td>
<td>Open</td>
<td>Akagawa and Fukuda (1991)</td>
</tr>
<tr>
<td>Brézé chalk</td>
<td>47</td>
<td>0</td>
<td>-11</td>
<td>Length change</td>
<td>Closed</td>
<td>Pissart et al. (1993)</td>
</tr>
<tr>
<td>Brézé chalk</td>
<td>47</td>
<td>-0.2</td>
<td>-2</td>
<td>Length change</td>
<td>Open</td>
<td>Mutton et al. (2000)</td>
</tr>
</tbody>
</table>

The apparent relationship between porosity and cracking temperature is due to the following conditions. Firstly, the freezing point of pore water is generally lowered with decreasing porosity, depending on the amount of smaller pores. Secondly, ice segregation is optimized by the balance between the suction force, which increases with lowering temperature, and the water permeability, which decreases with lowering temperature (e.g. Williams and Smith, 1989). Furthermore, the strength of rock generally increases with decreasing porosity, so that larger suction force (hence, lower temperature) is required for low porosity rocks to crack (Matsuoka, 2001b).

Several investigators emphasize the role of microfractures in bedrock as the method by which water penetrates bedrock and subsequently freezes to cause rock disintegration. But microfracturing may not require freeze-thaw cycles or even falling temperatures but instead a constant sub-zero temperature. Walder and Hallet (1985, 1986) emphasized that a high frequency of freeze-thaw cycles is not necessary for crack propagation, which can occur at temperatures between $-4^\circ$C and $-15^\circ$C (French, 1996).

4.4 Ice pressure

Measuring the magnitude of the ice pressure generated by ice growth in a crack is a difficult task, because the ice pressure is a result of the rigidity of the adjacent material. Thus, the more rigid the material is, the greater the magnitude of the ice pressure (e.g. Krus and Sundquist,
Expansion of ice in an edge crack is mechanically similar to compression of material between two rigid plates as they move together. In either case, extrusion of the confined material occurs if its tensile strength is exceeded and when the crack walls are pressed away from each other, the ice pressure reduces (Tharp, 1987). Tharp (1987) found out that the cracks that are most susceptible to frost weathering are cracks with aspect ratio (maximum aperture/crack length) greater than 0.01 and terminating in sharp crack tips.

Ice pressure increases almost linearly from 0 MPa at 0 °C to a maximum of 207 MPa at -22 °C (see also section 3.2), which is attained if water pressure equals ice pressure during freezing (Tharp, 1987). In reality the maximum value of the ice pressure is almost certainly never reached or even approached, since a number of factors operate to reduce the pressure. Firstly, the water or ice must be contained within a closed system for high pressures to develop. This usually means conditions of extremely rapid freezing from the surface and downwards, which seals the pores and cracks in the rock. Secondly, air bubbles in the ice and pore spaces within the rock reduce pressures considerably. Thirdly, and probably the most important of all, the rock itself and certainly the soil mantle is not strong enough to withstand such extreme pressures, especially since it is a tensile force rather than a compressive force, which is being considered. As a result, the actual pressure developed by the freezing of water in rocks is much less than the theoretical maximum (French, 1996).

The magnitude of the ice pressure from different experiments varies a great deal in the literature. Some of the test results are summarized below:

- Heaving pressures associated with water migration may reach and exceed 20 MPa in freezing soils (Radd and Oertle, 1973; Takashi et al, 1980). Such pressure in small (grain-scale) cracks would be ample to cause crack growth in practically all rocks near the Earth’s surface (Walder and Hallet, 1985).

- Battle (1960) examined as early as 1960 the relationship between the geometry of water-filled cracks, temperature and pressure exerted during freezing in a granite crack. Theoretically, the pressure set up at the bottom of a water-filled crack, which is freezing from top downwards, varies directly with the length of the ice column in the crack, and inversely with the width of the crack. Examination of the results suggests that pressure in excess of 19.6 MPa could be exerted at the base of a crack 1 mm wide and 100 mm deep in a saturated non porous rock as granite if freezing took place from the top downwards (McGreevy, 1981).

- According to Blyth and de Freitas (1984) ice in pores, cracks and fissures can exert a pressure of about 14 MPa if the freezing occurs in a confined space.
- Davidson and Nye (1985) performed a laboratory test where the main interest was to measure the resulting stresses at an artificial crack, a slot in a lucite block. The maximum pressure which was measured in this experiment was 1.1 MPa, caused by volumetric expansion of ice and the confined water in the slot (see also section 5.1.1).

- Walder and Hallet (1985) developed a theoretical model for the breakdown of porous rock by the growth of ice within cracks (see also section 5.4). The maximum pressure, 12 MPa, was achieved when a crack with initial radius 5 mm freezes to a temperature of -12 °C.
5 FREEZING TESTS

In this chapter some of the most interesting freezing experiments of rock are gathered. Most of the experiments are conducted as freeze-thaw tests with or without access to water. Rock with low porosity is relatively resistant to frost shattering and therefore freezing tests are very seldom preformed on this kind of rocks. Instead porous rocks, which are less resistant to weathering, are used so that the result of the weathering processes can be reached within reasonable time limits. The results can be used for Swedish conditions, even though most of the rock where Banverket build their railway tunnels consists of hard, crystalline rock with low porosity, but the difference must be taken into consideration. For sections with crushed or heavily weathered rock, where the conditions resemble high porosity rock according to rock strength and water conditions, similarities to the results from weathering tests appears more often.

5.1 Laboratory tests

Simulating natural weathering conditions in the laboratory have its limitations but as long as these limitations are kept in mind, the simulations provide a powerful means of increasing our understanding of rock-weathering processes (McGreevy and Whalley, 1985). According to French (1996) another limitation of laboratory experiments is that there is relatively little information on the exact time and the temperature range under which actual breakdown occurs.

5.1.1 Ice pressure in a slot

Davidson and Nye (1985) preformed a test with the primary interest to measure the resulting stresses at an artificial crack, a slot. The slot was made in a transparent material (perspex/lucite) see Figure 5.1. It was filled with water and then frozen from the top down to create a pressure at the slot wall, while the water at the bottom of the slot was confined. It was not intended that the crack should propagate.
Figure 5.1 The slot made in perspex (Davidson and Nye, 1985)

Figure 5.2 shows the progression of the ice front down the slot. The x-axis shows the time in hours and the y-axis shows the position of the front, where l is the length of the slot and y is the distance measured from the bottom. Three lines are plotted: (I) the ice surface, showing a small amount of extrusion; (II) the position of the ice-water interface; (III) the position of a front dividing the cloudy ice from clear ice. This front (III) dividing the cloudy ice from the clear ice was formed parallel to the ice-water front, but some way behind, and the cloudy area was caused by ice separating from the slot walls.

Figure 5.2 Progression of the ice front versus time (Davidson and Nye, 1985)

Figure 5.3 shows both the positions of the ice-water front and the cloudy front (explanation in Figure 5.3b), and also the stresses along a line near to the slot wall and parallel to it. The normal stress, $\sigma_x$, is drawn with a solid line and the shear stress, $\tau_{xy}$, is drawn with a dotted line. Figure 5.3 shows that the highest stress occurs at the ice-water front and that the pressure decreases when the front passes. This is, as explained earlier, caused by the ice separating from the slot walls in the cloudy area.

The letters in parentheses refer to the ice front position in Figure 5.3.

Block: $70 \times 70 \times 32$ mm
Slot: width B=1 mm
depth h=32 mm
length l=25 mm
Figure 5.3  Distribution of shear stress $\tau_{xy}$ and normal stress $\sigma_x$ on the walls of the slot at different stages (a)-(g) of freezing (Davidson and Nye, 1985)

The maximum pressure measured in this experiment was 1.1 MPa (11 bar as shown in Figure 5.3f). This pressure is caused by the confined water in the slot. The contact area between ice and the slot wall is of great importance. If the contact is not sufficient the water pressure
decreases because the water is not confined in the bottom. It is true that the expansion of the crack as freezing progresses tends to release the ice plug, but the analysis of Davidson and Nye’s experiments suggest that this is a small effect of the total expansion. Most of the expansion of the slot occurs in the water-filled region below the level of the ice front.

There are two main differences between the experiment with perspex and conditions in real rocks. Firstly, the shape of the crack is different; the slot walls in the experiment were plane and fairly smooth rather than sinuous and rough as a crack in rock. Secondly, perspex is elastically softer than most rocks. The pressure developed in a crack depends essentially on the elastic and strength properties of the rock, because these control the opening of the crack (Davidson and Nye, 1985).

5.1.2 Crack widening

In 1995 Matsuoka conducted an experiment designed to simulate frost shattering of a granite block with an artificial crack (Figure 5.4). A parallel-sided slot was cut in the granite block and a closed system crack was produced by sealing both ends of the slot with silicon rubber to prevent water escaping from the slot. The rock block was cooled step-by-step in a cold chamber. The crack was empty at the beginning of the cooling and, when the rock temperature reached +1 °C, it was filled with water at +1 °C; otherwise part of the crack water may be lost by desiccation before freezing. Then, cooling was performed only from the upper surface.

Figure 5.4 Recording of freezing expansion in a crack (Matsuoka, 2001b)

Freezing began simultaneously in the super cooled crack water and ice progressed from the top towards the bottom. The temperature dip in Figure 5.5 arises from emission of the latent heat. After that, abrupt expansion occurred upon the ice nucleation. Figure 5.5 shows that
expansion was most rapid between 0 °C and -1 °C and decelerated below -1 °C. Cooling below -2 °C rarely contributed to the crack widening, because below this temperature ice starts to contract more than the rock.

As seen in Figure 5.5b the expansion of the crack is greater during the thawing phase (dotted line) than during the freezing phase (solid line). This arises from the fact that ice expands more rapidly than rock when thawing, causing the crack to widen.

Matsuoka also tested different rates of freezing, but no relations were found between the maximum expansion and the rate of freezing in his tests. He explained that the relationship between the two seemed to have been nullified by other effects; like the slight difference in the initial water level and the flexible silicon rubber absorbing part of the expansion (Matsuoka, 1995). The maximum expansion was about 0.5 %, which is much lower than predicted from the phase change (9 %). Matsuoka explained this by suggesting that the rigid crack wall forced ice to extrude both upwards and side wards (Matsuoka, 2001b).

### 5.1.3 Access to water

Matsuoka (1990a) demonstrated by laboratory tests the important role of water migration in the frost shattering process of the rock. He showed that expansion of the rock samples increased when the samples had access to water during freezing.

Matsuoka determined the freezing behaviour and frost shattering of 47 different samples of saturated rocks (sedimentary, igneous and metamorphic). The 5×5×5 cm samples were submerged one third of their heights in water for the open system experiments. The samples
were subjected to the repetition of half-daily freeze-thaw cycles with the maximum and minimum room temperatures at +20 °C and -20 °C respectively. This temperature cycle is useful to facilitate rock breakdown and hence data collection, despite the greater range and shorter period compared to the diurnal freeze-thaw cycled held in the field.

For a saturated specimen of tuff-b the expansion (shown in Figure 5.6 as increase of strain) occurred rapidly with decreasing temperature in a closed system. The expansion slowed down below -5 °C and stopped at about -10 °C. As seen in Figure 5.6b the thawing contraction curve follows the freezing expansion relatively well and both expansion and contraction occurred mostly between 0 °C and -5 °C, while most of the free water are freezing or thawing.

![Figure 5.6](image)  

**Figure 5.6** Strain data for tuff-b: closed system, cooling rate of 6 °C/h (Matsuoka, 1990a)

When Matsuoka instead tested the same rock in an open system, it showed a similar behaviour except for the difference in total expansion (Figure 5.7). Comparing these two conditions, the result is that the maximum freezing strain $\varepsilon_{L_{\text{max}}}$ in an open system is $4.7 \cdot 10^{-3}$ and in a closed system $2.8 \cdot 10^{-3}$. These results explain why a rock subjected to freeze-thaw action is more rapidly broken down in an open system.
Figure 5.7 Strain data for tuff-b: open system, cooling rate of 6 °C/h (Matsuoka, 1990a)

Figure 5.8 shows comparisons between the closed and the open systems for some rock specimens with different porosity. The rock specimen with the lowest porosity shows the smallest difference in expansion between the open and closed systems.

![Strain curve for several rocks against rock temperatures: comparisons between closed and open systems (Matsuoka, 1990a)]

<table>
<thead>
<tr>
<th>Sample</th>
<th>Porosity %</th>
<th>( \varepsilon_{L, max} ) open ( 10^{-3} )</th>
<th>( \varepsilon_{L, max} ) closed ( 10^{-3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shale-g</td>
<td>33.7</td>
<td>3.9</td>
<td>3.8</td>
</tr>
<tr>
<td>Tuff-c</td>
<td>45.5</td>
<td>2.2</td>
<td>3.8</td>
</tr>
<tr>
<td>Tuff-g</td>
<td>45.3</td>
<td>3.9</td>
<td>5.5</td>
</tr>
<tr>
<td>Andesite-e</td>
<td>22.5</td>
<td>1.3</td>
<td>1.4</td>
</tr>
</tbody>
</table>
5.1.4 Saturation

To understand the mechanisms of rock breakdown by frost shattering and weathering in cold regions, the influence of water saturation was tested by Chen et al. (2004). In this study, rock specimens were prepared from welded tuff (porosity 43 %) with a degree of saturation from 0 % to 95 %. The specimens were frozen in a temperature-controlled chamber where the temperature varied from +5 to -18 °C and then thawed in distilled water at 14 °C. The freeze-thaw test was conducted for only one cycle including 2 h of freezing and 1.5 h of thawing. The specimens were then dried at room temperature before testing. The deterioration of the specimens was examined by the changes in the uniaxial compressive strength, P-wave velocity, porosity and the appearance of crack pattern on the specimens.

Figure 5.9 shows that the uniaxial compressive strength of the specimen was not affected by the freeze-thaw cycle when the initial degree of saturation was below 60 %. But when the initial degree of saturation exceeded 70 %, the uniaxial compressive strength of the specimen was lowered by the freeze-thaw cycle, greatly if the initial degree of saturation was over 80 % (Chen et al., 2004).

![Figure 5.9](image)

Figure 5.9 Relationship between the initial degree of saturation and the uniaxial compressive strength after one freeze-thaw cycle (Chen et al., 2004)

Figure 5.10 shows the relationship between the initial degree of saturation and the P-wave velocity of the specimens. If the P-wave velocity reduces in the rock sample it is exposed for deterioration. When the initial degree of saturation was below 60 %, the P-wave velocity was not affected by the freeze-thaw cycle. When the initial degree of saturation was above 70 %, the P-wave velocity decreased with increasing initial degree of saturation (Chen et al., 2004).
The porosity of a rock specimen can be determined by the water saturation method. In this method, a specimen is immersed in distilled water under vacuum condition for 72 h. The saturated specimen was weighed, and then dried in an oven at 105 °C for 48 h, and weighed again. Figure 5.11 shows the relationship between initial degree of saturation and change in porosity. The change in the porosity of a specimen was determined by comparing the weight of the specimen before and after the freeze-thaw test, using the water saturation method. When the initial degree of saturation was below 60 %, the increase in porosity was less than 1 %. When the initial degree of saturation was over 70 % the porosity increased by 2 % to 6.3 % after one freeze-thaw cycle (Chen et al., 2004).
Figure 5.13 shows the relationship between the initial degree of saturation and the maximum crack width. The appearance of cracks was observed with a camera before and after freezing (Figure 5.12). When the initial degree of saturation was 78 % or lower, no cracks were observed but when the initial degree of saturation was between 80 % and 90 %, cracks were formed in a linear pattern. When the initial degree of saturation was higher than 90 %, the crack pattern became radial and the maximum crack width increased with increasing initial degree of saturation, reaching 1.0 mm when the initial degree of saturation was 95 % (Chen et al., 2004).

Figure 5.12  Cracks on specimen surfaces after one freeze-thaw cycle, $S_r =$ initial degree of saturation (Chen et al., 2004)

Figure 5.13  Relationship between the initial degree of saturation and the maximum crack width (Chen et al., 2004)
The experimental results preformed by Chen et al. show that when the initial degree of saturation was maintained below 60 % in the welded tuff, the properties did not change, but when the initial degree of saturation exceeded 70 %, the rock was damaged significantly. Therefore, the critical degree of saturation is about 70 % for this particular rock.

The deterioration of the rock occurs because the volumetric expansion of water/ice exerts pressure within a pore. If this pressure exceeds the tensile strength of the material, the material will be damaged. A fully saturated rock yields to frost action regardless of its strength, while a partially saturated rock can resist this action even when its strength is low. This implies that a critical degree of saturation exists for a rock and this critical value varies from rock to rock according to pore size distribution, pore connectivity and initial porosity, which all influence the tensile strength (Chen et al., 2004).

5.1.5 The influence of pre-existing flaws

Nicholson and Nicholson (2000) exposed ten types of sedimentary rocks to repeated cycles of freezing and thawing with the intention to consider the influence of material flaws and planes of weakness on breakdown due to frost weathering.

The results indicated that the weakest (lowest compressive strength) rocks were least durable and the strongest rocks were most durable. The result was not unexpected since compressive strength relates closely to tensile strength and porosity. These properties had been identified by Matsuoka (1990a) for their role in frost susceptibility. Nicholson and Nicholson (2000) observed four different modes of deterioration:

Rapid, severe disintegration: occurred in the weakest rocks. Breakage along pre-existing flaws appeared to be incidental.

Grain loss: occurred only in rocks with a granular texture, particularly those of coarser grain size. Initial breakdown by grain loss was followed by cracking, which subsequently developed into flaking.

Fracturing and large-scale fragmentation: occurred in the strongest rocks, where deterioration almost entirely consisted of fracture and breakage along pre-existing flaws. That is comparable to the results of an experimental frost weathering study by Brockie (1972) in which several highly resistant schists broke along pre-existing “fissures or lines of weakness” early in the test procedure.

Scaling: (the detachment and peeling off of single surface layers) occurred in three samples in
addition to other deterioration modes, but never in isolation. Scaling did not appear to relate to pre-existing flaws and its occurrence is, therefore, likely to relate directly to the frost weathering process.

The results from the tests conducted by Nicholson and Nicholson (2000) suggested that the presence or absence of rock flaws alone do not control the deterioration mode, but rather that it is the coupled relationship between these flaws, the rock strength and the textural properties which impose the greatest influence. The influence of pre-existing flaws is particularly important in the deterioration of stronger rocks. But in weaker rocks their direct influence diminishes, as the influence of other rock properties and environmental factors increases. For strong rocks, these findings support the conclusions of Tharp (1987) and Douglas (1981) that environmental conditions are subordinate to discontinuities in terms of their effect on weathering. For weak rocks, the findings indicate that macroflaws are of less importance to frost susceptibility than other rock properties (Nicholson and Nicholson, 2000).

5.2 Field tests

5.2.1 Frost wedging in alpine bedrock

Matsuoka (2001a) observed frost wedging in alpine bedrock during three years. The width and temperature of rock joints were automatically monitored on a sandstone rock face which showed two seasonal peaks of joint widening in autumn and spring (Figure 5.14).

Figure 5.14 Temporal variations in the width and temperature of the monitored joint A) crack movement B) crack-top temperature (Matsuoka, 2001a)
Matsuoka found that the autumn events were associated with short-term freeze-thaw cycles where the magnitude of joint’s widening was proportional to the subzero temperatures recorded at the top of the joint. He suggested that this correlation occurs due to an expansive force that rises with ice front propagation in the joint. The spring events were associated with a rise in the rock surface temperature to 0 °C beneath the seasonal snow cover, and the expansion appear to originate from refreezing of melt water, which had entered the joint. The largest widening occurs in the thawing period (see expansion in Figure 5.14). One theory of why this phenomenon occurs is as follows. When the ice in a joint starts melting, a space between the frozen rock wall and the ice in the joint is formed. This space is filled up by melt-water entering from the surface. When this water refreezes in the joint, the water gets confined and produces a large expansion (Matsuoka, 2001a).

The results of the measurements demonstrate that joint widening during the autumn freeze-thaw period reflects the intensity and duration of freezing. In Figure 5.15A the relationship between expansion and temperature are presented and the expansion increase with falling temperature (Matsuoka, 2001a).

![Figure 5.15](image_url)

**Figure 5.15** The maximum expansion reached during each freeze-thaw cycle in autumn, as a function of (A) the subzero crack-top temperature and (B) the frost penetration depth (Matsuoka, 2001a)
One interpretation for the correlation in Figure 5.15A was that the downward ice growth was responsible for progressive widening of the joint. When the crack-top temperature falls to -5 °C, the ice front may already be at a depth of a few decimetres. In order to examine this possibility, Matsuoka estimated the frost depth from penetration of the 0 °C isotherm into the bedrock when the joint width reached a maximum during each freeze-thaw cycle. Figure 5.15B demonstrates that the magnitude of joint widening (expansion) was nearly proportional to the frost depth, indicating a rise in expansive force with ice front propagation (Matsuoka, 2001a). This relationship may indicate that the pressure of the confined water beyond the ice front rises with ice propagation (Davidson and Nye, 1985).

5.2.2 Freeze-thaw events

Hall reported in an article in 2004 that many studies so far have used the crossing of 0 °C, as the basis for determining the number of freeze-thaw events. Despite the almost ubiquitous assumption of freeze-thaw weathering, clear proof of interstitial rock water actually freezing and thawing is lacking.

In order to assess the weathering regime at a site in northern Canada, temperatures were collected at the surface, 1 cm and 3 cm depth of 24 concrete paving bricks. To observe the influence of solar radiation the paving bricks were exposed vertically and at 45° and were oriented to the four cardinal points. Temperature data were collected at 1 min intervals for 1 year. In Figure 5.16 the surface temperature for vertical faces are plotted during a few days in the winter of 2001. There is a small temperature peak to the right of every big peak. This is more accurately shown in Figure 5.17.
Hall’s data provide obvious proof for the occurrence of the freezing and thawing of water on and within the rock (freeze-thaw events). The freeze event is evidenced by the exotherm associated with the release of latent heat as the water actually freezes. Figure 5.17 indicates that the freezing of water actually took place just below the brick surface. It is a clear demonstration of water freezing taking place and because this event occurs again on the subsequent night, it indicates also that thawing had taken place in between. This field test was thought to be the first record of such events. The short duration of the exothermic event also demonstrates the need for high-frequency data acquisition (Hall and André, 2001) for any meaningful understanding of the weathering process that may or may not be taking place. Hall’s tests also showed that the temperature at which freezing occurred varied considerably throughout the year (Hall, 2004).
Figure 5.17  Detail regarding a surface exotherm (south aspect, vertical brick) for January 2001 (Hall, 2004)
5.3 The gap between laboratory and field weathering

Though laboratory studies have documented some empirical relationships between the frost sensitivity of rocks and such variables as temperature, moisture and rock properties, there seems to remain a significant gap between laboratory and field weathering. Most laboratory studies use intact rock samples with medium (5-20 %) to high (>20 %) porosity, while many field studies concern weathering of low porosity (<5 %) rocks with joints (Matsuoka, 2001b). Laboratory data are not common for rocks with low porosity, because these rocks rarely fail even after hundreds of freeze-thaw cycles under an optimum moisture condition in the laboratory, unless they containing pre-existing flaws (e.g. Lautridou and Ozouf, 1982; Matsuoka, 1990a).

In a paper Matsuoka (2001b) aimed at bridging the gap between laboratory and field studies, by describing the differences between microgelivation\(^1\) and macrogelivation\(^2\). Matsuoka’s paper reviews dynamic approaches to frost weathering by comparing the two contrasting conditions: (i) microgelivation of soft, intact rocks and (ii) macrogelivation of hard, jointed rocks. Matsuoka stated that most of the laboratory studies simulate microgelivation, while field studies mainly deal with macrogelivation. Figure 5.18 illustrates how laboratory conditions differ from field conditions.

![Figure 5.18](image_url)  
Figure 5.18 Contrasts between laboratory and field frost weathering (Matsuoka, 2001b)

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\(^1\) Microgelivation = degradation of material in small scale, which involves granular disintegration or flaking. This process leads to production of fine debris (μm-to-cm scale of fragmentation).

\(^2\) Macrogelivation = degradation of material in a greater scale than microgelivation. Macrogelivation is opening or wedging of pre-existing macrofractures or joints. It tends to produce pebble-size or coarser materials (cm-to-m scale).
The difference between micro- and macrogelivation is essential in connecting laboratory results with field observations. The most intensive disintegration of bedrock in the field uses pre-existing macrofractures, while laboratory studies mainly define conditions for the breakdown of intact rocks (e.g. McGreevy and Whalley, 1982). To prove that ice action causes disintegration, direct evidence is needed. This can be obtained in the field by simultaneous monitoring of the rock disintegration and associated parameters. This is so far rare for soft, intact rocks, because of technological immaturity and the need for long term operations. Therefore laboratory criteria will continue to be useful for interpretation of field observations. Field problems which can be simulated by laboratory models involve small-scale fragmentation of soft rocks on a rock outcrop (Matsuoka, 2001b).

To study micro- and macrogelivation in the field the researches should at least examine three factors: water accessibility (e.g. distance from open water or depth of the water table), dominant freeze-thaw regime (diurnal, annual or inter-annual) and rock properties (e.g. porosity; strength). When comparing field with laboratory values, the use of laboratory criteria should include the following limitations:

1. Most of the criteria are applicable only to porous, soft rocks (porosity more than 5 %).
2. Thresholds (e.g. critical temperature) may vary with rock properties.
3. Laboratory criteria account mainly for fragmentation up to the centimetre scale.
4. Even though a proper criterion is chosen for the field rock, it is applicable only to the surficial fragmentation; in fact, ice lens formation at a certain depth requires cooling below the critical temperature experienced not at the rock surface but at this depth (Matsuoka, 2001b).

5.4 Mathematical model

Walder and Hallet (1985) presented a mathematical model for breakdown of porous rock by growth of ice. The model was founded upon well-established principles of fracture mechanics and soil physics, along with assumptions that progressive crack growth resulted from water migration to ice bodies in cracks, like water migrates to ice lenses in soil. The model is based on material parameters (elastic modules, fracture-mechanical properties, crack size, grain size and shape) and environmental properties (temperature, temperature gradient, water pressure).

For a granite and a marble the most effective crack growth appear when temperatures range from -4 °C till -15 °C, due to the fact that the frozen fringe still is relative permeable. At higher temperatures, thermodynamic limitations prevent ice pressure from building up
sufficiently to produce significant crack growth – thermodynamic equilibrium has appeared
between ice crystals and unfrozen water adsorbed to mineral particles. At lower temperatures
the ice pressure don’t build up, while the permeability of the frozen fringe is extremely low
and the water migration is strongly inhibit (Walder and Hallet, 1985).

**Assumption in the model**

All rocks contain pore spaces of various shapes and sizes. In their model, Walder and Hallet
chose a crack which was circular in plan view (radius c) and had a very small opening which
decreased from a maximum of w at the centre to 0 at the edges (Figure 5.19).

![Figure 5.19 Idealization of freezing of cracked rock (Walder and Hallet, 1985)](image)

They assumed that (i) cracks are widely enough spaced that they grow independently of each
other and that (ii) all crack growth is within the plane of the crack and is induced only by
uniform, internal ice pressure. Under these conditions, crack propagation in brittle, elastic
solids is governed by the magnitude of the stress-intensity factor $K_I$. For a crack with radius $c$,
Sneddon and Lowengrub (1969) calculated $K_I$ as follows:

$$K_I = \left(\frac{4c}{\pi}\right)^{\frac{1}{2}} \cdot p_i$$

where;

- $K_I = \text{stress-intensity factor}$
- $c = \text{crack radius}$
- $p_i = \text{internal pressure}$
Cracks propagate unstably when $K_I$ exceeds $K_c$ ($K_c$ is a characteristic of the material known as the fracture toughness). For values $K_I < K_c$, it is well known that slow, stable crack growth occurs.

More assumptions of the model are as follows:

- pore spaces of unfrozen rock is assumed to be completely saturated at all times
- the rock is cooled progressively from the surface
- the crack lengths are large compared to the grain size, but small enough that the temperature differences from end to end are negligible.

As the freezing front propagates past any given crack, a thin zone in which the pore space is only partially ice-filled will form adjacent to the warm side of the crack. This zone is called the frozen fringe. Below this frozen fringe the rock is completely unfrozen.

**Results**

The model deals with cracks in granite with initial radii of 5 and 50 mm. The result of the study showed that crack propagation were most rapid for the crack with the larger initial radius, while the highest pressure 12 MPa, developed in the crack with the smaller initial radius.
Figure 5.20 shows the crack growth rate in granite as a function of crack wall temperatures. For the crack with the larger initial radius (50 mm) the crack propagation rate are higher than for the crack with the smaller initial radius (5 mm). The crack growth rate for the 50 mm crack, are most rapid from 0 °C to -5 °C, and after that the rate decreases. For the smaller crack, the most rapid crack growth rate develops between -6 °C to -15 °C.

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Freezing for 50 h

Freezing for 100 h

\(c_0 = \text{initial radius}\)

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Figure 5.20 Crack growth rate in granite as a function of crack wall temperatures (Walder and Hallet, 1985)
Figure 5.21 shows crack growth in granite and ice pressure within crack as a function of time. The highest pressure, 12 MPa, develops when the crack with the initial radius of 5 mm freezes at -12 °C. Notice that the crack propagation only is 0.3 mm, but interestingly the crack propagation continues in spite of constant ice pressure. The largest crack propagation, 16 mm and still growing at constant pressure (see Figure 5.21b), develops in the crack with the initial radius of 50 mm, which freezes at -6 °C. The ice pressure rises to 4 MPa and stays constant.

Figure 5.21 Cracking history during sustained freezing of granite (Walder and Hallet, 1985)

The effect of crack growth depending on the cooling rate is illustrated in Figure 5.22. Each crack growth was terminated when the temperature reached -25 °C. It is obvious for a fixed final temperature that the lower the cooling rate the greater crack growth. For example; when cooling the crack with an initial radius of 5 mm in granite from -1 °C to -25 °C, it grows almost 2 mm if the cooling rate is 0.025 °C/h, but only 0.4 mm if the cooling rate is 0.1 °C/h (look at the dotted line in Figure 5.22). The crack growth reflects the increase of flow resistance at the frozen fringe during cooling. If the cooling rate is high, the crack growth is small or non-existent, due to the fact that the frozen fringe is impermeable. With low cooling rate, the frozen fringe still is permissible and water can migrate to the crack, which continues to grow. Crack growth occurs even if the ice pressure drops.
Figure 5.22 Cracking history during monotonic cooling as function of time (Walder and Hallet, 1985)

Walder and Hallet (1985) emphasized that water migration is the limiting factor in crack growth and that frost shattering doesn’t necessarily require freeze-thaw oscillations.
6 CONCLUSIONS AND PROPOSAL FOR FURTHER RESEARCH

The reason for the degradation of rock and shotcrete in tunnels is not clear but this report and literature review shows that water in combination with negative temperatures is one of the major reasons for degradation. The problem with ice formation in tunnels is a problem of itself. The problems with frost action in rock and shotcrete resemble those in soil, and as in soil water tends to migrate to the frozen fringe in rock. This water migration can cause frost shattering of rock and shotcrete.

The hypothesis as to why shotcrete and rock falls down in Banverket’s railway tunnels is that an ice pressure develops in cracks and in the interface between the rock and the shotcrete due to (i) volumetric expansion of existing water during freezing and (ii) to the fact that water tends to migrate to the frozen fringe and causes ice growth. If the ice pressure exceeds the tensile strength of the rock material or the adhesive strength of the shotcrete, the material will yield to the pressure and cracks will appear. This can cause damage to the tunnel construction and in the worse case, fall outs of rock and shotcrete.

To verify the hypothesis and to see how water migration influences the growth of ice in the interface between rock and shotcrete, laboratory studies consisting of freeze tests of shotcrete/rock samples are recommended. The freezing tests should be performed both with and without access to water during freezing, to see the difference in degradation for different water conditions. To find out if degradation occurs, adhesive strength between shotcrete and rock can be tested. The results of adhesion tests should reveal if the ice pressure causes more damage to the shotcrete/rock sample which has access to water during freezing, than the shotcrete/rock sample which has not access to water during freezing.
REFERENCES


Ehara, S. et al., 1985. Thermal expansion of saturated rocks subjected to cyclic temperature change between 100 K and 300 K. *Zairyo* 34(382) 864-870. ISSN 0514-5163. [From Glamheden, 2001]


