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

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PREFACE

This licentiate thesis forms part of a research project that was initiated in 2002 by the Swedish Rail Administration (Banverket) and undertaken as collaboration between it and the Division of Mining and Geotechnical Engineering at the Luleå University of Technology from 2004 to 2009. Supervision was provided by Professor Lars-Olof Dahlström and Professor Erling Nordlund at Luleå University of Technology, with financial support provided by the Swedish Rail Administration in Borlänge.

First of all, I would like to thank Professor Lars-Olof Dahlström for his valuable contributions to discussions and for all the support he has given me in my work. I would also like to thank my project reference group for their support and their many suggestions as to how to improve my work, which have been of great importance. The individuals involved are: Professor Erling Nordlund, Professor Lars-Olof Dahlström and Professor Sven Knutsson at the Division of Mining and Geotechnical Engineering, Luleå University of Technology; Dr. Tommy Olsson at I&T Olsson AB for participation during the literature review, and Tommy Ellison at BESAB for participation during the laboratory tests. Thanks are also due to TESTLAB, and especially Roger Lindfors at Luleå University of Technology for performing the freeze-thaw tests; to Ganesh Mainali at Luleå University of Technology for the acoustic emission measurements; Göran Kindahl at BESAB AB in Gothenburg, Owe and David Persson at Jerneviken Maskin AB in Gothenburg for helping me with the test panels. I would also like to thank all the people who contributed ideas and inspiration at the start of this research project.

I would like to give special thanks to my family for the never-ending encouragement, love and support.

Finally, I want to express my gratitude to my beloved life partner, David, a million thanks for everything!

Luleå, March 2009

Anna Andrén
SUMMARY

In recent years the Swedish Rail Administration has observed an increased incidence of shotcrete and rock fall-outs in its tunnels, for which reason it has initiated several research projects, of which the present project entitled “Degradation of rock and shotcrete due to ice pressure and frost shattering” is one. The aim of this licentiate project was to bring together experience and information relating to ice formation and the effect of ice pressure on fault zones, cracks and, in particular, the shotcrete/rock interface. Furthermore, the hypothesis from the literature review is tested and the results of the laboratory tests are presented.

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 this ice pressure exceeds the tensile strength of the adjacent material or the adhesive strength of the shotcrete/rock interface, the material will be damaged. The degree of damage depends, among other factors, on the degree of saturation of the material. A partially saturated material can resist breakage despite its low tensile strength, because ice expansion and pore-water distribution can occur in pores which were initially filled with air. A fully saturated material however yields to frost action regardless of its tensile strength, because it has none of the free space this expansion requires.

Volumetric expansion is not the only cause of frost shattering and 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 ice bodies to grow inside pores and cracks. This water migration takes place because a thin film of adsorbed water occurs at the surface of mineral particles and it is in this water film that water is able to migrate towards the frozen zones. Experimental work has shown that a considerable amount of adsorbed water remains unfrozen at sub-zero temperatures not only in soils, but also in rocks, which enable water migration.

Water migration and ice growth thus depend not only on access to water and freezing temperatures, but also on the duration of these temperatures and the freezing rate. If rock or shotcrete is subjected to rapid freezing, the thickness of the water film is quickly reduced and the water migration is inhibited, which

1 frozen fringe – layer between frozen and unfrozen rock or soil
limits frost damages to rock and shotcrete. By contrast, a slow freezing allows water migration to occur over a longer period, which can result in greater frost damage to rock and shotcrete. The field investigations found changes to the freezing periods as well as their duration to be of major importance to ice formation growth. If the freezing period was of long duration, several of the cracks and the leakage spots freeze. If leakage is subjected instead to short periods of freezing and thawing, the water in the crack will never freeze and will continue to leak, resulting in ice formation growth. In cold areas, such as the north of Sweden, this problem takes place even far inside the tunnels. This phenomenon occurs because the leakage water transports heat from the rock mass to the cold tunnel wall. The heat content of the water keeps the rock around the crack opening from freezing despite sub-zero tunnel air temperatures. Hence, the leakage spot will continue to leak, until a certain temperature and temperature duration is achieved, which results in ice formations when the water meets the cold tunnel air. Another experience in the field investigations was that the rock and shotcrete fall-outs often occurred in areas with leakage problems.

The results of the laboratory tests performed in this licentiate project also show that water in combination with freezing temperature can cause degradation problems. The tensile tests undertaken, showed that the adhesive strength decreased about 50% when the shotcrete/rock samples had been subjected to freeze-thaw cycles. Furthermore, acoustic emission measurements (AE) showed that more events\(^2\) took place when the shotcrete/rock panels had access to free water during freezing.

The literature review, field investigations of railway tunnels and the laboratory tests shows that access to water during freezing can cause damage to the shotcrete/rock interface. This confirms the hypothesis that shotcrete and rock fall-outs can occur because ice pressure in a crack or at the interface exceeds the tensile strength of the material or the adhesive strength between rock and shotcrete. One thing that the laboratory tests failed to provide a satisfactory answer to, was whether these fall-outs could occur due to widening of an initially small area of poor adhesion around a rock crack opening. However, the laboratory test showed a lot of activity during freezing in those areas prepared with poor adhesion. It thus it appears that small areas of poor adhesion can be a factor.

\(^2\) event – a local material change which gives rise to acoustic emission, for example adhesion failure, crack development, etc.
adhesion in some way affect deterioration of the adhesive strength of the shotcrete/rock interface.

Keywords: Ice pressure, rock and shotcrete degradation, water migration, adhesion, frost shattering, shotcrete fall-out, freeze-thaw tests, adhesive strength, railway tunnel.
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 detta licentiatprojekt var att samla erfarenhet och information om hur is bildas samt hur istryck påverkar krosszoner, sprickor och framför allt skiktet mellan berg och sprutbetong. Vidare testas hypotesen från litteraturstudien och resultaten från laborationsförsök redovisas.

När vatten fryser till is sker en 9 % volymsutvidgning och denna expansion kan orsaka att ett tryck uppstår mot det omgivande materialet. Det omgivande materialet kommer att utsättas för brott om trycket från isen överstiger materialets draghållfasthet eller vidhäftningshållfasthet i skiktet mellan berg och sprutbetong. 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.

Det är inte bara isens volymsutvidgningen som orsakar frostsprängning. Forskning visar att om berg har tillgång till fritt vatten under nedkylningen sker en process som liknar den i jord, där vatten vandrar fram mot frysfronten och bildar islinser. På ett liknande sätt vandrar vatten i berg och orsakar tillväxt av issektorer. Vattenvandringen sker på grund av att det finns en tunn vattenfilm av adsorberat vatten längs mineralkornens ytor 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 möjliggör vattenvandringen.

Vattenvandring och istillväxt är inte bara beroende av tillgången till vatten och frystemperatur, utan även av frystabilitet och varaktighet av köldgrader. Om berg och sprutbetong utsätts för snabb nedkylning minskar vattenfilmenς tjocklek och vattenvandringen förhindras, vilket begränsar frostsprängningen.
av materialet. Om istället berget kyls ned långsamt, tillåts vattenvandringen att ske under en längre period, vilket kan resultera i större frostsprängning. I de utförda fältundersökningarna visade det sig att varaktigheten och förändring av frysperioderna var av stor vikt för tillväxten av isformationer. Om frysperioden hade lång varaktighet frös vissa av sprickorna och läckagepunkterna. Om läckagen istället utsattes för kortare perioder av fyrning och tining frös aldrig sprickorna och vatten fortsatte att läcka med växande isformationer som följd. För kalla områden, som i de norra delarna av Sverige, uppstår dessa problem även långt in i tunnlarerna. Problemen uppstår på grund av att läckagevatten transporterar fram värme från bergmassan till den kalla tunnelytan. Värmen från läckagevattnet håller bergmassan kring spricköppningen ofrusen trots att tunneluften är kall. Därför fortsätter sprickan att föra fram vatten med konsekvensen att det bildas is när vattnet väl kommer ut i den kalla tunneluften. En annan erfarenhet från fältundersökningarna var att utfallen av berg och sprutbetong ofta förekom i sektioner som hade problem med vattenläckage.

Resultaten från laborationsförsöken utförda i det här licentiatprojektet visar också att vatten i kombination med negativa temperaturer kan orsaka nedbrytningsproblem. De utförda dragtesterna visade att vidhäftningshållfastheten minskar med ungefär 50% när sprutbetong/bergproverna hade utsatts för fyrning. Vidare visade mätningarna av akustisk emission (AE) att fler AE-händelser3 skedde när sprutbetong/bergproverna hade tillgång till vatten under fyrningen.

Litteraturstudien, fältundersökningarna i järnvägstunnlarerna och laborationsförsöken pekar på att tillgången på vatten under fyrning kan orsaka skador på skiktet mellan berg och sprutbetong. Detta bekräftar hypotesen att utfall av berg och sprutbetong kan uppstå på grund av att istryck i en spricka eller i skiktet mellan berg och sprutbetong överskrider draghållfastheten för materialet eller vidhäftningshållfastheten mellan berg och sprutbetong. En sak som laborationsförsöken inte kunde ge ett bra svar på var ifall utfallen kunde ske på grund av spridning av en liten yta som redan från början hade dålig vidhäftning runt en spricköppning. Men försöken visade att det förekom mycket aktivitet under fyrningen i de områden som preparerats med dålig vidhäftning. Så det verkar som att små områden med

3 AE-händelser – lokal material förändring som ger upphov till akustisk emission (ljud företeelse), t.ex. vidhäftningsbrott, material uppsprickning, etc.
dålig vidhäftning kan påverka försämringen av vidhäftningen mellan berg och sprutbetong.

Nyckelord: Istryck, nedbrytning av berg och sprutbetong, vattenvandring, vidhäftning, frostsprängning, nedfall av sprutbetong, frezelförsök, vidhäftningshållfasthet, järnvägstunnel.
**LIST OF SYMBOLS AND ABBREVIATIONS**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$\alpha$</td>
<td>thermal expansion or contraction</td>
</tr>
<tr>
<td>$\varepsilon_{L,\text{max}}$</td>
<td>maximum freezing strain</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>thermal diffusivity</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>thermal conductivity</td>
</tr>
<tr>
<td>$\rho$</td>
<td>density</td>
</tr>
<tr>
<td>$B$</td>
<td>slot width</td>
</tr>
<tr>
<td>$c$</td>
<td>crack radius</td>
</tr>
<tr>
<td>$c_0$</td>
<td>initial crack radius</td>
</tr>
<tr>
<td>$c_p$</td>
<td>specific heat</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter of boreholes in the test panels</td>
</tr>
<tr>
<td>$D$</td>
<td>diameter of test samples</td>
</tr>
<tr>
<td>$H$</td>
<td>slot depth</td>
</tr>
<tr>
<td>$l$</td>
<td>slot length</td>
</tr>
<tr>
<td>$L_0$</td>
<td>length at temperature $T_0$</td>
</tr>
<tr>
<td>$L_T$</td>
<td>length at temperature $T$</td>
</tr>
<tr>
<td>$p_i$</td>
<td>internal ice pressure in crack</td>
</tr>
<tr>
<td>$S_r$</td>
<td>initial degree of saturation</td>
</tr>
<tr>
<td>$T_{\alpha}$</td>
<td>temperature at the start</td>
</tr>
<tr>
<td>$T_\alpha$</td>
<td>temperature after alteration</td>
</tr>
<tr>
<td>$T_H$</td>
<td>upper temperature limit</td>
</tr>
<tr>
<td>$T_L$</td>
<td>lower temperature limit</td>
</tr>
<tr>
<td>$V_p$</td>
<td>longitudinal wave velocity</td>
</tr>
<tr>
<td>$w$</td>
<td>crack width at point of widest opening</td>
</tr>
</tbody>
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**Definitions**

- **closed system** = no access to water during the freezing period
- **event** = a local material change which gives rise to acoustic emission, for example adhesion failure, crack development, etc.
- **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
- **P-wave velocity** = longitudinal wave velocity
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1 INTRODUCTION

Every winter a number of railway tunnels in Sweden are affected by problems related to water leakage and freezing temperatures, which causes ice to form along the tunnel contour in the form of icicles and pillars that can damage the tunnel structure, handrails, cable racks, installations and drainage system. Furthermore, the ice formations obstruct train passages and can cause time delays. Such problems necessitate a considerable maintenance effort during winter. The problem of ice is of greater concern when it occurs in the fracture network close to the tunnel contour or in the interface between the rock and shotcrete, since this can cause fall-outs in both materials.

The purpose of the initial literature review (Andrén, 2006) was to collect experience and information as to how ice forms, how ice pressure develops and affects the rock mass including its discontinuities and the interface between rock and shotcrete. A summary of the literature review is included in this licentiate thesis, and furthermore, hypothesis from the literature review is tested and the results from laboratory testing to study the influence of ice formation on shotcrete/rock adhesion are presented.

1.1 Background

Leakage and ice formation have always been a problem in Swedish railway tunnels. In the past few years, the Swedish Rail Administration has observed an increase in the number of incidents involving shotcrete and rock fall-outs in both older and newer railway tunnels. The problem of ice results from unsuccessful efforts to prevent water reaching the cold tunnel air, while successful prevention of water leakage eliminates problems of ice formation in the tunnels. Ensuring a completely dry tunnel without the use of an impermeable tunnel construction, such as lining, is difficult. In Sweden grouting is traditionally employed to prevent water leakage, in preference to the more expensive alternative of an impermeable tunnel construction.
1.1.1 Leakage

Leakage into a tunnel depends on factors such as the rock mass properties, tunnel position below ground surface, groundwater level, and others. Rocks are classified into three different groups based upon their formation process, i.e. igneous, sedimentary and metamorphic. In Sweden, igneous and metamorphic rocks (crystalline rocks) are the most common (Loberg 1993). Crystalline rocks are often dense and water occurrence is concentrated in crack and fissure systems (Fairhurst et al., 1993). Microcracks occur between the mineral particles, but on the whole, porosity and hydraulic conductivity are low (Gustafson, 1986). Sedimentary rocks can be porous and have high hydraulic conductivity, and the groundwater occurs in the pores and is uniformly distributed throughout the rock mass (Fairhurst et al., 1993).

When a tunnel is excavated, the characteristics of the rock mass closest to the tunnel may change (Pusch, 1989). Excavation 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 an increase or decrease in the normal stress across the crack. An increase in compressive stress can cause closure of a crack, while others cracks can open up due to a decrease in compressive or shear stress (Hakami, 1988). Shooting and blasting in a tunnel can widen the cracks and cause an increase in leakage and thus, smooth blasting is employed nowadays in the construction of most tunnels to avoid such problems. Leakage is also affected by topography and the distance from the groundwater level. If a tunnel is located below the groundwater level, it will always have access to water, while leakage into a tunnel close to or above groundwater level depends on such factors as precipitation and ground frost (e.g. Andrén, 1995).

Experience shows the crack frequency along a tunnel in Sweden to be approximately 1-3 cracks with an aperture of 0.1-1 mm per meter. 75 % of the water leakage is thought to originate from only a few larger cracks, while the remaining 25 % leakage originates from a large number of small cracks (Vägverket, 1994).

1.1.2 Prevention of water leakage

Water leakage into tunnels must be prevented to reduce the problems caused by ice. The permissible amount of daily water leakage level varies with each
specific tunnel and such leakage into it must not be allowed to effect its surroundings, i.e., lower the groundwater level.

The most common method used in Sweden to prevent water leakage is grouting and whenever it proves inadequate, other methods must be employed to address water leakage. These include watertight linings or diversion of water from the tunnel using insulated drainage or geotextile to prevent water from reaching the tunnel. If leakage causes a lowering of the groundwater level in areas where this is prohibited, infiltration can be used. However, diversion and infiltration increase operative outlays, and life-cycle costs thus have to be analysed when selecting a method of elucidating the most suitable solution for each specific site (Banverket, 2004).

Grouting is the most common method in the Nordic countries, unlike other European countries, which use impermeable tunnel constructions to prevent water leakage. Such constructions are far more expensive than grouting, but, on the other hand, they completely eliminate any problems associated with ice formation.

1.1.3 Tunnel regulations

Ice that forms in a tunnel can generate a pressure or a load on the tunnel constructions and installations. According to rules and regulations issued by the Swedish Road Administration (Vägverket), tunnel constructions and installations must be designed to withstand an ice load when there is a risk of freezing. The value of the ice load is 3 kN/m² on the assumption that it is a free load that acts perpendicularly to the construction, and this value includes both ice pressure and drop load from ice (Vägverket, 2004). The load originates from the requirements in Håndbok 163 published by the Norwegian Public Roads Administration (Statens vegvesen, 1995). The Norwegian National Rail Administration (Jernbaneverket) employs a “general payload” of 3 kN/m² in order to deliberately raise the capacity of these constructions to handle ice loads, drop loads and special conditions arising from pressure and suction loads from traffic (Jernbaneverket, 2004).

Choosing a value for the ice load can be wearisome, since the magnitudes of ice load and ice pressure depend on factors such as access to water, the rigidity of the adjacent material and temperature conditions. The Swedish Rail Administration has chosen not to design the tunnel constructions to take


account of ice load, but it lays down that a tunnel should be designed in such way as to avoid damage due to freezing (Banverket 2004).

1.2 Objective

This licentiate thesis is aimed to provide an understanding of the factors and processes that govern the growth of ice, the development of ice pressure and frost shattering of rock and shotcrete.

The factors governing the growth of ice in tunnels include the freezing rate, duration of freezing temperatures, temperature fluctuations (see section 3.3), rock mass properties, rigidity of the adjacent material (see section 3.2) and access to water (see section 3.3.2). All these factors effect the processes in progress, such as the manner in which the ice freezes and how the ice pressure develops.

The focus is on the problem of rock and shotcrete degradation due to ice pressure and frost shattering, which can occur as a result of material weathering processes or poor adhesion between the rock and the shotcrete. Poor adhesion on its own does not cause a degradation problem. However, voids may form as a result of the poor adhesion in the interface between rock and shotcrete, which, when filled with water, can lead to the development of ice pressure at freezing temperatures. Furthermore the ice exerts pressure on the interface, causing cracking and degradation of the shotcrete.

1.3 Scope

The first part of the study consists of a summary of the comprehensive literature review, which brings together experience and information relating to ice formation and the manner in which ice pressure affects fault zones, cracks and the shotcrete/rock interface. This review is followed by a discussion, after which a hypothesis is presented on the effect of ice pressure on the shotcrete/rock interface. The following chapters discuss and present the results of field investigations and laboratory tests, while the concluding chapter presents the results of the licentiate project in its entirety.
2 PROBLEM STATEMENT

2.1 General

This licentiate thesis and the initial literature review shows that water in combination with freezing temperatures is one of the major causes of degradation of rock and shotcrete in tunnels. Ice formation in tunnels is a problem of its own, but the process of frost action is more problematic. Frost action in rock and shotcrete resemble that in soil, and as in soil, water in rock tends to migrate to the frozen fringe. This migration may cause frost shattering in the rock and shotcrete and can result in fall-outs when the material undergoes degradation by the weathering processes.

These fall-outs often occur in tunnel sections where there are problems of water leakage. Hence, a probable scenario is that in cracks near the tunnel contour or in the interface between the rock and shotcrete, the water subsequently freezes and expands. This process can produce a high pressure which can cause pieces of rock to break lose from the tunnel wall and roof and also lead to shotcrete cracking, which will then diminish its load-bearing capacity.

Ice pressure is a complex problem and our present level of knowledge of its magnitude and effect between the rock and shotcrete is inadequate. Hence, further research is needed to achieve a better understanding of the processes of ice growth.

2.2 Hypothesis

The hypothesis behind the rock and shotcrete fall-outs in Swedish 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) the tendency of water to migrate to the frozen fringe.
and cause ice growth. If the ice pressure exceeds the tensile strength of the material or the adhesive strength between rock and shotcrete, the material will yield to this pressure and cracks will appear.

If there is adequate adhesion between rock and shotcrete, degradation due to frost shattering should not present a problem, but if it is poor, even on merely a small area around a rock crack opening, a small void will form between the rock and the shotcrete. Here water can accumulate and ice will develop at freezing temperatures, thus exerting a pressure on the interface and causing the shotcrete to crack and degrade.

2.3 Methodology

In this licentiate project, laboratory studies comprising freeze-thaw tests of shotcrete and rock samples have been performed with a view to verifying this hypothesis and to examining how water migration affects ice growth in the interface between rock and shotcrete. These freeze-thaw tests were performed both with, and without, access to water during freezing, so as to determine the difference in degradation under varying water conditions. In order to note the effect of a small area of poor adhesion, a number of the shotcrete/rock samples subjected to freeze-thaw tests were prepared with minor areas with no or poor adhesion before shotcreting.

In order to establish whether the freeze-thaw tests caused degradation, acoustic emission measurements were taken during the tests and adhesive strength between shotcrete and rock was subsequently tested. The hypothesis stated that the results of adhesion tests would reveal whether the ice pressure causes greater damage to the shotcrete/rock sample which had access to water during freezing, than to one that did not.

Besides the freeze-thaw tests, two separate research projects with connection to freezing temperatures and its consequences in tunnels were carried out. The first project aimed to identify problems of leakage and ice formation in tunnels (Andrén, 2008a), while the second project was intended to measure temperature flow and penetration depth of the freezing zone in tunnels (Andrén, 2008b and 2008c).
2.4 Limitation

Degradation of rock material is caused by different weathering processes and weathering is the decomposition of geological materials through mechanical or chemical processes. This licentiate thesis is limited to focus solely on mechanical weathering as a result of frost shattering and ice pressure.
3 LITERATURE REVIEW

In order to better understand the degradation processes of rock and shotcrete that is caused by frost shattering, a comprehensive literature review was performed, which is summarised in chapter 3 (Andrén, 2006).

3.1 Rock and shotcrete degradation

3.1.1 Weathering

Weathering refers to the decomposition of geological materials through mechanical or chemical processes in which exposed materials at the earth’s surface undergo constant alteration by water, air, temperature fluctuations and other environmental factors. Mechanical weathering includes processes that physically break down material into smaller pieces without changing the chemical composition, i.e., the minerals remain unchanged. Chemical weathering refers to decomposition of materials by exposure to water or to atmospheric gases, whereby some of the original minerals are chemically transformed into different ones (Plummer and McGeary, 1996).

This licentiate thesis will focus solely on mechanical weathering as a result of frost shattering and ice pressure.

3.1.2 Adhesion

Poor adhesion can occur between rock and shotcrete, which can cause the latter to fall out. However, it is not clear whether these fall-outs result from poor adhesion, which occurs as soon as the shotcrete is applied, or whether poor adhesion is an effect of degradation.

The problems arising from poor adhesion between rock and shotcrete may be due to the following:
- rock type and mineral composition
- excess leakage from the rock during shotcreting
- low adhesive strength due to poor rock conditions
- uneven rock contour – which makes it difficult to achieve good contact with the rock surface.

Hahn (1983) described two tests undertaken to observe how the adhesion of shotcrete was affected by moisture of the rock surface. Hsu and Slate (1964) showed that the difference between adhesions on dry and wet ballast, respectively, was negligible at only 3%. Their tests were undertaken on ballast that had been (i) in a water bath for 24 hours and (ii) heated in an heating oven. Karlsson (1980) demonstrated by means of field tests that shotcrete adhesion is unaffected whether the rock surface is dry or wet when shotcreting.

However, in areas with complex water conditions, for example, where there are relatively open joints and fissures, the shotcrete may cause a stability problem itself. It is important to obtain good adhesion between shotcrete and rock. Otherwise the latter, when exposed to frost, may be subjected to an ice pressure that develops in the interface between them, thus causing the shotcrete to crack. Consequently, load-bearing capacity is lowered, and fall-outs of shotcrete fragments may occur (Selmer-Olsen and Broch, 1976).

Further, in order to prevent poor adhesion it is important that the rock surface is thoroughly cleaned prior to shotcreting, since the shotcrete cover is entirely dependent on absolute adhesion between it and the rock (Selmer-Olsen and Broch, 1976). Malmgren (2001) showed that when the rock surface was cleaned by water-jet scaling (at a pressure of 22 MPa) instead of normal treatment (a water pressure of 0.7 MPa), adhesive strength increased from 0.21 MPa to 0.61 MPa.

Hahn (1983) conducted tensile tests on 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 both on the material and the location of the highest stress concentration. The most frequent breakage was an adhesion breakage, except in the case of porous sandstone and limestone, where breakage occurred in the rock and the shotcrete, respectively. The results showed that adhesive strength was dependent on the roughness of the rock
surface. As shown in Figure 3.1, a rough surface gives a higher value for adhesive strength than a smooth one. Hahn's result also showed that the type of feldspar in the granite samples affected adhesive strength and his tests proved that both the mineral composition and the roughness of the rock surface affected the adhesive strength measured, from which he concluded that the mineral composition had a greater effect than roughness.

Figure 3.1  Adhesive strength of different rock types and rock surfaces (Hahn, 1983 – from Malmgren, 2001)
3.1.3 Case studies of fall-outs

There has been an increase in the incidence of rock and shotcrete fall-outs in recent years in Swedish railway tunnels. Some installations have been damaged (Andrén, 2008a) and an ice fall-out seriously damaged a passenger train passing through the Glödberget tunnel (Nilsson, 2008). This section collates a number of cases of rock and shotcrete fall-outs, several of which occurred even though tunnels were inspected as prescribed in the regulations laid down by the Swedish Rail Administration. This indicates that the problem of ice is more complicated than previously assumed and, furthermore, that the regulations may need to be revised accordingly.

According to these regulations, safety and maintenance inspection of the tunnels is mandatory. A safety inspection should be undertaken twice a year and in addition to which maintenance inspection is to be undertaken at regular intervals that are based on the needs of each specific tunnel.

The safety inspection (Banverket, 2005a) includes:
- checking for any rock fall-outs
- checking for any risk of rock fall-outs
- checking for presence of damage, cracks or other signs of movements in the shotcrete.

The maintenance inspection (Banverket, 2005b) includes:
- checking to see if there is any need for rock mechanical measurement or scaling
- checking damage to reinforcement due to degradation processes such as frost shattering, rust shattering, leaching, corrosion, deposition, among others.

Rock fall-outs

The Bergträsk tunnel

In November 2005 a rock fall-out was reported from the Bergträsk tunnel in Älvsbyn, when two blocks, each with a diameter of 1 m, came away from the tunnel wall. The rock surface above the fall-outs was fractured and the section was reinforced with rock bolts. The tunnel had been scaled as recently as 2002 (Banverket BRN, 2005). The cause of the fall-outs was not clear, but
since the ground freezing period had started, frost action seems to be the most obvious reason 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\times0.5 m) had fallen off the roof of the tunnel, where upon inspection, it was noted that the rock surface consisted of open oxidised 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, here too, 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 (diameters from 0.2 to 0.4 m) were reported from the Herrljunga tunnel near Uddevalla. The blocks had disintegrated crack surfaces, some of which 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).

Shotcrete fall-outs
Shotcrete usually falls out as sheets, some of which are covered with ice on the interface side, i.e., the side that has been attached to the tunnel roof or wall. Furthermore, the exposed rock surface is often also covered by ice, which implies that water accumulates in the interface between rock and shotcrete. According to the reports summarised below, these fall-outs occurred despite the inspections undertaken in recent years, which, at the time this was done, failed to reveal any indication of faults in the shotcrete. The question is whether the shotcrete can degrade to such a considerable degree that it can fall out in just a year or two, or if the inspection missed any defects.

The Gårda tunnel
The Gårda tunnel was built in the late 1960s and a station was added at the start of 1990. In January 2003 there was a fall-out of shotcrete and a thin sheet of rock with an area of 1 m² in the tunnel, and an additional 2 to 3 m² of loose material was removed by scaling the damaged area. Upon inspecting the entire tunnel, further areas of reduced surface stability were detected. The inspectors found that the shotcrete had degraded and that this was a continuing and accelerating process (Bergab, 2003a).
The Nuolja tunnel

The Nuolja tunnel was built in 1990 and in January 2003 a 2 m² area of shotcrete fell onto the track and the overhead wire was slightly damaged. In 1995 there had been another fall-out of 1 m² of shotcrete in the same area. In the 2003 fall-out an ice layer had formed on the interface side and one was noted in the tunnel roof in the interface between the shotcrete and the rock. The tunnel had been scaled in July 2001 and according to the report produced, the tunnel was in a good condition at the time and the shotcrete was intact throughout the tunnels entire length (Banverket BRN, 2003a).

The Bergträsk tunnel

The Bergträsk tunnel was built in 1982 and a fall-out of shotcrete with an area of 1 to 2 m² was detected during a routine check in September 2003. The tunnel had been inspected in the spring of 2002 and was scaled the same year. The section where the fall-out took place showed several spots of water leakage and according to the Swedish Rail Administration report (Banverket BRN, 2003b) it was caused by frost shattering.

Ice fall-outs

In 2008 the fall-out of an ice pillar in the Glödberget tunnel seriously damaged both the locomotive and the coaches of a passenger train. The coachwork was damaged and several windows were crushed by the ice pillar bouncing numerous times between the train and the tunnel wall. Fortunately all the passenger compartments were located on the opposite side of the train and thus all the pieces of broken glass fell into the corridor (Nilsson, 2008).

3.2 Thermal and thermomechanical properties

Some knowledge of the thermal and thermomechanical properties of rock and shotcrete is required in order to understand the degradation process in these materials.

3.2.1 Introduction

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

The process of excavating a tunnel in a cold region destroys the original stable thermodynamic condition of the rock, which is replaced by a new thermodynamic system. During the winter the rock walls and roof of the tunnel are exposed to freezing temperatures and as the temperature drops, water in fissures and pores freezes and volumetric expansion of the water and ice occurs. This expansion is restrained by the tunnel lining or reinforcement, such as shotcrete or bolts, thereby resulting in pressure on the reinforcement. The ice pressure acting on the tunnel reinforcement may result in cracking and flaking and can pose a serious threat to tunnel stability (Lai et al., 2000).

3.2.2 Properties of water and ice

Ice is an important factor that, when formed in pores and fissures, affects several rock properties. Although there is a great difference between rock and ice, as ice deformation is time-dependent and can be both elastic and viscoplastic. Dahlström (1992) compiled the results of several experiments on the properties of ice, which show that the magnitudes of Young's modulus, Poisson's ratio and compressive and tensile strength increase with decreasing temperature. For example, compressive strength of water saturated porous rocks may 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 during the water-ice phase transition as during freezing from 0 °C down to -22 °C, ice expands by 13.5 %. Theoretically, if constrained in a perfectly rigid body, 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 3.3.4 (Tharp, 1987). At temperatures below this, the pressure decreases because the ice begins to contract (French, 1996).

3.2.3 Latent heat

To get water to transform into ice, heat must be removed without any change in its temperature. This is called latent heat, L (J/kg), and has a magnitude of
334 kJ/kg. In the same way, heat is needed to melt ice at 0 °C (Fransson, 1995).

While considerable heat is either released or consumed during phase transition, latent heat has a great effect on frost action in soil and rock. The frost depth in a material of low water content is much greater than in one where the content is higher. In the latter case, a larger amount of heat has to been removed to place the material in a frozen condition than in the case of a material of lower water content (Knutsson, 1981).

### 3.2.4 Thermal properties

The thermal properties of a material change according to temperature and depend on its water content, which is because water and ice have very different properties.

#### Thermal conductivity

The thermal conductivity, $\lambda$ (W/m·K), of a material can be described as its ability to conduct heat and it varies with temperature, porosity and water content, among other factors.

High porosity normally implies low thermal conductivity. If a porous material contains water, its conductivity will change when the temperature falls and the water turns into ice. This is because the thermal conductivity of ice is 2.25 W/m·K while that of water at ±0 °C is 0.56 W/m·K (Knutsson, 1981).

The bedrock in Sweden is dominated by gneiss and granite, which are highly quartzose rock types. An average value for the thermal conductivity of these rocks is about 3.5 W/m·K (Dahlström, 1992). The typical thermal conductivity of shotcrete may vary depending on material content, but an average value is 1.4 W/m·K (Schwarz, 2004).

#### Specific heat

Specific heat, $c_p$ (J/kg·K), is the amount of heat needed to raise the temperature by one degree K for each kilogram of the material and it is temperature dependent, decreasing with decreasing temperature. In a composite material such as rock, the overall specific heat comprises that of the individual components. The specific heat of water is 4200 J/kg·K and of
ice, 2040 J/kg K at ± 0 °C, while that of granite varies in the literature although an average value is about 730 J/kg K (Dahlström, 1992).

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

\[
\kappa = \frac{\lambda}{c_p \cdot \rho}
\]

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

Specific heat decreases with decreasing temperature while thermal conductivity increases with decreasing temperature. Dahlström (1992) pointed out that the thermal diffusivity of granitic and gneissic rocks increases with decreasing temperature, while concrete behaves in a similar way as rock material.

### 3.2.5 Thermomechanical properties

**Strength**
Nicholson and Nicholson (2000) analysed freeze-thaw, wetting and drying, salt weathering and frost durability tests undertaken on the same group of sedimentary rocks. Their 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 effect of environmental conditions. These also suggested that rock strength provides the basic resistance to the mechanical weathering of rocks, although this is overridden by the presence of pre-existing flaws.

The tensile strength of hard, intact rocks may be of the order of 10 MPa although it decreases significantly in jointed bedrock of the same lithology (e.g. Matsuoka, 1990b). The mechanical strength of rock depends on its porosity and the intrinsic low mechanical strength of weak rocks usually equates to high porosity (Winkler, 1994) and hence greater frost susceptibility (McGreevy, 1982).
Uniaxial compressive strength
The effect of temperature on uniaxial compressive strength depends on rock mass, texture, mineral composition, porosity and water content. The porosity and water content have at normal temperature a reducing effect on compressive strength, but at freezing temperatures these properties behave differently. Uniaxial compressive strength increases with decreasing temperature down to -120 °C and is then relative constant. For highly porous rocks such as sandstone and limestone, this increase in strength is greater than for those of lower porosity, i.e., granite. The effect of porosity and water content is highly significant and causes a considerable increase in 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 and the behaviour of tensile strength resembles compressive strength during freezing, and the increase in strength is of the same magnitude (Dahlström, 1992).

Young’s modulus
Porosity and water content are of considerable significance for Young’s modulus when the temperature decreases. Dahlström (1992) summarised tests that showed that Young’s modulus remain relatively constant for air-dry rock when the temperature decreases, but increases with decreasing temperature for saturated rock. For saturated granite Young’s modulus increases down to -120 °C because absorbed water continues to solidify down to these temperatures.

Poisson’s ratio
Poisson’s ratio does not exhibit the same behaviour as the above parameters do when the temperature changes. Various 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).
**Thermal expansion and contraction**

When the temperature of a body alters, its length and volume will change (Glamheden, 2001). The definitions for this thermal linear expansion or contraction, \( \alpha \), is as follows:

\[
\alpha = \frac{L_T - L_0}{L_0 (T - T_{00})} 
\]

where;

- \( L_0 \) = length at temperature \( T_{00} \)
- \( L_T \) = length at temperature \( T_T \)
- \( T_{00} \) = temperature at the start
- \( T_T \) = temperature after alteration

Dahlström (1992) demonstrated the difference in behaviour in terms of thermal expansion between rock and soil, by means of an experiment undertaken by Boulanger and Luyten (1983). The difference among clay, gneiss and limestone is shown in Figure 3.2. Clay has a higher water content than rock and exhibits expansion down to a temperature of -50 °C, when a small contraction starts. Limestone starts to expand, and then contract, while gneiss shows only contraction.

![Figure 3.2](image)

**Figure 3.2** Schematic contraction and/or expansion of clay, limestone and gneiss with decreasing temperature (After Boulanger and Luyten, 1983 – from Dahlström, 1992)

In cracks, the joint filling may consist of clay minerals and clay, with its high water content, can be problematic during freezing as it expands (see Figure
3.2) and exerts a pressure at the adjacent rock surfaces. This problem can also be encountered in fault zones, where the rock material is often weathered and has transformed into clay minerals.

The thermal contraction of ice at -20 °C is $48 \times 10^{-6}/°C$ (Powell, 1958 – from Glamheden, 2001) compared to that of crystalline rock which is about $5 \times 10^{-6}/°C$ (Dahlström, 1992). Hence, the ice contracts more than the rock does, when the temperature falls below -22 °C, which means that, when the ice has expanded as much as possible (13.5 % at -22 °C, see section 3.2.2) it starts to contract and thus the pressure at the rock surfaces decreases.

### 3.3 Frost phenomena

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

A tunnel exposed to freezing temperatures does not exhibit any problems in those sections where the rock mass is intact and of good quality, while the opposite applies where rock conditions are bad and there is water leakage. Frost action in crushed or heavily weathered rock resembles that 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 despite of extensive literature on the subject in general, there appears to be no quantitative theoretical analysis of the basic process.

#### 3.3.1 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 cause 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 rock temperatures, moisture content and
disintegration rate as well as the role of ice segregation as a weathering mechanism and the effect of freeze-thaw cycles (French, 1996).

Rock weathering as a result of freezing is of primary interest for this licentiate thesis. Among the mechanical weathering processes in rock, it is frost action that 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 applicable to frost action in soil.

Frost action in soil is caused by two processes; (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 3.3). 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 is being pressed out of the soil by ice formation. The predominant part of the frost heaving come from the growth of ice lenses and the ability of soils to grow them depends on grain-size distribution, permeability, specific surface, mineral content and capillarity, among other factors. Ice lenses are generally formed perpendicularly to the direction of the thermal flow, while their thickness is governed primarily by access to water. In soils of low permeability, the water cannot migrate to the frozen fringe at the rate required for frost heaving and thus it is of limited occurrence in this kind of soil (Knutsson, 1981).

Figure 3.3 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 until an alternative mechanism was first presented by Everett (1961). He suggested that capillary suction caused water migration towards the freezing front, and when this water froze, ice pressure increased which 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) and these experiments showed that water migration could be responsible for frost shattering. Matsuoka’s laboratory tests (1990a) on the effect of access to water during freezing, led to a hypothesis that frost shattering was controlled by a combination of the two processes, volumetric expansion and water migration, which also was suggested by Tharp (1987).

The leakage into the tunnel can be affected by the frost action in rock. For instance, frost action can change the hydraulic conductivity of the clay-filled cracks in the rock mass. When soils consisting of clay freeze, a restructuring of the clay particles occurs, which is known as freeze consolidation, see Figure 3.4. During each freezing period the soil becomes more and more consolidated, causing settlement and an 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 freezing temperature of the cold air in the tunnel, which can cause problems in newly excavated tunnels. A crack that appeared to be impermeable at the time the tunnel was excavated may start to leak after a single freezing period, due to the restructuring of the clay particles (Andrén, 2008a). In contrast to the above information, other cracks may stop leaking, as a result of natural clogging by the precipitation of calcium oxide and by precipitations containing iron (Statens vegvesen, 2004).

Figure 3.4 Restructuring of clay particles during freezing (after Chamberlain and Gow, 1978 – from Johansson, 2005)
3.3.2 Access to water

The ice formation and ice pressure, which can occur in a crack, are affected by access to water and if this is the case when a crack freezes, the thickness of the ice layer may increase due to water migration. However, if there is no access to water, only the existing water in the crack will expand 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).

Water migration

Frost heaving in soils occurs due to the migration of water from unfrozen parts of the soil to the frozen parts, with the ice usually being concentrated as ice lenses perpendicular to the direction of the thermal flow. The growth of ice lenses is the predominant part of frost heaving. In rock the water can migrate in a similar manner inside a crack, where it does so in the thin water film at the interface between ice and rock. This allows expansion of the ice layer, which exerts pressure on the rock surfaces through the water film and 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 sub-zero temperature there are ice crystals as well as unfrozen water in the soil, both adsorbed and free water (water that has not been absorbed by mineral particles). Adsorbed water has a lower energy level compared to free water, and the adsorbed water requires lower temperatures in order to freeze. When the temperature decreases and all the free water has been frozen, water with a lower energy level starts to freeze. The proportion of unfrozen water is reduced (see Figure 3.5) and causes the thinning of the water film that separates the ice from the solid particles (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 the unfrozen water at different energy levels due
to temperature variations. Water always endeavours to achieve the lowest energy possible and this causes unfrozen water to migrate from warmer to colder zones, as the energy level there is lower (Knutsson, 1999).

Figure 3.5 Energy level of water in proportion to the distance from the mineral particle (modified after Knutsson, 1981)

Experimental work has shown that at subfreezing temperatures a considerable amount of water remains unfrozen, not only in soil, but also in rock; and that unfrozen water tends to migrate towards a freezing centre in rock as well as in soil (Walder and Hallet, 1985). Walder and Hallet (1985) developed a theoretical model of ice grow within cracks (see section 3.4.4) on 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) this water migration creates an expansion of the ice layer, which can generate a pressure at the crack surfaces.

Matsuoka (1990a) used 47 rock samples in freeze-thaw experiments to demonstrate that water migration plays an important role in the frost shattering of rock (see also section 3.4.1 Access to water) and these samples were exposed to both an open system\(^4\) and a closed system\(^5\). The water

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\(^4\) open system – access to water during the freezing period

\(^5\) closed system – no access to water during the freezing period
migration in the open system led to an increase in ice volume, which caused
great damage to the rock, while in the closed system the resultant damage was
rather small, because only pre-existing water became ice. Specimen
deterioration was detected through a reduction in longitudinal wave velocity,
termed P-wave velocity or $V_p$, which is commonly used in laboratory tests to
prove rock deterioration. If the longitudinal wave velocity in the rock sample
is reduced, then it has been subjected to deterioration.

Figure 3.6 shows reduction of $V_p$ in tuff (a volcanic sedimentary rock) and
sandstone specimens in the course of 50 freeze-thaw cycles. The solid dots are
the results for the reduction in $V_p$ during freezing in a closed system, while the
unfilled dots represent an open system. By comparing the reduction in $V_p$ for
the same specimen but with varying access to water, Matsuoka showed that
an open system is much more susceptible to breakdown through 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).

![Figure 3.6 Reduction in longitudinal wave velocity $V_p$ of rocks in the course of 50 freeze-thaw cycles; comparisons between open and closed systems (Matsuoka, 1990a)](image_url)
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 a redistribution of pore water. Chen et al. (2004) conducted freeze-thaw tests on rock specimens prepared from welded tuff to determine the deterioration of the rock in relation to saturation (see also section 3.4.1 Saturation). Specimens were examined by means of the changes in 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 it exceeded 70 %, the rock was significantly damaged. Hence, the critical degree of saturation for this particular rock was about 70 % and it should be noted that this changes according to the rock material (Chen et al., 2004).

The degree of saturation may be important for ice lens growth in concrete since increases in the degree of saturation cause water absorption in air pores. When they 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 expansion, which can cause damage to the concrete construction (Fridh, 2005).

McGreevy and Whale (1985) found that in intact rock 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 through the swelling of clay minerals (McGreevy, 1982). Swedish rocks often have a high content of mica, which has a higher tendency to absorb moisture than other minerals.

Between 1990 and 1991 Matsuoka reported from field observations in the Japanese Alps, Svalbard and Antarctica that the annual frost shattering rate is not only a function of annual freeze-thaw frequency, but also of the degree of water saturation and bedrock tensile strength. The most effective environmental factor controlling the shattering rate was moisture content (Matsuoka, 1990b and 1991). If the water migration is prevented during freezing and the conditions are favoured by short-term, rapid freezing or the lack of an external water source, it is the initial degree of saturation that control rock expansion (Matsuoka, 1990a). Such a situation occurs with rocks
at some distance from open water (e.g. a stream, lake or sea) or far above the subsurface water table (Matsuoka, 2001b).

Konishchev and Rogov (1993) reported from an experiment from which they learned 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 on 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, ranges from a high of 3.5 mm in marl to a low of 30 to 50 \(10^{-5}\) mm in sandstone and porphyry. For dry samples of these rocks the disintegration layer was 600 \(10^{-5}\) mm for marl and 6 to 11 \(10^{-5}\) mm in sandstone and porphyry. Konishchev and Rogov also reported that in the Ukraine, the frost weathering rate for 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 0.1 to 0.01 mm/year.

### 3.3.3 Freezing

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

**Freezing intensity**

Freezing intensity is represented by the minimum sub-zero temperature that the rock surface experiences during a freeze-thaw cycle. It provides a basis for counting effective freeze-thaw cycles occurring in the bedrock (Matsuoka, 2001b).

**Freezing rate**

The freezing rate (decrease in temperature per unit of time, °C/h) primarily controls the magnitude of the expansion of rocks, and its effect varies in conjunction with moisture content (Matsuoka, 2001b). The freezing rate affects water accessibility and thus the rate of ice growth and the magnitude of the ice pressure generated.

Frost damage can occur in initially unsaturated rocks when slow freezing drives water migration from surrounding rock or an external moisture source. However, when a rock undergoes rapid freezing, frost damage can occur only
when the degree of saturation is high (>80% in this particular case) 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 damage. By contrast, long-term slow freezing which permits water migration can result in greater frost damages (Matsuoka, 2001a).

Walder and Hallet (1985) developed a theoretical model (see section 3.4.4) for the breakdown of rock by the growth of ice within cracks. They found that in open systems, crack-growth rates during continuous cooling were generally greatest at the slow freezing rate of less than 0.1-0.5 °C/h. At a more rapid freezing rate, the influx of water into growing cracks was significantly inhibited. For example, during cooling from -1 °C to -25 °C, a crack with a 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, which occurs when water become confined due to rapid freezing; thus in this case rapid freezing is more harmful than slow freezing. However, on the other hand, the other main destruction mechanism in shotcrete is ice lens growth, which is worse when the freezing rate is slow (Fridh, 2005).

Observations of Swedish railway tunnels have shown that water can continue to leak for a long time if the freezing rate is slow. The growth of ice formations depends on access to water. It is known that icicles usually stop growing when the temperature drops rapidly (rapid freezing rate), which is because the crack, which supplies the icicle with water, freezes (Andrén, 2008a).

**Duration and frost index**

The duration and intensity of the temperature drop below 0 °C affects the rate and degree of the freezing of soil and rock (French, 1996). The duration of freezing affects both frost depth (Matsuoka, 2001b) and internal damage to both rock and shotcrete. The latter occurs because longer duration at a constant low temperature allows the water migration mechanism more time to effectively produce an ice pressure (Fridh, 2005).

Temperature and duration vary among the different climate zones of our country, which leads to a different frost depth within each respective zone. Climate data provide every area of Sweden with a specific “frost index” based on the number of days of freezing temperatures. Calculation of the days when
it is either freezing or thawing in any specific area is based upon cumulative
total air temperatures above or below zero during any one yea, using data
derived from mean daily air temperatures (French, 1996).

Observations of Swedish railway tunnels have shown that changes in the
freezing periods and their duration have a major effect on the number and
location of leakage spots in a tunnel. If the freezing period is of long duration,
some of the leakage spots become frozen and no ice formations will grow. If
the leakage is subjected instead to shorter periods of freezing and thawing,
the crack will never freeze and water will continue to leak, resulting in
growing icicles (Andrén, 2008a).

**Freeze-thaw cycles**

Discussions of weathering in cold environments generally centre on
mechanical processes such as frost wedging, rock shattering, and the freeze-
thaw mechanism in particularly. 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 temperature transition across
0 °C as the basis for determining the number of freeze-thaw cycles. However,
Hall (2004) proved that the freezing of rock water did actually take place but
found that the temperature at which freezing occurred, varied considerably
throughout the year (see also section 3.4.2 Freeze-thaw events).

In his 1990 laboratory tests (see also section 3.4.1 Access to water) Matsuoka
used 47 saturated rock samples in freeze-thaw experiments. Specimen
deterioration was detected through a reduction in longitudinal wave velocity;
$V_p$. Figure 3.7 shows the reduction in the longitudinal wave velocity of
sandstone specimens in the course of 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 of a lower porosity (Matsuoka, 1990a).
30

Figure 3.7  Reduction in the longitudinal wave velocity $V_p$ of sandstone specimens in the course of 300 freeze-thaw cycles (Matsuoka, 1990a)

**Temperature range**

In Table 3.1 Matsuoka (2001b) summarises temperature ranges for effective frost weathering from different reports. A degree of care should be exercised when comparing the listed values, as methodology varies among different researchers. A comment on the table states that laboratory data are not available for rocks of low porosity, and that 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 3.1 high porosity rocks begin to crack at 0 to -1 °C and terminate at about -5 °C, while in medium porosity rocks the cracking temperature is between -3 °C and -6 °C 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 experimentally validated. The experiments in Table 3.1 show that cracking progresses most rapidly just below the upper temperature limit $T_u$, and decelerate toward the lower temperature limit $T_l$ (Matsuoka, 2001b).
The apparent relationship between porosity and cracking temperature is due to the following conditions. Firstly, the freezing point of pore water is generally lowered as porosity decreases, depending on the number of smaller pores. Secondly, ice segregation is optimised by the balance between the suction force, which increases with falling temperatures, and hydraulic conductivity, which decreases with falling temperatures (e.g. Williams and Smith, 1989). Furthermore, rock strength generally increases with decreasing porosity, so that a greater suction force (hence, a lower temperature) is required for low porosity rocks to crack (Matsuoka, 2001b).

Several investigators emphasise the role of microfractures in the bedrock as the method in which waters penetrate it and subsequently freezes to cause rock disintegration. However, microfracturing may not require freeze-thaw cycles or even falling temperatures but a constant sub-zero temperature instead. Walder and Hallet (1985, 1986) emphasised that a high incidence of freeze-thaw cycles is not necessary for crack propagation, which can occur at temperatures between -4 °C and -15 °C (French, 1996).

### 3.3.4 Ice pressure

Measuring the magnitude of the 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, 1996). Expansion of

### Table 3.1 Temperature range for effective frost weathering, defined by the upper limit (TH) and lower limit (TL) (Matsuoka, 2001b)

<table>
<thead>
<tr>
<th>Lithology</th>
<th>Porosity (%)</th>
<th>TH (°C)</th>
<th>TL (°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>Walder 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>Meller (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>Meller (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>Olid tuff</td>
<td>38</td>
<td>-1.4</td>
<td>-5</td>
<td>Ice lens formation</td>
<td>Open</td>
<td>Akagawa and Fukushima (1994)</td>
</tr>
<tr>
<td>Breccia 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>Breccia chalk</td>
<td>47</td>
<td>-0.2</td>
<td>-2</td>
<td>Length change</td>
<td>Open</td>
<td>Murton et al. (2000)</td>
</tr>
</tbody>
</table>
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 decreases (Tharp, 1987).

Ice pressure increases almost linearly from 0 MPa at 0 °C to a maximum of 207 MPa at -22 °C (see also section 3.2.2), 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 it. Firstly, the water or ice must be contained within a closed system for high pressures to develop, which 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 are not strong enough to withstand such extreme pressures, especially since it is a tensile force rather than a compressive one, 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 derived from different experiments varies a great deal in the literature. Some of the test results are summarised 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 able to cause crack growth in practically all rocks near the Earth’s surface (Walder and Hallet, 1985).

- As early as 1960 Battle (1960) examined the relationship between the geometry of water-filled cracks, the temperature and the pressure exerted during freezing in a granite crack. Theoretically, the pressure established at the bottom of a water-filled crack, which freezes from the 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 such 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 objective was to measure the resulting stresses at an artificial crack, a slot in a Lucite block. The maximum pressure measured in this experiment was 1.1 MPa, caused by volumetric expansion of ice and the confined water in the slot (see also section 3.4.1 Ice pressure in a slot).

- Walder and Hallet (1985) developed a theoretical model for the breakdown of porous rock through the growth of ice within cracks (see also section 3.4.4). The maximum pressure, 12 MPa, was achieved when a crack with an initial radius of 5 mm freezes down to a temperature of \(-12 \, ^\circ C\).

3.4 Freezing tests

A number of interesting rock freezing experiments are compiled here, most of which are performed as freeze-thaw tests with or without access to water. Rock of low porosity is relatively resistant to frost shattering and therefore freezing tests are very seldom performed on this kind of rock. 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 the Swedish Rail Administration build their tunnels consists of hard, crystalline rock of low porosity, but this difference must be taken into consideration. Similarities to the results of weathering tests occur more often for sections of crushed or heavily weathered rock, where the conditions resemble those that apply to high porosity rock, in terms of rock strength and water conditions.
3.4.1 Laboratory tests

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

Ice pressure in a slot

Davidson and Nye (1985) performed a test whose the primary objective was to measure the resulting stresses on an artificial crack, a slot cut into a transparent material (Perspex or Lucite) see Figure 3.8. It was filled with water and then frozen from the top down, so as to exert a pressure on the slot wall, while the water at the bottom of the slot was confined. Crack propagation was not intended.

Figure 3.8 The slot cut into the Perspex (Davidson and Nye, 1985)

Figure 3.9 shows the progression of the ice front down the slot. The x-axis shows the time in hours and the y-axis, 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 cloudy ice from clear ice. This front (III) was formed parallel to the ice-water front, but some way behind it, and the cloudy area was caused by ice separating from the slot walls.
Figure 3.9 Progression of the ice front over time (Davidson and Nye, 1985)

Figure 3.10 shows both the positions of the ice-water and cloudy fronts (explained in Figure 3.10b), and also the stresses along a line close to the slot wall and parallel to it. The normal stress, $\sigma_x$, is shown by a solid line and the shear stress, $\tau_{xy}$, by a dotted line. Figure 3.10 shows that the highest stress occurs at the ice-water front and that the pressure decreases as the front passes. This, as previously explained, is caused by the ice separating from the slot walls in the cloudy area.
The maximum pressure measured in this experiment was 1.1 MPa (11 bar as shown in Figure 3.10f). This pressure is caused by the water confined within the slot. The contact area between the ice and the slot wall is of great importance, as if there is insufficient contact, the water pressure will decrease, 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 suggests 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 the 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 since these controls the opening of the crack (Davidson and Nye, 1985).

**Crack widening**

In 1995 Matsuoka conducted an experiment designed to simulate the frost shattering of a granite block with an artificial crack (Figure 3.11). A parallel-sided slot was cut into the granite block and a closed system crack was produced by sealing both ends of the slot with silicon rubber to prevent water escaping. The rock block was then 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 might have been lost through desiccation prior to freezing. Cooling was then undertaken from the upper surface only.

![Diagram of freezing expansion in a rock crack](image)

**Figure 3.11** Freezing expansion in a rock 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 3.12 arises from emission of the latent heat. After that, abrupt expansion occurred with the ice nucleation. Figure 3.12 shows that expansion was most rapid between 0 °C and -1 °C and decelerated below -1 °C.
Figure 3.12  a) Temporal variations in crack temperatures and crack expansion
b) Freezing expansion as a function of temperature at the top of the crack (Matsuoka, 1995)

As shown in Figure 3.12b 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 freezing rates, but his tests showed no relationship between maximum expansion and freezing rate. He explained that the relationship between the two seemed to have been nullified by other effects such as the slight difference in initial water level and the absorption by the flexible silicon rubber of part of the expansion (Matsuoka, 1995). Maximum expansion was about 0.5 %, which is much lower than that predicted from the phase change (9 %). Matsuoka explained this by suggesting that the rigid crack wall forced ice to extrude both upwards and sideways (Matsuoka, 2001b).

**Access to water**

Matsuoka (1990a) demonstrated through laboratory tests the important role of water migration in the frost shattering process in rock. He showed that expansion of the rock samples increased when they 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 $5 \times 5 \times 5$ cm samples were submerged to 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 at maximum and minimum room temperatures of $+20 \, ^\circ C$ and $-20 \, ^\circ C$ respectively. This temperature cycle is useful for facilitate rock breakdown and hence data collection, despite the greater range and shorter period as compared to the diurnal freeze-thaw cycled occurring in the field.

For a saturated specimen of tuff-b expansion (shown in Figure 3.13 as an increase of strain) occurred rapidly with decreasing temperature in a closed system. This expansion slowed down below $-5 \, ^\circ C$ and stopped at about $-10 \, ^\circ C$. As shown in Figure 3.13b the thawing contraction curve follows freezing expansion relatively well and both expansion and contraction occurred primarily between $0 \, ^\circ C$ and $-5 \, ^\circ C$, while most of the free water is freezing or thawing.

![Figure 3.13 Strain data for tuff-b: closed system, cooling rate of 6 °C/h (Matsuoka, 1990a)](image)

When Matsuoka tested the same rock in an open system instead, it showed similar behaviour except for the difference in total expansion (Figure 3.14). The result obtained from comparing these two conditions is that the maximum freezing strain $\varepsilon_{L_{\text{max}}}$ in an open system is $4.7 \times 10^{-3}$ and in a closed system, $2.8 \times 10^{-3}$. These results explain why a rock subjected to freeze-thaw action is more rapidly broken down in an open system.
Figure 3.14  Strain data for tuff-b: open system, cooling rate of 6 °C/h (Matsuoka, 1990a)

Figure 3.15 shows comparisons between the closed and the open systems for some rock specimens of differing porosity. The rock specimen of the lowest porosity shows the smallest difference in expansion between the open and closed systems.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Porosity</th>
<th>$\varepsilon_{L_{\text{max}}}^{\text{open}}$</th>
<th>$\varepsilon_{L_{\text{max}}}^{\text{closed}}$</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>

Porosity 39.3 %  
$\varepsilon_{L_{\text{max}}} = 4.7 \times 10^{-3}$

Figure 3.15  Strain curve for rocks against rock temperatures: comparisons between closed and open systems (Matsuoka, 1990a)
Saturation

To understand the mechanisms of rock breakdown in cold regions through frost shattering and weathering, the effect 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 varying from 0% to 95%. They were frozen in a temperature-controlled chamber, where the temperature varied from +5 to -18°C, before being thawed in distilled water at 14°C. The freeze-thaw test was conducted for only one cycle, which included 2 hours of freezing and 1.5 hour of thawing. The specimens were then dried at room temperature before testing. Their deterioration was examined through changes in the uniaxial compressive strength, P-wave velocity, porosity and the appearance of a crack pattern.

Figure 3.16 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%. However, when this exceeded 70%, it was lowered after one freeze-thaw cycle and considerably so, if the initial degree of saturation was over 80% (Chen et al., 2004).

![Figure 3.16](image)

Figure 3.16 Relationship between the initial degree of saturation and the uniaxial compressive strength after one freeze-thaw cycle (Chen et al., 2004)

Figure 3.17 shows the relationship between the initial degree of saturation and the P-wave velocity of the specimens, which demonstrate a reduced P-wave velocity when subjected to deterioration. When the initial degree of saturation
was below 60 %, the P-wave velocity was not affected by the freeze-thaw cycle. When it was above 70 %, the P-wave velocity decreased with the increasing initial degree of saturation (Chen et al., 2004).

![Figure 3.17 Relationship between the initial degree of saturation and the P-wave velocity after one freeze-thaw cycle (Chen et al., 2004)](image)

The porosity of a rock specimen can be determined by the water saturation method, where a specimen is immersed in distilled water under vacuum conditions for 72 hours. The saturated specimen was weighed, then dried in an oven at 105 °C for 48 hours and re-weighed. Figure 3.18 shows the relationship between the initial degree of saturation and the change in porosity. The change in the porosity 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 it was over 70 % the porosity increased by 2 % to 6.3 % after one freeze-thaw cycle (Chen et al., 2004).
Figure 3.18  Relationship between the initial degree of saturation and the change in porosity after one freeze-thaw cycle (Chen et al., 2004)

Figure 3.20 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 3.19). When the initial degree of saturation was 78 % or lower, no cracks were observed, but when it was between 80 % and 90 %, a linear pattern of cracks formed. When the initial degree of saturation was higher than 90 %, the crack pattern became radial and the maximum crack width increased with an increasing initial degree of saturation, reaching 1.0 mm at an initial degree of saturation of 95 % (Chen et al., 2004).

Figure 3.19  Cracks on specimen surfaces after one freeze-thaw cycle, $S_r =$ initial degree of saturation (Chen et al., 2004)
Figure 3.20 Relationship between the initial degree of saturation and the maximum crack width (Chen et al., 2004)

The experimental results obtained by Chen et al. show that when the initial degree of saturation was maintained below 60 % in the welded tuff, its properties did not change, but when it exceeded 70 %, the rock suffered significant damage. Hence, the critical degree of saturation is about 70 % for this particular rock.

Rock deterioration occurs because the volumetric expansion of water/ice exerts pressure within a pore. If this pressure exceeds the tensile strength of the material, it 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 there exists a critical degree of saturation for a rock and that this critical value varies from rock to rock according to pore size distribution, pore connectivity and initial porosity, which all affect tensile strength (Chen et al., 2004).

The influence of pre-existing flaws

Nicholson and Nicholson (2000) exposed ten types of sedimentary rocks to repeated freezing and thawing cycles with the aim of considering the effect 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. Such a result was not unexpected since compressive strength correlates closely with tensile strength and porosity. These properties were identified by Matsuoka (1990a) for their role in frost susceptibility.
The results of the tests conducted by Nicholson and Nicholson (2000) suggested that the presence or absence of rock flaws alone does not control the deterioration mode, but rather that it is the mutual relationship among these flaws, rock strength and textural properties. The effect of pre-existing flaws is particularly important in the deterioration of stronger rocks. Their direct effect in weaker rocks diminishes, as the effect of other rock properties and of 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.

### 3.4.2 Field tests

**Frost wedging in alpine bedrock**

Matsuoka (2001a) observed frost wedging in alpine bedrock for three years. Here, the width and temperature of rock joints were automatically monitored on a sandstone rock face that showed two seasonal peaks of joint widening in the autumn and spring (Figure 3.21).

![Temporal variations in the width and temperature of the monitored joint](image)

**Figure 3.21** 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 widening was proportional to the freezing 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 results of the measurements demonstrate that joint widening during the autumn freeze-thaw period reflects the intensity and duration of freezing.

The spring events were associated with a rise in the rock surface temperature to 0 °C beneath the seasonal snow cover, and expansion appear to originate from the refreezing of melt water that had entered the joint. The largest widening occurs during the thawing period (see expansion in Figure 3.21). One theory on the occurrence of this phenomenon is as follows. When the ice in a joint starts melting, a space forms between the frozen rock wall and the ice, which is filled by melt water entering it from the surface. When this water refreezes in the joint, the water becomes confined and produces considerable expansion (Matsuoka, 2001a).

**Freeze-thaw events**

Hall (2004) reported that many studies have so far set transition across 0 °C, as the basis for determining the number of freeze-thaw events. Despite the almost ubiquitous assumption of freeze-thaw weathering, clear proof is lacking with regard to interstitial rock water actually freezing and thawing.

In order to assess the weathering regime at a site in northern Canada, the temperatures of 24 concrete paving bricks were collected at the surface and at depth of 1 cm and 3 cm. Temperature data were collected at 1 minute intervals for 1 year. In Figure 3.22 the surface temperature is plotted during a few days in the winter of 2001 and a small temperature peak can be seen to the right of every big peak.

Hall’s data provide obvious proof for the occurrence of water freezing and thawing 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 3.22 indicates that the freezing of water actually took place just below the brick surface and clearly demonstrates that this event did occur. Since it also did so the subsequent night, this indicates that thawing also took place between these two events. 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) in order to gain any meaningful insight into 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 3.22 Detail regarding a surface exotherm (Hall, 2004)
3.4.3 The gap between laboratory and field weathering

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

Matsuoka (2001b) aimed to bridge the gap between laboratory and field studies by describing the differences between microgelivation and macrogelivation. 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 deal mainly with macrogelivation. Figure 3.23 illustrates how laboratory conditions differ from those in the field.

Figure 3.23 Contrasts between laboratory and field frost weathering (Matsuoka, 2001b)

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6 Microgelivation – degradation of material on a small scale, which involves granular disintegration or flaking. This process leads to production of fine debris (μm-to-cm scale of fragmentation).

7 Macrogelivation – degradation of material on a greater scale than microgelivation. Macrogelivation is the 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 linking laboratory results to 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). Direct evidence is needed to prove that ice action causes disintegration and this can be obtained in the field through simultaneous monitoring of rock disintegration and associated parameters. For soft, intact rocks this is so far rare because of technological immaturity and the need for long-term operations. Laboratory criteria will therefore continue to be useful for the interpretation of field observations. Field problems that can be simulated by laboratory models involve the small-scale fragmentation of soft rocks on a rock outcrop (Matsuoka, 2001b).

### 3.4.4 Mathematical model

Walder and Hallet (1985) presented a mathematical model for the breakdown of porous rock by ice growth. The model was based on 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, in the same manner as water migrates to ice lenses in soil. This model is based on material parameters (elastic modules, fracture-mechanical properties, crack size, grain size and shape) and environmental properties (temperature, temperature gradient and water pressure).

For a granite and a marble the most effective crack growth appears when temperatures range from -4 °C to -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 ice pressure does not build up, while the hydraulic conductivity of the frozen fringe is extremely low and water migration is strongly inhibited (Walder and Hallet, 1985).
All rocks contain pore spaces of various shapes and sizes. In their model, Walder and Hallet chose a crack that 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 3.24). 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 and below it the rock is completely unfrozen.

Figure 3.24 Idealisation of the freezing of cracked rock (Walder and Hallet, 1985)

The model deals with cracks in granite with initial radii of 5 and 50 mm. The results of the study showed that crack propagation progressed most rapidly for the crack with the larger initial radius, while the highest pressure of 12 MPa, developed in the crack with the smaller initial radius.
Figure 3.25 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 is higher than for the crack with the smaller initial radius (5 mm). The crack growth rate for the 50 mm crack progresses most rapidly from 0 °C to -5 °C after which the rate decreases. For the smaller crack, the most rapid crack growth rate develops between -6 °C and -15 °C.

Figure 3.25 Crack growth rate in granite as a function of crack wall temperatures (Walder and Hallet, 1985)
Figure 3.26 shows crack growth in granite and the ice pressure within the 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. Note that crack propagation is only 0.3 mm, but interestingly it continues despite a constant ice pressure. The largest crack propagation of 16 mm, which still grows at constant pressure (see Figure 3.26b) develops in the crack with the initial radius of 50 mm, which freezes at -6 °C. The ice pressure rises to 4 MPa and remains constant.

Figure 3.26 Cracking history during sustained freezing of granite (Walder and Hallet, 1985)

- a: initial crack radius = 5 mm
  Crack growth is < 0.05 mm for freezing at -6 °C
- b: initial crack radius = 50 mm
  Crack growth is < 0.5 mm for freezing at -12 °C

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*increase in crack radius from initial value*
*ice pressure within crack*
The effect of crack growth that depends on the cooling rate is illustrated in Figure 3.27. 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 the crack growth. For example, when the crack with an initial radius of 5 mm in granite cools 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 it is 0.1 °C/h (look at the dotted line in Figure 3.27). Crack growth reflects the increase of flow resistance at the frozen fringe during cooling. If the cooling rate is high, crack growth will be small or non-existent, due to the fact that the frozen fringe is impermeable. At a low cooling rate, the frozen fringe is still permeable and water can migrate to the crack, which continues to grow. Crack growth occurs even if the ice pressure drops.

Figure 3.27 Cracking history during monotonic cooling as function of time (Walder and Hallet, 1985)

Walder and Hallet (1985) pointed out that water migration is the limiting factor in crack growth and that frost shattering does not necessarily require freeze-thaw oscillations.
4 FIELD INVESTIGATIONS

During the winter seasons ice causes great problems in many Swedish railway tunnels, which applies to both older and newer tunnels. Ice, rock and shotcrete at roof and walls can come loosen and fall down, installations and cables are break down under the ice load and the tracks become covered by ice because freezing of water leakage from the roof and the walls.

To maintain safety and prevent traffic disturbance, many tunnels requires frequent maintenance efforts, and the removal of ice, loose rock and shotcrete is expensive and also connected with risks for the maintenance workers. To reduce maintenance cost it is necessary to improve the knowledge of frost penetration along tunnels as well as the effect of ice pressure and frost shattering on the load-bearing construction. Therefore, two field investigations have been undertaken and several Swedish railway tunnels have been investigated regarding ice problems and temperature flow.

4.1 Field observations

4.1.1 Background

The extent of the problems of water leakage and ice formation were not known to the Swedish Rail Administration and the aim of the first investigation was to gather information about problems of water leakage and its effect on a tunnel when it was subjected to freezing temperatures.

There are many factors that determine whether frost or ice formations will appear in tunnels. These include the quality of the rock mass (crushed or fractured zones or cracks), rock overburden, groundwater level, precipitation, tunnel design, operations in the tunnel such as grouting and location of frost-insulated drainage and, of course, temperature conditions in the region where the tunnel is located.
In order to collect information on ice formation problems, field observations were undertaken in five of Sweden's railway tunnels from autumn 2004 until summer 2005. These tunnels were the Nuolja tunnel, the Laduberg tunnel and the Glödberget tunnel in the north of Sweden, while in the Stockholm area observations were undertaken in the Rönninge tunnels, which consist of three shorter tunnels close to each other, and in the Kvedesta tunnels, two shorter tunnels (Andrén, 2008a).

These tunnels were inspected on four different occasions, in order to determine variation in the problems throughout the year. The first occasion was in the autumn while water was still leaking into the tunnel. The second occasion was at the beginning of the freezing period, when the leakages began to freeze and ice formations were formed. The third occasion was during the winter, when the tunnel had been frozen for a while, in order to tell whether there were any active leakages and how the ice formations had developed. The fourth and last occasion was in the spring, when the tunnel had begun to thaw and the leakage spots were active again.

In the Glödberget tunnel the inspections were carried out during three consecutive years with the intention of recording the variations during the entire period years and not just those throughout any one particular year.

### 4.1.2 Observations

The observations in Swedish railway tunnels have shown that water can continue to leak for a long time if the freezing rate is slow. This can produce ice formations such as icicles and ice pillars in the tunnels that can constitute a major problem (Figure 4.1). Ice layers can spread out over the tunnel floor and on the rails, which can cause derailment, and ice pillars on the tunnel walls can grow so large that they intrude on the clearance gauge. Another problem in railway tunnels is that icicles can grow so long that they reach the overhead contact line, which can cause a short-circuit. They are also a hazard because they can fall down and injure personal working in the tunnel and cause damage to installations. An icicle that falls down when a train passes through the tunnel can easily break a window. Frost shattering is another problem and this process can cause fall-outs of rock and shotcrete.
Figure 4.1  

- a) Icicles in the tunnel roof  
- b) Ice layer spread out over the tunnel floor  
- c) Ice pillars on the tunnel wall

This project has shown that many problems in the tunnels are directly linked to the frost-insulated drainage. Leakage and ice formation occurs at the edge of the drainage, in joints and by inlets for cable rack, handrail or other installations. In drainages covered with shotcrete, cracking of the material can be a problem.

The observations reveal that there is a difference as to where and when leakage spots appear throughout the year, and also with regard to the degree of leakage water fluctuation. For example, see section 1512+840 in Figure 4.2, where there were some leakage spots in October (yellow filled columns), while in December there were numerous of ice observations (red dashed columns). The result is that it is difficult to estimate where the insulated drainage should be located along the tunnel. In some tunnel projects it is unfortunately so that drainages are installed along the tunnel after just one inspection. Based on experience gained from this investigation, the determination of location for
drainages should be carried out after several inspections and especially after a winter period, when the actual problems with ice formations occur.

Figure 4.2 Quantity of leakage spots and ice observations along the Nuolja tunnel (Andrén, 2008a)

These observations have also shown that ice formations appear along the entire tunnel, even in tunnels with a length in excess of 1000 m. In these tunnels an even larger number of ice formations occur in the inner sections of the tunnel than in those around the tunnel entrances. See Figure 4.2 which shows that a great number of ice formations (dashed columns) appear in the inner sections as opposed to the tunnel entrances. Previous thinking about this problem was that ice formation occurred only at the tunnel entrances and outer sections. Therefore, one solution have been to install insulated drainage on the entire tunnel area over a distance of 200-300 m from each tunnel entrance, in an effort to complete eliminate the problem of ice formation. However, this is not an effective solution to the problem as the insulation does not only prevent the cold from reaching the leakage spot. It also prevents heat from the rock mass passing into the tunnel and warming up the cold air entering it from outside. The cold can now can reach further into the tunnel and thus move the problem of ice further along the tunnel.
Another insight from the project is that the changes in the freezing periods and also their duration have a major effect on the number and location of leakage spots in a tunnel. If a freezing period is of 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 is subjected instead to short periods of freezing and thawing, the crack will never become frozen and water will continue to leak, resulting in growing icicles. This phenomenon is most common in the central and southern parts of Sweden, where the temperature often fluctuates around 0°C during the winter. In the north of Sweden the climate is colder and the leakage spots close to the tunnel entrance almost always become frozen. The problem of growing icicles in these sections appears only in the autumn and spring and not during the winter. But it can occur further along the tunnels, where the tunnel air temperature is higher due to heat transport from the rock. Another problem is that the leakage water transports heat from inside the rock mass to the cold tunnel wall. The heat content of the water keeps the rock unfrozen despite of freezing tunnel air temperatures. Hence, the leakage spot will continue to leak and as a consequence ice formations grow when the water meets the cold tunnel air (Andrén, 2008a).

4.1.3 Photo documentation

Many photos were taken during the tunnel inspections and the same sections were photographed during all four inspections to allow a comparison between the different occasions. To take one example, at section 816+345 in the Glödberget tunnel there was a leakage spot at the edge of a covered drainage. The drainage is shown at the right side of the photo (Figure 4.3) and cracks in the shotcrete appear distinctly along the outer edge of the drainage.

During the first inspection in November the shotcrete around the leakage spot was wet and the handrail was broken, which led to this section being flagged as a potential maintenance problem. When the second inspection was undertaken in January, a large ice pillar had developed beside the edge of the drainage and the handrail was covered with ice. When comparing the first and the second inspection, the photos show that the shotcrete was much moister during the second inspection. During the third inspection in March, the large ice pillar had been removed from the tunnel, leakage had stopped and there was no longer any moisture on the tunnel wall. During the fourth inspection
in May, leakage was active again, the shotcrete was moist and the ice pillar was slowly melting (Andrén, 2008a).

Figure 4.3 Differences between the four inspections in the Glödberget tunnel, section 816+345 (Andrén, 2008a)
To see how the occurrence of ice formation varies over different years, further follow-up inspections of the Glödberget tunnel were undertaken in March 2005, 2006 and 2007. These inspections showed that an annual variation in the moisture and quantity of ice in tunnels can occur. Figure 4.4 shows the difference in size of ice formation at a certain section during the three annual inspections. The photos also show the difference in moisture conditions in the tunnel wall. During the 2007 inspection, the shotcrete was much moister than in the two previous years, which can partly be explained by the fact that the temperature had fluctuated around 0 °C for about a week before the inspection. This may have caused warm outside air to penetrate the tunnel, thawing the rock mass nearest the tunnel wall and causing leakage at the leakage spots that had previously been frozen (Andrén 2008a).

Figure 4.4  Comparison at section 816+665 in the Glödberget tunnel in March 2005, 2006 and 2007 (Andrén, 2008a)
4.2 Monitoring of temperatures etc.

4.2.1 Background

The second investigation aimed to investigate temperature flow in railway tunnels and to verify a model test undertaken by the University of Gävle and the Royal Institute of Technology. Verification involves monitoring temperatures, air pressure and wind velocity in two railway tunnels in Sweden. The objective of the model test was to find a method to determine tunnel temperature conditions. The method should be used for designing frost insulation needed to protect the drainage system and prevent ice formation. Frost penetration in a tunnel occurs as a result of the fact that the tunnel air is set in motion by (i) thermal produced air flow, (ii) train passing through the tunnel and (iii) wind pressure.

The final report of the model test showed that in majority of tunnels, the predominate cause of frost penetration was the thermal produced air flow because it is constantly in progress (Sandberg et al., 2002). The results of the model test comprise a number of temperature diagrams for tunnels with different inclinations. Figure 4.5a shows the variation in temperature along a tunnel, where frost penetration is the distance from the tunnel entrance to the section where the temperature rise above 0 °C. The frost penetration distance ($X_0$), depends on the air temperature outside the tunnel ($T_0$) and the rock mass temperature ($T_b$). The final report presents a number of diagrams that show how frost penetration varies with different tunnel inclination. In Figure 4.5b one of the diagrams is shown that is valid for frost penetration at the lower entrance of an inclined tunnel.
Figure 4.5  a)  Variation in temperature along a tunnel  
b)  Frost penetration at the lower entrance of an inclined tunnel  
(modified from Sandberg et al., 2002)

4.2.2 Field test configuration

The first monitoring system was installed in the Åsa tunnel, south of Gothenburg, in April 2006 (Figure 4.6) and the second system was installed in the Glödberget tunnel, south of Umeå, in February 2007 (Figure 4.8).

In the Åsa tunnel the system measures air and rock surface temperatures in nine sections along the tunnel, and in five of these sections temperatures 10 cm into the rock mass are measured (Figure 4.7). Beside these temperatures, the air pressure in two sections is measured as well as wind velocity in three sections along the tunnel. Outside the northern tunnel entrance there is a meteorological station that measures air temperature, wind velocity and direction, and atmospheric humidity (Andrén, 2008b).
Figure 4.6 Monitoring system installed in the Åsa tunnel

Figure 4.7 Installation of temperature sensors at section 46+650 in the Åsa tunnel (Andrén, 2008b)
The Glödberget and Åsa tunnels have almost identical monitoring systems. One difference being that the rock temperature is measured at two depths, 10 cm and 50 cm. Another difference being that temperature measurements are also taken in the ballast bed at depth of 0.5 m, 1 m and 2 m in two sections in the tunnel, temperature measurements behind an insulated drainage and lastly, the air and rock surface temperature in the adjoining service tunnel, are also taken (Andrén, 2008c).

Figure 4.8 Monitoring system installed in the Glödberget tunnel
4.2.3 Results

The results of these measurements have shown that frost penetration reaches further into the tunnels than previous assumed. Though the Glödberget tunnel is 1680 m long, the temperature is below 0 °C in the entire tunnel although the temperature outside the tunnel is just a few degrees below 0 °C (Figure 4.9).

![Figure 4.9 Frost penetration along the Glödberget tunnel in March 2007 (Andrén and Dahlström, 2008)](image)

4.2.4 Comparison with the model test

To allow a simple comparison with the model test, temperature values from the field measurements in Figure 4.9 are inserted in the model test diagram. With an outside temperature of -12 °C and a rock mass temperature of +3 °C (which is the same as the annual mean temperature value for the region where the Glödberget tunnel is located), the dashed lines in Figure 4.10 show that, according to the model test, maximum frost penetration should be found at approximately 500 m from the lower tunnel entrance. But measurements show that the temperatures are sub-zero in the entire tunnel.
One reason for the failure the field measurements and the model test to correspond can be that model test theories are based on a completely uninsulated tunnel. In the Glödberget tunnel a lot of insulated drainage has been installed, and as previously explained (see section 4.1.2) its function is to prevent the cold tunnel air from reaching a leakage spot and causing water to turn into ice. However, the insulation does not only prevent the cold air from reaching the rock, but also prevents the heat from the rock mass from entering the tunnel and warming up the cold tunnel air. Consequently, the frost penetrates further into the tunnel than it would do if the heat from the rock mass were allowed to warm up the outside air on its way into the tunnel (Andrén, 2008c).
5 LABORATORY TESTS

5.1 Introduction

Laboratory studies consisting of freeze-thaw tests on shotcrete/rock panels were performed to study how water migration affects the growth of ice and ice pressure in the shotcrete/rock interface. These were performed on saturated panels both with and without access to water during freezing as a means of determining the difference in degradation under varying water conditions.

A small area of poor adhesion around a rock crack opening can initiate a void in the shotcrete/rock interface. In this void ice pressure can develop during freezing if there is access to water, which can cause adhesion failure and shotcrete degradation. To simulate this action, the rock panels were prepared with small areas of poor adhesion prior to shotcreting.

The shotcrete/rock interface in the panels was subjected to 20 freeze-thaw cycles. This is not a standard method, and the number of freeze-thaw cycles having been selected as a result of air temperature field measurements in the northern Sweden (Umeå) during a single winter season, when temperature was found to transit the zero mark some 20 times (Andrén, 2008a). Hence, in the laboratory tests, the temperature at the interface was made to oscillate around 0 °C 20 times in order to cause potential water in the interface to freeze and subsequently thaw and allow more water to reach the interface area through water migration. These laboratory tests thus represent deterioration in the shotcrete/rock interface when subjected to approximately a single winter season of freeze-thaw cycles.

To evaluate whether degradation occurs, both acoustic emission (AE) measurements and a direct tensile strength test were undertaken. The AE measurement is a non-destructive test and can be used continuously during the freezing test without influencing the results. The direct tensile strength
test is a destructive test, which is used to determine the adhesive strength between shotcrete and rock (SIS, 2005).

5.2 Test panels

The test panels were made of kuru granite, which is a fine-grained, equigranular and isotropic granite (Figure 5.1). The panel dimension was 80×800×800 mm, with a tolerance of ± 3 mm for length and thickness and ± 1 mm for plane-parallel surfaces.

Figure 5.1 Test panel of kuru granite

Boreholes to imitate fractures were drilled through the rock panels to induce the water to reach the shotcrete/rock interface. These were drilled in a particularly pattern in order to utilise the maximum numbers of test samples in each panel (Figure 5.2). The distance between each two boreholes was 2.5 D, where D represents the diameter of the test samples, which after the freeze-thaw test, were to be used for tensile strength tests. Drilling was done by TESTLAB at the Luleå University of Technology.
The boreholes were drilled with a 5 mm diameter drill. Initially, it was intended to use a 2 mm drill to imitate the test carried out by Matsuoka in 2001 (Matsuoka, 2001b), but 5 mm proved to be the smallest diameter that could be used for this thickness of rock (80 mm).

After drilling the test panels were sandblasted and covered with 50 mm of shotcrete, prior to which, they were prepared with small areas of poor adhesion. Shotcreting was undertaken by BESAB AB in Gothenburg.

5.2.1 Evaluation of material to provide poor adhesion

Various materials were tested for their suitability in providing a small area of poor adhesion around the boreholes. Such a material had to fulfil two criteria, namely an ability to provide poor adhesion and also robustness so as to resist being worn away by the effect of the sandblasting during shotcreting.
Panel no. 1 was prepared with three different materials; geotextile (Typar 3377 with a pore size 180 μm and a thickness 0.46 mm), and a red and a white wax crayon (Figure 5.3). The red one provided a thick, greasy surface, as did the white wax crayon, although in this latter case the surface provided was less thick. The geotextile was fixed to the rock surface by four small dots of glue (Tremco Tremsil 600) at the outer edge of the circle (diameter 30 mm).

![Figure 5.3 Panel no. 1 prepared with materials to provide poor adhesion](image)

After preparation the panel was covered with frost-resistant shotcrete (Maxit C 100/300 Anl 4 mm), which had to harden for at least 28 days before further testing. After hardening, shotcrete/rock cores with a diameter of 74 mm were drilled out from panel no. 1 and the cores were subjected to a bending load to separate the shotcrete from the rock surface (Figure 5.4). This was done to expose the surfaces and thereby estimate whether the material caused poor adhesion and/or if the material had been worn down by the effect of sandblasting during shotcreting.

The test result confirmed that the white wax crayon provided the worst adhesion, and that the abovementioned sandblasting had not affected either the crayon layer or the position of the geotextile. Therefore, the white wax
crayon and the geotextile were selected to be used on the test panels. Drilling and separation of test samples were carried out in Gothenburg by the firm of Jerneviken Maskin AB.

Figure 5.4  a, b) One of the cores from panel no. 1 subjected to a bending load  
   c)  Rock and shotcrete surface prepared with white wax crayon after separation

5.2.2 Preparation of test panels for the freeze-thaw test

After the materials that provided the worst adhesion were selected, all the other panels were prepared according to a particular pattern (Figure 5.5), with a total of eight boreholes in each panel. Two boreholes were left unprepared to establish some reference samples with good adhesion. Three boreholes were prepared with geotextile and three, with the white wax crayon.

The geotextile was used to imitate the conditions close to a fracture opening and to simulate the properties of the fracture filling material. Fracture filling can contribute to poor adhesion due to the involvement of weathered material and it can also carry water from the rock mass towards the tunnel wall. The wax crayon was used to imitate the behaviour of rock surfaces with poor adhesion, which may be due to the mineral composition of the rock mass or
inadequate cleansing of the rock surface with water or through sandblasting prior to shotcreting.

Figure 5.5 Preparation of the test panels

After preparation, the panels were covered with shotcrete and transported back to TESTLAB in Luleå and were kept wet during the transportation. Back in Luleå they were placed in a water reservoir for at least two weeks to ensure their water saturated condition before the freeze-thaw test started.

Figure 5.6 shows borehole numbering and preparation of each hole, viewed from the rock side.
Figure 5.6  Borehole numbering and preparation, viewed from the rock side

5.3 Test samples

Once the test panels had been exposed to the freeze-thaw tests, eight cores of rock and shotcrete with a diameter of 94 mm were drilled out from each panel (Figure 5.7). The cores were drilled centrically by over-coring the 5 mm boreholes according to Figure 5.2. The shotcrete/rock samples were then used to test the adhesive strength of the interface through a direct tensile strength test. Drilling was undertaken by TESTLAB in Luleå.

Figure 5.7  Test sample of shotcrete/rock core
5.4 Experimental arrangement

5.4.1 Arrangement of the freeze-thaw experiment

The freeze-thaw tests were performed by TESTLAB in Luleå in a cooling chamber at a constant temperature at +4 °C. The test panels were installed in a box of cellular plastic insulation and cooling took place from the shotcrete side of the panel, as in a tunnel situation. The cooling system consists of a cryostat (water with 20-30 % ethanol) that circulates in copper pipes at the bottom of the box at a temperature around -15 to -10 °C. To even out the temperatures across the test panel, a steel plate was placed on top of the pipes, and a layer of sand then placed on the plate to level the test panel in the box (Figure 5.8).

![Diagram of freeze-thaw test experiment](image)

Figure 5.8 Arrangement of the freeze-thaw test experiment

To ensure identical freezing and thawing cycles for all the test panels, the cooling system was controlled by a temperature relay that started or stopped cryostat circulation at a pre-set temperature of the temperature sensors placed at the shotcrete/rock interface. The intention was to induce the freezing front to oscillate close to the interface, to enable water migration.
Temperature sensors were installed in boreholes at two locations on each panel, A and B, and they were placed in eight positions at each location (Figure 5.9). Sensor no. 1 was placed directly on the rock surface and sensor no. 8 on the shotcrete surface. Sensor no. 5 was placed at the shotcrete/rock interface and the other sensors were located in the rock and shotcrete as shown in Figure 5.9b. Afterwards the boreholes used for the installation was grouted.

![Figure 5.9 a) Location of temperature sensors, A and B, viewed from the rock side](image1)

![Figure 5.9 b) Temperature sensor positions and numbering, cross section](image2)

At each borehole (1 to 8 in Figure 5.9a) a water access hose was installed (Figure 5.10) and these hoses were connected to a bucket filled with water about 1 m above the test panel, to create a water pressure with a one-meter water head, above the level of the shotcrete/rock interface (Figure 5.11). The water head around the periphery of a grouted tunnel varies. However, one-meter water head was selected as a reasonable value.
Figure 5.10 Water access hose sealed with silicone paste

Figure 5.11 Photo of the experimental arrangement
5.4.2 Arrangement of the acoustic emission monitoring

Acoustic emission monitoring (AE) was used to follow up the progress of the failure of the shotcrete/rock interface. The test was carried out in 2D but during measuring and processing of the data, the system takes into account the depth of the events. Consequently, it is possible to ensure that the location of the events will appear from the shotcrete/rock interface.

AE was continuously monitored during the freeze-thaw test of the test panels. The system consisted of eight sensors, eight preamplifiers, an AE acquisition system and a computer (Figure 5.12). The ESG program, UltrACQ, was used for the real-time monitoring of the AE events for all the tests and was set to record all events that triggered more than four piezoelectric sensors. More information about the AE system can be read in Appendix 4.

Figure 5.12 Arrangement of AE monitoring (Photo: Ganesh Mainali)

All the sensors were mounted at eight different locations on the top surface of the test panels (Figure 5.13) using silicon high vacuum grease as a coupling material. In addition to coupling, the sensors were kept stationary by using metal brackets and a piece of insulation material to hold them in position (Figure 5.14).
Figure 5.13 Location of the sensors on test panel no. 4 (Photo: Ganesh Mainali)

Figure 5.14 Sensor mounted on the test panel
5.4.3 Arrangement of the direct tensile strength test

To test the adhesive strength of the shotcrete/rock interface, a direct tensile strength test was performed by TESTLAB in Luleå, using a Dartec 50 kN machine (Figure 5.15). The load increment for the test was 0.02 kN/second.

After the freeze-thaw tests, shotcrete/rock samples with a diameter of 94 mm were cored out from the panels. The samples were face-grounded and one side of the test sample was glued to a steel plate (Figure 5.16a). When the glue had hardened the steel plate was installed in the machine for direct tensile strength testing (Figure 5.16b). The other end of the core was then glued to another steel plate, already in contact with the machine and after the glue hardened, the direct tensile strength test was performed (Figure 5.17).
Figure 5.16  a) Test sample glued to the first steel plate
           b) Test sample installed in the machine

Figure 5.17  a) Test sample glued to the second steel plate in the machine
           b) Test sample separated after the direct tensile test
5.5 Results of the freeze-thaw tests

In total, four test panels were tested, and three of them were subjected to the freeze-thaw tests at different water conditions. In Table 5.1 the conditions for each panel are given.

Table 5.1 Conditions for each test panel

<table>
<thead>
<tr>
<th>No. of test panel</th>
<th>Subjected to freeze-thaw test</th>
<th>Saturated with access to water during freezing</th>
<th>Saturated without access to water during freezing</th>
</tr>
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<tbody>
<tr>
<td>6 - Reference</td>
<td>No</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>7</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
</tbody>
</table>

5.5.1 Temperature monitoring

The test panel was placed in the cooling chamber at a temperature of +4 °C and the freeze-thaw test started when the panel had reached the same temperature as the chamber. The cryostat in the first freezing test had a temperature of -15 °C. The system was controlled by the temperature sensors at position 5A and 6A (Figure 5.9). When the freezing front had passed the shotcrete/rock interface, and the temperature sensor at position 5A had reached -0.5 °C the cooling system circulation stopped and the thawing phase could begin. When the temperature at sensor 6A was +0.5 °C the cooling system circulation was re-initiated and the freezing phase started all over again. These low temperatures were selected to enable water migration, which can be inhibited if temperature is too low or if freezing rate is too rapid (Walder and Hallet, 1985)

Panel no. 5 – The first test

The first test panel was no. 5 which had access to water during freezing and was exposed to 20 freeze-thaw cycles. Figure 5.18 shows temperature monitoring by some of the sensors and for further details please refer to Appendix 1a.
Figure 5.18 Temperature at the shotcrete/rock interface and in the cryostat for the first test on panel no. 5 – cycle 1 to 20

Discussion after the first test

When the results from the AE measurements was analysed (see section 5.5.2), they showed that the AE events did not spread out over the test panel as expected, but were concentrated around borehole no. 5. One explanation could be that only borehole no. 5 had access to water and for the other boreholes the water access was prevented. Therefore, a check on the calibration of the temperature sensors were performed, which showed that they were incorrectly calibrated and displayed a temperature that was 0.5-0.7 °C higher than the actual temperature. There was an immediate risk that the interface had been frozen throughout the whole first test and therefore a second test was performed of the same test panel no. 5.

Panel no. 5 – The second test

The second test performed on panel no. 5, which had equally access to water during freezing. The panel was subjected to a further 20 freeze-thaw cycles. To ensure that the interface froze and thawed correctly, the start and stop temperatures for the cooling system circulation were changed from +0.5 °C and -0.5 °C to +1 °C and -1 °C. There was also a temperature discrepancy at the two different locations for the temperature sensors A and B (compare the
blue and the red lines in Figure 5.18). Sensor no. 5A had a higher temperature than sensor no. 5B, which reveals that the temperature was not uniformly distributed across the panel. The result of temperature measurement in the second test is shown in Figure 5.19 and here the temperature at sensor no. 5A and no. 5B was almost the same. As a consequence of the incorrectness of the temperatures, the first test with its 20 cycles actually was a single long freeze-thaw cycle. The second test had 20 cycles and the cycles were numbered 21 to 41, but were actually 2 to 22. For further details please refer to Appendix 1b.

![Panel no. 5 - The second test, cycle 21 to 41](image)

Figure 5.19 Temperature at the shotcrete/rock interface and the cryostat for the second test on panel no. 5 – cycle 21 to 41

As shown in Figure 5.19 the temperature in the cooling chamber was disturbed from 2008-05-20 to 2008-05-22, due to a power cut at the university.

**Proceeding of the laboratory tests**

When the tests on panel no. 5 were over, fresh discussion of the test arrangement took place. The primary idea was to perform a freeze-thaw test on another test panel without access to water to compare with that done on panel no. 5. However, because of the problem with the first and second tests, it was decided to perform two completely new tests – one with access to water
during freezing and one without. Hence, the next test was undertaken on panel no. 7 which had access to water during freezing, and the following test on panel no. 4, which did not. To ensure that freezing and thawing would occur, start and stop temperatures were changed once more, first to +1.5 °C and -1.5 °C, and then to +2 °C and -2 °C after eight cycles during the test on panel no. 7. Subsequently the temperature was more uniformly distributed across the whole panel. In addition, the cryostat temperature was changed to -10 °C and the difference in regularity is quite clear when comparing Figure 5.19 with Figure 5.20 (cryostat = the black line).

**Panel no. 7**

Panel no. 7 had access to water during freezing, and as described above the temperature range was changed, which caused longer freeze-thaw cycles. The panel was subjected to 20 cycles and this test was performed without any interruptions or disturbances. Figure 5.20 shows the temperature monitoring of some of the sensors. For further details please refer to Appendix 2.

![Figure 5.20 Temperature at the shotcrete/rock interface and in the cryostat for the test on panel no. 7](image-url)
Panel no. 4

Panel no. 4 had no access to water during freezing and was subjected to the same temperature range as panel no. 7. The panel was subjected to 20 freeze-thaw cycles and in Figure 5.21 the temperature monitoring of some of the sensors is shown. For further details please refer to Appendix 3.

![Graph showing temperature monitoring](image)

Figure 5.21 Temperature at the shotcrete/rock interface and in the cryostat for the test on panel no. 4

After two freeze-thaw cycles a cooling system problem occurred. The thawing phase did not start although the temperature sensor had reached -2 °C. This led to a longer freezing period than expected (see dates 2008-11-01 to 2008-11-04 in Figure 5.21).

5.5.2 Acoustic emission measurements

In the following figures illustrating AE events, every dot indicates a recorded event that triggered more than four sensors. If the events triggered four or less sensors that event is not recorded, which gives better data quality and a high accuracy in the locations of these events. The colour of the dot represents the number of sensors that was triggered during that specific event. Red dots mean that AE signals from eight sensors are used to locate the event, yellow
dots indicate AE signals from seven sensors, blue dots AE signals from six sensors and finally purple dots AE signals from five sensors.

Panel no. 5 – The first test

The first test was performed with access to water during freezing. Figure 5.22 shows total numbers of AE events (1640) for the first 20 cycles and there was an obvious concentration of events around borehole no. 5.

Figure 5.22 Location of AE events for the first test on panel no. 5
The increase in number of events for different freezing cycles is shown in Figure 5.23. Each picture shows accumulated number of events and event locations widened outwards from the centre of borehole no. 5.

Figure 5.23  Accumulated AE events according to different freezing cycles for panel no. 5
Figure 5.24 shows the number of AE events for each cycle, which decreased gradually during the test.

Panel no. 5 - first 20 cycles

Figure 5.24 Number of AE events for each freezing cycle for the first test on panel no. 5

**Panel no. 5 – The second test**

As described in section 5.5.1 it seemed that only one borehole had access to water, which led to performance of another 20 freeze-thaw cycles for panel no. 5. The second test was, just as the first test, performed with access to water during freezing, but the start and stop temperature were changed. Figure 5.25 shows the total numbers of AE events (82 events) for cycles 21-41. In this test there was some concentration of events around boreholes nos. 4 and 5 but not nearly as many as in the first test.
Figure 5.25 Location of AE events for the second test on panel no. 5 – cycles 21-41

Figure 5.26 shows the number of AE events for each cycle in the second test, which is considerably fewer than in the first test.

Figure 5.26 Number of AE events for each freezing cycle for the second test on panel no. 5
Panel no. 7

After the first two tests on panel no. 5, the discussions recorded in section 5.5.1 led to changes to the temperature range. The two following tests on panels nos. 7 and 4 were then performed in the same way, but with different water access.

Panel no. 7 had access to water during freezing and Figure 5.27 shows the total numbers of AE events (1603) for the 20 cycles. There was a concentration of events around boreholes nos. 7, 5 and 3 and in the area between boreholes nos. 8, 6 and 4.

Figure 5.27 Location of AE events for panel no. 7
Figure 5.28 shows the increase in the number of events for different freezing cycles. Each picture shows an accumulated number of events and event location widened out from the centre of boreholes nos. 7, 5 and 3.

In Figure 5.29 the number of AE events for each cycle is shown. There were many events in the first six cycles but then their number decreased radically.

Figure 5.29 Number of AE events for each freezing cycle for panel no. 7
Panel no. 4

The test on panel no. 4 differed from the other two panels. It was water saturated but panel no. 4 had no access to water during freezing. This panel had far fewer AE events than the two other, only 129 events compared to over 1600. Figure 5.30 shows the location of the total numbers of AE events for panel no. 4. There was no obvious concentration of events around any borehole, which was expected, while this panel had no access to water through the boreholes.

Figure 5.30 Location of AE events for panel no. 4
Figure 5.31 shows the increase in the number of events for different freezing cycles. Each picture shows an accumulated number of events.

Figure 5.31  Accumulated AE events according to different freezing cycles for panel no. 7

Figure 5.32 shows the number of AE events for each cycle. There were some events in the first four cycles but only a few during the rest of the test.

Figure 5.32  Number of AE events for each freezing cycle for panel no. 4
5.5.3 Adhesive strength test

The tables below show the results of the direct tensile strength test. The information in the different columns is as follows:

1. number of the test sample
2. maximum force from the direct tensile strength test
3. how the sample was prepared
4. the area used in calculating of the adhesive strength – it represents the area of the test sample which had good adhesion, i.e. the total area of the test sample (diameter = 94 mm) minus the area of the borehole (diameter = 5 mm) or the area of poor adhesion (diameter = 30 mm)
5. adhesive strength for the sample = maximum force/area

Panel no. 6 – the reference panel

The results of the tensile tests for reference panel no. 6, which was not subjected to the freeze-thaw test, are shown in Table 5.2. There was a great variation of the results and some of them were surprisingly low.

Table 5.2 Results of the tensile tests for the reference panel no. 6

<table>
<thead>
<tr>
<th>No. of the sample</th>
<th>Maximum force (kN)</th>
<th>Preparation</th>
<th>Area (m²)</th>
<th>Adhesive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.98</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.58</td>
</tr>
<tr>
<td>2</td>
<td>2.74</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.44</td>
</tr>
<tr>
<td>3</td>
<td>9.07</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>1.31</td>
</tr>
<tr>
<td>4</td>
<td>2.83</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.45</td>
</tr>
<tr>
<td>5</td>
<td>1.33</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.21</td>
</tr>
<tr>
<td>6</td>
<td>1.63</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.26</td>
</tr>
<tr>
<td>7</td>
<td>6.77</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>1.09</td>
</tr>
<tr>
<td>8</td>
<td>5.18</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.83</td>
</tr>
</tbody>
</table>

The results for panel no. 6 give a mean value of the adhesive strength for:

- unprepared samples = 0.94 MPa
- wax crayon = 0.38 MPa
- geotextile = 0.71 MPa
Panel no. 5

The results from the tensile tests for panel no. 5, which had access to water during freezing, are shown in Table 5.3. For two samples, nos. 2 and 6, the shotcrete came away from the rock immediately when the tensile test started.

Table 5.3  Results of the tensile tests for panel no. 5

<table>
<thead>
<tr>
<th>No. of the sample</th>
<th>Maximum force (kN)</th>
<th>Preparation</th>
<th>Area (m²)</th>
<th>Adhesive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.42</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.35</td>
</tr>
<tr>
<td>2</td>
<td>0.00</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.00</td>
</tr>
<tr>
<td>3</td>
<td>3.76</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.54</td>
</tr>
<tr>
<td>4</td>
<td>1.43</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.23</td>
</tr>
<tr>
<td>5</td>
<td>1.91</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.31</td>
</tr>
<tr>
<td>6</td>
<td>0.00</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.00</td>
</tr>
<tr>
<td>7</td>
<td>4.23</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.68</td>
</tr>
<tr>
<td>8</td>
<td>4.35</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.70</td>
</tr>
</tbody>
</table>

The results for panel no. 5 give a mean value of the adhesive strength for:
- unprepared samples = 0.45 MPa
- wax crayon = 0.08 MPa
- geotextile = 0.56 MPa

Panel no. 7

The results from the tensile tests for panel no. 7, which also had access to water during freezing, are shown in Table 5.4. Sample no. 2 had an unrealistic value for the maximum force, which led to a breakage in the shotcrete instead of an adhesion failure. It is not likely that such a high value for the adhesive strength was achieved as compared with the other samples and therefore sample no. 2 was excluded from further evaluations.
Table 5.4 Results of the tensile tests for panel no. 7

<table>
<thead>
<tr>
<th>No. of the sample</th>
<th>Maximum force (kN)</th>
<th>Preparation</th>
<th>Area (m²)</th>
<th>Adhesive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.56</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.51</td>
</tr>
<tr>
<td>2</td>
<td>16.96*</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>2.72*</td>
</tr>
<tr>
<td>3</td>
<td>2.56</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.37</td>
</tr>
<tr>
<td>4</td>
<td>1.42</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.23</td>
</tr>
<tr>
<td>5</td>
<td>1.36</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.22</td>
</tr>
<tr>
<td>6</td>
<td>2.72</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.44</td>
</tr>
<tr>
<td>7</td>
<td>1.50</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.24</td>
</tr>
<tr>
<td>8</td>
<td>3.13</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.50</td>
</tr>
</tbody>
</table>

* Sample excluded from further evaluations

The results for panel no. 7 give a mean value of the adhesive strength for:
- unprepared samples = 0.44 MPa
- wax crayon = 0.33 MPa (sample no. 2 excluded)
- geotextile = 0.32 MPa

Panel no. 4

The results from the tensile tests for panel no. 4, which had no access to water during freezing, are shown in Table 5.5. When sample no. 3 was fixed to the tensile machine, it was exposed to an accidental movement, which probably damaged the sample and therefore this value was excluded from further evaluations.

Table 5.5 Results of the tensile tests for panel no. 4

<table>
<thead>
<tr>
<th>No. of the sample</th>
<th>Maximum force (kN)</th>
<th>Preparation</th>
<th>Area (m²)</th>
<th>Adhesive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.57</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.23</td>
</tr>
<tr>
<td>2</td>
<td>2.48</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.40</td>
</tr>
<tr>
<td>3</td>
<td>0.01*</td>
<td>Unprepared</td>
<td>0.00692</td>
<td>0.00*</td>
</tr>
<tr>
<td>4</td>
<td>1.75</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.28</td>
</tr>
<tr>
<td>5</td>
<td>1.87</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.30</td>
</tr>
<tr>
<td>6</td>
<td>4.60</td>
<td>Wax crayon</td>
<td>0.00623</td>
<td>0.74</td>
</tr>
<tr>
<td>7</td>
<td>2.46</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.40</td>
</tr>
<tr>
<td>8</td>
<td>3.36</td>
<td>Geotextile</td>
<td>0.00623</td>
<td>0.54</td>
</tr>
</tbody>
</table>

* Sample subjected to accidental movement
The results for panel no. 4 give a mean value of the adhesive strength for:
- unprepared samples = 0.23 MPa (only one sample considered – sample no. 3 excluded)
- wax crayon = 0.47 MPa
- geotextile = 0.41 MPa

The value for the unprepared sample is low, 0.23 MPa, and it has the lowest adhesive strength of all the samples even though it was unprepared.

5.6 Analysis of the results

5.6.1 Acoustic emission measurements

The difference in the number of AE events between the panels which had access to water during freezing (nos. 5 and 7) and the one that did not (no. 4) shows that water causes more events, which may confirm that water contributes to material deterioration and may also do so in the case of reduction of the adhesive strength.

Some of the events are concentrated at the location of the boreholes, which was expected for panels with access to water during freezing. There was no concentration of events at any particular borehole for panel no. 4, which had no access to water. Most of the events appeared in the area between the boreholes, and these phenomena occurred with regard to panels both with and without access to water. As these panels were saturated, these events might indicate that the freezing of the pore water, which probably migrated towards the frozen areas of the panel, caused micro adhesive failure of the shotcrete/rock interface. For panel no. 4, most of these events took place at the beginning of the test. An explanation might be that after some cycles all the free water had migrated, and therefore there was no further increase in tension in the interface. The lack of access to water in this panel no. 4, meant that the number of events decreased considerably compared to panels nos. 5 and 7, as all the free water had already migrated during the early cycles. However, most of the events took place where the boreholes were located, and almost no events took place near the panel borders. Hence, it is clear that the boreholes and their preparation cause disturbance at the shotcrete/rock interface.
Figure 5.33 shows the number of AE events in each cycle of the test for panel no. 7. There were many events in the first six cycles but their number then decreased radically.

![Figure 5.33 Number of AE events in each freezing cycle for panel no. 7](image)

The total number of events was about the same for panel no. 5 and panel no. 7, 1722 and 1603 respectively. However, their distribution over the freezing cycles differs completely from one to the other. For panel no. 5 Figure 5.34 shows that the distribution of events was more constant throughout all the first 20 cycles, compared with panel no. 7, where the concentration of events occurs during the first cycles. This may be due to panel no. 5 experiencing problems in the temperature range which caused constant freezing of the interface. In other words, the area around the interface neither froze nor thawed, which probably resulted in the absence of any access to free water for most of the boreholes. Therefore, there were fewer events at the beginning of the test, except for borehole no. 5 which probably had access to free water.
Figure 5.34 Number of AE events in each freezing cycle for the first and second test on panel no. 5

Freezing and thawing of panel no. 7 occurred as planned and the interface had access to water during the entire test and thus many events were taking place at the beginning of the test. After six cycles the number of events decreased dramatically. A possible cause of this decrease may be the fact that ice formations in the interface had reached their maximum size under these circumstances, i.e. access to free water under the prevailing temperature conditions. To get the ice to grow further, the free water need more time to migrate towards the freezing zone, i.e. the freezing cycles needs to be longer. These ice formations caused adhesion breakage of the interface, which in turn caused a large number of events at the beginning of the test. When the ice growth halted despite continued freezing cycles, damage to the interface stopped, and the number of events decreased.

Figure 5.34 shows the number of AE events in each cycle for both the first and second test on panel no. 5, while the number of events decreased notably after the first test (20 cycles). This might be a result of all the stresses in the material having been released during the first test, when the panel was subjected to a single long freezing cycle. This release of stress can be described as a relaxation after tension occurred in the panel, which tension
may be produced by ice growth in the shotcrete/rock interface after which an assumed adhesion failure can cause relaxation. However, tension can also be produced through differences in thermal contraction between two different materials, e.g., rock and shotcrete. If one of the materials contracts faster than the other, tension will occur between them. After a further temperature decrease, the other material starts to contract and relaxation can take place. But both materials have almost the same value for thermal contraction $5.3 \times 10^{-6}/°C$ (Glamheden, 2001), so this explanation is not credible as the temperature decrease must be much faster than in this test, if tension between these two materials is to arise. Furthermore, the result from panel no. 4 shows that there were far fewer events happening during that test, just 129 compared to over 1600 for the other panels. If thermal contraction had been the cause of tension and stress release, the result for panel no. 4 would resemble those for panels nos. 5 or 7. Therefore, it is more likely that the tension and the stress release are caused by ice formation and adhesive failure in the shotcrete/rock interface.

Another observation during the AE measurement was that for some of the boreholes with access to water during freezing there was a widening of the location of events out from the centre of the borehole. This was most evident for borehole no. 5 in panel no. 5, which was prepared with geotextile. For panel no. 7 there was also a widening around boreholes nos. 7 and 5, which were prepared with geotextile, as well as smaller widening around borehole no. 3, which was unprepared. The widening of the location of events indicate that microcracks appeared in the shotcrete/rock interface and caused adhesion failure.

5.6.2 Adhesive strength

Table 5.6 shows the mean values of adhesive strength for all the tests as well as a calculation of the deterioration in adhesive strength compared to the reference panel no. 6, regardless of preparation. This deterioration is calculated as the adhesive strength of the reference panel minus that of the test panel divided by the adhesive strength of the reference panel. Panels nos. 5 and 7 had access to water during freezing while panel no. 4 did not, for which reason the results for panels nos. 5 and 7 have been amalgamated into one value. Adhesive strength deterioration in the samples with access to water was 47 % as against 45 % for the samples without such access.
The results of the tensile tests show that adhesive strength deteriorates when the panels have been subjected to freeze-thaw cycles, but also that there is no great difference whether the panel had access to water or not during freezing. The reason for this may be that all the panels were water saturated when they were tested. Consequently, the pores in panel no. 4 could have contained so much water that it could migration to the interface and cause damage during freezing, although the panel did not had access to free water during the freeze-thaw test. It would thus have been interesting to perform freeze-thaw tests on a complete dry panel. This was discussed at an early stage in the project, but the idea was rejected because conditions in a tunnel situation are never completely dry. However, it would have been a good idea to test a dry panel in order to understand the problem of water migration. Consideration of the obvious difference in the number of AE events between the two different water conditions makes it clear that some activity is in progress when the panel has access to free water during freezing.
In Table 5.7 the results of every type of preparation are compared within its own type, e.g. samples of geotextile in the test panel are compared with those in the reference panel. In evaluating the results for different panels with the same preparation, the decrease in adhesive strength in comparison with the reference panel exhibits a considerable distribution from an increase of 23 % to a decrease of 80 %.

Table 5.7 Mean values of adhesive strength and deterioration compared to the reference panel no. 6 – for each preparation

<table>
<thead>
<tr>
<th>No. of panel</th>
<th>Access to water</th>
<th>Preparation</th>
<th>Adhesive strength (MPa)</th>
<th>Deterioration of adhesive strength (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference no. 6</td>
<td>-</td>
<td>Unprepared</td>
<td>0.94</td>
<td>-</td>
</tr>
<tr>
<td>No. 5</td>
<td>Yes</td>
<td>Unprepared</td>
<td>0.45</td>
<td>53</td>
</tr>
<tr>
<td>No. 7</td>
<td>Yes</td>
<td>Unprepared</td>
<td>0.44</td>
<td>53</td>
</tr>
<tr>
<td>No. 4</td>
<td>No</td>
<td>Unprepared</td>
<td>0.23</td>
<td>76</td>
</tr>
<tr>
<td>Reference no. 6</td>
<td>-</td>
<td>Wax crayon</td>
<td>0.38</td>
<td>-</td>
</tr>
<tr>
<td>No. 5</td>
<td>Yes</td>
<td>Wax crayon</td>
<td>0.08</td>
<td>80</td>
</tr>
<tr>
<td>No. 7</td>
<td>Yes</td>
<td>Wax crayon</td>
<td>0.33</td>
<td>14</td>
</tr>
<tr>
<td>No. 4</td>
<td>No</td>
<td>Wax crayon</td>
<td>0.47</td>
<td>-23</td>
</tr>
<tr>
<td>Reference no. 6</td>
<td>-</td>
<td>Geotextile</td>
<td>0.71</td>
<td>-</td>
</tr>
<tr>
<td>No. 5</td>
<td>Yes</td>
<td>Geotextile</td>
<td>0.56</td>
<td>21</td>
</tr>
<tr>
<td>No. 7</td>
<td>Yes</td>
<td>Geotextile</td>
<td>0.32</td>
<td>55</td>
</tr>
<tr>
<td>No. 4</td>
<td>No</td>
<td>Geotextile</td>
<td>0.41</td>
<td>42</td>
</tr>
</tbody>
</table>

For the unprepared samples, panel no. 4 without access to water shows more deterioration than the samples with access to water (panels nos. 5 and 7). However, adhesive strength for panel no. 4 is based only on one unprepared sample and therefore this observation is rather uncertain and may not be used in the evaluation.

The converse applies to the wax crayon samples, where panel no. 4 shows no deterioration but a higher adhesive strength instead than the reference panel no. 6. Panels nos. 5 and 7 show deterioration compared with the reference panel. One explanation for the difference due to water access between unprepared samples and wax crayon samples can be that water can accumulate more easily in the void between the rock and the shotcrete formed
by the poor adhesion characteristics of the wax crayon, than close to the unprepared borehole with good adhesion for the shotcrete.

However, consideration of the results of the samples with geotextile yields a mixed result. The geotextile should have formed a similar void in the interface between rock and shotcrete as the wax crayon. But one of the panels with access to water (no. 7) caused more deterioration than the panel without access to water (no. 4), while another one (no. 5) caused less deterioration than the panel without access to water.

5.6.3 Summary

According to the hypothesis the results from the tensile test would reveal whether ice growth causes more damage to the shotcrete/rock sample which had access to water during freezing, than to the sample that did not.

Figure 5.35 and Figure 5.36 shows the total results of the laboratory tests with regard to the preparation of the boreholes, adhesive strength and the AE measurements.

Figure 5.35  a)  Results of adhesive strength for reference panel no. 6
   b)  Results of AE measurement and adhesive strength for panel no. 4
Figure 5.36  

a) Results of AE measurement and adhesive strength for panel no. 5  
b) Results of AE measurement and adhesive strength for panel no. 7

For panel no. 5 most events appeared around borehole no. 5 (Figure 5.36a). The results of the tensile test were expected to show that this sample would achieve the lowest adhesive strength, but that was not the case. The value for sample no. 5 was 0.31 MPa, which is not particular good but neither is it the lowest value.

No events happened around borehole no. 2 for any of the test panels, although it has the same wax crayon preparation as boreholes nos. 4 and 6. One reason may be that borehole no. 2 is closest to the cryostat pipe inflow. In the temperature measurement figures, cryostat temperature decreases rapidly when circulation starts. The absence of events around borehole no. 2 may be caused by the rapidly freezing of this area of the panel and the speed of this process may hamper water access to borehole no. 2 and thus no events will occur in this area. The adhesive strength of these borehole varies from 0 to 0.4 MPa.

The results of the laboratory tests have shown that more AE events take place in a panel when it has access to water during freezing, and also that the adhesive strength between rock and shotcrete decreases after the test panels
have been subjected to freeze-thaw cycles. The reduction of adhesive strength for the test samples was about 50% as shown in Table 5.6.
6 DISCUSSION AND CONCLUSIONS

6.1 Discussion

During the field investigations of railway tunnels, several cases of rock and shotcrete fall-out were observed. The fall-outs often occurred in wet areas, i.e., in tunnel sections with groundwater leakage problems. This is probably because access to water during freezing deteriorates the rock and shotcrete material and consequently reduces the load-bearing capacity of the tunnel.

Water migration and thus ice growth depend not only on access to water and freezing temperatures, but also on duration as well as the freezing rate. Tharp (1987) explained that in rock, water can migrate in a manner similar to its action in soil, which causes ice bodies to grow inside pores and cracks. Water migration occurs in the thin water film in the interface between ice and rock (the water in question being adsorbed water that freezes at low temperatures). If the temperature falls further, this adsorbed water in the water film starts to freeze and the quantity of water that has not yet frozen is reduced. The water film becomes thinner, which thereby reducing the hydraulic conductivity of the material and obstructing water migration (Tharp, 1987). If the rock mass is subjected to rapid freezing, water film thickness is rapidly diminished and water migration is obstructed, while frost damage of rock and shotcrete is not so pronounced. By contrast, slow freezing allows water migration to occur over a longer period, which results in greater frost damage of the material.

Walder and Hallet discovered that a crack with access to water during freezing exhibited the greatest growth at a slow freezing rate. At a more rapid freezing rate, the influx of water into growing ice bodies inside cracks was significantly inhibited. For example, during cooling from -1 °C to -25 °C, a crack with a 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 (Walder and Hallet, 1985).
The field investigations found changes to the freezing periods as well as their duration to be of major importance to ice formation growth. If the freezing period is of long duration, most of the cracks and the leakage spots freeze depending on the magnitude of the leakage. If the leakage of a specific magnitude is subjected instead to short periods of freezing and thawing, the water in the crack will never freeze and will continue to leak, resulting in ice formation growth. In cold areas, such as the north of Sweden, this problem occurs even far in the tunnels and does so because the leakage water transports heat from the rock mass to the cold tunnel wall. The heat content of the water keeps the rock around the crack openings from freezing despite sub-zero tunnel air temperatures. Hence, the leakage spot will continue to leak until a certain temperature and temperature duration is achieved, which results in ice formations when the water meets the cold tunnel air.

It is well known that water in rock in combination with freezing is a major degradation factor. For example, Matsuoka showed that rock samples subjected to freeze-thaw cycles deteriorate, especially when the sample had access to water during freezing (Matsuoka, 1990a). The results of the laboratory tests performed in this licentiate project also points in that direction with regard to deterioration in adhesive strength between rock and shotcrete. The tensile tests undertaken, showed that adhesive strength decreased when the shotcrete/rock samples were subjected to freeze-thaw cycles. The reduction of adhesive strength for the test samples was about 50%. Furthermore, acoustic emission (AE) measurements showed that more AE events took place when the shotcrete/rock panels had access to water during freezing. There was an obvious difference between the two different water situations, as when the panels had access to water the number of events was over 1600 as compared to 130, when water access was absent.

The adhesive strength of the shotcrete/rock interface in the test samples appears to be affected by the ice pressure that develops in the interface. This pressure derives from the 9% volumetric expansion of freezing water and during freezing from 0 °C down to -22 °C the expansion of ice is 13.5%. In a perfectly restrained body, the ice pressure increases almost linearly from 0 MPa at 0 °C to a maximum of 207 MPa at -22 °C (Tharp, 1987). Under normal conditions the maximum value of the ice pressure is almost certainly never reached or even approached. For example, 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, which seals
the pores and cracks in the rock (French, 1996). Fridh explained that this is one of the main destructive mechanisms for shotcrete itself, although this requires freezing to take place from all sides at the same time (Fridh, 2005). In a tunnel situation freezing takes place from one side. As a result the water is able to drain into unfrozen parts of the rock mass. Water can also drain into pores in the rock or shotcrete, which were initially filled with air, as long as the material is not completely saturated. If the material is saturated and the water is secluded (for example, by ice) it can not drain, which may damage the material during freezing. Chen et al. (2004) demonstrated how the initial degree of water saturation affected different rock properties. If the test samples had a high saturation, their properties deteriorate when subjected to freezing, but when water saturation was low, deterioration did not occur. Rock or shotcrete deteriorates because the volumetric expansion of water/ice exerts pressure within pores and cracks. If this pressure exceeds the tensile strength of the material, it will be damaged. A fully saturated material yields to frost action regardless of its strength, while a partially saturated material can resist this action even when its strength is low (Chen et al., 2004).

In the laboratory tests performed in this project, the samples were subjected to only approximately one winter season of freeze-thaw cycles. The result of the laboratory tests for panels nos. 7 and 4 indicated that reduction in adhesive strength was greater during the first cycles, since the number of AE events was greater in the first part of the test. However, the freeze-thaw cycles were relatively short compared to what actually occurs in a tunnel situation. If compared with panel no. 5, which was subjected to one long freezing cycle, the constant distribution of AE events over the freezing cycles indicated that a process was going on during the entire freezing test. For this long freezing cycle, the water was able to migrate towards the freezing zone during the entire test and thus AE events were spread out over time. This situation resembles a tunnel situation much better than the shorter freeze-thaw cycles do. For panel no. 7, the decrease in AE events after just a few cycles, might have been caused by the short duration of the freezing cycles. Although panel no. 7 had access to free water, ice growth in the interface was restricted because the freezing part of the freeze-thaw cycle was too short to allow the water migration to reach the interface. Therefore, the AE events decreased after just a few cycles. Hence, the duration of the freezing temperature is of great importance as indicated by the differing distribution of the AE events over the freezing cycles in the various tests.
6.2 Conclusions

The field investigations revealed that rock and shotcrete fall-outs often occur in areas with leakage problems. The results of the laboratory tests showed that more AE events took place in a panel, when it had access to water during freezing, and also that the adhesive strength between rock and shotcrete decreased after the test panels had been subjected to freeze-thaw cycles. The test samples showed that this reduction of adhesive strength after freezing was about 50%. These results endorse the hypothesis that access to water affects ice growth and ice pressure in the shotcrete/rock interface when it is exposed to freezing temperatures, which contributes to material deterioration. When the tunnel construction is subjected to water leakage, it must be protected from freezing to prevent the adhesive strength between shotcrete and rock to deteriorate. This is usually done by insulated drainage systems. However, it is difficult to determine the exact location for drainages during one particular inspection, because locations of leakage spots can change over the years. Therefore, based on experience gained from the field investigations undertaken in this project, the determination of location for drainages should be carried out after several inspections and especially after a winter period, when the actual problems with ice formations occur.

If there is good adhesion between rock and shotcrete, degradation due to frost shattering should not be a problem. But maintenance reports reveal that areas of poor shotcrete adhesion can appear and propagate rather quickly, when this area also has leakage problems. Therefore, the Swedish Rail Administration might need to revise their regulations according to safety and maintenance inspections, which regulates the time period between the inspections. The laboratory test of poor adhesion on a small area around a rock crack opening was unable to provide any obvious answers, although more events took place in the area that had been prepared with poor adhesion than in the areas of good adhesion. It is clear that a reduction in adhesive strength occurs in the saturated samples subjected to freezing, but the direct tensile tests could not explain how a small area of poor adhesion around a crack opening affects adhesive strength. This is probably due to the inaccurate test method, normal distribution of the results in combination with too few test samples.

The field observations have shown that problem with ice formations and fall-outs of rock and shotcrete appear along the entire tunnels, even in longer
tunnels (over 1000 m). In these long tunnels an even larger number of ice formations occur in the inner sections of the tunnel than in those around the tunnel entrances. Previous thinking about this problem was that ice formation occurred only at the tunnel entrances and outer sections. Therefore, one solution have been to install insulated drainage on the entire tunnel area over a distance of 200-300 m from each tunnel entrance, in an effort to complete eliminate the problem of ice formation. However, this is not an effective solution to the problem as the insulation does not only prevent the cold from reaching the leakage spot. It also prevents the heat from the rock mass from entering the tunnel and warming up the cold air. Consequently, the frost penetrates further into the tunnel than it would do if the heat from the rock mass were allowed to warm up the outside air on its way into the tunnel and thus move the problem of ice further along the tunnel.

6.3 Suggestions for further research

A comparison of the results of the direct tensile strength test revealed no great difference whether or not the panel had access to water during freezing. A possible explanation might be that the panels when tested were water saturated. Hence, water migration from the water-filled pores to the interface could have occurred, and ice growth could have caused damage during freezing, although one panel had no access to free water during the freeze-thaw test. One suggestion for further research is therefore to perform freeze-thaw tests on a complete dry panel. This condition never occurs in a tunnel situation, but it would be a good idea to test a dry panel in order to understand the problem of water migration.

Another suggestion is to test a panel and vary the temperature range and durations so as to simulate a variety of different situations. For example;

- more than one winter season — could possibly reveal whether deterioration increases when samples are subjected to more freeze-thaw cycles
- longer freezing temperature durations — so as to simulate a tunnel situation and to allow water to migrate over a longer period
- very low freezing rate — so as to examine whether water migration could cause a higher ice pressure at the shotcrete/rock interface.

Finally, it would be interesting to thoroughly link the field measurements and investigations to the theories and laboratory tests presented in this project.
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APPENDIX 1a: Temperature monitoring of test panel no. 5 – First test
APPENDIX 1b: Temperature monitoring of test panel no. 5 – Second test

Panel no. 5 - The second test, cycle 21 to 41

Temperature

Time

1A - rock surface
5A - interface
6A
8A - shotcrete surface
1B - rock surface
5B - interface
6B
8B - shotcrete surface
Cryostat
APPENDIX 2: Temperature monitoring of test panel no. 7
APPENDIX 3: Temperature monitoring of test panel no. 4
APPENDIX 4: Acoustic emission monitoring

By Ganesh Mainali

Acoustic emission (AE) was continuously monitored during the freeze-thaw test of the test panels using a Hyperion Ultrasonic System manufactured by the Canadian Engineering Seismology Group (ESG), which consisted of eight sensors, eight preamplifiers, an AE acquisition system and a computer. The acquisition system has a resolution of 14 bit at up to 10 MHz sampling rates. The Windows-based software allows for real time monitoring through eight channels and is capable of filtering acoustic emission events (AE events) by setting a threshold trigger value and the number of channels that need to be triggered. The ESG program, UltrACQ, was used for the real-time monitoring of the AE events for all the tests and was set to record all events that triggered more than four piezoelectric sensors.

AE measurements were achieved using eight piezoelectric sensors (R6α model) manufactured by Physical Acoustics Corporation (PAC). These have a diameter of 19 mm and are 22.4 mm high with a resonant frequency of 55 kHz and operate in the 35-100 kHz frequency range. The sensor operating temperature range is from -65 ºC to 175ºC and the sensors converted the waves detected into electrical signals, which were amplified by a 40 dB fixed gain 2/4/6 preamplifier and then sent to the ISA Analog/Digital boards for processing. Each waveform was digitised into 4,096 samples at a sampling rate of 5 MHz. The digitised signals were then stored in the computer and analysed later. Prior to AE monitoring of each test panel, UltrACQ was triggered once to ensure that the system and sensors were all working correctly. A pencil lead test was also carried out to ensure the coupling properties of the sensor and its sensitivity to the specimen and also check the accuracy of source locations.

All the sensors were mounted at eight different locations on the top surface of the test panels using silicon high vacuum grease as a coupling material. In addition to coupling, the sensors were kept stationary by using metal brackets and a piece of insulation material to hold them in position.