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Durability of low carbon concrete verified by various curing conditions for freeze-thaw testing

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Foreword

It is with great pleasure that I present this thesis on the durability of low-carbon concrete made with alternative supplementary cementitious materials (SCMs). The production of cement has long been recognized as a major contributor to greenhouse gas emissions, and reducing its carbon footprint is a critical challenge facing the construction industry today.

As a master's student at UiT Narvik, I have for my master thesis investigated how the incorporation of SCMs such as fly ash, silica fume, dolomite, and blast-furnace slag can improve the performance of concrete. By testing the durability of SCM concrete using the freeze-thaw (F-T) resistance test, chloride migration, and compressive strength, this research has the potential to uncover new, more sustainable options for concrete production.

I would like to thank Iveta Novakova for great guidance as my supervisor during this thesis, and Boy-Arne Buyle for help when needed. Lastly, I would also like to forward a thanks to Klevis Xhura for help with the physical labor.

Abstract

This study investigated the feasibility of producing low carbon concrete (LCC) using supplementary cementitious materials (SCMs) and evaluated their impact on concrete F-T durability. Variations in the curing conditions were done to see the effect it had on the F-T resistance. These results were then supported by compressive- and chloride migration testing. The performance of different concrete mixtures containing silica fume (SF), fly ash (FA), limestone powder (LP), dolomite (DO), and ground granulated blast furnace slag (GGBS) was evaluated.

The study also examined the limitations of current F-T testing methods and the challenges they face in assessing F-T durability. The results showed that concrete mixtures containing up to 47% SCM's performed well, especially if let cure for 56 days instead of the standard 28 days, with one mixture containing 20% dolomite outperforming the reference mixture on all tests performed on hardened concrete.

List of abbreviations

AEA Air-entraining agent

DO Dolomite powder

FA Fly ash

F-T Freeze-thaw

GGBS Ground granulated blast furnace slag

LCC Low carbon concrete

LP Limestone powder

OPC Ordinary Portland cement

SCM Supplementary cementitious materials

SF Silica fume

TBC Ternary blended concrete

W/B-ratio Water-to-binder ratio

1 Introduction

1.1 Background

Concrete is the most widely used construction material in the world, owing to its abundant availability, cost-effectiveness, durability, and high strength. However, its quality depends heavily on several factors, including the quality of the input materials, appropriate hardening conditions, and proper handling.

Humans have been using concrete for thousands of years. The essential ingredients of concrete, including sand, gravel (aggregate), a cementitious binder, and water, were mixed in ancient times, as evidenced by Egyptian and Roman constructions. The Romans, in particular, were experts in concrete and used it to create wonders such as the Pantheon in Rome, which features a 43.3-meter dome that still stands today, two millennia later [1].

However, after the fall of the Roman Empire, concrete technology declined, and it was only about 200 years ago that concrete re-emerged as a viable construction material. The concrete that we know today, with its most crucial ingredient, Portland Cement, was discovered in the early 1800s. It involves roasting and grinding limestone and clay into clinker, which is then ground into a fine powder, to which gypsum is added. By adding water and aggregates with Portland cement, regular concrete is formed through a hydration process.

1.1.1 Concretes environmental impact

Furthermore, with the rising awareness towards reducing CO_2 emissions, attention has been directed towards cement manufacturing which accounts for as much as 5-10% of global CO_2 emissions[2]–[4]. Such a large CO_2 footprint is caused by greenhouse gas emission both directly and indirectly in the production of cement. First, the heating of the limestone directly releases CO_2, while the need to burn fossil fuels to heat the kiln also indirectly contributes to CO_2 emissions. To put the numbers into context, one study found that for every ton of cement manufactured, about 800 kg of CO_2 is produced [4], this number however will vary depending on factors like what type of energy source is used for the heating of the kiln or whether partial replacement with supplementary cementitious materials (SCMs) are used in the cement production. Furthermore, according to estimates, approximately 4 billion tonnes of cement are produced annually [5].

According to a report on concretes sustainability performance, a CO_2 reduction of 29.3% was found between 2019 (79 kg CO_2/tonne of concrete) when compared to a 1990 baseline of 102.6 kg CO_2/tonne of concrete [6]. However, this reduction is largely due to cement manufacturers making their processes more energy efficient, using less fossil fuels to make cement. This reduction in CO_2 emissions per tonne cement is still not satisfactory, and therefore, some cement manufacturers like Heidelberg Materials in Norway are as of 2023 researching and building CO_2 capturing facilities which aims to reduce the CO_2 output of the cement production factories by as much as 50% [7]. The knowledge gained from projects like these will help immensely with incorporating solutions worldwide, thus leading to a significant reduction of the CO_2 footprint of concrete.

Another way to reduce the CO_2 emissions associated with cement production is by replacing a portion of the cement with SCM's such as fly ash (FA), ground granulated blast furnace slag (GGBS), dolomite powder (DO), limestone powder (LP), or silica fume (SF), which can sometimes improve the performance of concrete while also reducing the amount of cement required. However, this requires the cement manufacturers to take responsibility for this incorporation, which they have done with products like FA cements, GGBS cements and so on. However, an alternative to this is the method of incorporating SCM's directly into concrete while mixing, thus reducing the need for ordinary Portland cement (OPC) in the finished concrete product. This is what this thesis will be investigating.

1.2 Goal for research

The goal of this research is to investigate the freeze-thaw (F-T) durability, chloride migration and compressive strength of low carbon concrete (LCC) using specifically FA, SF, DO, and GGBS. As mentioned in the previous chapter, the production of cement is a significant contributor to global CO_2 emissions, which has led to increased interest in reducing the carbon footprint of concrete. The use of SCMs has been shown to be an effective way to reduce the amount of cement required in concrete, and its carbon footprint. Since these materials are often waste materials from various industries, finding ways to effectively use them in building materials are often good environmental actions on its own as it.

1.3 Research objectives

The specific objectives of this research are:

- To investigate the effect of FA, SF, DO, and GGBS on the durability of low carbon concrete.
- To compare the performance of low carbon concrete with conventional concrete in terms of freeze-thaw resistance, compression strength and chloride migration resistance.
- To investigate alternative curing methods to the CEN/TR 12390-9 standard for freeze-thaw testing and determine if various curing conditions affect the performance of SCMs in terms of freeze-thaw resistance.
- To provide recommendations for the optimal use of SCMs in low carbon concrete to achieve desired freeze-thaw durability and other performance characteristics.

1.4 Significance of research

The significance of this research lies in its potential to contribute to the development of more sustainable construction practices by reducing the carbon footprint of concrete without sacrificing performance significantly. The use of SCMs such as FA, SF, DO, and GGBS can lead to significant reductions in CO_2 emissions associated with concrete production. By investigating the freeze-thaw durability of low carbon concrete containing these materials and exploring alternative testing methods, this research aims to provide valuable information for the design and use of sustainable concrete mixtures in construction projects - especially in harsh environments with high number of freeze-thaw cycles. Additionally, this research can contribute to the growing body of knowledge on the use of SCMs in concrete and provide guidance for their optimal use in achieving desired performance characteristics.

2 Theory

2.1 Supplementary cementitious materials

Supplementary cementitious materials are being widely researched at the time of this paper as a means of reducing the carbon footprint of concrete while also utilizing waste materials. SCMs are often waste materials from various industries, and because of their ability to exhibit cement like properties, they can partially replace some of the ordinary Portland cement in concrete. But different types of SCMs can have drastically different properties and chemical compositions.

Some SCMs are pozzolans; these materials are defined as siliceous or siliceous and aluminous materials that have little or no cementitious properties in themselves; but, when finely ground and in contact with calcium hydroxide and water (which is produced in the hydration process between OPC and water), they will chemically react [8]. In the case of OPC, the products formed by the reaction with water are calcium silicate hydrate (C-S-H gel) and calcium hydroxide ($Ca(OH)_2$). The C-S-H gel is the desirable product which provides strength, while the calcium hydroxide is undesirable because it provides little strength [9]. Therefore, pozzolanic SCM's consumes the undesirable calcium hydroxide and produces additional C-S-H gel and C-A-S-H or C-A-H. This pozzolanic reaction is the underlying reason why SCMs contribute to durability and can improve strength. Examples of pozzolanic SCM's are fly ash, silica fume and metakaolin.

Other SCM's may have hydraulic properties, this means that the material can chemically react with water to form cementitious compounds, which can harden and set like hydraulic cement. GGBS is an example of a hydraulic SCM. This chapter will delve deeper into some of the different types of SCMs available and used in this thesis, their benefits, and applications.

While SCMs can be a useful tool in reducing CO_2 emissions associated with cement production, it is important to note that it is not a one-solution fix-all. Instead, finding waste materials that are locally available is also key to reduce the emissions and costs of transportation. This can include using industrial by-products from nearby power plants or steel mills as SCMs. Additionally, with the use of local waste materials, a reduction in the amount of waste that is sent to landfills is also reduced.

2.1.1 Fly ash

Fly ash is a commonly used SCM in concrete and is produced by dust-collection systems that remove particles from the exhaust gases of power plants that burn pulverized coal. FA consists mostly of small spheres of glass of complex composition involving silica, ferric oxide, and alumina [10]. When used in concrete, FA can replace a portion of the cement, typically ranging from 15-25% while maintaining or even improving the strength, workability and durability of the concrete [11].

Two types of FA are commonly used in concrete: Class C and Class F. Class C are often high-calcium fly ashes with carbon content less than 2%; whereas Class F are generally low-calcium fly ashes with carbon contents less than 5% but sometimes as high as 10%. In general, Class C ashes are produced from burning sub-bituminous or lignite coals and Class F ashes bituminous or anthracite coals. Performance properties between Class C and F ashes vary depending on the chemical and physical properties of the ash and how the ash interacts with cement in the concrete. Many Class C ashes when exposed to water will react and become hard just like cement (hydraulic effect), but not Class F ashes.

Low calcium fly ash (Class F) is a very fine dust, mainly with spherical grains, presenting pozzolanic properties [12]. Thermodynamic calculations indicate that for complete consumption of the calcium hydroxide, approximately 35 wt% of the total binder mass have to be replaced with class F fly ash [13].

Pros of using FA is first and foremost that it is finer than OPC and can therefore inhabit smaller voids than cement particles, which are normally inhabited by water; this in turn leads to a reduction in water which improves strength. Furthermore, it aids in creating a stronger concrete with less permeability and better workability [14], which helps with durability factors like resistance to chloride intrusion. Because some FA contains larger or less reactive particles than OPC, significant hydration can continue for up to six months or longer, leading to much higher ultimate strength than concrete without fly ash [15].

According to literature on the use of FA in concrete, there is a consensus that the use of fly ash in high volumes can impact the concrete negatively. These effects are mainly longer setting time which results in lower early strength (to a great degree at 1 day) and reduced durability, especially with regards to F-T resistance and carbonation [11], [12], [16]. However, an important factor to consider is that concrete is usually tested at 28 days. For many FA concretes, a longer curing period will often make a stronger and more durable concrete than a regular OPC concrete. Testing FA concrete at 56 days instead of 28 days may therefore show critically different results.

There is additionally a trend of phasing out coal-powered energy plants and replacing them with greener power sources. This will have an impact on the availability of FA – especially in Europe. Therefore, investigating alternative SCMs are necessary for the concrete industry to have viable replacements for FA in concrete production in the future.

The Norwegian annex in NS-EN 206 [17] recommends that the total amount of FA cannot exceed 35% of the total binder mass. In addition, if over 20% fly ash is used for concretes that requires frost resistance, the F-T resistance also needs to be documented.

2.1.2 Ground granulated blast furnace slag

Ground granulated blast furnace slag (GGBS) is a by-product in the production of pig iron in a large blast furnace at a temperature of 1300-1500 °C. It is formed by melting the waste rock of iron ore, flux (limestone, dolomite, etc.) and inorganic parts from the combustion of fuel (coke). Slag, being a material of lower density, flows out over the surface of molten iron in the blast furnace and is removed at regular intervals. The slags temperature at the blast furnace output reaches about 1400-1450 °C and is then cooled slowly by means of air or quickly by waterjet. The latter process is called granulation and leads to the formation of the product called GGBS.

GGBS is commonly used as a supplementary cementitious material in concrete. This is done either by adding the GGBS separately when mixing the concrete or using cement that already from the factory have a certain percentage of its clinker content replaced with GGBS. GGBS have glassy and crystalline phases; the glassy phases consist of alumina-silicates of calcium, which are responsible for the cementitious properties of GGBS. Activation of GGBS is necessary for the hydration reaction (production of C-S-H gel) in GGBS concrete, and because activation of GGBS is possible only with other alkali materials or OPC, complete replacement of cement with GGBS is not possible. The formation of hydrates leads to a pore blocking effect, and this effect is the main reason for the chemically stable and hardened concrete [18]. It is found through thermodynamic calculations that for the complete consumption of calcium hydroxide ($Ca(OH)_2$) into C-S-H gel the required ratio between GGBS to total binder content is approximately 75wt%.

GGBS is commonly known for properties such as high durability, high resistance to chemical attacks, chloride migration, low permeability, high sulphate resistance, lower CO_2 footprint and reduced shrinkage and cracking when used in concrete [19]. GGBS can therefore be used to replace a portion of the cement in a concrete mixture, typically up to 50%. However, a study found a replacement ratio up to 70% possible without causing problems with the resulting concrete [20]. Another study done by Prasanna (2021) [21] found the compression strength of GGBS concrete up to 40% replacement ratio to be stronger by a small amount (<2%) at 28 days compared to concrete only using OPC. However, the same study showed that when looking at 90 days results compression strengths, up to 80% replacement ratio of GGBS was only 5% weaker than the reference OPC concrete.

However, GGBS must be properly ground and blended with cement to ensure optimal performance [22]. The fineness of GGBS is also an important factor to be considered, as it affects the rate of the pozzolanic reaction, which in turn affects the strength development of concrete. It is well documented that the early strength of the GGBS-blended concrete is negatively affected, but the strength of the concrete improves at later stages, usually matching the compressive strength of OPC concrete at 56 days onward [22], [23].

In ASTM C 989, GGBS are classified into three grades (80, 100, and 120) based on their respective mortar strengths when blended with an equal mass of OPC. Grade 80 has a low activity index and is used primarily in massive structures because it generates less heat than OPC. Grade 100 has a moderate activity index, is like OPC with respect to cementitious behaviour and is readily available. Grade 120 has a high activity index and is more cementitious than OPC [24].

According to the Norwegian annex in NS-EN 206 [17], the maximum recommended replacement ratio of GGBS to total binder mass is 80%.

2.1.3 Dolomite

Due to the scarceness of high-quality limestone as required for CEM II Portland-limestone cements, other carbonate sources, like dolomite ($CaMg(CO_3)_2$), are in the focus as alternative mineral replacement for cement clinker. However, DO is assumed to undergo the so-called dedolomitization reaction in high-pH environments. In this reaction, dolomite reacts with calcium hydroxide ($Ca(OH)_2$) to form calcium carbonate (calcite) and magnesium hydroxide (brucite) as shown in equation 1 [25].

$$CaMg(CO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3 + Mg(OH)_2 \quad (1)$$

$CaCO_3$ affects early strength positively in concrete and adding magnesium powder reduces the overall water to binder (w/b) ratio of the concrete and increases the workability [26]. DO powder being very fine (i.e., < 90 µm) it turns out to be a good replacement for OPC up to certain percentages. In a study done by Gusain (2022) [26] different percentages of the OPC were replaced with DO. The result from this study is illustrated in Figure 1 where it can be observed that a replacement ratio up to 15% increases the compressive strength, but above 15% and up to 30% a decreasing trend can be observed. However, all mixtures above 15% exhibits a higher compressive strength compared to the reference mixture where 0% DO was used. This same study also found the flexural strength of the concrete to be improved by adding dolomite, and the same peak at 15% replacement ratio was found.

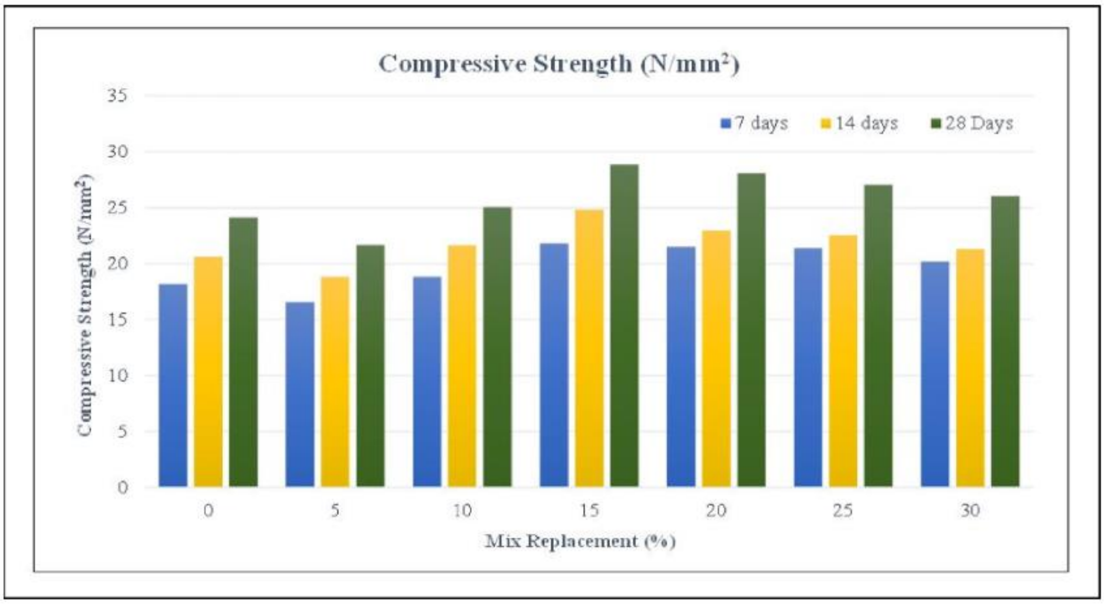

Figure 1 Compression strength of mixtures containg various amounts of dolomite, Gusain (2022)

In another study done by Dahme (2019) [27], a maximum compressive strength was found when 10% of OPC was replaced with DO, with a decreasing trend in compressive strength found when increasing the DO content up to 20% replacement ratio. Even though this optimal replacement ratio is quite small, the author concluded that this was still a step forward as dolomite is both freely available and cheaper than OPC.

However, when considering the effects of DO on concrete, the particle size of DO after grinding is crucial for how it reacts when used in concrete. In a review study performed by D. Wang et al. (2018) [28] it was found that dolomite from quarry waste *reduced* the workability of the concrete, while DO from direct grinding *increased* the workability. This difference was contributed to the fine particle size and smoother surface morphology of the direct grinded powder compared to the coarser fineness and rougher surface morphology of the quarry waste.

Challenges posed by the addition of limestone in cement include potential contamination by clay or other organic materials, which can have a negative impact on the F-T resistance of concrete, as well as the concrete's resistance to chemically aggressive environments. Addition rates above 5% can also lead to lower long-term strengths and raise the risk of steel reinforcement corrosion [29].

There are no recommendations in the NS-EN 206 standard for dolomite. However, due to its similarities to limestone powder, the same recommendations may apply. The maximum replacement ratio of limestone powder to total binder mass is 5%. However, this thesis will show that a much higher replacement ratio of DO to total binder mass can go as high as 20% and still produce a superior concrete over the reference mixture when tested for compressive strength, freeze-thaw, and chloride migration.

2.1.4 Silica fume

Silica fume, also known as micro silica, is a fine, powdery substance that is a by-product of the production of silicon metal or ferrosilicon alloys. SF is known for the high content of amorphous silicon dioxide (SiO_2) and consists of very fine spherical particles which can be up to 100 times smaller than OPC particles [30]. Small amounts of magnesium, iron, and alkali oxides can also be found in SF [31]. Because of these properties SF has through many studies been found to be beneficent when used as a SCM in OPC concrete. A normal replacement ratio of OPC replaced with SF is often in the range 5-10% [32].

Pros of using SF as a partial replacement in OPC concrete can be [31], [32]:

- High early compressive strength
- Heigh tensile, flexural strength, and modulus of elasticity
- Improvement of ITZ zones
- Enhanced durability
- Very low permeability to chloride and water intrusion
- Increased abrasion resistance
- High resistance to chemical attack from chlorides, acids, nitrates, and sulphates
- Low permeability

- Higher F-T resistance

The reason for so many pros when utilizing SF in concrete is because of the material's extreme fineness (<1 μm) and very high SiO_2 content. This leads to three separate mechanism taking place when used in combination with OPC concrete:

- Pore size-refinement and matrix densification.
- Cement paste aggregate ITZ refinement.
- Reaction with free lime ($Ca(OH)_2$) from the hydration process.

However, because the high surface area of SF, the amount of water needed in the mixture is increased, so it is recommended to use it along with superplasticizer (SP) in order to reach the desired workability. Another disadvantage to utilizing silica fume in cement is the cost. To compare, SF costs $400-$1,000/ton, which is significantly higher than OPC which costs about $90/ton [33].

Standard specifications for SF used in cementitious mixtures can be found in ASTM C1240 and EN 13263. The author of this paper was not able to get access to these documents. The amount of SF is regulated in the Norwegian annex in NS-EN 206 [17], the maximum recommended replacement ratio of SF to total binder mass is 11%.

2.1.5 Ternary blended concrete

As the previous chapters show, there are often some negative consequences by utilizing SCMs in concrete. However, one potential solution to this is to adapt ternary blended concretes (TBC's). TBC's is typically defined as concrete containing OPC with two other SCMs such as FA, GGBS, SF and DO. TBC has the potential of significantly improving the performance of OPC concrete or binary blended concrete by combining the benefits of each SCM, and simultaneously minimizing the adverse effects on fresh and hardened concrete [34].

As previous chapters have shown, the use SF as a partial replacement of OPC in concrete is limited due to its high cost and increased water demand to get the right workability. However, if FA and SF are used together, the negative effect SF have on workability can be mitigated by FA positive contribution to workability. Further, SF can improve the early strength of concrete which may be important for certain concrete applications, therefore reducing the negative effect that FA have by its retarding effect on concrete.

Chang-Seon Shon (2018) [34] looked into making TBC with FA and SF, an introduction of 5% SF into concrete managed to make a F-T resistant concrete with a 45% content of FA in cement mass, this was not possible without the SF. This result was mainly contributed by the author to the effect SF has on concrete by reducing the pore size, thus making the freezing-point lower and the concrete less permeable [35].

In another study on the use of TBC containing various amounts of GGBS and SF done by A.R. Bagheri et al. (2012) [36], results showed that simultaneous use of SF has only a moderate effect in improving the slow rate of strength gain of binary mixes containing low reactivity GGBS. However, it improves the durability considerably. By using an appropriate combination of low reactivity GGBS and SF, it is possible to obtain TBC mixtures with 28-day strength comparable to the control mix as well as improving the durability particularly in the long term. Ternary mixes also have the added advantage of reduced water demand.

Although recent interest in ternary blended concrete and studies on their use, little data is available on the effect of using ternary blended concretes containing various proportions of OPC, SF, FA, GGBS, DO regarding F-T resistance. This thesis will therefore look into the F-T resistance of TBC mixtures and assess their overall durability when compared to a reference mixture.

2.2 Cold climates

In cold regions of the world where the temperatures go sub-zero and freezing occurs, F-T degradation is the biggest contributor to concrete failure [37]. This physical deterioration is caused, by two processes: F-T cycles and salt induced scaling [38]. In short, according to previous research, the F–T failure of concrete structure mainly includes three stages: water absorption, water freezing and structural failure. As external water now can enter the concrete through these previous F-T cycle damages, the risk of steel bar corrosion is higher, and more F-T damage will accumulate with continual F-T cycles. More on this in chapter 2.3.

Since concrete is naturally porous and the weather around the world varies, its resistance to F-T damage depends on several factors which can be split into two different types. It is outside the scope of this thesis to expand upon every factor mentioned in this section, it will however be showed how some of these factors have an effect through a short case study. First there are environmental factors that depends on the location; this includes rate of cooling, minimum freezing temperature, minimum temperature duration, number of F-T cycles, presence of chlorides and water saturation [39]. The second type of factors that affect F-T resistance are the characteristic properties of the concrete which includes w/b-ratio, age, type and distribution of aggregates, binder composition (SCM's) and distribution and size of pores and capillaries.

To expand upon the environmental factors, the effect of the rate of cooling and duration on minimum freezing temperature were looked at by Jacobsen et al. (1997) [40]. In this study they measured internal cracking by resonance frequency and scaling damage with the Borås test. It was found that for air-entrained concrete, a rapid rate of cooling resulted in more internal cracking than a slow rate of cooling but did not affect scaling noticeably. For the same concrete, a prolonged duration at minimum freezing temperature did not affect the internal cracking or scaling noticeably.

Another important environmental factor that will affect the amount F-T degradation in concrete is the minimal freezing temperature. Most laboratory F-T tests uses -20° C as the minimum temperature, but in most countries where F-T degradation is a problem for concrete; temperatures rarely or never reach -20° C. Therefore, in a study done by Gehlen et al. (2012) [41] it was found that increasing the minimum temperature from -20° C to -10° C reduces the scaling rate close to 50 %. Another important finding from the same study was the effect initial moisture content of the concrete had on F-T resistance. It was found that a reduction of the initial moisture content leads to a postponed evolution of F-T damage. Additionally, the same study investigated the effect of intermediate dry periods in between F-T cycles. Here it was found that the scaling rate after an intermediate dry period is reduced by about 20 %.

Moving on to the concrete properties that affects F-T durability, the w/b ratio is one of the key factors that can significantly affect the freeze-thaw durability of concrete. When the w/b ratio is high, more water is in the mix, which can result in more capillary pores in the hardened concrete (more on damage mechanisms in chapter 2.3). These capillary pores can absorb more water during freeze-thaw cycles, leading to internal pressure and damage to the concrete. In addition, the high w/b ratio can also result in a weaker, more porous concrete matrix, which is more susceptible to damage from F-T cycles. On the other hand, when the w/b ratio is low, there is less water in the mix, which can lead to a more dense and stronger concrete matrix. This can reduce the number and size of capillary pores, and hence, reduce the potential for F-T damage. Several studies have shown that lower w/b ratios generally result in better F-T durability of concrete. However, the optimal w/b ratio for F-T resistance may vary depending on several other factors such as the type of cement, aggregate characteristics, and curing conditions. Figure 2 illustrates how the capillary pores increases as the w/b ratio increases.

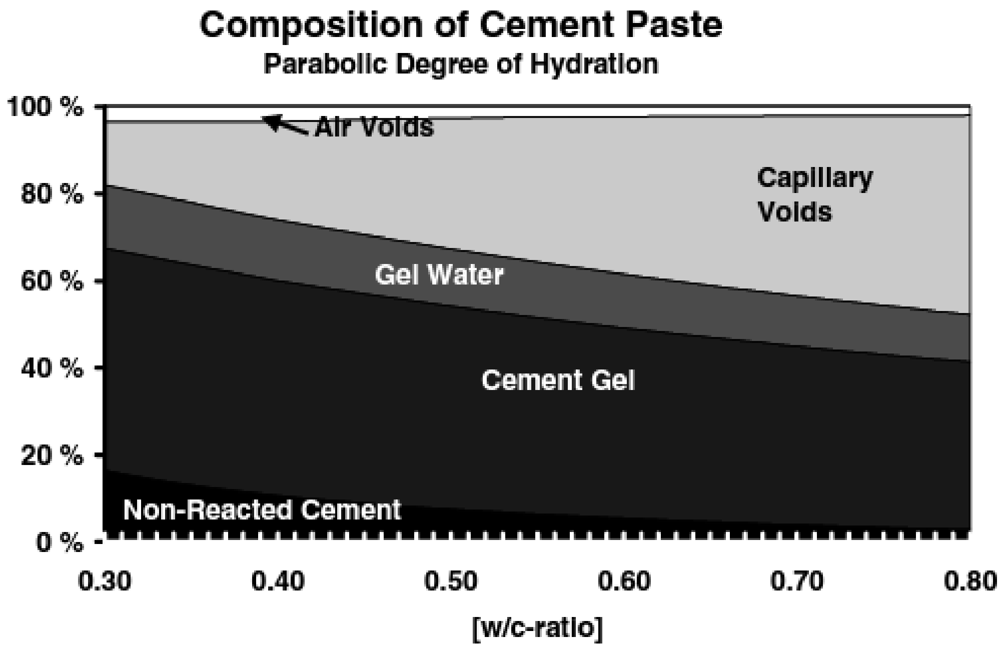

Figure 2 The effect of w/b-ratio on pores, Rønning (2001)

The Norwegian annex in NS-EN 206 recommends a maximum w/b ratio of 0.45 for concrete that are subjected F-T environments. This means that the amount of water used in the mixture should not exceed 45% of the amount of cementitious material used.

The age of concrete is another concrete property that has been found to have a significant effect on its F-T resistance. As concrete cures, the strength and durability of the material increase. Therefore, concrete that has been allowed to cure for a longer period generally exhibits better resistance to F-T cycles. This thesis will show that curing age affects how LCC withstands F-T attacks significantly. The reason being that today's F-T tests have been using OPC concrete when making the tests, which generally have 90% of its strength at 28 days. However, when using SCM concrete, reaching 90% of its final strength may take considerably longer. Therefore, it is important to consider the curing age of concrete when evaluating its F-T durability; especially when utilizing SCMs that have slower strength development.

One more concrete property that largely affects the F-T resistance is the presence of well-dispersed air-pores in a certain diameter. It has been found that at least 3 percent of air, by volume, in the fresh concrete is necessary to protect concrete from freezing and thawing. However, this required air content depended on the paste content, which is largely a function of aggregate size and gradation and of minimum cement content requirements. Therefore, 3 %

air per unit of concrete volume may be sufficient for a mild exposure classes but not for a harsh exposure classes [42].

Today, chemical air-entraining agents (AEAs) are almost always used as a common and popular measure to mitigate F-T damage to concrete. AEAs works by stabilizing the air bubbles generated during concrete mixing and prevents tiny bubbles from collecting and escaping. These small, well-dispersed micro-bubbles allows pore water to flow during F-T, thereby reducing the internal stresses in the concrete due to hydrostatic pressures. Figure 3 depicts the mechanism of action of air bubbles introduced by the AEA. The initial assumptions are shown in Figure 3a depicts an idealized system of water-free concrete specimen incorporated with AEA, where the solid space is represented by the light grey area and the pores by the dark grey area, consisting of capillary pores and tiny air bubbles. When the sample is immersed in water, as shown in the Figure 3b, the water enters the pores and fills them. The capillary tube will quickly be filled with water due to capillary suction. The saturated pores are represented in blue. The dark grey pore section in Figure 3c indicates that when the concrete suffers F-T damage, its pore volume can absorb the flowing water moved by the ice growth, acting as a release zone for the water pressure [43].

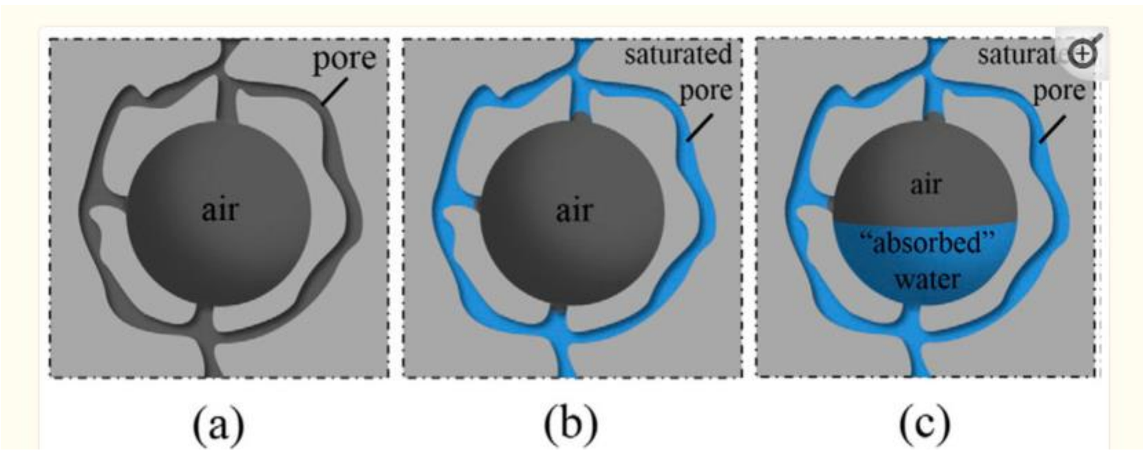

*Figure 3 Schematic depicting capillarity and absorption of water by air diffusion: **(a)** Before immersion in water; **(b)** After immersion in water; **(c)** During F-T, Luo (2022)*

The spacing factor of the bubble network is also essential for F-T damage mitigation efficiency. A commonly accepted air-void spacing factor of 0.20 mm or less is usually good enough for the concrete to withstand F-T degradation and is more conservative than several early research studies that reported that a spacing factor of approximately 0.25 mm or less signified adequate freeze-thaw resistance [42].

However, air content in concrete containing chemical AEAs is highly variable and influenced by temperature, transport time, mixing, pumping, and internal vibrations [44]. This can be a problem when the method of measuring air-content in concrete is taken into consideration. The standard norm is to use the pressure method in EN 12350-7. Unfortunately, these methods provide only a measurement of the total air volume, not the size or distribution of the air voids. Furthermore, these tests are often performed before the completion of construction operations (such as placing, consolidating, and finishing) that can alter the air void system. Therefore, the actual in-place air content in hardened concrete and other air void system parameters may differ significantly from those in the fresh concrete. Chapter 2.4 mentions some of the laboratory tests that are used to assess the spacing factor and air content more accurately than the pressure method.

Figure 4 illustrates an overview over some of the factors that may influence air-content in concrete.

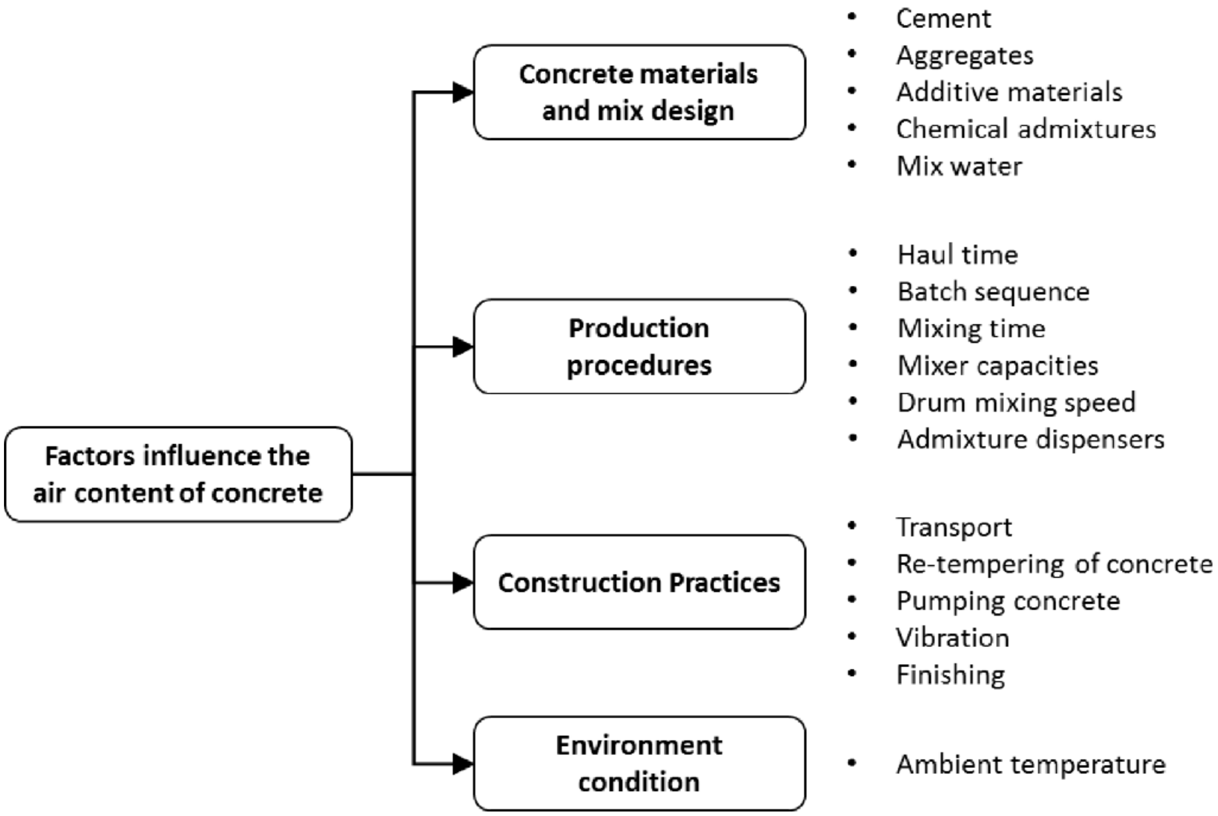

Figure 4 Factors influencing the air content, Whiting and Stark (1983)

2.3 Freeze-thaw degradation

The deterioration of concrete microstructures in F-T cycles is the primary reason for the reduction in the service life of concrete. This chapter will investigate the different theories of the damage mechanisms and some of the damage models of concrete exposed to F-T cycles.

2.3.1 Hydraulic pressure theory

The types of pores in concrete are classified based on their pore size into gel pores, capillary pores, and air voids. These pores are not mutually exclusive, and they contain different types of water, such as bulk water, capillary water, gel pore structure water, and gel pore adsorption water. In natural conditions, bulk water and capillary water can freeze. However, saturated gel pore water can only freeze at extremely low temperatures between -30°C to -80°C. Therefore, the variation in freezing points among these different types of pores is considered the primary factor contributing to frost damage in concrete [45].

When concrete is exposed to freezing temperatures, the water present in the pores on its surface freezes, leading to a 9% volume expansion. This expansion causes the liquid water in the capillary pores to migrate, and as the temperature decreases, the volume of ice continues to increase and compress the liquid water. This leads to compressive stress in the pores and tensile stress in the concrete, causing capillary water flow that is proportional to the stress produced in the concrete. As the tensile stress increases, microcracks occur inside the concrete, ultimately leading to its failure. The hydrostatic pressure in the pores is determined by the pore length, permeability coefficient, icing amount, and cooling rate. Concrete with a high icing rate, small permeability, and long pore size is more vulnerable to F-T damage. The hydraulic pressure theory is the most widely accepted explanation for F-T degradation in concrete, highlighting the importance of a well-distributed air void system in reducing the maximum distance of water transport from any location, thereby relieving the pressure [45].

Figure 5a illustrates hydraulic pressure in a pore, and how the air void helps to reduce the pressure created.

2.3.2 Osmotic pressure theory

Numerous experimental studies and observations in real-world applications have shown that the presence of salt solutions in concrete significantly increases the severity of F-T damage compared to pure water. In 1953, Powers et al. [46] proposed the osmotic pressure theory, which explains the phenomenon. The theory suggests that concrete contains salt ions, and when macropores and capillaries freeze, the concentration of salt solution in the unfrozen pores increases. This, in turn, causes the supercooled water in the gel pores to migrate. Due to the complex pore structure of concrete, the migration permeability rate varies, leading to osmotic pressure that damages the interior of the material (as depicted in Figure 5b). Moreover, the saturated vapor pressure of water is higher than that of ice, which generates osmotic pressure when unfrozen water moves to the ice. The concentration of salt solution has a direct relationship with the osmotic pressure, whereas the amount of ice is inversely proportional to it. The coupling of the two pressures generates a maximum value at a certain concentration of around 3% NaCl concentration.

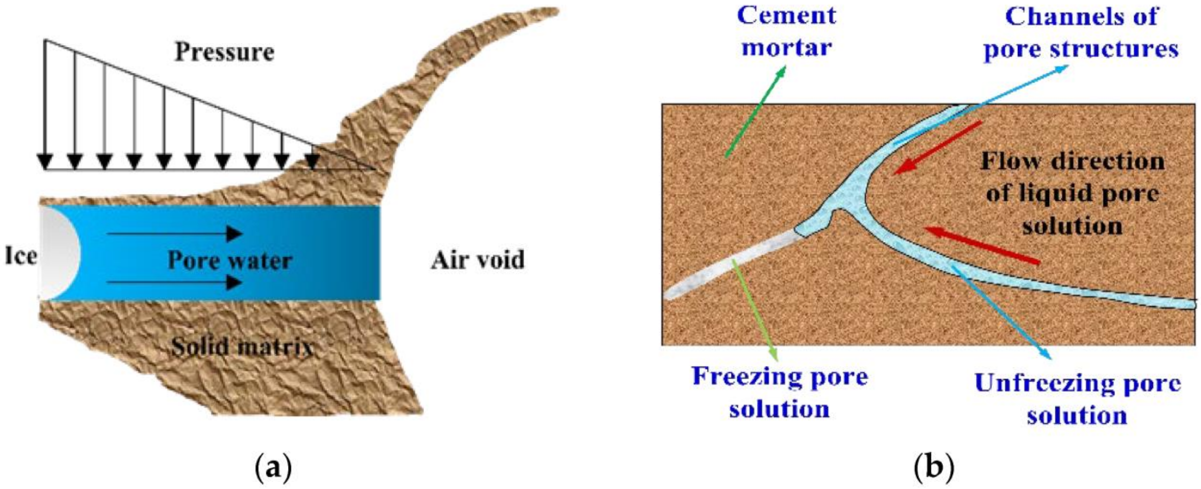

Figure 5 hydrostatic pressure and osmotic pressure model. ***(a)*** *Hydrostatic pressure principle ;* ***(b)*** *Osmotic pressure principle, Guo et al. (2022)*

2.3.3 Temperature shock

The thermal shock theory of F-T damage in concrete suggests that the physical expansion and contraction of water when it freezes and thaws in concrete causes internal stresses that lead to cracking and deterioration of the material. When water freezes, it expands, putting pressure on the surrounding materials. When the water thaws, it contracts, which can cause the material to pull apart and form cracks. This process is known as thermal shock. The repeated cycles of freezing and thawing in concrete can cause significant damage to the material, especially if the concrete is porous or contains voids that allow water to enter and freeze. The thermal shock theory of F-T damage is one of the most widely accepted theories for explaining the degradation of concrete in cold climates [47]. The principle is illustrated in Figure 6.

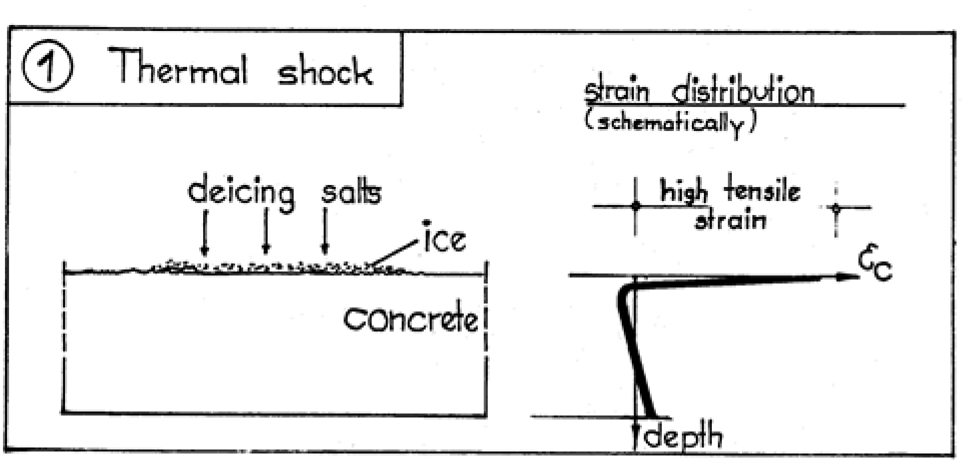

Figure 6 Thermal shock to a concrete surface layer, caused by application of a de-icer to a frozen surface covered by ice, Rønning (2001)

2.3.4 Synopsis

The process of F-T degradation in concrete is a complex phenomenon that involves multiple mechanisms simultaneously. While the hydraulic pressure theory, osmotic pressure theory and the thermal shock theory have been proposed to explain some aspects of this degradation, it is likely that there are other mechanisms at play as well. The interplay of these different mechanisms and their combined effects make it challenging to fully understand and predict the behaviour of concrete under F-T cycles. Depending on the environmental factors and concrete properties, different mechanisms may cause severely different F-T degradations. Therefore, it is important that the researchers working to understand these degradation mechanisms also understand that it is not as black and white as saying that one mechanism stands for all the degradation.

Rønning (2001) [47] states very clearly the issue of having multiple deterioration mechanisms, and how they the different mechanisms often will work simultaneously at any given time during F-T cycles.

> *There is evidently no single deterioration mechanism that can account for the damage - or resistance to such – by concrete subjected to freeze-thaw, with or without the presence of de-icing salt. Nor have any of the single mechanisms described above been rejected by convincing arguments, nor established clearly.*
>
> *Obviously depending on the actual, environmental - as well as material parameter in the specific case and exposure, several of these simultaneously and partly*

counteracting mechanisms will play a role. The key action in material testing, material design and future product development under such circumstances must be to conduct studies, enabling the (potential) mechanisms to act as closely related to those of field exposure as possible.

2.4 Freeze thaw tests overview

Finding good ways to determine the concretes F-T durability is a challenging task because of the complexity the F-T degradation presents. Because F-T degradation may take decades to turn severe in real world environments, accelerated tests to determine F-T resistance of any given concrete mixture is necessary. There are two primary European standards for F-T testing of concrete are used today for assessing a concrete mixtures freeze-thaw resistance.

These are:

1. CEN/TS 12390-9 "Freeze-thaw resistance with de-icing salts — Scaling"
2. CEN/TR 15177 "Testing the freeze-thaw resistance of concrete - Internal structural damage".

Both standards mentioned above involve the so called direct methods. Direct methods subject samples to repeated freezing and thawing conditions usually by having concrete specimens in a climate chamber. Normally a salt solution is used on top of the samples to simulate real environments where free water freezes and thaws on top of the specimens. However, as mentioned above all direct tests are accelerated tests which removes most of the real-world factors which may also have an influence on freeze-thaw degradation, and the environmental load that age and other conditions have on the concrete will therefore not occur. This risk is present in all of the direct methods that exists today [48].

For the CEN/TS 12390-9 (hereby abbreviated to CEN 12390-9), three methods are proposed for assessing scaling resistance under F-T attacks. These are:

1) Slab test (Reference method)
2) Cube test (Alternative method)
3) CF/CDF test (Alternative method)

In short, the difference between these methods are how the concrete slabs are placed in the climate chamber, the rate of cooling, the duration at which minimum temperatures are kept, thawing conditions, as well as some other minor differences.

In the CEN/TR 15177 standard elastic-wave methods are applied to estimate the frost damage non-destructively. In the slab test according to CEN/TR 15177 length change measurement by extensometer is the reference measurement procedure but there are also two alternative elastic-wave methods for that. One alternative method is based on the ultrasonic test, which measures the transmission time of the longitudinal wave (P wave, Ultrasonic pulse transit time, UPTT), while the other alternative method measures the resonant frequencies of vibrations (Fundamental transverse frequency, FF).

Having looked at some of the direct methods, there are also what's called indirect methods. The idea in indirect methods is to measure properties that is affecting F-T degradation of concrete. These indirect tests usually take less time than direct methods to perform, and normally give acceptable accuracies for the assessment of F-T degradation. However, the risk when doing an indirect test for F-T degradation is that it focuses on one property at the time, and other properties which may have an even greater effect on F-T degradation may not be measured.

Examples of indirect methods can be optical thin and plane sections and air-pore analysis. In optical thin and plane sections, the idea is that because of the porous nature of concrete and cement paste, it is possible to fill the voids and pore spaces with resin (epoxy) that has been mixed with a fluorescent dye. As fluorescent epoxy is in original voids and cracks in the concrete, including the large capillary pores of cement paste, this method strongly highlights the presence of all kinds of pores and cracks in cement paste [48].

In optical concrete air pore analysis performed on hardened concrete, the main determined parameter is the spacing factor (L). It is an attempt to calculate the fraction of paste within some distance of an air void (paste-void proximity). Also, other parameters can be determined, and are also used for the calculation of Powers´ spacing factor, or for the evaluation of concretes air pore structure, specific surface area of the air pores, amount of air pores in different size classes i.e., air pore size distribution or volume of air pores below some size [48]. There are multiple standards for this test, but the most used one in Europe is EN 480-11 and NT BUILD 381.

There are many other indirect methods, but this thesis will not look further into these as this thesis focuses on the direct method given in CEN/TR 12390-9.

2.5 Problems with today's F-T tests

As mentioned in chapter previous chapters, many different factors are going to affect how any given concrete will resist F-T degradation. It is therefore of crucial importance to note the conclusion in the report done by Kuosa (2013) [48] on the different F-T tests:

> *[…] the selection of a suitable test method depends on the target of research or quality control. There is no "right" freezing and thawing resistance test method. All of them have been made for their own purpose and use in any other purpose must be done in great care. It must be kept in mind that the criteria for durability is also greatly dependent on environmental loading and test methods used.*

The conclusion drawn from this study highlights the significance of addressing the challenges associated with F-T testing of concrete. As shown previously in chapter 2.2, environmental factors like minimum freezing temperature, water saturation, minimum temperature duration and rate of cooling will affect how the concrete degrades due to F-T exposure. Tests that more accurately replicates the actual environment and location that the concrete is going to be put in, will most likely make concrete mixtures containing SCM's usable at least in some F-T environments. Afterall, it is not logical that concrete structures that throughout its life cycle never see temperatures beyond -10°C and often have intermediate dry periods in between cycles shall have to endure -20°C and constant water saturation like the CEN 12390-9 requires it to do to pass as a F-T resistant concrete.

This point is well put in a study by Gehlen (2012) [41]:

> *Neither the role of dry periods nor the variety of existing freeze-thaw-loads (e.g., the majority of F-T in Europe is within a temperature range from +5° c to -5° C) is considered in most test methods. Consequently, the transferability of current laboratory results to field performance of concrete structures exposed to free weathering conditions without continuous moisture should be further investigated.*

Investigating various ways to test F-T degradation in concrete, and making tests which replicates field performances more accurately is going to be crucial going forward so that a broader use of SCM concrete can be proven to work also in F-T environments. It is well overdue to find these improved methods, and as Rønning [47] already said back in 2001:

> *It is no secret that freeze-thaw testing in accordance with various standardised or non-standardised test methods may destroy any concrete material. The phrase "in the lab, we can destroy anything" is evident in this field, while there is a severe lack of correlation between laboratory and field performance. Hence, as for any accelerated (laboratory) testing, it is essential to:*
>
> *a) create artificially conditions simulating field conditions, or*
>
> *b) in other ways calibrate the test results against field experience. For freeze-thaw testing, the most crucial boundary conditions relate to the temperature and moisture vs. time testing regime.*

Rønning then goes on to mention a very important point that also requires attention:

> *However, test procedures with enhanced precision are not necessarily more relevant to field exposure than those with poorer precision. Since field exposure and its correlation to testing has not received very much attention, there is a risk that laboratory tests may be adopted based on precision alone. Therefore, field exposure should receive more attention.*

As part of the upcoming Ar2CorD [49] project at UiT Narvik, this thesis contributes to the project's aim of developing low carbon concrete for arctic climate with excellent sustainability and durability. One crucial aspect of the project involves setting up a field exposure station where concrete mixtures can undergo long-term exposure to real F-T environments. The results from these stations will provide valuable insights into the actual F-T degradation occurring in the field and will help to develop laboratory tests that accurately reflect these conditions.

On a relevant note, CEN 12390-9 does make a case for adjustments to the tests, and states the following in annex A:

> *The application of these test methods for the performance assessment of concrete under different exposure severity is addressed in this annex. The applied testing conditions are sometimes considered to be extreme compared to practical temperature conditions and degree of saturation. This may cause confusion regarding setting of acceptance limits for local conditions. Also, some regions may experience series of mild winters in a row and suddenly one with extensive*

use of de-icers, a situation for which it is very difficult to establish a direct laboratory versus field correlation.

Annex A lists several potential alternative modifications that can be made to laboratory testing methods with the aim of better simulating field conditions. These modifications include, but are not limited to:

a) Geometry of samples
b) Other curing conditions
c) Other de-icing agents
d) Number of F-T cycles
e) Rate of cooling / heating

This brings on another issue that needs to be addressed. The CEN 12390-9 standard opens for potential adjustments to be made; however, the knowledge necessary to confirm that alternative testing methods means adequate F-T resistance for concrete is both challenging and time consuming to research. This strengthens the case that projects like Ar2CorD mentioned above have a huge potential to impact how we assess F-T resistance of concrete. And necessary for the next generation SCM concrete to be used in F-T environments.

In conclusion, there is a need for further research to develop laboratory testing methods that better resemble field conditions when assessing F-T durability. Current tests may have been too focused on precision, making it difficult to evaluate the suitability of products like LCC in F-T environments. However, ongoing research and the establishment of field exposure stations in different locations and environments are providing valuable data for the development of more realistic laboratory tests. Ultimately, these efforts will lead to improved methods for assessing the F-T resistance of concrete in real-world conditions.

As a contributor to this work, my thesis will be investigating the effect of different curing conditions on different SCM concretes, using the reference method in CEN/TS 12390-9. These different curing conditions described in chapter 3.3.2 are prolonged curing and curing with 1% CO_2. The prolonged curing is done to see if longer curing periods may improve the SCM mixtures F-T durability (because of the slower strength development that concrete containing SCM's often have). The CO_2 curing was done to simulate real world environments where over time, concrete structures with access to air will carbonate with the CO_2 in the atmosphere. This carbonation process will often take decades to infiltrate deep into the concrete, mainly because of the low CO_2 concentration in the atmosphere of about 0.04% [50]. Therefore, this study attempts to accelerate this process by using a concentration of CO_2 which is 25 times higher than the concentration in the atmosphere (1% CO_2).

3 Experimental program

3.1 Mix design

In this work, a total of four mixes were studied. For all the mixes in this trial CEM II/B-M (V-L) 42,5 R cement was used. This cement contains 18% fly ash and 6% limestone powder. Total amount of cementitious materials was kept constant at 300 kg/m3. Concrete with different combinations of SCMs were designed as listed in Table 1.

REF is a control mixture with only 10% replacement of CEM II replaced with SF. For the other three mixtures (FA+SF, DO+SF and GGBS+SF), 30% of the CEM II were replaced with different combinations of the chosen SCMs for this trial. Table 1 takes the amount of FA and LP in the CEM II into consideration.

Table 1 Mix design with overview over materials and their content.

Mix	REF		FA+SF		DO+SF		GGBS+SF	
	[kg/m3]	[w%]	[kg/m3]	[w%]	[kg/m3]	[w%]	[kg/m3]	[w%]
Water/cementitious materials ratio	0,45		0,44		0,45		0,50	
Portland Cement	201	67 %	160	53 %	160	53 %	160	53 %
Fly ash	52	17%*	86	29%**	38	13%*	38	13%*
Limestone powder *	17	6 %	13	4 %	13	4 %	13	4 %
Silica fume	30	10 %	42	14 %	30	10 %	30	10 %
Dolomite	-	-	-	-	60	20 %	-	-
Granulated blast furnace slag	-	-	-	-	-	-	60	20 %
Total SCM content	99	33 %	140	47 %	141	47 %	141	47 %
Total cementitious materials	300	100 %	300	100 %	300	100 %	300	100 %

*Content from CEM II

**17% of content from CEM II

3.1.1 Input materials

Four different SCMs were added to the concrete during the mixing process in various replacement ratios and combinations. These were fly ash, dolomite, silica fume and granulated blast furnace slag.

- Properties of the CEM II cement is given in Table 2.
- The fly ash is distributed by Heidelberg materials and its technical data is given in Table 3.
- The dolomite originates from Franzefoss Minerals, Norway and its technical data is given in Table 4.
- The silica fume originated from Elkem, Norway and its technical data is given in
- Table 5.
- The GGBS originates from Bremen, Germany and its technical data is given in
- Table 6.

Sand and coarse aggregates both originated from northern parts of Norway. Their physical properties are given in Table 7.

The superplasticizer (SP) used in this trial came from Mapei (DYNAMON SX-23) and the air entrainer from the same company (MAPEAIR 25).

Table 2 Technical data for CEM II/B-M (V-L) 42,5 R cement

Density (g/cm3)	Specific Surface Area (m2/kg)	Setting time (min)	Compressive Strength (MPa)				Contents		
		Initial	1d	2d	7d	28d	OPC (%)	FA (%)	LP (%)
300	450	150	19	29	40	53	76	18	6

Table 3 Technical data for fly ash

FLY ASH		
Cl^-	<0,1	%
SO_3	<0.1	%
Free CaO	<1.5	%
Reactive CaO	<10	%
Particle density	2300	kg/m^3

Table 4 Technical data for dolomite

ARCTIC DOLOMITE		
CaO	33	%
MgO	18,5	%
Density	2860	kg/m^3

Table 5 Technical data for silica fume

ELKEM MICROSILICA® 940 (SF)		
SiO_2	>90	%
H_2O	<1	%
Density	200-350	kg/m^3

Table 6 Technical data for GGBS

SLAGG BREMEN (GGBS)		
CaO	40	%
SiO_2	35	%
Al_2O_3	12	%
MgO	7	%
Blaine fineness	400	m^2/kg

Table 7 Density and WA% of aggregates

Material	**Density [kg/m3]**	**Water absorption [%]**
Sand 0-8 mm	2580	0,8
Coarse 8-22 mm	2770	0,5
Crushed 4-8 mm	2700	0,5

3.2 Tests on fresh concrete

Slump were measured on the fresh concrete according to NS-EN 12350-2 [51], air content according to NS-EN 12350-7 [52], and fresh concrete density according to NS-EN 12350-6 [53]. Table 8 illustrates the different samples and their curing condition.

Table 8 Sample overview

Curing regime	Type of specimens	Specimens per mix	Testing days	Test conducted	Specimens in total
Water	100 mm cubes	9	2, 28 and 56	Compression	36
CEN 12390-9	150 mm cubes	3	7, 14, 28, 42, 56*	Freeze-thaw	12
CEN 12390-9 + CO_2 curing	150 mm cubes	3	7, 14, 28, 42, 56*	Freeze-thaw	12
CEN 12390-9 + prolonged curing	150 mm cubes	3	7, 14, 28, 42, 56*	Freeze-thaw	12
Water	200 x 100 cylinder	2	28 and 56	Chloride migration	8
			***number of F-T cycles**	Total	80

3.3 Tests on hardened concrete

3.3.1 Compressive strength

The compressive strength were measured using cubic specimens with 100 mm sides according to NS-EN 12390-3 [54]. All specimens were demoulded after 24 h before being cured in water at 20 ± 2°C until testing. The testing ages were 2, 28 and 56 days so that the effect of SCM's on both early age and long term could be investigated. At each age, three samples were tested from each mixture, and the average value was taken to represent the strength at each age.

3.3.2 Freeze-thaw testing

Freeze-thaw testing were carried out according to SN-CEN/TS 12390-9:2016 [55]. This standard mentions 3 different methods for F-T testing and this thesis used the reference method (slab test). In order for a concrete mixture to be regarded as F-T resistant according to NS-EN 206 [17], the standard requires the scaling after 56 F-T cycles to be $\leq$ 0,50 kg/m^2. This limit is used in this thesis to decide whether the concrete mixture is F-T resistant or not.

Planned deviations from the CEN 12390-9 standard:

1. Prolonged curing
    a. The reference method in CEN 12390-9 uses a standard curing time of 31 days before exposing the samples to F-T attacks. In this thesis, two sets of samples were tested: one set was cured for 31 days, while the other set was cured for a prolonged time of 62 days, achieved by extending the time in the water bath.
2. 1% CO_2 exposure
    a. For each concrete mixture, 3 samples were exposed to a CO_2 curing for 7 days after sawing. This is done by having the same conditions of the climate chamber as the reference method (20°C & 65% humidity) but with an additional controlled concentration of 1% CO_2. Standard curing conditions are also applied to have comparative references.
3. 3 samples were used per mixture instead of the standardized 4 stated in CEN 12390-9. This was done as the climate chambers for F-T testing lacked space, and to get more mixtures tested, a reduction of 1 sample per mix were done.

Table 9 shows the 3 different methods chosen for curing as described above.

For the application of the salt solution on top of the samples, the method of weighing 67ml of solution was used instead of the method of pouring until a 3 mm layer on top of the sample is reached. Not only does this method mean a constant water content on every sample, but it also removes the problem of accurately measuring 3 mm from the top of the sample. Thirdly, it also solves the problem of how much salt solution that should be added to a sample with extreme scaling Figure 7 shows a heavily pitted sample where it would be problematic to measure a 3 mm salt solution on top.

Figure 7 Heavily pitted F-T samples

Description of method Age of sample (days)	Reference method CEN 12390-9	Reference method + CO_2 curing in climate chamber	Reference method + prolonged curing in water bath
1	Water bath (20°C)	Water bath (20°C)	Water bath (20°C)
7	Climate chamber (20°C, 65% humidity)	Climate chamber (1% CO_2, 20°C, 65% humidity)	
21	Specimens are sawn and returned to climate chamber	Specimens are sawn and returned to climate chamber	-
25	Specimens glued and placed in forms	Specimens glued and placed in forms	-
28	Water saturation	Water saturation	-
31	Start of F-T testing	Start of F-T testing	
38			Climate chamber 65% hum.
52			Specimens are sawn and returned to climate chamber
56			Specimens glued and placed in forms
59			Water saturation
62			Start of F-T testing

Table 9 Overview over F-T curing conditions

3.3.3 Chloride migration

Chloride migration testing was carried out according to NT-BUILD 492 [56]. Concrete cylinders with diameter of 100 mm and height of 200 mm were casted. Testing were then carried out at 28 and 56 days, and the samples were cut 1 day before testing.

4 Results

4.1 Results from fresh concrete

In this chapter, the fresh properties of the concrete mixtures are presented. Table 10 shows the fresh density, amount of SP used, slump measurements and air content of the fresh concrete.

Table 10 Fresh properties of concrete mixtures

Mix name	Density [kg/m3]	Super plasticizer [kg/m3]	Slump [mm]	Air [%]
REF	2331	4,29	215	5,6
FA+SF	2463	3,90	210	5,6
DO+SF	2375	4,53	210	6
GGBS+FA	2306	3,30	200	6

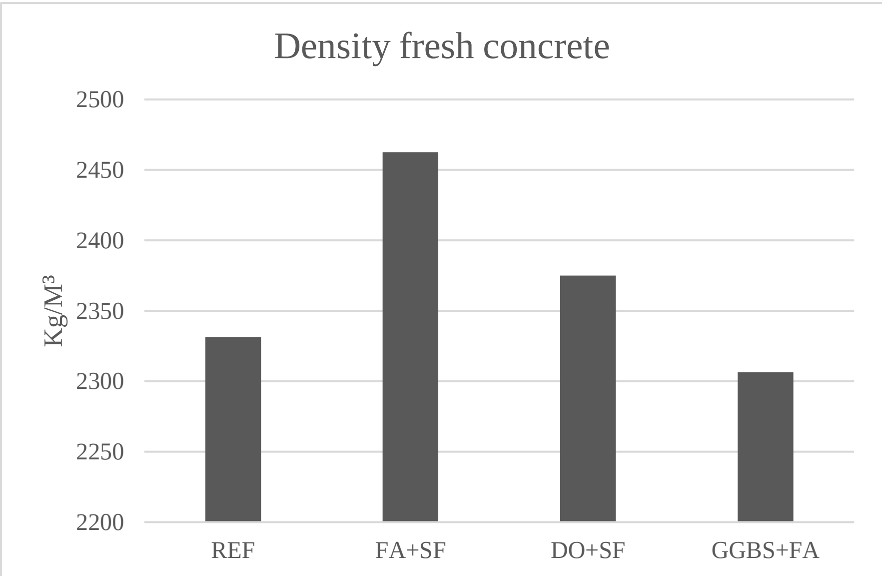

Figure 8 Density of fresh concrete in this trial

4.2 Results from hardened concrete

4.2.1 Freeze-thaw

4.2.1.1 Regular curing

Figure 9 illustrates the F-T results for the samples being tested according to the reference method in CEN 12390-9 with no adjusted curing conditions. The red line is the limit value to be considered a F-T resistant concrete according to NS-EN 206 (≤ 0,50 kg/m2). Already at 28 days the FA+SF and GGBS+SF surpasses this requirement, and consequently does not classify as a F-T resistant concrete according to this trial.

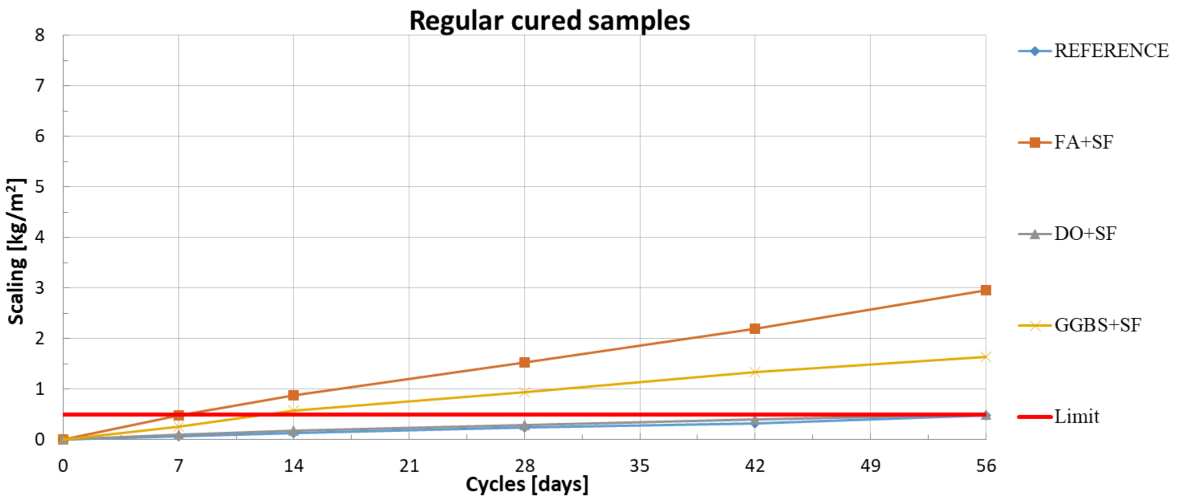

Figure 9 F-T results for regular cured samples

Table 11 shows the values for F-T scaling at 7, 28 and 56 cycles. At 56 cycles, the FA+SF and GGBS+SF have 2,99 kg/m2 and 1,63 kg/m2 of scaling respectively. Looking at the REF and DO+SF mixture at 56 cycles, these two mixes make the F-T requirement set for this thesis as the total scaling is 0,47 kg/m2 and 0,48 kg/m2 respectively.

Mix name	7 cycles [kg/m3]	28 cycles [kg/m3]	56 cycles [kg/m3]
REF	0,06	0,24	0,47
FA+SF	0,50	1,56	2,99
DO+SF	0,10	0,28	0,48
GGBS+SF	0,25	0,94	1,63

Table 11 F-T results at 7, 28 and 56 cycles, reference cured

4.2.1.2 CO_2 Curing

Figure 10 illustrates the F-T results for the mixtures that were subjected to 7 day 1% CO_2 concentration in the climate chamber phase. The results show that a drastic increase in scaling takes place across all mixtures except for the DO+SF mixture after being carbonated when compared to the regular cured samples presented in the previous chapter. Not only does none of the mixtures make the requirement in NS-EN 206 of ≤ 0,50 kg/m2 scaling, but three of the mixtures (REF, FA+SF and GGBS+SF) surpasses this limit drastically. Therefore, from this study it can be concluded that carbonation affects the F-T resistance of concrete negatively.

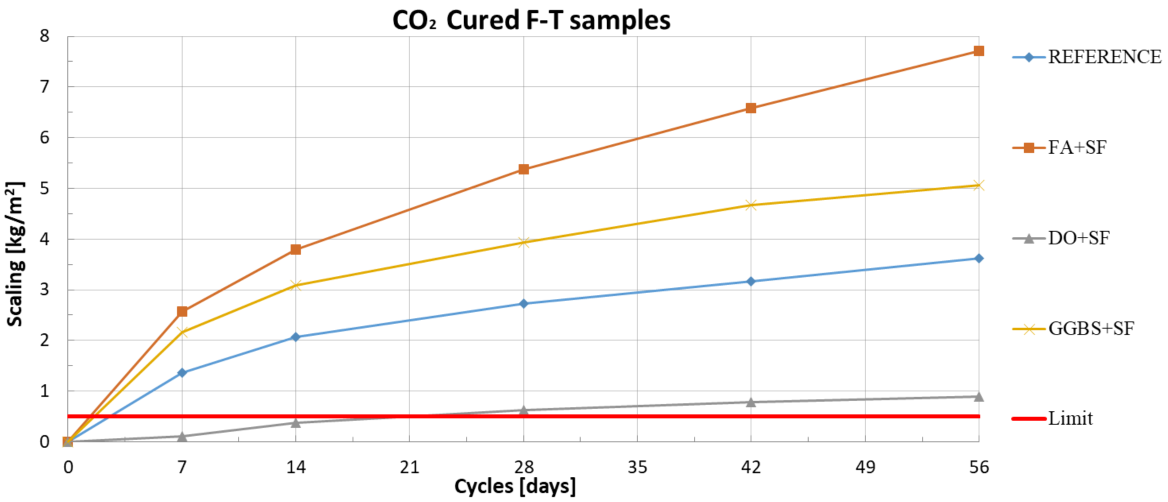

Figure 10 F-T results for CO2 cured samples

The graph clearly demonstrates that the DO+SF mixture outperforms the other mixtures by a significant margin. However, it is worth noting that this mixture was not subjected to the same curing process as the rest of the samples due to an error made by the author. Instead of being cured for 7 days in a 20 °C water bath before being placed in the climate chamber, the DO+SF mixture was cured for 19 days in the water bath, resulting in exposure to 100% RH rather than the intended 65% RH in the climate chamber. As a result, the extended exposure to moisture is likely to have impacted the curing process, causing the sample to become saturated with water. This may have had an impact on the carbonation process, as carbonation does not typically occur to the same extent when water is present in the pores of the material.

4.2.1.3 Prolonged curing

Due to the delivery of this thesis before 56 cycles in the F-T chamber were reached for the prolonged cured samples, results up to 28 cycles were recorded for the REF, FA+SF and DO+SF. For the GGBS+SF mixture, 14 cycles were recorded before the delivery of the thesis. The results are illustrated in *Figure* 11.

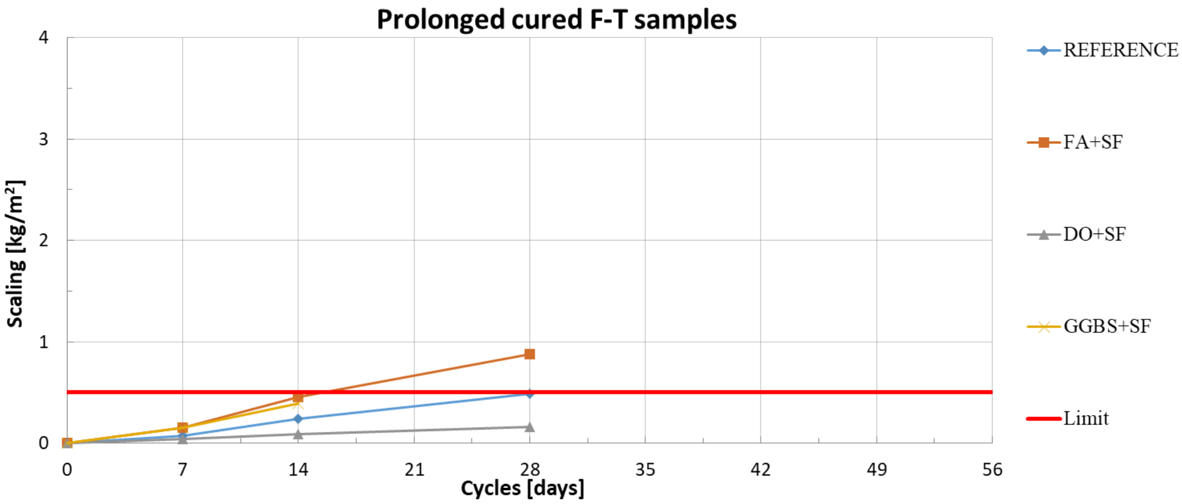

Figure 11 F-T results for prolonged cured samples

Although a reduction in the scaling is observed for the FA+SF mixture when compared to the regular cured, it quickly goes beyond the limit chosen for this trial, and accordingly not F-T resistant. The GGBS+SF mixture is also close to the limit value already at 14 days and will most likely go past the limit over the course of the 42 F-T cycles it has left.

Interestingly, the reference mixture already surpasses the limit value and 28 days and performs much worse than when only regular cured. The author was unable to find an explanation for this result.

Lastly, the DO+SF mixture performs better when prolonged curing is used, and based on the low scaling at 28 cycles, it will most likely make it under the limit value for a F-T resistant concrete.

4.2.2 Compression strength

Figure 12 and Table 12 illustrates the compression strength results from this trial. Additionally, the table shows the difference in strength when compared to the reference mixture at every test day. As expected for the concrete mixtures containing FA and GGBS, a delayed strength development is observed at 2 days, and both mixtures are about 33% weaker at 2 days when compared to the reference.

Interestingly, the opposite trend is seen for the concrete containing DO where this mixture shows an almost 29% higher compression strength at 2 days when compared to the reference mixture. Because of the delayed hardening of the FA and GGBS concretes, it can be problematic to reach such a high target strength as required in this trial (C45/55) at 28 days. The FA mixture does not reach this target strength of 55 MPa at 28 days; but is close at 53 MPa. The GGBS mixture surpasses this target strength with minimal margins (57 Mpa). The reference mixture also makes the target strength. The concrete containing DO surpasses this target strength with exceptional margin and reaches 72 MPa at 28 days, thus meeting the strength requirement for a C55/65 strength class.

For 56-day compression results, all mixes meet the required target strength set for this trial. Further, all mixes have continued their strength development, but the FA and GGBS concretes have a higher strength development between 28 and 56 days than the reference and DO mixture.

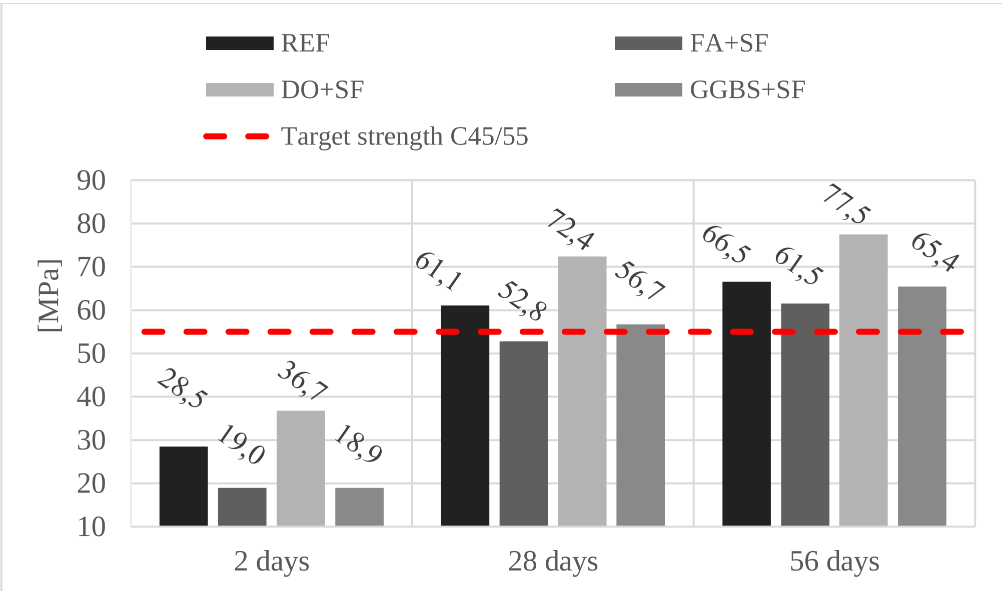

Figure 12 Compressive results

Mix name	2 days [MPa]	Δ REF	28 days [MPa]	Δ REF	56 days [MPa]	Δ REF
REF	28,5	-	61,1	-	66,5	-
FA+SF	19,0	-34 %	52,8	-14 %	61,5	-8 %
DO+SF	36,7	29 %	72,4	19 %	77,5	17 %
GGBS+SF	18,9	-34 %	56,7	-7 %	65,4	-2 %

Table 12 Compressive results

4.2.3 Chloride migration

Figure 13 shows the chloride migration results for all concrete mixtures in this trial. The reference mixture was not tested at 56 days due to a mistake by the author. The red line shows the limit value to classify as a “Extremely high” chloride migration resistance. All limit values to evaluate chloride migration resistance is shown in Table 13. From this table it can be observed that the REF or FA+SF mixture classifies as a “very high” chloride migration resistance at 28 days. The DO+SF and GGBS+SF mixture however both classifies as “extremely high” in chloride resistance.

The development at 56 days however shows that all tested mixtures are well within the “extremly high” chloride migration resitance. Since high chloride migration correlates highly with lower permeability, it can be concluded that the DO+SF and GGBS+SF mixtures are less permeable than the reference and FA+SF mixtures.

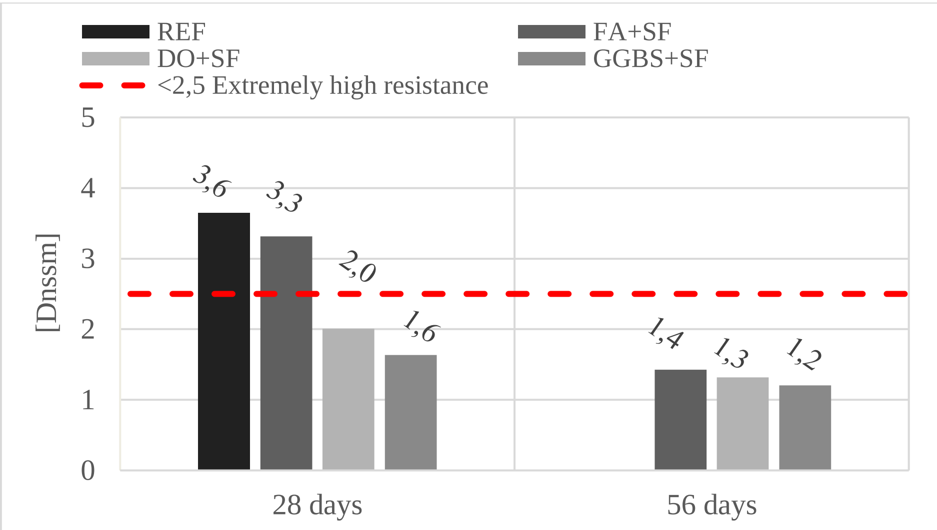

Figure 13 Chloride migration results

Table 13 Chloride migration resistance requirement from NTBUILD-492

Resistance to chloride migration	
Dnssm	28 days
>15	Low
10-15	Moderate
5-10	High
2,5-5	Very high
<2,5	Extremely high

5 Discussion of results

In this thesis, different low carbon concrete (LCC) mixtures with various supplementary cementitious materials were tested for compressive strength, chloride migration resistance and F-T resistance. Tests on air, density and slump were also tested on the fresh concrete.

5.1 Fresh concrete tests

1. **The amount of SP is highly dependent on the type of SCM used.**

From both Figure 8 and Table 10, it can be observed that the fresh density is similar in the REF, DO+SF and GGBS+SF mixtures, while the FA+SF mixture have quite a high fresh density of 2463 kg/m^3. Further, to reach the desired workability of the mixtures around 200-220 mm slump, SP was added. In accordance with available previous research, adding FA and GGBS to concrete improves the workability, and less SP must be added (only 3,90 kg/m3 and 3,30 kg/m3 respectively). However, the opposite was true for the DO+SF mixture which needed 4,53 kg/m^3 to reach the correct workability which is higher than the reference mixture where 4,29 kg/m^3 SP were used. Table 14 shows this difference in SP amounts compared to the amount used in the reference mixture. While the GGBS difference of 30% less SP is significant, it needs to be noted that the higher w/b ratio of this mix (0,50 from Table 1) will have had a positive impact on the workability since the rest of the mixtures had a lower w/b ratio of 0,45-0,46.

Table 14 SP amount and comparison of amount up against reference mixture.

Mix name	SP [kg/m3]	ΔREF
REF	4,29	-
FA+SF	3,90	-10 %
DO+SF	4,53	5 %
GGBS+FA	3,30	-30 %

5.2 Hardened concrete tests

1. **The use of SCM in F-T environments can be problematic.**

Based on the F-T results presented in this thesis, a big variation can be observed when looking at the scaling for the reference cured samples presented in Table 15. Here only the 7, 28 and 56 cycles are presented, and the difference in scaling when compared to the reference at every respective cycle is calculated (Annotated as Δ). Looking only at the difference in scaling at 56 cycles, the FA+SF and GGBS+SF scales 84% and 71% more than the reference, while the DO+SF only scales 3% more at 56 cycles. Given this big variation based on which SCM is used in the concrete points towards the fact that for more SCM's to be used in F-T environments, the materials need to be investigated closely in relation to F-T resistance before being used for construction purposes.

Based on this thesis, using dolomite in concrete can make a F-T resistant concrete if tested according to the reference method in CEN 12390-9. Furthermore, the FA and GGBS mixtures increases the scaling that takes place under F-T environments when compared to the reference, and being over the limit set for this trial cannot be used in F-T environments.

Mix name	7 cycles [kg/m3]	Δ REF	28 cycles [kg/m3]	Δ REF	56 cycles [kg/m3]	Δ REF
REF	0,06	-	0,24	-	0,47	-
FA+SF	0,50	88 %	1,56	85 %	2,99	84 %
DO+SF	0,10	39 %	0,28	13 %	0,48	3 %
GGBS+SF	0,25	76 %	0,94	74 %	1,63	71 %

Table 15 F-T results at 7, 28 and 56 cycles with calculated difference against reference mixture

2. **LCC concretes containing FA, DO and GGBS benefits greatly from prolonged curing.**

This finding can be backed up by the compressive-, chloride migration- and F-T results presented in the previous chapter.

Table 16 shows the strength development for the compression strength for each mix. The 28-day strength is used as a 100% reference, and the 2- and 56-day strength is compared to the 28 days strength. It can be observed that the FA+SF and GGBS+SF mixes have significantly lower 2-day strength development compared to the REF and DO+SF mixture. However, the FA+SF and GGBS+SF mixes both have a significant strength development after 28 days and up to 56 days with 17% and 15% respectively, compared to the REF and DO+SF mixtures where a <10% strength development between 28 and 56 days is observed.

Mix name	2 d	28 d	56 d
REF	47 %	100 %	109 %
FA+SF	36 %	100 %	117 %
DO+SF	51 %	100 %	107 %
GGBS+SF	33 %	100 %	115 %

Table 16 Strength development, compressive strength

Further, moving on to the chloride migration results the GGBS mixture performed extremely well at both 28- and 56 days, while the FA mixture benefitted drastically from increasing the curing time to 56 days (44% decrease in Dnssm value). The DO+SF mixture also benefitted from prolonged curing in respect to chloride migration resistance.

Lastly, looking at the F-T results, Table 17 shows the F-T results for regular cured and prolonged cured samples in tabular form, while also comparing the difference between regular and prolonged curing for each mixture (Delta). The first trend that can be noted, is that all the mixtures containing 47% SCM (FA+SF, DO+SF and GGBS+SF) benefits greatly from prolonged curing for this test. Although, the FA+SF mixture still goes beyond the limit value chosen for this trial already at 28 days with 0,90 kg/m^3 of scaling, it performs 41% better in terms of reduced scaling at 28 days. For the DO+SF mixture, a reduction of 48% is observed at 28 days, and outperforms the reference mixture in terms of F-T scaling resistance. Lastly, a decrease of about 30% scaling is observed for the GGBS+SF mixture but being at 0,41 kg/m^3 already at 14 days, it will most likely not stay under the limit value chosen for trial as it reaches 56 F-T cycles.

	REF			**FA+SF**			**DO+SF**			**GGBS+SF**		
	Scaling [kg/m3]		Delta	Scaling [kg/m3]		Delta	Scaling [kg/m3]		Delta	Scaling [kg/m3]		Delta
	Regular	PC*		Regular	PC*		Regular	PC*		Regular	PC*	
7d	0,06	0,04	**-34 %**	0,49	0,14	**-71 %**	0,10	0,01	**-88 %**	0,25	0,17	**-31 %**
14d	0,14	0,21	**54 %**	0,88	0,46	**-47 %**	0,19	0,07	**-61 %**	0,58	0,41	**-29 %**
28d	0,25	0,46	**84 %**	1,53	0,90	**-41 %**	0,29	0,15	**-48 %**	NO DATA		
	*Prolonged curing											

Table 17 Comparison between regular cured and prolonged cured F-T samples for each respective concrete mix

3. Carbonation affects the F-T durability negatively.

As mentioned in the results for the F-T tests, it can be seen a very significant increase in scaling due to F-T attacks on samples that were exposed to 1% CO_2 for 7 days. This finding was found for all mixtures except for the DO+SF mixture.

Based upon previous research articles, the negative effect of carbonation could be due to the chemical reactions that occur during the carbonation process. When carbon dioxide reacts with the hydrated cement paste in concrete, it forms calcium carbonate crystals (equation 2) [39]:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O \quad (2)$$

Because these crystals tend to occupy more volume than the original hydrated cement paste, it leads to an increase in internal stress within the concrete. This stress, combined with other environmental factors such as F-T cycles, can result in the concrete's surface layer flaking or scaling off. In a study by Hasholt et al. (2022) [57], a pore refinement was found where non-carbonated concrete had more pore volume in the range 1-10 nm and less pore volume in the range 100-1000 nm (Figure 14), compared to carbonated concrete. Because of the importance of pore size distribution mentioned in chapter Cold climates2.2, this change in pore size distribution brought on by carbonation may have affected the F-T durability of the concrete.

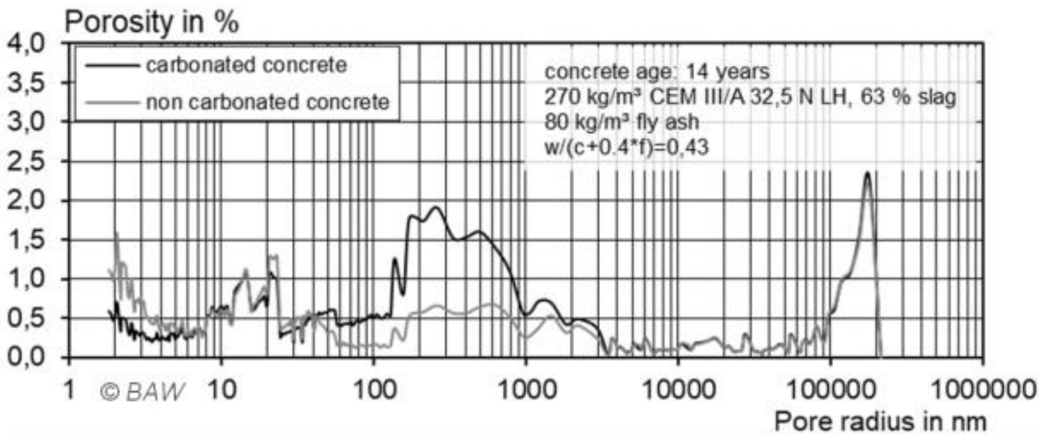

Figure 14 Changes in pore distribution on carbonated concrete samples, Hasholt et al. (2022)

This theory can be further backed by looking at *Table* 18 which shows the difference in scaling between the regular and CO_2 cured samples for each mix. In this table the ratio is how many times more the CO2 cured samples scaled compared to the reference cured samples at each respective test day. As explained in chapter 2.1, all SCM materials used in this trial reacts with the portlandite ($Ca(OH)_2$) which is also what the CO_2 reacts with in the carbonation process (equation 2). Because of this, depending on how reactive the SCM's are, different amounts of portlandite will be available in the different mixtures, and from this line of theory it can be derived that the reference mixture which has the lowest amount of SCM's will have the most portlandite available to be carbonated. This may explain why the difference in scaling between regular cured samples and CO_2 cured samples are at the largest for the reference mixture (22 times larger at 7 days).

	REF			FA+SF			DO+SF			GGBS+SF		
	Scaling [kg/m3]		Ratio	Scaling [kg/m3]		Ratio	Scaling [kg/m3]		Ratio	Scaling [kg/m3]		Ratio
	Regular	CO2		Regular	CO2		Regular	CO2		Regular	CO2	
7d	0,06	1,36	**22**	0,49	2,57	**5**	0,10	0,11	**1**	0,25	2,16	**9**
14d	0,14	2,06	**15**	0,88	3,80	**4**	0,19	0,37	**2**	0,58	3,08	**5**
28d	0,25	2,73	**11**	1,53	5,37	**4**	0,29	0,62	**2**	0,95	3,93	**4**
42d	0,33	3,16	**10**	2,20	6,58	**3**	0,40	0,78	**2**	1,33	4,66	**4**
56d	0,48	3,62	**8**	2,96	7,72	**3**	0,50	0,90	**2**	1,64	5,06	**3**

Table 18 Difference in scaling between regular and CO2 cured F-T samples.

Further, to continue this line of reasoning, the GGBS+SF mixture have the second largest amount of available portlandite (and also second the largest increase in scaling at 7 days), which is also in line with previous studies that shows that up to 75%wt (chapter 2.1.2) of GGBS needs to be used in order to consume all available portlandite (this trial only used 20%). While the FA is known for being more reactive than the GGBS and have a lower difference in scaling between regular and CO_2 curing, the absolute lowest is the DO+SF mixture. If this theory holds, it can be derived that a lot of the dolomite have reacted with the portlandite as explained in chapter 2.1.3, and may also explain why it performs so well on both compressive- and chloride migration testing.

Another finding that supports the fact that carbonation leads to decreased F-T resistance can be seen as the concrete remains longer in the F-T chamber, *Figure* 15 presents the scaling that have taken place between each test day. Looking at the regular cured samples in the graph, the scaling is quite consistent between the different test days, pointing to the fact that the concrete is of uniform quality as the F-T scales deeper into the sample. However, for the CO_2 cured samples, a decreasing trend can be observed as the scaling goes deeper into the sample.

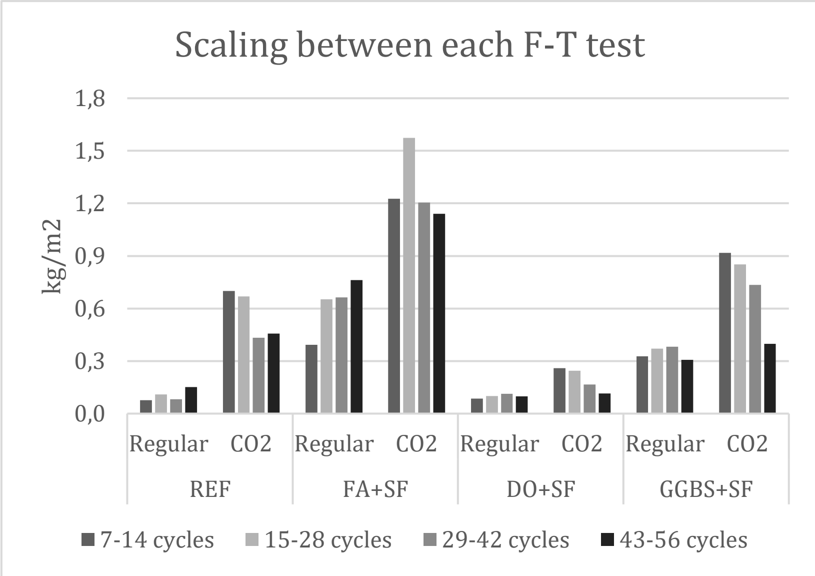

Figure 15 Scaling between F-T test days

This trend can be attributed to the fact that carbonation affects only the outer layer of the concrete, which is the first to scale. As scaling penetrates deeper into the sample, it eventually reaches the uncarbonated concrete, which can resist F-T attacks better than the carbonated layer. Thus, the scaling intensity decreases over time.

4. **Dolomite mixtures should be investigated further.**

Based on all the results in this thesis, the mixture containing 20% dolomite outperforms the reference in both compression strength and chloride migration. For the compression results it even exceeds all mixtures in this trial at both 28- and 56 days. Additionally, it performs very well in F-T environments, and the only one of the LCC in this trial which makes the requirement for a F-T resistant concrete according to EN-206. Further, the F-T results for the CO_2 cured sample is the only mixture in this trial which performs close to the regular cured samples (but this need to be confirmed by future studies because of the mistake in curing conditions explained in chapter 4.2.1).

To sum up, given the dolomite mixtures good performance in this trial, and the pros of good availability and low cost described in chapter 2.1.3, this SCM should be investigated for further use in concrete.

6 Conclusion

In conclusion, this thesis has provided valuable insights into the performance of low carbon concrete (LCC) containing different supplementary cementitious materials (SCMs) under freeze-thaw (F-T) durability testing. The results have demonstrated that LCC with SCMs up to a 47% replacement ratio of total binder mass can be used for constructive purposes. However, it is important to test each new SCM before use, as their performance can differ significantly, especially in F-T environments.

The findings of this study also showed that prolonged curing of concrete before subjecting it to F-T attacks can lead to significant improvements in F-T durability, resulting in reduced scaling. However, it is worth noting that despite the increase in F-T resistance, the outcome on whether the concrete mixture is classified as F-T resistant according to NS-EN 206 was not affected.

Furthermore, this thesis highlights the negative impact of carbonation on the F-T durability of LCC, as carbonated concrete exhibited a drastic reduction in scaling resistance when subjected to F-T attacks.

Lastly, the DO+SF mixture was found to be the most effective in terms of F-T resistance, exhibiting excellent performance compared to other mixtures. However, the difference in curing conditions of the DO+SF mixture may have impacted its performance, and further research is required to fully understand the reasons for its superior performance.

Overall, the findings of this study suggest that careful consideration of the SCM selection and curing conditions is necessary to ensure the F-T durability of LCC for use in construction.

7 Further research

This thesis has demonstrated that the F-T performance of low carbon concrete (LCC) is affected by different curing conditions, such as exposure to 1% CO_2 and prolonged curing. However, current F-T tests, such as CEN 12390-9, may not fully resemble the actual F-T conditions that concrete is exposed to in the environment. To address this issue, it is necessary to establish field test stations that resembles the F-T conditions in real environments and collect data on the performance of concrete. This data can then be used to develop laboratory tests, including accelerated tests like CEN 12390-9, that better correlate with the F-T damage caused by actual F-T environments.

By improving the correlation between laboratory tests and field conditions, the use of LCC in F-T environments may be feasible to a larger degree in the future.

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Appendixes

APPENDIX A – Product data sheet dolomite

Produktdatablad

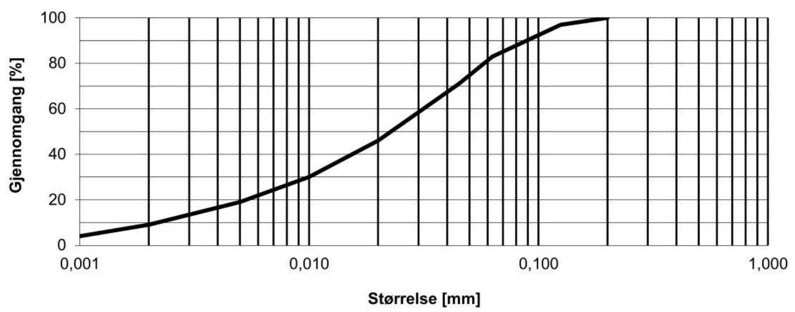
BETOFILL AD

Franzefoss Minerals AS
Postboks 53
NO-1309 Rud
Telefon : **+47 05255**
E-post, Web: post@kalk.no www.kalk.no

Materiale:	Dolomitt		**Produsent:**	Franzefoss Minerals AS, avd. Eydehavn
	$CaMg(CO_3)_2$		**Råmaterial:**	Dolomitt fra Hekkelstrand, Ballangen i Nordland
Reg.nr.:	Sertifikat CPR	1111-CPR-0726	**Fremstilt:**	Nedmaling av dolomitt
	Produktregistrert	33850	**Versjon:**	1/23
	REACH nr	-		
Anvendelse:	Tilslag for betong (system 2+)			
Krav:	NS-EN 12620: Tilslag for betong			

Parameter		**Metode**	**Enhet**	**Statistikk**		**Krav**		
				Snitt	**s**	**L**	**H**	**Toleranse +/-**
CaO	Kalsiumoksid	WD-XRF	[%]	33	-	-	-	±2
MgO	Magnesiumoksid		[%]	18,5	-	-	-	±2
Ca	Kalsium	Beregnet fra	[%]	24	-	-	-	
Mg	Magnesium	WD-XRF	[%]	11,2	-	-	-	
Klorider		Våtkjemisk	[%]	0,02	-	-	-	
Syreløselige Sulfater			[%]	<0,01	-	-	-	
Totalt innhold av svovel			[%]	<0,003	-	-	-	
Alkali silika reaktivitet			[%]	0	-	-	-	
Vanninnhold		NS-EN 12048	[%]	<0,2	-	-	-	
Masstetthet		Pyknometer	[kg/dm^3]	2,86	-	-	-	
0,001 mm		Microtrac	[%]	4	0,4	-	-	
0,002 mm			[%]	9	0,5	-	-	
0,005 mm			[%]	19	0,7		-	
0,010 mm			[%]	30	1,1	-	-	
0,020 mm			[%]	46	1,8	-	-	
0,045 mm			[%]	71	2,8	-	-	
0,063 mm			[%]	83	2,7	70	100	
0,125 mm			[%]	97	0,9	85	100	
0,200 mm			[%]	100	0,2	-	-	

Kornfordeling



Råmateriale:	Råmaterialet er et naturprodukt med variasjoner innenfor visse grenser
Levering:	Bulk
Volumvekt:	1,31 kg/dm³
Lagring:	Produktet må oppbevares tørt
SDS:	Se produktets sikkerhetsdatablad for informasjon angående helse, miljø og sikkerhet. Les denne informasjonen og iverksett eventuelle sikkerhetstiltak før produktet tas i bruk.

Franzefoss Minerals AS PDB (BA-90)

APPENDIX B – Product data sheet Micro Silica

Elkem

Product Data Sheet
ELKEM MICROSILICA® 940
Concrete applications

Elkem ASA Silicon Products
Drammensveien 169 P.O. Box 334
Skøyen NO-0213 OSLO
Fax: +47 22 45 01 11
www.elkem.com

Description

ELKEM MICROSILICA® 940 is a dry silica fume powder. In use, ELKEM MICROSILICA® 940 improves the properties and performance of high performance concrete and specialist mortar formulations.

It acts physically to optimise particle packing of the mixture, and chemically as a highly reactive pozzolan.

ELKEM MICROSILICA ® 940 is available in two forms: Undensified (U) with a typical bulk density of 200 to 350 kg/m³ and Densified (D) with a typical bulk density of 500 to 700 kg/m³.

Chemical & physical data

Property	Unit	Value
SiO_2	%	Minimum 90.0
H_2O (when packed)	%	Maximum 1.0
Loss of ignition	%	Maximum 3.0
Retained on 45µm	%	Maximum 1.5*
Bulk density U	kg/m³	200 - 350
Bulk density D	kg/m³	500 - 700

Test methods are available on request. For typical analysis of minor elements, please contact our Elkem representative.
**Tested on undensified*

Packaging

The product is available in 25 kg bags, various other size bags and bulk road tanker. Please contact our representative for more details.

Storage and handling

ELKEM MICROWILICA® 940 should be stored in dry conditions and not exposed to moisture.

Health, safety & environment

Refer to Product Safety Information (PSI) document on our website: www.elkem.com

Quality assurance

Elkem Silicon Products management system for development, processing and supply of ELKEM MICROSILICA® is certified to ISO 9001:2015. The chemical and physical properties of ELKEM MICROSILICA® 940 are regularly tested.

Sustainability

For Elkem, sustainability is central to our business strategy. Our mission is to provide advanced material solutions shaping a better and more sustainable future, adding value to our stakeholders globally. We are committed to reducing embodied carbon emissions. For further information, please visit www.elkem.com/sustainability/ or contact us to learn more about our sustainability roadmap.

Additional Information and contact

See additional Elkem data sheets and technical papers on our website: www.elkem.com
All data listed are reference values subject to production related tolerances. Although reasonable care has been taken in the preparation of the information contained herein, it still remains the duty of the user to prove the suitability of this material for their application.

APPENDIX C – Product data sheet Fly ash

PRODUCT DATA SHEET

FLY ASH

ADDITIONAL COMPONENT

LATEST REVISION MARCH 2023

The Fly ash is certified according to the requirements in NS-EN 450-1, Class A

Properties	Declared values	Requirements according to NS-EN 450-1
Loss on ignition (%)	≤5.0	Satisfies requirements in NS-EN 450-1
Chloride (% Cl^-)	≤0.10	Satisfies requirements in NS-EN 450-1
Sulfate (% SO_3)	≤3.0	Satisfies requirements in NS-EN 450-1
Free Calcium oxide (% free CaO)	≤1.5	Satisfies requirements in NS-EN 450-1
Reactive Calcium oxide (% reactive CaO)	≤10	Satisfies requirements in NS-EN 450-1
Particle density (kg/m³)	2300	Declared value +/- 200 kg / m³
Other chemical and physical parametres		Satisfies requirements in NS-EN 450-1

Heidelberg Materials Sement Norge AS handles sales and distribution of Fly ash for cement and concrete production. Fly ash is a processed residual product derived from coal power plants. Fly ash contains silicate, and is a pozzolan which together with cement and water, makes concrete denser. Combined with cement, fly ash has been used in Norway since the 1980s. All Heidelberg Materials' FA -cements contain fly ash.

Heidelberg Materials

Heidelberg Materials Sement Norge AS
PO.Box 143, Lilleaker, NO-0216 Oslo
Tlf. +47 22 87 84 00
www.sement.heidelbergmaterials.no

APPENDIX D – Product data sheet GGBS

PRODUKTDATABLAD

Reviderad 2022-12-12OE. Gällande version kan laddas ner från www.thomasconcretegroup.com

Slagg Bremen

Mald granulerad masugnsslagg (ggbs) för användning i betong och bruk

0402

Thomas Cement AB, Box 5162, SE-402 26

2014

0402-CPR-SC0324

SS-EN 15167-1:2006

Mald granulerad masugnsslagg
för användning i betong, bruk och injekteringsbruk

GGBS – Slagg Bremen

Övriga specifikationer om produkten se aktuellt Produktdatablad (på www.thomasconcretegroup.com)

Typ och ursprung

Slagg Bremen är ett mineraliskt tillsatsmaterial (typ II) med latent hydrauliska egenskaper, för användning som bindemedel i betong och bruk. Slagg Bremen kommer från järntillverkning i Tyskland där den flytande slaggen avskiljs, kyls snabbt (granuleras), torkas och mals. Slagg Bremen är ett kvalitetssäkrat och CE-märkt tillsatsmaterial som uppfyller kraven i SS-EN 15167-1 och -2.

Egenskaper

Som ett bindemedel har slagg effektivitetsfaktor k=0,7 men en högre faktor (k=0,8 eller 0,9) kan tillämpas med aktuellt cement om kraven uppfylls enligt SS 137003 och SS-EN 15167-1. Slagg Bremen är godkänd för k=0,9 med följande cement:

- Bygg (Skövde) CEM II/A-LL 42,5 R
- BAS (Slite) CEM II/A-LL 42,5 R
- ANL (Brevik) CEM I 42,5 N SR3/MH/LA
- ANL (Slite) CEM I 42,5 N SR3/MH/LA
- ANL FA (Slite) CEM II/A-V 42,5 N NSR/MH/LA
- Schwenk Komposit CEM II/A-M (S-LL) 52,5 N

Godkännandebevis kan laddas ner på www.thomasconcretegroup.com. För övriga cement gäller k=0,7.

Användningen av Slagg Bremen som ett bindemedel erbjuder följande möjligheter:

- Minskad cementmängd (minskad CO_2)
- Förbättrad pump- och arbetbarhet samt minskat vattenbehov
- Minskad värmeutveckling och risk för temperatursprickor vid massiva konstruktioner
- Ökad beständighet (ökad sulfatbeständighet, ökat motstånd mot alkalikiselreaktioner, syra och kemiska angrepp, samt minskad inträngning och ökad bindning av klorider)
- Ljusare betong och högre ytfinish

Största mängd slagg som får tillsättas för respektive exponeringsklass och cementtyp redovisas i SS-137003.

Hantering

GGBS kan vid kontakt verka irriterande på hud, andningsorgan och ögon, och kan vara skadligt att förtära. GGBS motsvarar cements egenskaper varför transport och förvaring bör ske på motsvarande sätt. För fullständig information se Säkerhetsdatablad.

Fysikaliska data

Egenskap	Riktvärde	Variation	Enhet	Krav enligt SS-EN 15167-1	Standard
Specifik yta (Blaine)	400	±20	m^2/kg	≥ 275	SS-EN 196-6
Kompaktdensitet	2900	±50	kg/m^3	-	SS-EN 196-6
Skrymdensitet	1150	±200	kg/m^3	-	
Aktivitetsindex* 7/28/91d	≥ 55/75/100		%	≥ 45/70/-	SS-EN 15167-1
Bindetid*	≤ 1,3		-	≤ 2,0	SS-EN 196-3

** Standard bruk med 50% OPC (Cem I 42,5) och 50% GGBS, relativt 100% OPC.*

Kemiska data

Egenskap	Riktvärde	Variation	Enhet	Krav	Standard
Kalcium (CaO)	40	±5	vikt-%	-	
Kisel (SiO_2)	35	±5	vikt-%	-	
Aluminium (Al_2O_3)	12	±3	vikt-%	-	
Magnesium (MgO)	7	±3	vikt-%	≤ 18,0	SS-EN 196-2
Titan (TiO_2)	≤ 1,2		vikt-%		
Mangan (Mn_2O_3)	≤ 0,6		vikt-%		
Sulfat (SO_3)	≤ 0,2		vikt-%	≤ 2,5	SS-EN 196-2
Klorid (Cl^-)	≤ 0,02		vikt-%	≤ 0,10	SS-EN 196-2
Alkalitet (Na_2O_{eq})	≤ 1,2		vikt-%	-	SS-EN 196-2
Glödförlust	≤ 2,0		vikt-%	≤ 3,0	SS-EN 196-2
Glashalt	≥ 90		%	≥ 67	

Vid enstaka tillfällen kan värdena avvika från angivna gränser och spann. Om så sker utfärdas larm till berörda.

Thomas Cement AB
Box 5162, 402 26 Göteborg, 0104-50 50 00, www.thomasconcretegroup.com