Prediction of early age and time dependent deformations in a massive concrete structure

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Abstract

The heat development that occurs due to the hydration of cement is important to consider during casting of massive concrete structures. By using computer programs that are based on finite element methods (FEM), simulations can be performed on the heat- and strength development. In this project, a FE program called ConTeSt has been used in order to predict the temperature- and strain development in a massive concrete wall. If the potential risks in a concrete structure are evaluated before casting, economical savings, including a better casting plan could be obtained. The structure under investigation was a concrete wall behind one of the spillways in the hydro power dam of Storfinnforsen. Due to a re-construction of the wall, an opportunity occurred to develop a measurement plan of the casting and perform simulations on the wall.

A sensitivity analysis was performed in order to investigate the effects on the temperature- and strain development, by varying the cement content, ambient temperature, wind speed and degree of restraint in translation. The results showed, that a higher cement content increased the rate of hydration and hence the temperature in the concrete. Higher wind speeds contributed to more cooling of the concrete which, in some cases, resulted in cracking due to contraction of the material. Cracking due to contraction also occurred when the ambient temperature was decreased. The ambient temperature did not have a significant impact on the rate of hydration, but instead the impact was larger from the initial temperature of the fresh concrete. A higher initial temperature of the fresh concrete increased the rate of hydration, which increased the temperature in the material. The degree of restraint could only be varied in translation in ConTeSt and hence the effect on the strain development was not that significant.

A crack risk analysis was performed where the developed tensile stresses were compared with the tensile strength of the concrete. The same factors were varied as in the sensitivity analysis. The results showed that the tensile strength was exceeded for most of the cases and thus that the crack risk was high.

The required equipment, in order to perform the measurements on site, consisted of 7 strain gauges of the module KM-100B from TML Tokyo Sokki Kenkyujo, 2 data loggers of the module Spider-8 from HBM, at least a 25 m $\phi$9 mm 5-core shielded cable and a computer with the software Catman Easy.

Keywords: Early age concrete, hydration of cement, Storfinnforsen, ConTeSt, crack risk, temperature development, strain, mass concrete structures, measurement plan.
Sammanfattning

Värmeutvecklingen som uppstår på grund av hydratationen av cement är viktig att beakta vid gjutning av massiva betongkonstruktioner. Detta brukar göras genom simuleringsar av värmeutvecklingen och hållfasthetstillväxten med hjälp av olika finita element (FE) program. I detta projekt har programmet ConTeSt använts för att på förhand kunna förutse temperatur - och töjningsutvecklingen i en massiv betongvägg. I och med detta kan bl.a. gjutningen planeras bättre samtidigt som ekonomiska besparingar kan åstadkommas om eventuella risker kan kartläggas innan gjutningen påbörjas. En ledmur bakom ett av utskoven i Storfinnforsens kraftverk undersöks närmare i samband med en ombyggnad. Möjligheten uppstod att planera en mätning av gjutningen av ledmuren samt att utföra simuleringsar av väggen i ConTeSt.

En känslighetsanalys utfördes för att undersöka effekterna på temperatur- och töjningsutvecklingen genom att variera cementhalten, omgivningstemperaturen, vindhastigheten och graden av tvång i förskjutningen i långdiktningen av väggen. Resultaten visade att högre cementhalter ökade graden av hydratation vilket ökade temperaturen i betongen. Högre vindhastigheter bidrog till snabbare kylning av betongen vilket i vissa fall lett till sprickor på grund av kontraktion av materialet. Sprickor till följd av kontraktion uppstod även då omgivningstemperaturen sänktes. Omgivningstemperaturen hade ingen större påverkan på hydratationen, utan istället var det temperaturen av den färska betongmassan som visade större påverkan. Högre temperatur av den färska betongmassan ökade graden av hydratation vilket ökade temperaturen i betongen. Graden av tvång kunde i ConTeSt endast varieras i förskjutningen i långdiktningen av väggen vilket inte hade någon större effekt på töjningsutvecklingen.

En sprickrisk analys utfördes där den utvecklade dragspänningen jämfördes med draghållfastheten. Analysen utfördes genom att variera samma faktorer som varierades i känslighetsanalysen. Resultaten visade att draghållfastheten överskreds i de flesta fall och att därmed sprickrisken var hög.

För att genomföra mätningen blev slutsatsen att det behövs 7 st töjningsgivare av modell KM-100B från TML Tokyo Sokki Kenkyujo, 2 st data logger av typ Spider8 från HBM samt minst en 25 m φ9 mm skärmad 5-kärnkabel, inklusive en dator med programvaran Catman Easy.

Nyckelord: Ung betong, hydratation av cement, Storfinnforsens kraftverk, ConTeSt, sprickrisk, temperatur, töjning, massiva konstruktioner, mätningsplanering.
Preface

This degree project has been carried out at KTH Royal Institute of Technology at the Department of Civil and Architectural Engineering and the Division of Concrete Structures, in collaboration with the consultant company WSP.

First off, we would like to thank our WSP supervisor, Ph.D student Rikard Hellgren, for initiating the project and for the continuous valuable assistance and guidance.

We would also like to express sincere gratitude to our KTH supervisor Dr. Richard Malm for the help, feedback and involvement in the project.

Furthermore, we are grateful to the Department of Hydropower and Dam Safety at WSP for providing us with office space and computers. We would also like to thank all of the staff in the department for the enjoyable working atmosphere and for inviting and making us feel welcomed to all of the fun activities outside the office hours.

An additional gratitude is directed to WSP employee Jim Larsson who provided and assisted us with drawings and for handling the communication between us and the contractor.

Last but not least, we would like to thank our friends and families for supporting us throughout all the years at KTH, including for the duration of this degree project.

Stockholm, June 2018

Ali Aghili
Haris Ribac
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Chapter 1

Introduction

1.1 Background

A massive concrete wall connected to one of the spillways in the hydro power dam of Storfinnforsen was re-constructed for strengthening purposes during the summer of 2018. In connection to the casting of the new structure, there was an interest in investigating the early age concrete behavior in terms of temperature- and strain development, including an estimation of the cracking risk.

For massive concrete structures, variations in e.g. ambient temperature can sometimes be more demanding for the structure than mechanical loads. This is especially true during the beginning of the hydration process, when the chemical reaction between cement and water generates a large amount of heat. As a result, thick structures can sometimes create a large temperature difference between the outer surface and the core. This also causes strain development within the structure, which in turn result in self-induced stresses in the material, that could lead to crack initiation. [1]

In Europe, concrete structures are designed according to Eurocode 2 [2] in the countries that uses the regulations. However, in order to successfully use the Eurocode one must, in some cases, start by generating the section forces acting on a structure by finite element (FE) modelling [3]. Furthermore, calculations on the behavior of early age concrete are difficult to perform since the physical phenomenons are time dependent [3]. Heat transfer and time dependent deformations for instance, are usually described with partial differential equations (PDEs) which are difficult to solve analytically [1]. The solutions can instead be approximated with finite element methods. The approach of FE modelling is, especially nowadays, also a very common choice when the actual behavior of a structure is to be predicted. When utilizing it for concrete, it is often desirable to predict the mechanical effects from various mechanical loads acting on the structure. However, in order to obtain a more accurate prediction of the behavior, it is sometimes important to consider the physical phenomenons during the early age of the concrete [1]. In this project, the FE program ConTeSt Pro was used in order to investigate the behavior at early age for the massive concrete wall.
1.2 Aim and research questions

The overall aim of the project is to predict strain- and temperature development in critical points of a massive concrete wall during its early curing stage. Furthermore, the report also aims to create a suggestion for a practical measurement plan. The research questions addressed in this project are listed below.

1. Are there crack risks during early age for the concrete wall?
2. How does the cement content, wind speed, ambient temperature and degree of restraint affect the temperature- and strain development at early age?
3. What is the effect from the factors mentioned in research question 2 on the cracking risk?
4. What is a suitable measurement plan to investigate the temperature development, strain development and cracking risk in early age for the concrete wall, in practice?

1.3 Limitations

To be able to perform the investigation within a reasonable time frame, certain limitations are necessary. The investigated structure is massive and varies in dimensions and reinforcement content along its length. Therefore, only a certain cross section was investigated through 2D simulations. Also, the material properties of the concrete that was used at casting were not known at the time of modelling and hence could not be defined correctly in the simulations.

Finally, it should be noted that the initial plan of the project was to not only plan the measurements, but to actually implement them in practice to be able to compare it with the numerical results. However, due to delays on the construction site, this part of the project was forced to be neglected.

1.4 Outline of the report

In Chapter 2, early age concrete is described and this includes its behavior after casting, both instantaneously and during the first days.

Chapter 3 explains how the chosen FE tool works mathematically and how it takes the factors mentioned in Chapter 2 into account.

The case study of the project is presented in Chapter 4 and this includes a description of the investigated structure and the modelling approach, respectively.

Chapter 5 outlines the obtained numerical results including a discussion.
The practical measurement plan is described in Chapter 6, which also covers a suggestion of points in the studied structure that ought to be investigated, based on the numerical results.

Chapter 7 highlights the conclusions of the project and outlines suggestions for a potential further research.
Chapter 2

Early age concrete

There is no exact definition of "early age concrete" in terms of a specific time period during the hardening of concrete. This is partly because the time for curing varies depending on type of material and thickness of the structure respectively. Roughly speaking, early age concrete is the condition during the first few days of concreting, characterized by the hydration process of cement. During hydration, rapid temperature development occurs as a result of the heat generated in the structure due to the chemical reactions of cement with water. This can create large temperature differences in mass concrete structures between the outer surface and inner core, which may result in internal stresses and/or volume changes [1]. The American Concrete Institute (ACI) [4] defines mass concrete as "any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change, to minimize cracking."

Higher temperatures yields larger movement of the molecules in the concrete material which increases the kinetic energy [1]. This increase of movement causes volume changes in the concrete material, which in turn can induce stresses, especially for restrained structures. However, to describe how these factors are initiated, it is necessary to describe the initial hydration process after concrete mixing, see Section 2.1. The generated heat from the hydration of cement is explained in Section 2.2. The purpose of the concrete is to develop strength and withstand certain loads. The most important mechanical properties developed during the hydration are mentioned and explained in Section 2.3. Finally, the volume of the concrete can increase or decrease during the early age, which is explained in Section 2.4.

2.1 Hydration process of cement

Portland cement is the most commonly used cement and consists mainly of the four compounds alite (C$_3$S), belite (C$_2$S), aluminate (C$_3$A) and ferrite (C$_4$AF) [1]. Table 2.1 shows the oxide composition and the respective abbreviations with the cement chemist notation (CCN). The CCN notations use one letter for each oxide,
i.e. \( C = \text{CaO}, \ S = \text{SiO}_2, \ A = \text{Al}_2\text{O}_3 \) and \( F = \text{Fe}_2\text{O}_3 \). Similarly, the notation for water is \( H \). The reaction of the compounds in Table 2.1 with water, the so called hydration process, creates the hardened cement paste [5]. The reaction is roughly described according to Gasch [1] with the Eq. (2.1) - (2.3) where Eq. (2.1) and (2.2), containing the calcium silicates, describes the major part of the behavior.

Table 2.1: The oxide composition of the Portland cement compounds and their respective CCN abbreviation. [5]

<table>
<thead>
<tr>
<th>Name</th>
<th>Oxide composition</th>
<th>CCN abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium silicate</td>
<td>3CaO.SiO(_2)</td>
<td>C(_3)S</td>
</tr>
<tr>
<td>Dicalcium silicate</td>
<td>2CaO.SiO(_2)</td>
<td>C(_2)S</td>
</tr>
<tr>
<td>Tricalcium aluminate</td>
<td>3CaO.Al(_2)O(_3)</td>
<td>C(_3)A</td>
</tr>
<tr>
<td>Tetracalcium aluminoferrite</td>
<td>4CaO.Al(_2)O(_3).Fe(_2)O(_3)</td>
<td>C(_4)AF</td>
</tr>
</tbody>
</table>

\[
\begin{align*}
2C_3S + 6H & \rightarrow C_3S_2H_3 + 3Ca(OH)_2 \quad (2.1) \\
2C_2S + 4H & \rightarrow C_3S_2H_3 + Ca(OH)_2 \quad (2.2) \\
C_3A + 6H & \rightarrow C_3AH_6 \quad (2.3)
\end{align*}
\]

A perhaps better and visual description of the hydration process is illustrated in Figure 2.1 where the process is divided into five stages.

![Figure 2.1](image.png)

Figure 2.1: The hydration process of Portland cement divided in five stages. Reproduced from the version given by Gasch [1].

Stage I is the initial reaction of cement with water where the dissolution of ions in the water creates the reaction between aluminate (C\(_3\)A) and gypsum, i.e. ettringite formation, following a rapid heat evolution. As a result of the ettringite formation, the reaction between the cement and water slows down. Stage II is represented by
the so called dormant period since the hydration rate is low. The dormant period do not usually last longer than 5 hours [6] and it ends at Stage III where the hydration rate increases again. Here, the reaction described by Eq. (2.1) is dominating and a rapid production of C-S-H and CH occurs [1]. When the rate of heat evolution is at its peak, Stage IV begins and the heat generation starts decreasing again due to the protective layer created over the unhydrated particles by the hydrates [6]. This continues at Stage V where the cement hydrates dominates the space initially containing water and the hydration is almost at the same level as during the dormant period [6]. Stage IV and V are characterized by the diffusion of the water through the pores and the reactions described in Eq. (2.3) is common for most cements in stage V [1]. The setting time describes the rigidity development of the cement paste and is the change of an element from a fluid to a solid [5]. Also, the initial set, as shown in Figure 2.1, describes a rapid rise in temperature and the final set corresponds to the peak temperature [5].

The process in Figure 2.1 is exothermic meaning that the energy is released as heat. The reactions depend largely on the temperature where the rate of hydration and heat evolution is increased with increasing temperature causing volume changes in the structure. During the reactions described in Eq. (2.1) - (2.3), a large amount of water is chemically bound. Along with the hydration process, an increase of the solid areas occurs causing further adsorption of the water. Eventually, as the rigidity of the structure increases, the large consumption of water induce volume changes referred to as autogenous shrinkage, see Section 2.4. [1]

Since the hydration is thermally governed, it is affected by the ambient temperature but also the amount of free water that is present [1]. According to Neville [5], the water-cement (w/c) ratio needs to be greater than 0.38 in order for the cement to fully hydrate. The surrounding humidity also plays an important role, where it has been shown that, for a humidity below 80 % in the pores, the hydration has in some cases completely stopped or experienced a slow performance. [1]

2.1.1 Aging of concrete

The degree of hydration, \( \alpha \), is used to describe how much of the cement that has reacted with water. The development of the degree of hydration is dependent on the w/c ratio. Lower w/c ratio gives higher degree of hydration at early age, but lower degree of hydration at high age, as shown in Figure 2.2
CHAPTER 2. EARLY AGE CONCRETE

2.2 Temperature development

2.2.1 Heat of hydration

The concept of concrete maturity is based on the principle that the properties of concrete (such as strength) have a direct relation to both age and temperature history. This method is used to determine the strength of the concrete during the construction process. This is a relatively simple approach to optimize the workflow on the construction site by determining e.g. when to demolish the formwork. [8]

The degree of hydration, and therefore the strength development, is affected by the temperature [9]. To convert the actual age of the concrete to its equivalent age (in terms of strength gain), an equivalent maturity age, $t_e$, is introduced. This is based on a reference temperature of usually 20 °C in Europe. A maturity factor, $\beta_t$, is required to be able to determine the equivalent maturity age. The degree of hydration, and therefore the heat development, can be determined when the equivalent age is obtained. [10]

The heat development is governed by the hydration of cement and hence dependent on the cement content [9]. Generally, a higher cement content causes increased heat development. A report from the American Concrete Institute [11] states that the hydration of cement increases the temperature in the concrete by roughly 5 - 7 °C for every 50 kg/m$^3$ of cement content. In the case of having concretes with the same cement content but different w/c ratios, the concrete with the higher w/c ratio would have a higher rate of heat development. This is because more water is available for the cement to react with which also increases the rate of hydration. Figure 2.3 shows the effect of different w/c ratios on the heat evolution. The effect
on the rate of heat development by increasing the cement content is more significant than the same effect from a higher w/c ratio. The heat generation also increases by lowering the w/c ratio as a result of increasing the cement content. The rate of heat generation is also influenced by the cement composition. Higher contents of tricalcium silicate (C\textsubscript{3}S) and tricalcium aluminate (C\textsubscript{3}A) in the cement increases the rate of heat generation. Cements that have higher fineness have higher rate of heat generation than other cements. [12]

Figure 2.3: The effect from the w/c ratio on the heat evolution (referera).

A high ambient temperature accelerates the hydration and leads to a faster setting of the concrete, resulting in lower long-term strength. The same effect occurs for higher temperatures on the fresh concrete, which also increases the rate of hydration. [5]

Wind is another important factor to consider when concreting since it contributes to the liberation of heat and water from the concrete surface. Higher wind speeds causes more cooling of the concrete surface which may result in more contraction. Generally, wind increases the drying of the concrete, which can lead to crack development as a result of plastic shrinkage. [13]

### 2.2.2 Thermal properties and heat transfer

The thermal properties of the concrete is partly affected by the environmental factors, which includes e.g. ambient - and casting temperatures and wind speed, respectively. In addition, external factors such as form material and type of insulation could play a decisive role as well. [9]

The thermal properties affected includes e.g. thermal conductivity, heat capacity,
heat transfer coefficient and coefficient of thermal expansion, respectively. The thermal conductivity, \( k \), represents the ability of the material to transmit heat. This property is dependent on the moisture content and type of aggregate, where the conductivity decreases with decreasing moisture content. The heat capacity, \( C_t \), describes the amount of heat necessary to increase the temperature with one degree in the material. As opposed to the thermal conductivity, the heat capacity increases with increasing moisture content and temperature [1]. The heat transfer coefficient, \( h \), describes the heat exchange due to convection and depends on the wind speed for surfaces exposed to air [14]. The coefficient of thermal expansion, CTE, describes the material's propensity to change in volume due to a change in temperature. However, it is necessary to describe the coefficient of thermal expansion of the concrete as a function of both the coefficients for the cement paste and aggregates since these are usually different. [1]

### 2.2.3 Thermal cracks and restraint

As a consequence of the heat generated during the hydration process, the temperature in a new concrete structure rises while it simultaneously expands. After some time, the heat generation decreases following a natural cooling and contraction of the concrete as shown in Figure 2.4. [15]

![Figure 2.4: Temperature variation at the centre of a massive concrete element at early ages showing the expansion phase (I) and the contraction phase (II). Reproduced from the version given by Bamforth [16].](image)

When the heat rises within the new structure, a temperature difference between the core and the outer surface occurs. This, in combination with the boundary conditions, generates tensile stresses and cracking may occur [9]. These cracks are often categorized as either *through-* or *surface* cracks as shown in Figure 2.5.
2.2. TEMPERATURE DEVELOPMENT

The difference between the type of cracks is explained in Table 2.2 along with examples when these may occur during the expansion - and contraction phase, respectively.

Table 2.2: Crack types in the expansion - and contraction phase, respectively. [17]

<table>
<thead>
<tr>
<th></th>
<th>Expansion phase (heating)</th>
<th>Contraction phase (cooling)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Through cracks</td>
<td>Occurs if the difference in mean temperature is large between different adjacent parts of a casting stage.</td>
<td>Normally occurs in relation to restraint from an adjacent structure.</td>
</tr>
<tr>
<td>Surface cracks</td>
<td>Normally occurs if the temperature difference between the core and the outer surface is large.</td>
<td>Could occur during sudden cooling, e.g. if the formwork is demolished in cold weather.</td>
</tr>
</tbody>
</table>

During the curing, the inner parts of the structure acts like a restraint which prevents contraction. This is referred to as internal restraint and can cause surface cracking during heating and cooling. Cracks developing during heating usually selfheal whereas cracks during cooling are permanent. However, these are usually surface cracks and do not usually propagate towards the inner parts [9]. External restraint is from e.g. the environment, such as foundation conditions or from an adjacent older structure. The external conditions prevents movement of the structure, which can initiate crack propagation through the structure and be the cause of surface cracks. [9]

2.2.4 Cooling and heating

To eliminate cracks and other durability problems, the most important factor is to control the temperature during the curing of the concrete [18]. For this, there are several methods depending on the size and requirements of the project.
To keep the temperature at a lower magnitude, one option is to cool the concrete, which could be achieved in different ways. For instance, the aggregates could be cooled prior to mixing. One option is to spray the coarse aggregates with chilled water while the finer aggregates could be placed in a tank and cooled with cold air. The water that is used for the mixing could be cooled as well, either separately or combined with ice [19]. When acting on one of these options prior to mixing, especially if cooled water or ice are used, one needs to always take the w/c ratio into account. Adding water to the mix in an uncontrolled way could create durability problems in the concrete.

The use of cooling pipes is a common option as well, especially since no additional ingredients in the recipe are required. Cooling pipes are embedded in the formwork and when the concrete is casted, a cooling media is allowed to flow through the pipes to keep the temperatures down. Although cooled air could be used, the diameter of the pipes would have to be increased to have an effect, making it a less popular option. [20]

Another option to minimize the risks of cracks for a new structure is to pre-heat adjacent structures with heating cables which decreases the usually large temperature difference in the concrete. In addition, the similar effect could be obtained by using insulation in the formwork. [21]

The heat development can be reduced by lowering the cement content and replacing some of it with additives [9]. Fly ash can for instance reduce the rate of heat development significantly [12]. The effects of the cooling methods on the temperature development are shown Figure 2.6.

![Figure 2.6: Results from an investigation on the temperature development for different cooling methods. Reproduced from the version given by Lagundžija [22].](image-url)
2.3 Strength development

2.3.1 Fresh concrete

When concrete is newly cast, it is considered to be fresh, with barely any hardening taking place. Afterwards, the hardening process begins in the early age concrete, which is characterized by a rapid rise in strength. This pattern more or less continues until about 28 days after casting, see Figure 2.7. At this stage, the concrete is considered to be hardened and sufficiently strong, even if a certain smaller growth in strength still continues for years to come [23].

Both the strength development and the durability depends on the concrete curing which implies maintaining control over temperature- and moisture conditions. There are various methods of concrete curing. Normal curing is one of these methods and has the main objective to keep the concrete saturated, or as nearly saturated as possible. The moisture conditions are maintained until the cement hydrates occupy the initially water-filled space. It is necessary for the capillary pores to be filled with water for cement to hydrate. Hence, it is important to prevent water loss through evaporation from the capillary pores. This can be achieved by sealing the surface of the concrete by e.g. an impermeable membrane, but that will prevent adding water to the concrete in order to replace the water loss by self-desiccation. To achieve a sufficient strength development, it is not required for the cement to fully hydrate and in practice also hard to accomplish. However, if the curing proceeds until the capillary pores are completely filled, the concrete would become impermeable and obtain a good durability [5]. The influence of moist curing on the compressive strength is shown in Figure 2.8 for six cases. The first case (solid line) represents the development of strength when the concrete is continuously saturated. The other five cases (dashed lines) represents the influence when the concrete is exposed to air (20 °C, 50 % RH) continuously, after 3 days, after 7 days, after 14 days and after 28 days, respectively.

Figure 2.7: Strength increase of concrete. Reproduced from the version given by Burström [23].
The temperature of the fresh concrete is another major factor affecting the strength development. For instance, a higher temperature results in a more rapid initial strength development, but weaker concrete in the long-term. The reason is that a higher temperature of the fresh concrete increases the initial rate of hydration which causes uneven distribution of the cement gel, giving a structure with lower quality. To avoid this, it is necessary to keep a lower temperature of the fresh concrete when e.g. concreting in hot weather as described in Section 2.2.4. [5]

The rate of the strength development of the hardened cement paste also depends on the fineness of the cement particles, where a high fineness yields a more rapid strength development. The fineness also have an effect on other factors such as workability of the fresh concrete or the long term behavior and must thus be selected with care. [5]

### 2.3.2 Cement compounds

The strength development of the cement compounds is shown in Figure 2.9 where the silicates alite (C\textsubscript{3}S) and belite (C\textsubscript{2}S) are most important in the strength development of the cement paste. It can be noted from Figure 2.9 that C\textsubscript{3}S has the largest influence on the strength during the first couple of weeks whereas C\textsubscript{2}S reaches the same contribution as C\textsubscript{3}S after approximately one year [5]. Aluminate (C\textsubscript{3}A) has little influence on the strength development and the experienced contribution occurs at early ages and when the cement paste is hardened during the presence of sulfates. Ferrite (C\textsubscript{4}AF) has an even smaller contribution on the strength but contributes in accelerating the hydration of the silicates [5].
2.3.3 Mechanical properties

The development of mechanical properties includes the compressive strength, tensile strength and elastic modulus, respectively, and they are necessary to predict and model for assessment of early age concrete. Although all of these increase as the hydration proceeds, they do so at different rates. [24]

The main purpose of concrete structures is to carry compressive forces. Hence, the compressive strength is the most important and most studied property of concrete. The development of the compressive strength for different cement finenesses are shown in Figure 2.10. Other than the varied cement fineness, the recipes of the different concrete used in the experiment are identical. It can be concluded that a higher rate of cement fineness generates higher compressive strength [24]. In addition, it has an effect on workability of the fresh concrete, as mentioned in Section 2.3.1.

Figure 2.10: Development of compressive strength with w/c = 0.40. The recipes are identical except the different cement finenesses S = 742 m^2/kg, R = 490 m^2/kg and O = 277 m^2/kg. [6]
The tensile strength is a property that, in comparison to compressive strength, has been investigated and tested to a small extent. The reason for this is because of its complexity. It is difficult to actually grip a concrete specimen in a satisfactorily manner when testing the behavior for direct (uniaxial) tension. Although tensile stresses are unavoidable in practice, structures are in general designed to not rely on their tensile strength due to it being significantly lower than the compressive strength [24]. According to Neville [5], the theoretical compressive strength is eight times larger than the tensile strength. However, this factor could vary depending on the level of strength of the concrete. In addition, there is in fact a close relation between the compressive and tensile strength, but not a direct proportionality. The ratio of tensile to compressive strength is generally lower the higher the compressive strength is. [5]

If a structure is restrained and uninsulated, compressive stresses develops during heating [16]. These compressive stresses will reach its maximum about one day after casting before they decrease just as rapidly and turn into tensile stresses during the contraction stage of the concrete [9]. It is when these tensile stresses exceeds the tensile strength of the concrete that cracking occurs.

It is important to consider the elastic modulus as well, since it is dependent on the degree of hydration and is therefore connected to the risk of cracking [25]. During an experiment by Oluokun [26], four different concrete mixes with w/c ratios 0.33, 0.39, 0.53 and 0.76, respectively, were tested to investigate the rate of development for the elastic modulus. The results showed that, for all mixes, the elastic modulus developed with the fastest rate compared to the compressive - and tensile strength. The compressive - and tensile strength developed at approximately the same rate with < 5 % difference. It was also concluded that the elastic modulus develops extremely rapid the first three days and then slows down in rate, as can be seen in Figure 2.11. According to Neville [5], one of the main differences between the elastic modulus and the compressive strength is that the elastic modulus is influenced by the properties of the aggregates to a larger extent than the compressive strength.
2.4 Time dependent deformations

2.4.1 Shrinkage and swelling

Shrinkage is a term that sometimes is divided into many parts. However, according to Eurocode 2, the total shrinkage strain is the sum of the drying shrinkage strain and the autogenous shrinkage strain.

Drying shrinkage occurs as a slow process due to the loss of water from the concrete and continues many years after the concrete has hardened [1]. This mainly occurs due to withdrawal of stored water in unsaturated air from the hardened concrete [5]. The effect of this can be seen in Figure 2.12. It can be concluded that, for concrete that has been allowed to dry in air at a certain relative humidity and then placed in water (or at a higher humidity), it will swell. Swelling is a process that describes a volume expansion due to continuous supply of water during the hydration. However, not all of the initial drying shrinkage is recovered, i.e. there is a certain irreversible shrinkage. [5]
Autogenous shrinkage occurs even when there is no moisture transport to or from the set concrete. This makes it an important factor to consider when estimating the volume change at early age in the concrete, and is especially important when new concrete is cast against old [1]. However, the influence from autogenous shrinkage is usually much more significant in high performance concrete than in normal strength concrete, perhaps making it negligible in most cases [5]. The autogenous shrinkage normally has a significant influence when the w/c ratio is less than 0.42 [27].

### 2.4.2 Creep

Initially, when a first stress is applied on a concrete structure, it shows an elastic behavior, which is described by the elastic strain. Creep is defined as the increase in strain under a sustained constant stress, after other time dependent deformations (e.g., shrinkage and swelling) has been accounted for [5]. Depending on the ambient temperature, creep is normally divided into basic - and drying creep [24].

Basic creep occurs when there is no moisture exchange between the concrete and the environment. It can be divided into two stages, short-term and long-term basic creep. For early age concrete, the short-term basic creep is important since it is partly affected by the hydration. This is due to the volume growth created by the hydrated products in the capillary pores that leads to a decrease of creep with loading time [1]. Drying creep is defined as the additional deformation (after subtraction of the pure drying shrinkage and thermal deformations) caused by drying or elevated temperatures [1].
Creep is an important factor to consider since it reduces the developed stresses in a concrete structure as a result of relaxation [16]. The development of creep, shrinkage and elastic strain are shown in Figure 2.13.
Chapter 3

Mathematical modelling in ConTeSt

In the field of structural engineering, solving physical problems with finite element (FE) methods has become a highly accepted approach and usually, these consist of calculating the response of a structure or a structural component due to mechanical loads. The problems are described with a mathematical model, which in turn is governed by differential equations. This includes assumptions regarding e.g. geometry, loading and boundary conditions. The model, which needs to be verified for high reliability, is then solved in a FE analysis [28]. Heat transfer and deformations could, for instance be difficult, and sometimes impossible to solve analytically. Therefore, FE tools are used, where the geometry is divided into small elements. For each element, the partial differential equations are able to be approximated and a solution, localized in the nodes of these elements, can be obtained. [1]

In this project, the FE tool ConTeSt has been used, which is developed by JEJMS Concrete, in collaboration with Luleå University of Technology, Cementa AB and PEAB AB. The software is developed for concrete structures and the main factors of interest are temperature - and strength development, including an assessment of the potential cracking risk. In practice, large economical and technical benefits are gained if measures to avoid cracking can be determined prior to casting [29]. The structure that is addressed in this project has been investigated in the early stages of the hardening process [14].

This chapter consist of an in-depth description of the mathematical modelling in ConTeSt considering the heat of hydration, heat flow, boundary conditions, strength growth and stress analysis. All of the presented equations in the following chapter are collected from the user manual [14] if not stated otherwise.

3.1 Heat of hydration

According to Hösthagen [29] the equivalent maturity age, $t_e$, is defined as

$$t_e = \beta_\Delta \cdot \int_0^t \beta_t \cdot dt + \Delta t_e^0$$

(3.1)
where $\beta_{\Delta}$ is a parameter that describes how a certain admixture affects the speed of the hydration process ($= 1$ in most cases), $t$ is the reference temperature [K], $\beta_t$ is the maturity factor expressed by Eq. (3.2) and $\Delta t^0$ is a possible adjustment parameter which shifts the time for the start of the hydration (e.g. if retarders are used).

$$
\beta_t = \exp \left( \theta_{\text{ref}} \cdot \left( \frac{30}{T + 10} \right)^{\kappa_3} \cdot \left( \frac{1}{293} - \frac{1}{T + 273} \right) \right) \quad (3.2)
$$

where $\theta_{\text{ref}}$ is a reference maturity parameter based on the cement type [K], $T$ is the initial temperature of the concrete [$^\circ$C] and $\kappa_3$ is a parameter reflecting the variation of the activation energy, based on cement type [-].

Once the equivalent maturity age is obtained, the heat energy development is [29] approximated with

$$
q_{\text{cem}}(t) = \exp \left( - \left( \ln \left( 1 + \frac{t}{t_1} \right) \right)^{-\kappa_1} \right) \cdot q_u \quad (3.3)
$$

where $t_1$ [s] and $\kappa_1$ [-] are decided experimentally and vary depending on the type of cement. Typical values for standard Portland cement are $t_1 = 5.52$ and $\kappa_1 = 1.07$, respectively [9]. $q_u$ is the total heat energy by cement weight [J/kg].

Finally, the generated heat per concrete volume [W/m$^3$], $Q_h(t)$, is calculated according to [29]

$$
Q_h(t) = \frac{dq_{\text{cem}}(t)}{dt} \cdot C \quad (3.4)
$$

where $C$ is the cement content [kg/m$^3$].

### 3.2 Heat flow

The simulations are performed by analyzing a certain cross section in the xy-plane, see Figure 3.1. It is assumed that the length of the structure is sufficient enough for the heat flow to be neglected in the z-axis, i.e.

$$
q_z = -k_z \cdot \frac{\partial T}{\partial z} \approx 0 \quad (3.5)
$$

where $q_z$ is the heat flow in z-axis [W/m$^2$], $k_z$ is the thermal conductivity in z-axis [W/mK] and $T$ is the temperature in the structure [$^\circ$C or K].
The heat flow in the xy-plane is given by

\[ \rho c \frac{\partial T}{\partial t} = \frac{\partial}{\partial x} \left( k_x \frac{\partial T}{\partial x} \right) + \frac{\partial}{\partial y} \left( k_y \frac{\partial T}{\partial y} \right) + Q_H \]  

(3.6)

where \( \rho \) is the density [kg/m\(^3\)], \( c \) is the specific heat per unit weight [J/(kgK)], \( k_x \) is the thermal conductivity in x-axis [W/(mK)], \( k_y \) is the thermal conductivity in y-axis [W/(mK)] and \( Q_H \) is the generated heat [W/m\(^3\)].

The thermal conductivity is assumed to be isotropic, i.e.

\[ k_x = k_y = k \]  

(3.7)

where \( k \) is the (isotropic) heat conductivity [W/(mK)] of the material.

### 3.2.1 External boundary conditions

The heat flow for the external boundary conditions of the structure is described with

\[ q_n = h_{\text{surface}} (T_{\text{surface}} - T_{\text{ambient}}) - I \]  

(3.8)

where \( q_n \) is the heat flow from the body to the external boundary along the normal of the boundary surface in the xy-plane [W/m\(^2\)], \( h_{\text{surface}} \) is the heat transfer coefficient of the external boundary [W/(m\(^2\)K)], \( T_{\text{ambient}} \) is the ambient temperature, usually air temperature [°C or K] and \( I \) is the heat radiation on the external boundary [W/m\(^2\)].

The heat transfer coefficient for external boundaries that are exposed to air are described with

\[ h_{\text{free}} = \begin{cases} 
5.6 + 3.95v & \text{for } v \leq 5 \text{ m/s} \\
7.8v^{0.78} & \text{for } v > 5 \text{ m/s}
\end{cases} \]  

(3.9)

where \( h_{\text{free}} \) is the heat transfer coefficient of a free surface exposed to air [W/(m\(^2\)K)] and \( v \) is the wind speed [m/s].
For different layers $i$, the heat flow from the external boundary to the surrounding environment can be described as a composite heat transfer coefficient. The different layers of the material are inserted in the model as separate blocks. The heat transfer coefficient for each material can then be calculated according to

$$h_{\text{surface}} = \left( \frac{1}{h_0} + \frac{1}{h_{\text{free}}} + \sum_{i=1}^{n} \frac{l_i}{k_i} \right)^{-1} \quad (3.10)$$

where

$$\frac{1}{h_0} = \begin{cases} \frac{1}{500} & \text{when the external boundary condition is set to "free surface"} \\ 0 & \text{when it is not set to "free surface"} \end{cases} \quad (3.11)$$

and

$$\frac{1}{h_{\text{free}}} = \begin{cases} \frac{1}{h_{\text{free}}} & \text{where } h_{\text{free}} \text{ is acc. to Eq. (3.9) when wind is simulated} \\ 0 & \text{when wind is neglected in the simulation} \end{cases} \quad (3.12)$$

Through this, the "free surface" is treated as a thin layer with the fictive value $\frac{1}{h_0} = \frac{1}{500}$ set to be numerically negligible compared to $\frac{1}{h_{\text{free}}}$. The user chooses whether or not to take the wind into consideration. However, when using Eq. (3.10) and the external boundary condition is set to "free surface", it is necessary to take the wind into account in the simulation.

### 3.2.2 Internal boundary conditions

Internal boundary conditions are necessary if embedded cooling pipes are to be simulated. In that case, the heat transfer coefficient is calculated according to

$$h_{\text{int}} = \left( \frac{1}{h_{\text{fl}}} \cdot \frac{r_v}{r_i} + \frac{r_v}{k} \cdot \ln \left( \frac{r_v}{r_i} \right) \right)^{-1} \quad (3.13)$$

where $h_{\text{int}}$ is the total heat transfer coefficient for the cooling pipe [W/(m²K)], $h_{\text{fl}}$ is the heat transfer coefficient from the flowing medium (air or water) to the inner surface [W/(m²K)], $r_v$ is the outer radius of the pipe [m], $r_i$ is the inner radius of the pipe [m] and $k$ is the thermal conductivity of the pipe material [W/(mK)].

### 3.3 Mechanical properties

#### 3.3.1 Strength growth

According to Hösthagen [29], the reference strength growth of concrete is defined in the three stages below.
3.3. MECHANICAL PROPERTIES

1. Fresh concrete \((0 \leq t_e < t_S)\).

2. Between initial and final setting \((t_S \leq t_e < t_A)\).

3. Hardened concrete \((t_e \geq t_A)\).

t_e is calculated according to Eq. (3.1), \(t_S\) is the equivalent time at initial setting, i.e. when the concrete starts to transform from a "liquid" to a "solid" state [h], and \(t_A\) is equivalent time at final setting, i.e. when the concrete has a "solid" surface [h].

The reference strength growth is calculated depending on the stage it is in [29] according to

\[
f_{cc}^{ref} = \begin{cases} 
0 & \text{for } 0 \leq t_e < t_S \\
\left(\frac{t_e-t_S}{t_A-t_S}\right)^{n_A} \cdot f_A & \text{for } t_S \leq t_e < t_A \\
\exp\left(s \cdot \left(1 - \left(\frac{672-t^*}{t_e-t^*}\right)^{n_{cc,28}}\right)\right) \cdot f_{cc,28} & \text{for } t_e \geq t_A
\end{cases}
\]  

(3.14)

where \(s\), \(n_A\) and \(n_{cc,28}\) are adjustment parameters without any physical representation [–], \(f_A\) is the concrete strength at final setting [Pa], \(t^*\) is calculated by Eq. (3.15) [h] and \(f_{cc,28}\) is the 28 days strength of the concrete [Pa].

\[
t^* = \frac{672 - \delta_c \cdot t_A}{1 - \delta_c}
\]

(3.15)

with

\[
\delta_c = \left(1 - \frac{1}{s} \cdot \ln \frac{f_A}{f_{cc,28}}\right)^{n_{cc,28}}
\]

(3.16)

where \(s\) is a parameter influencing the curve shape in time of the hardening concrete [–].

The tensile strength of the concrete, \(f_{ct}\), is related to the compressive strength, \(f_{cc}\), according to

\[
f_{ct} = \left(\frac{f_{cc}}{f_{cc}^{ref}}\right)^{\beta_1} \cdot f_{ct}^{ref}
\]

(3.17)

where \(f_{cc}^{ref}\) is calculated according to Eq. (3.14), \(\beta_1\) is a connection parameter for the strength according to Eurocode 1992-1-1 [–] and \(f_{ct}^{ref}\) is the reference tensile strength [Pa].

### 3.3.2 Stress development

The stress calculation is based on the perpendicular direction of the cross section that is under investigation, i.e. in the z-direction in Figure 3.1. This stress development can either be interpreted as uniaxial, i.e. that \(\sigma_x = \sigma_y = 0\) or cylindrical, i.e. that either \(\sigma_x\) or \(\sigma_y\) is equal to \(\sigma_z\) and the other one is equal to zero. The latter case...
arises when e.g. studying the risks of vertical cracks for a new slab that is casted on top of an old slab. However, the uniaxial stress case normally arises when analyzing the risk of cracks in a structure that is long in the z-direction.

The calculations are performed according to the Linear Line Model (LLM). This can be compared to a beam cross section during bending around one of its axes. In Figure 3.2, an example is shown where the stress variation in the y-direction of the wall is investigated. In other words, where we have maximum stress in the z-direction, we can expect the first crack to be perpendicular against the z-axis, i.e. in the y-direction. This is equivalent to determining the design for the cross section, for, in this case, bending around the x-axis.

![Figure 3.2: Left: A wall being casted on a slab. Right: The cross sectional forces of the wall for a fictive cut in the z-direction based on regular beam theory [14].](image)

If the element $dz$ from Figure 3.2 is studied more closely, it can be concluded that the wall can move in two possible ways. It can either deform through translation ($\varepsilon_{NL}$) in the z-direction or through rotation ($\omega_M$) around the x- or y-axis, respectively, as shown in Figure 3.3.

![Figure 3.3: The wall part with thickness $dz = 1$. O represents the original state and D is the deformed state, respectively. Reproduced from the version given by ConTeSt [14].](image)

If it is assumed that the cross section is completely constrained, then the forces that
are required to keep it fixed can be expressed as

\[
\begin{align*}
N^0 &= \int_A -E\varepsilon^0 \, dA \quad \text{for } \varepsilon_{NL} = 0 \\
M^0 &= \int_A -E\varepsilon^0(u - u_{NL}) \, dA \quad \text{for } \omega_M = 0
\end{align*}
\]  
(3.18)

where \( A \) is the cross sectional area [m\(^2\)], \( E \) is Young’s modulus [Pa], \( \varepsilon^0 \) is the non-elastic strain (caused by change in temperature or humidity), \( u \) is the x- or y-coordinate depending on direction of bending [m] and \( u_{NL} \) is the position of the neutral axis during rotation [m].

In relation to Eq. (3.18), the cross sectional forces \( N \) and \( M \) can be expressed as

\[
\begin{align*}
N &= k_N N^0 \\
M &= k_M M^0
\end{align*}
\]  
(3.19)

where \( k_N \) is the translation restraint factor and \( k_M \) is the rotation restraint factor. These factors are set to a value between zero (no resistance) and one (full resistance), depending on the boundary conditions of the structure, i.e. to which degree it is exposed to restraint from e.g. adjacent constructions or the foundation soil, respectively.

Once the resistance boundaries are determined, two equilibrium conditions can be introduced by combining Eq. (3.18) and Eq. (3.19) according to

\[
\begin{align*}
\int_A \sigma_z \, dA &= N = k_N N^0 \\
\int_A \sigma_z(u - u_{NL}) \, dA &= M = k_M M^0
\end{align*}
\]  
(3.20)

Subsequently, the translation, \( \varepsilon_{NL} \) and the rotation \( \omega_M \), respectively, are calculated and the stress variation in the cross section is then determined by

\[
\sigma_z = E(\varepsilon - \varepsilon^0) = E(\varepsilon_{NT} - \omega_M(u - u_{NL}) - \varepsilon^0)
\]  
(3.21)

At last, the stress - and strain ratio, respectively, is calculated according to

\[
\begin{align*}
\xi &= \frac{\sigma_z}{f_{\text{ct}}} \\
\eta &= \frac{\varepsilon_m}{\varepsilon_1}
\end{align*}
\]  
(3.22)

where \( f_{\text{ct}} \) is the tensile strength [Pa], \( \varepsilon_m \) is the strain associated to the stress and \( \varepsilon_1 \) is the strain associated with the tensile strength in the case of a linear (proportional) stress-strain relationship. The safety factor, \( S \), for the cracking risk is calculated, according to the Swedish guidance on building and civil engineering works (AMA) [30] with \( S = \frac{1}{\eta} \) or \( S = \frac{1}{\xi} \). The limit for the safety factor is, for a structure subjected to one-sided water pressure, \( S = 1.42 \), i.e. a strain/stress ratio of maximum 0.7 [30].
Chapter 4

Case study of a massive concrete wall

4.1 The power station of Storfinnforsen

The hydro power plant of Storfinnforsen is located in the community of Ramsele in the county of Västernorrland, north of Sweden, see Figure 4.1. The construction started in 1948 and the facility was taken into operation in 1954 [31]. The power plant consists of a 800 m long concrete buttress dam with 81 monoliths and a height varying between 6 and 41 m [32], as shown in Figure 4.1. A total of three turbine units produce on average 536 GWh combined annually [33]. Five spillways are placed along the dam. One sector gate and two segment gates are placed on high elevation and there are two bottom outlets, as shown in Figure 4.2.

Figure 4.1: Location of Storfinnforsen hydro power plant (a) and a view over the dam, including the river Faxälven (b). [31,34]
4.1.1 The bottom outlets

The purpose of the bottom outlets was to be able to drain water from the river while the dam was constructed. Once the dam was completed, these outlets were not used anymore. However, during an inspection years later, the left bottom outlet was deemed to be in unfit condition to serve as a proper outlet. It was considered to be more effective in terms of cost and time consumption to close it off with large concrete elements, since the discharge rate of the dam was fulfilled anyway. As for the right bottom outlet, a decision was made to renovate it to have the possibility to drain water on low elevation during a potential emergency. Behind this gate, in the downstream direction, a massive concrete wall served the purpose of diverting water masses. This particular wall, which from here on will be referred to as "the previous wall", was divided in six segments as seen in Figure 4.3.

![Figure 4.2: An overview of the spillways in the downstream direction. The vertical lines represents the monoliths [33].](image)

![Figure 4.3: View over the previous wall (a) and its six segments (b) in the downstream direction. [33]](image)

Due to increased demands of spilling through the outlets, the previous wall was required to be upgraded in terms of design and strength, to be able to safely withstand
potential extreme situations without being flooded. Another factor in consideration was the curvature of it, which would divert more water than expected in the left direction if the gate was opened, creating an increased and unnecessary water load on that side. Numerous actions were taken in the process of solving these problems by designing a new wall, which are described in Section 4.2.

4.2 The new wall

4.2.1 Design procedure

A laboratory experiment was performed by Vattenfall AB and R&D Laboratories between 2014 and 2015, where a prototype of a predetermined part of the dam was built in scale 1:50. The purpose of the experiment was to investigate the behavior of the wall for a new design proposal during extreme situations. It was concluded that segments 1-3 of the previous wall needed to be demolished and replaced, see Figure 4.4.

![Figure 4.4: The previous wall and its six segments. The hatched part, i.e. the first three segments, were removed [35].](image)

The most significant design change of the new wall segments was that they were designed with no curvature, including an inclined cap construction on the top of the wall to minimize the risks of flooding. In order for these adjustments to be practically possible, rock masses behind the previous wall had to be blasted in order to make room for the new one, see Figure 4.5.
The new wall consists of four 10 m wide segments with vertical joints in between as shown in Figure 4.6. These replaced the previous segments denoted no. 1 to 3 in Figure 4.5. Because of the size of the new structure, it was necessary to cast it in different stages. The casting sequence is illustrated by numbers within circles in Figure 4.6. These should not be confused with the segment numbering from previous figures.

Within the framework of this project, only the new casting stage 5 was considered and investigated. By visualizing a vertical cut in the center of this part, its cross section can be seen in Figure 4.7. For practical reasons, horizontal construction joints were placed on levels +241.77 m and +242.55 m, respectively. Stage 5 of the casting process was up until the first joint, i.e. level +241.77 m.
4.2. THE NEW WALL

Figure 4.7: The studied cross section (a) and the corresponding reinforcement placement (b) [35]

The placement of reinforcement in the studied cross section is shown in Figure 4.7. It consists exclusively of K500C-T $\varnothing16$ mm bars. These are standard, hot-rolled and profiled steel bars with a characteristic yield strength of 500 MPa [36].

4.2.2 Practical procedure and technical data

Certain conditions were defined by WSP for the contractor, responsible for the concrete casting, to follow. The allowable casting temperature for the concrete was dependent on the Average Daily Temperature, ADT, see Table 4.1. In addition, there were also certain technical requirements for the material itself, see Table 4.2.

Table 4.1: Approximate allowable casting temperatures depending on the Average Daily Temperature, ADT [35].

<table>
<thead>
<tr>
<th>ADT</th>
<th>$T_{casting}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$&lt; 5^\circ\text{C}$</td>
<td>$10 - 12^\circ\text{C}$</td>
</tr>
<tr>
<td>$5 - 13^\circ\text{C}$</td>
<td>$15^\circ\text{C}$</td>
</tr>
<tr>
<td>$&gt; 13^\circ\text{C}$</td>
<td>$ADT + 2^\circ\text{C}$ but not $&gt; 20^\circ\text{C}$</td>
</tr>
</tbody>
</table>
Table 4.2: Required technical properties of the concrete [35].

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class</td>
<td>C32/40</td>
</tr>
<tr>
<td>Cement type</td>
<td>Portland CEM I</td>
</tr>
<tr>
<td>Maximum cement content</td>
<td>360 kg/m³</td>
</tr>
<tr>
<td>Allowable range of w/c ratio</td>
<td>0.48 − 0.53</td>
</tr>
<tr>
<td>Exposure class</td>
<td>XC4/XF3</td>
</tr>
<tr>
<td>Chloride class</td>
<td>CI 0.20</td>
</tr>
<tr>
<td>Highest nominal grain size for the aggregate</td>
<td>32 mm</td>
</tr>
<tr>
<td>Consistence class</td>
<td>S2 (viscous)</td>
</tr>
<tr>
<td>Maximum average casting rate</td>
<td>0.5 m/h</td>
</tr>
</tbody>
</table>

For the formwork, fir timber boards with the dimensions 22×95 mm were used. The formwork was demolished after seven days for all construction parts. For the vertical concrete surface, the formwork was non bearing but designed for the horizontal pressure. However, for the inclined cap construction on top of the wall, a bearing formwork system was required for safety reasons.

4.3 Numerical model

As mentioned in Chapter 3, the simulations in this project were performed using the software ConTeSt. Since the simulations were used as a prediction before the on-site casting, uncertainties were unavoidable. The weather, for instance, can only be estimated using statistics from meteorological data. Consequently, the assumptions made are listed below.

- Ambient temperature was assumed equal to 13°C since the casting was expected to occur in the beginning of June.
- The temperature of the surrounding rock was assumed as 10°C.
- The inclined concrete surface against the rock (see Figure 4.7) was assumed to be perfectly flat to simplify the geometry.
- The wind speed was defined as 1 m/s.
- Full restraint between the concrete and the rock was assumed.
- The rock thickness under the wall was assumed to 1 m.

A sensitivity analysis was performed in order to evaluate the effects from the uncertainties in the temperature development, strain development and the cracking risk. This was performed by varying parameters listed below.
4.3. NUMERICAL MODEL

1. Ambient temperature.
2. Wind speed.
3. Cement content.
4. Degree of restraint in translation.

Each parameter was varied separately while the other factors were held constant in order to isolate the effect from every parameter. In order to perform this, some initial assumptions for each factor were needed and these are listed above. It was described in Section 3.3.2 that an element in ConTeSt can deform through movement in translation or rotation about the x- and y-axis. In the following model, the degree of restraint was only varied in translation.

4.3.1 Model and material definitions

The geometry in ConTeSt can be seen in Figure 4.8. It was designed with reference points inserted as coordinates. The concrete wall and the rock, respectively, were inserted as separate blocks.

![Figure 4.8: Geometry in ConTeSt with purple being the concrete wall and green being the surrounding rock.](image)

As previously mentioned, there is a horizontal construction joint at the level +241.77 m, as shown in Figure 4.7. Thus, the geometry of the wall was defined up to that level.
CHAPTER 4. CASE STUDY OF A MASSIVE CONCRETE WALL

In ConTeSt, there are pre-defined materials that could be chosen, but there is also the option of creating new materials with own definitions. In this project, a pre-defined material was chosen due to lack of information regarding the exact material properties. However, in practice, the needed parameters can be obtained through various material tests, see Section 4.4.

Materials are either defined as block materials or boundary conditions. Early age concrete, old concrete, rock, steel and ground are all examples of block materials. The only hydrating block material that was defined was "early age concrete" and its pre-defined properties are shown in Table 4.3. The w/c ratio of the pre-defined material was lower than the condition in Table 4.2. The thermal and mechanical properties in Table 4.3 are described in Sections 2.2.2, 3.1 and 3.3.1, respectively. The non-hydrating block materials are defined by their density, heat capacity and thermal conductivity. Boundary conditions often represents the formwork, other coverings, insulation or materials with own defined heat transfer coefficients. All of the boundary conditions are defined as materials with a heat transfer coefficient or with a combination of the thickness of the material and the thermal conductivity.

Table 4.3: Selected material properties for the concrete in ConTeSt.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength class</td>
<td>C32/40</td>
</tr>
<tr>
<td>Cement type</td>
<td>Portland Degerhamn</td>
</tr>
<tr>
<td>Cement content</td>
<td>360 kg/m³</td>
</tr>
<tr>
<td>w/c ratio</td>
<td>0.45</td>
</tr>
<tr>
<td>Density</td>
<td>2350 kg/m³</td>
</tr>
<tr>
<td>Heat capacity, $C_t$</td>
<td>1000 J/(kgK)</td>
</tr>
<tr>
<td>Total heat energy by cement weight, $q_u$</td>
<td>345 kJ/(kgK)</td>
</tr>
<tr>
<td>Free model parameter, $t_1$</td>
<td>4.5</td>
</tr>
<tr>
<td>Free model parameter, $\kappa_3$</td>
<td>1.71</td>
</tr>
<tr>
<td>Adjustment parameter, $\Delta t^0_c$</td>
<td>0</td>
</tr>
<tr>
<td>Adjustment parameter, $\beta_\Delta$</td>
<td>1</td>
</tr>
<tr>
<td>Reference maturity parameter, $\theta_{ref}$</td>
<td>4200 K</td>
</tr>
<tr>
<td>Parameter, $\kappa_3$</td>
<td>0.5</td>
</tr>
<tr>
<td>28 days strength of the concrete, $f_{cc,28}$</td>
<td>48 MPa</td>
</tr>
<tr>
<td>Adjustment parameter, $s$</td>
<td>0.349</td>
</tr>
<tr>
<td>The equivalent time at initial setting, $t_S$</td>
<td>5.719 h</td>
</tr>
<tr>
<td>The equivalent time at final setting, $t_A$</td>
<td>8.579 h</td>
</tr>
<tr>
<td>Adjustment parameter, $n_A$</td>
<td>1.803</td>
</tr>
</tbody>
</table>

The only block materials in this model are early age concrete and the surrounding rock. The concrete properties that was manually defined are listed below.

- The casting rate was defined as 0.5 m/h from where ConTeSt automatically calculates the end of the casting to approximately 12 hours.


- Following the conditions stated in Table 4.1, the initial temperature of the concrete mass was defined equal to 15°C since the ambient temperature was assumed to be 13°C.

The properties of the rock are pre-defined in ConTeSt and listed below.

- The temperature was defined as 10°C and constant since the daily variation was assumed to be insignificant.
- The density was defined as 2650 kg/m$^3$.
- The heat capacity was defined as 850 J/(kgK).
- The thermal conductivity was defined as 3.7 W/(mK).

4.3.2 Boundary conditions

The boundary conditions between two block materials, in this case the concrete and the rock, are defined by themselves when the two block materials are connected by their common reference points and have materials assigned to them. Other boundary conditions are defined as materials with their respective HTCs. The boundary conditions listed below were created.

- The top surface of the concrete wall, highlighted dark blue in Figure 4.9, was sealed with a plastic sheet and removed at the same time as the formwork, i.e. after 7 days. The boundary material "Sealing" was created with a cellular plastic material of 2 mm thickness.
- The formwork was built up by 22 mm thick wooden boards and placed at the left side of the wall, see the light blue marking in Figure 4.9. The boundary material "Formwork" was created with the respective properties assigned.
- The rock has two surfaces exposed to air and hence the boundary condition "Free surface" was created, see the red marking in Figure 4.9.
- In order to simulate that the ground is continuing downwards in Figure 4.9, a boundary condition was created with a material with the HTC of 900 W/m$^2$K at the bottom edge of the rock, see the white marking in Figure 4.9.

The external boundaries of the rock, marked purple in Figure 4.9, were not necessary to define since the software automatically recognizes them as adiabatic. The duration of the sealing was modelled in ConTeSt by creating a boundary condition that varies with time. Two different materials were needed to be defined. The sealing plastic sheet was created from the time 0 to 168 hours, i.e. up to 7 days. After the removal of the plastic sheet, the boundary condition "Free surface" was implemented after 168 hours. The same procedure was performed for the formwork.
where a material of 22 mm wooden boards was modelled to be present the first 168 hours.

Figure 4.9: Geometry in ConTeSt highlighting the external boundary conditions.

### 4.3.3 Convergence analysis of the element size

ConTeSt has a pre-defined element size of 0.150 m for a block and 0.040 m for an external boundary, but these can also be adjusted manually. A convergence analysis was performed in order to check the quality of the simulations. The convergence was checked for the maximum temperature and maximum strain ratio in the wall for the element sizes 0.100 m, 0.150 m and 0.200 m. The tolerance for the convergence was 1% and the convergence for the maximum temperature and maximum strain ratio is shown in Table 4.4. A higher element size than 0.150 m could be used for the maximum temperature but the maximum strain ratio did not converge for the element size 0.200 m. The pre-defined element size of 0.150 m would be enough to use in the simulations. However, since results can only be obtained in the nodes of an element, extracting results in specific points in the wall becomes difficult for larger element sizes. In order to extract results in a specific point in the wall, this point needs to be located in a node. By dividing the wall into smaller elements, more nodes are created and hence more points to extract results from. With this reasoning, the element size was decreased in order to be able to extract results in desirable points in the wall. The chosen element size was 0.035 m.
### 4.4 Material tests

The thermal and mechanical properties in Table 4.3 were obtained from a pre-defined concrete material in ConTeSt, as mentioned earlier. Material properties of the concrete used in the investigated wall would create a more accurate model and these were needed to be determined through different material tests. For this model, tests on the heat of hydration and the strength growth were needed.

Tests on the strength growth are performed in order to determine the reference strength growth, $f_{c}^{ref}$ (see Eq. (3.14) in Section 3.3.1) and the maturity factor, $\beta_t$ (see Eq. (3.2) in Section 3.1). The reference strength curve is dependent on the equivalent maturity age, $t_e$ (see Eq. (3.1) in Section 3.1) and the conditions for which the reference strength curve is defined are $\beta_t = 1$ ($T = 20$°C in Eq. (3.2)), $\beta_\Delta = 1$ and $\Delta t_0^e = 0$. The given values on $\beta_\Delta$ and $\Delta t_0^e$ are used when a recipe is tested for the first time. By testing the recipe a second time but with different type and/or amount of admixtures, the parameters are adjusted, i.e. $\beta_\Delta \neq 1$ and $\Delta t_0^e \neq 0$. The reference maturity parameter, $\theta_{ref}$, and the parameter $\kappa_3$ can be determined by testing the strength growth for different temperatures of the concrete. The tests on the strength growth are performed on several specimens that have the same recipe but different concrete temperature. [29]

The heat of hydration can be determined from measured temperatures in a semi-adiabatic test where the released heat energy at a certain time point is given by

$$q_{cem}(t) = \frac{\rho \cdot c}{C} \cdot \left( \eta \cdot (T(t) - T_{ambient}) + \int_0^t (T(t) - T_{ambient}) \cdot dt \right)$$  \hspace{1cm} (4.1)$$

where $\rho$ is the density [kg/m$^3$], $c$ is the specific heat per unit weight [J/(kgK)], $C$ is the cement content [kg/m$^3$], $\eta$ is a correction factor due to the heat stored in the test set-up, $T$ is the measured temperature of the concrete specimen [K or °C], $T_{ambient}$ is the ambient temperature [K or °C] and $a$ is a cooling factor [1/s]. As mentioned in Section 3.1, Eq. (3.3) is an approximation of $q_{cem}(t)$ used for computer calculations and hence the parameters $\kappa_1$ and $t_1$ are free model parameters used to get an acceptable fit with the data obtained from the semi-adiabatic test. [29]
According to the documents of the company, these tests were not performed on the concrete that was used for the wall. Thus, the material properties of the real concrete material were not known and hence a pre-defined material was used in ConTeSt.
Chapter 5

Results and discussion

The presented results from the sensitivity analysis in this chapter are obtained from ConTeSt for the maximum temperature point in the wall. Some of the results from other locations in the wall are also presented in order to have a broader discussion about the analyzed effects. The results that are not presented in this chapter can be found in Appendix A. The crack risk analysis was performed for the whole cross section and not for specific points as in the sensitivity analysis.

5.1 Sensitivity analysis

The maximum temperature in the wall was approximately 46°C and located in the middle of the wall at a height of about 6 m, as shown in the contour plot in Figure 5.1. This temperature was registered 62 hours after the casting was started, or 50 hours after the casting was completed since the casting time was about 12 hours, as explained in Section 4.3.1.

Figure 5.1: Contour plot from ConTeSt showing the temperature in the wall and surrounding rock 62 hours after casting was started.
5.1.1 Effects from the cement content

Figure 5.2 shows the influence from the cement content on the temperature development for the maximum temperature point in the wall. It can be noted that higher cement content increases the temperature in the concrete. This is due to the fact that the water has more cement to react with, which increases the heat generation during hydration as mentioned in Section 2.2.1. As the temperature in the concrete increases, the concrete expands followed by natural cooling and contraction, as described in Figure 2.4 in Section 2.2.3. The concrete cools down to approximately the same temperature in all of the cases in Figure 5.2, but the maximum temperature is higher for a higher cement content. This results in a larger temperature difference over time for a concrete with higher cement content.

As a result of the larger temperature difference in the concrete for a higher cement content, an increase in strain development occurs as shown in Figure 5.3. A higher maximum temperature yields more expansion, but also more contraction over time, since the concrete cools down to the same temperature for all the cases in 5.2, as mentioned. The strain ratio becomes unstable during a specific time period for the case when the cement content is equal to 370 kg/m³. ConTeSt calculates the strain/stress until cracking occurs, i.e. until the strain/stress ratio exceeds 1. The calculations after cracking becomes unstable and hence the result of the dotted curve in Figure 5.3. The instability of the calculations increases with increasing strain ratio, usually when it reaches \( \eta = 1.2 - 1.3 \). The time of cracking is illustrated
with the circular marking in Figure 5.3. The negative strain ratio of approximately −1.2 is not a sign of a crushing risk as one might suspect. Instead, it represents developed compressive stresses and is merely due to sign convention. The developed compressive stress at that time was approximately 2 MPa, whereas the compressive strength at the same time was approximately 20 MPa. Similar negative strain ratios were obtained for the other strain developments in this chapter. To reduce the cement content, one could use larger size of aggregates or use alternative binders such as e.g. fly ash, see Section 2.2.4.

![Figure 5.3: The evolution of the strain ratio over time for different cement contents. Cracking occurs at the circular marking in the graph.](image)

### 5.1.2 Effects from the ambient temperature

The temperature development in the concrete for the maximum temperature point and different ambient temperatures is presented in Figure 5.4. For an ambient temperature of 20°C, the temperature of the fresh concrete was also changed to 20°C according to the conditions mentioned in Section 4.2.2. Accordingly, the temperature of the fresh concrete was set to 15°C for the other ambient temperatures in Figure 5.4. As mentioned in Chapter 2, higher temperature of the fresh concrete yields greater rate of hydration and hence results in higher temperatures in the concrete as shown in Figure 5.4. This is also illustrated by the initially higher temperature development for the dotted curve in Figure 5.4 as a result of a higher rate of hydration. It can also be noted that the difference in maximum temperature between the dotted curve and the rest of the curves is approximately 6-7°C, whereas the difference in initial temperature is 5°C. Thus, the ambient temperature only contributes to a 2-3°C
increase in temperature and the initial temperature of the fresh concrete has the biggest influence on the increase in temperature of the concrete.

The maximum temperature in the concrete is approximately the same for both the 10°C and 13°C ambient temperatures. The concrete cools down until it approximately reaches the ambient temperature, which in this case results in more cooling when the ambient temperature is 10°C, as shown in Figure 5.4. This causes more contraction of the concrete since there is slightly more cooling. The effects from the different ambient temperatures are of course bigger for points closer to the outer surface of the wall, as illustrated in Figure 5.5. It is clear that there are larger differences in the temperature development between the cases, but also a more rapid cooling after the removal of the formwork compared to Figure 5.4.

Figure 5.4: Temperature development for different ambient temperatures for the maximum temperature point in the wall.
5.1. SENSITIVITY ANALYSIS

Figure 5.5: Temperature development for different ambient temperatures in a point closer to the outer surface of the wall. A more rapid cooling occurs when the formwork is removed after 168 hours.

The strain development for the maximum temperature point in the wall is presented in Figure 5.6. The same unstable behavior occurs as discussed earlier for the cement content in Figure 5.3 and hence cracking occurs for the ambient temperature 10°C. Since there was more cooling of the concrete for the ambient temperature 10°C (see Figure 5.4), more contraction is to be expected, hence a higher strain development occurs and also cracking. Similarly to the temperature development, the difference in strain development for different ambient temperatures in a point closer to the surface of the wall, was larger. However, the results showed no cracking of the concrete for those points.
CHAPTER 5. RESULTS AND DISCUSSION

5.1.3 Effects from the wind speed

The effect from different wind speeds on the temperature development in the concrete for the maximum temperature point is shown in Figure 5.7. The difference in temperature between the wind speeds 6 m/s and 12 m/s is not that significant, but both contribute to a more rapid cooling compared to a wind speed of 1 m/s. The maximum temperature is slightly lower for the wind speeds 6 m/s and 12 m/s, compared to 1 m/s and the cooling rate is approximately the same in the beginning. The cooling rate slightly increases after approximately 170 hours. The reason for this is the removal of both the formwork and the covering on the top surface that occurs after 168 hours, which exposes the concrete to air. The effect from the removal of the formwork and covering can be better visualized in Figure 5.8, for a point closer to the outer surface of the wall. By using some type of weather protection, the temperature in the concrete would increase and the cooling rate would decrease.

Figure 5.6: The evolution of the strain ratio for different ambient temperatures for the maximum temperature point. Cracking occurs for the lowest ambient temperature.
Figure 5.7: Temperature development for different wind speeds for the maximum temperature point.

Figure 5.8: The temperature development in a point closer to the outer surface of the wall. More rapid cooling occurs when removing the formwork and covering.
The strain development for the maximum temperature point is shown in Figure 5.9. It can be noted that cracking occurs for the wind speeds 6 m/s and 12 m/s as a result of more cooling and contraction of the concrete.

Figure 5.9: The evolution of the strain ratio for different wind speeds for the maximum temperature point. Cracking occurs for the wind speeds 6 m/s and 12 m/s.

5.1.4 Effects from the degree of restraint

Figure 5.10 shows the effect from the degree of restraint in translation on the strain development in the concrete. The strain increases for increasing degree of restraint as expected, even if the difference between the cases is not that large. The difference in strain development between the cases would probably be larger if the degree of restraint was varied along the width of the wall, i.e. the restraint from the rock and formwork at the sides. The calculations by ConTeSt on the strain ratio in Figure 5.10 are stable, compared to the Figures 5.3, 5.6 and 5.9, respectively. However, the strain ratio exceeds 1 and thus there is still a crack risk for all the cases.
5.1.5 Combination of factors

By combining the factors discussed in this section, the cases in Figure 5.11 are obtained. Combining all the factors that gives the highest temperature in the concrete gives Case a), i.e. the temperature development for the wind speed 1 m/s, cement content 370 kg/m³ and ambient temperature 20°C with the same temperature on the fresh concrete. Case b) represents the initial assumptions with the wind speed 1 m/s, cement content 360 [kg/m³] and ambient temperature 13°C. Similarly, by combining the factors giving the lowest temperature in the concrete, Case c) is obtained, i.e. the temperature development for the wind speed 12 m/s, cement content 335 kg/m³ and ambient temperature 10°C.

Even if one factor does not have a significant impact on the temperature development in the concrete, by combining the factors, these effects can increase as shown in Figure 5.11.
5.2 Crack risk analysis

Figure 5.12 shows the tensile strength and the maximum stress developed for different cement contents. According to the figure, cracks are expected to occur for the cement contents 360 kg/m³ and 370 kg/m³ since the maximum tensile stress exceeds the tensile strength for those cases. The cracking risk is marked with the two circles in Figure 5.12. Earlier, it has been discussed that higher cement content contributes to an increase of the temperature in the concrete and an increase of strain ratio, see Figures 5.2 and 5.3 respectively. It is thus expected for the tensile stresses to increase with higher cement content as the strain is directly related to the stress and dependent on the temperature in the concrete. By comparing Figures 5.12 and 5.3 for the cement content 370 kg/m³, it can be noted that the cracking in Figure 5.12 occurs at the same time as the unstable behavior of the strain ratio begins in Figure 5.3. This confirms the relation between cracking and the instability of the strain/stress calculations in ConTeSt discussed earlier. It can also be noted from the results in Figure 5.12, that there is a sudden increase in the tensile stress development after 168 hours. This is due to the removal of the formwork and covering that leaves the concrete exposed to air. As a result, there is more impact from the
environment as e.g. from different wind speeds, as shown in Figure 5.14. By using some type of insulation or weather protection after the removal of the formwork and covering, these effects could be decreased.

The maximum stress developed in the long-term increases with decreasing ambient temperature as shown in Figure 5.13. It was mentioned earlier that the cases with ambient temperatures 10°C and 13°C in Figure 5.4 reached approximately the same maximum temperature at the center of the concrete. However, the concrete cooling was greater for the lower ambient temperature of 10°C. This in combination with restraint could have caused the stress to increase and thereby the tensile strength was exceeded in Figure 5.13. Again, the relation between the unstable behavior of the strain ratio in Figure 5.6 and the cracking occurrence in Figure 5.13 can be visualized by comparing the respective Figures. It can be noted that the crack risk is very high for most of the cases and that the structure is on the limit of cracking. The cracking risk is illustrated in the figures with the circular marking. With the same principle as in the sensitivity analyses, the factors discussed were combined in order to obtain the two extreme cases showed in Figure 5.16.

Figure 5.12: The development of the tensile strength (dashed lines) and maximum stress (solid lines) for different cement contents.
CHAPTER 5. RESULTS AND DISCUSSION

Figure 5.13: The development of the tensile strength (dashed lines) and maximum stress (solid lines) for different ambient temperatures. The maximum stress increases for lower ambient temperatures.

Figure 5.14: The development of the tensile strength (dashed lines) and maximum stress (solid lines) for different wind speeds.
Figure 5.15: The development of the tensile strength (dashed lines) and maximum stress (solid lines) for different degrees of restraint in translation.
Figure 5.16: The development of the tensile strength (dashed lines) and maximum stress (solid lines) for combination of the factors giving the lowest stress development, case a) with the cement content 335 [kg/m$^3$], ambient temperature 20°C, wind speed 12 m/s and no restraint. The highest stress development is given by case b) with the cement content 370 [kg/m$^3$], ambient temperature 10°C, wind speed 1 m/s and full restraint.
Chapter 6

Measurement plan

6.1 Suggested placement of measuring equipment

Based on the results in Chapter 5, certain points in the cross section were considered interesting. In Figure 6.1, a compilation of all the points that were investigated are marked A-G.

Figure 6.1: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G.
The exact coordinates of points A-G are presented in Table 6.1, which are measured from the point at the left side of the wall at level +235.73 according to Figure 6.1.

Table 6.1: Coordinates of the measuring points A-G.

<table>
<thead>
<tr>
<th></th>
<th>x (mm)</th>
<th>y (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>70</td>
<td>1690</td>
</tr>
<tr>
<td>B</td>
<td>243</td>
<td>1690</td>
</tr>
<tr>
<td>C</td>
<td>70</td>
<td>3840</td>
</tr>
<tr>
<td>D</td>
<td>453</td>
<td>3840</td>
</tr>
<tr>
<td>E</td>
<td>70</td>
<td>5130</td>
</tr>
<tr>
<td>F</td>
<td>583</td>
<td>5130</td>
</tr>
<tr>
<td>G</td>
<td>1013</td>
<td>5130</td>
</tr>
</tbody>
</table>

The reasoning behind the choice of the points presented in Table 6.1 were based on the obtained numerical results. The maximum temperature point in Chapter 5 is point F in Figure 6.1. Point B is the maximum temperature point 24h after the start of casting and point D after 36h. It was concluded that the maximum temperature approximately moves along the line between B-D-F in Figure 6.1. Points A, C and E are chosen based on the location of reinforcement bars, i.e. 70 mm from the outer surface of the wall. Point G was placed to the right of point F in order to capture the temperature development closer to the rock.

### 6.2 Method of monitoring and instrumentation

A suitable plan for measurement execution is suggested in this section. It is worth noting that this content, which includes equipment products and a strategy for instrumentation, should be treated as a suggestion and not a requirement. Equivalent data could be acquired with other products as well. The measurements have solely been based on strain- and temperature development within the concrete structure, for the investigated cross section described in Section 4.2.

#### 6.2.1 Measuring equipment

**Strain gauges**

To be able to measure strain- and temperature development, a total of seven (7) strain gauges of module KM-100B from Tokyo Sokki Kenkyujo [37] are needed, see Figure 6.2 and Table 6.2. These transducers are especially suitable for materials which undergo a transition towards a hardened state, making it an ideal choice for the concrete structure that is under investigation in this project. With the help of a thin steel wire that is tensioned between the edges of the gauge, it reacts to
compression and elongation, enabling it to measure strain development in real time. Also, because of the waterproof design, they can be directly installed inside the formwork before casting. In addition, each unit includes a thermocouple sensor by which relative temperature can be measured [37].

Figure 6.2: Strain transducers of module KM-100B with specifications given in mm [37].

Table 6.2: Specifications of the KM-100B sensor [37].

<table>
<thead>
<tr>
<th>Specification</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Capacity</td>
<td>±5000 × 10^{-6} strain</td>
</tr>
<tr>
<td>Rated output (RO)</td>
<td>2.5 mV/V (5000 × 10^{-6} strain)</td>
</tr>
<tr>
<td>Non-linearity</td>
<td>1% of RO</td>
</tr>
<tr>
<td>Allowable temperature range</td>
<td>−20 ～ +80°C</td>
</tr>
</tbody>
</table>

**Data logger, laptop and software**

Two (2) data loggers of module Spider8 from HBM [38] are needed to collect data from the strain gauges, see Figure 6.3 and 6.4, respectively. The specifications of the logger are given in Table 6.3. Each logger has 8 channels available and each transducer will be using two of these each to measure strain - and temperature, respectively. However, this is based on the assumption that strain and temperature is to be measured at the same points. If one is interested in measuring temperature in additional points, other than where the transducers are to be located, there are many options on the market for this. Basically any cable with thermocouple sensors that is waterproof would be sufficient. In that case, it would probably require at least one more data logger, depending on how many other locations that temperature is to be measured.
A regular laptop, preferably with Windows as operating system, is needed. A software from HBM called Catman [40] is compatible with the equipment and can collect the desired results, making it an ideal choice. The software has two versions called Catman AP and Catman Easy, respectively. Simply put, Catman AP is more suitable for experienced users and especially for long-term experiments. Catman Easy, however, is considered to be more modern and user friendly, hence its name. In that sense, there is no advantage for a new user, especially for a short-term experiment like this, to use Catman AP. Therefore, Catman Easy is strongly recommended. [41]

6.2.2 Data acquisition

To be able to acquire the data from the transducers, it is important that they are mounted in a proper way. This includes the direction of which they are placed in, which affects how the strain is registered. It is preferable to connect them to reinforcement bars with cable ties where it is possible. If it is a necessity to place a device where there is no steel bar near, the experimenter has to manually solve the issue on site by hanging the device in a sufficiently proper way with the use of steel wires and cable ties.

9 mm 5-core shielded cables are recommended for this project, see Figure 6.5. These are included with the transducers but has to manually be cut and divided into two parts since both strain - and temperature wants to be measured. This is easily performed with the provided description. The exact cable length that is required for the whole experiment is difficult to predict but at least 25 m is recommended. [37]
6.2. METHOD OF MONITORING AND INSTRUMENTATION

There are two options to choose from when deciding how to divert the cables out of the form. The first option is to let the cables go up in the vertical direction, towards the upper surface. The second option is to drill suitable holes in the timber boards of the formwork to let the cables pass through in the left horizontal direction [41]. However, to minimize cable damage and to make the demolishing of the formwork efficient for the workers, the first option is recommended. Also, to avoid interference with the results generated by the transducers, it is recommended to keep the cables tied to reinforcement bars as much as possible on their way out of the form.

It is important to have a compilation of the exact locations of all of the transducers after they have been successfully mounted. Subsequently, it is also important to keep track of which cables that are connected to which transducer before plugging them in the data loggers. The experimenter needs to strategically plug in the cables to pre-determined sockets of data logger to be able to review the obtained results and know which transducer that generated them [41]. Once all the cables are outside the form and attached in the chosen sockets, they are preferably bundled together with cable ties for practical purposes.

To protect the equipment from weather, an instrumentation cabinet is required. The data loggers and the computer are kept in this cabinet, along with all the cables. In practice, this is usually a box made of stainless steel which can be purchased as a unit and easily mounted on-site. The most important factor to consider is the size of it to make sure that there is room for all of the equipment. For this project, with the equipment that is suggested, a box with at least the (inner) dimensions $h = 500$ mm, $d = 500$ mm and $w = 400$ mm is recommended. An exact position to mount the box is difficult to say beforehand since it is much dependent on the conditions on site. However, it is preferably mounted on the rock masses as close as possible to the concrete form. [41]

The last step is to use the computer software. In Catman, certain information is expected by the user, e.g. which type of transducers that are being used. The start time and the frequency of the measuring is also saved as input. At last, when the measuring process is started, values are saved for each device for the chosen frequency. This information can be imported to an excel sheet where it can be easily visualized with graphs. The experimenter is not required to be present on the construction site during the measuring process. The data that is saved can easily be accessed from another computer through the software. [41]
To perform this measurement, certain additional tools are needed to be able to do the practical installation. A few suggestions of this are listed below.

- Notebook, pen and drawings.
- Measuring tape.
- Cable ties and steel wires.
- Nipper.
- Power strip for the logger and laptop charger.
- Soldering iron and electrical tape in the case of e.g. cable repairs on site.
Chapter 7

Conclusions and further research

In this project, numerical analyses have been performed to predict the temperature development, strain development and the risk of cracking in a massive concrete wall during early age. In addition, sensitivity analyses have been performed to study the influence from parameters such as cement content, ambient temperatures, wind and degree of restraint.

It is evident from the results in Chapter 5 that there was a crack risk of the wall, since the strain ratio exceeded the limit of 0.7 for the maximum temperature point (point F in Figure 6.1). The strain ratios for the other points in the wall, that were investigated, also were either close to the limit, or exceeded it for all the cases. It can be concluded that cracking occurred in the maximum temperature point, for the case with the cement content of $370 \text{ kg/m}^3$, the case with the ambient temperature of $10^\circ\text{C}$ and the cases with the wind speeds of $6 \text{ m/s}$ and $12 \text{ m/s}$ respectively. This conclusion was based on the instability of the strain ratio calculations in ConTeSt, that occur due to cracking of the structure, as discussed earlier in Chapter 5. A higher cement content increased the rate of hydration and hence the temperature in the concrete. This effect had the most important impact at the core of the wall, since the temperature due to hydration is higher at the internal parts. As a result of the increased temperature at the core, the concrete cracked. An increase of the ambient temperature resulted in an increase of the temperature in the concrete and also lead to more expansion. However, a higher ambient temperature only contributed to an increase of approximately 2-3°C in temperature of the concrete at the core (point F). Instead, a larger contribution came from a higher initial temperature of the fresh concrete which increased the rate of hydration and hence the temperature in the material. However, a higher cracking risk occurred for lower ambient temperatures due to the internal restraint from a high temperature gradient between the outer surface and the internal parts of the wall. A higher wind speed contributed to more cooling of the concrete and hence more contraction. The cracking risk was higher for higher wind speeds. Even if the concrete did not crack in point F for the rest of the cases, it was still on the limit of cracking, since the strain ratios were close to 1. This can also be seen from the results of the crack risk analysis, where the crack risk was present for almost all of the cases. The results for the points closer to the surface of the wall (points A, C and E in Figure 6.1) showed no cracking, i.e. no
surface cracks. This is important since the water could not reach the reinforcement. However, the strain ratios were also high for those points and usually close to 1, see Appendix A.

The environment had more impact on the investigated points closer to the outer surface of the wall, since the difference in temperature- and strain development, for different ambient temperatures and wind speeds, was larger for those points. After the removal of the formwork and covering, these effects increased where a rapid cooling of the concrete occurred. To avoid this sudden increase of the cooling rate, some type of insulation could be implemented after the removal of the formwork and covering.

A suggestion of a measurement plan was established based on the results from the numerical analyses. By following the description of the measurement plan, the measurements on the temperature- and strain development can be performed on-site and used as a comparison with the numerical results. The equipment needed are 7 strain gauges that can measure temperature and strain, 2 data loggers to collect data, at least a 25 m 9 mm 5-core shielded cable and a computer with the software Catman Easy to process the data.

A pre-defined material from ConTeSt, with built in properties, was hard to apply on a specific case. In order to establish a more accurate prediction, informations on the material properties of the concrete that is used at casting, were needed. These can be determined from material tests on the heat of hydration and strength growth, as described in Section 4.4. For further research, it is recommended to perform these material tests in order to create a material in ConTeSt that more accurately describes the real material. To check the quality of the prediction, it is recommended to perform the measurements on-site, using e.g. the description of the measurement plan in Chapter 6 and compare the results with the results from the model. Also, the restraint from the sides of the wall would have to be modelled, i.e. the restraint from the rock and formwork on the expansion/contraction of the wall along the width.
Bibliography


[41] S. Trillkott. Laboratory Engineer, KTH, Department of Civil Engineering [Personal meeting; May 25 2018].
Appendix A

Numerical results

A.1 Measurement point G

Figure A.1: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point G are presented below.
A.1.1 Temperature development

Figure A.2: The temperature development in point G for different cement contents.

Figure A.3: The temperature development in point G for different ambient temperatures.
Figure A.4: The temperature development in point G for different wind speeds.

Figure A.5: Combining the factors in point G. Case a) wind speed 1 m/s, cement content 370 [kg/m³] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m³] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m³] and amb. temp 10°C.
A.1.2 Strain development

Figure A.6: The strain development in point G for different cement contents.

Figure A.7: The strain development in point G for different ambient temperatures.
A.1. MEASUREMENT POINT G

Figure A.8: The strain development in point G for different wind speeds.

Figure A.9: The strain development in point G for different degrees of restraint in translation.
A.2 Measurement point E

Figure A.10: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point E are presented below.
A.2. MEASUREMENT POINT E

A.2.1 Temperature development

![Graph showing temperature development](image1)

Figure A.11: The temperature development in point E for different cement contents.

![Graph showing combined factors](image2)

Figure A.12: Combining the factors in point E. Case a) wind speed 1 m/s, cement content 370 [kg/m³] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m³] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m³] and amb. temp 10°C.
A.2.2 Strain development

Figure A.13: The strain development in point E for different cement contents.

Figure A.14: The strain development in point E for different ambient temperatures.
Figure A.15: The strain development in point E for different wind speeds.

Figure A.16: The strain development in point E for different degrees of restraint in translation.
A.3 Measurement point D

Figure A.17: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point D are presented below.
A.3.1 Temperature development

Figure A.18: The temperature development in point D for different cement contents.

Figure A.19: The temperature development in point D for different ambient temperatures.
Figure A.20: The temperature development in point D for different wind speeds.

Figure A.21: Combining the factors in point D. Case a) wind speed 1 m/s, cement content 370 [kg/m³] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m³] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m³] and amb. temp 10°C.
A.3.2 Strain development

![Graph showing strain development for different cement contents](image1)

Figure A.22: The strain development in point D for different cement contents.

![Graph showing strain development for different ambient temperatures](image2)

Figure A.23: The strain development in point D for different ambient temperatures.
Figure A.24: The strain development in point D for different wind speeds.

Figure A.25: The strain development in point D for different degrees of restraint in translation.
A.4 Measurement point C

Figure A.26: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point C are presented below.
A.4.1 Temperature development

Figure A.27: The temperature development in point C for different cement contents.

Figure A.28: The temperature development in point C for different ambient temperatures.
A.4. MEASUREMENT POINT C

Figure A.29: The temperature development in point C for different wind speeds.

Figure A.30: Combining the factors in point C. Case a) wind speed 1 m/s, cement content 370 [kg/m³] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m³] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m³] and amb. temp 10°C.
A.4.2 Strain development

![Strain development diagram for different cement contents.](image1)

Figure A.31: The strain development in point C for different cement contents.

![Strain development diagram for different ambient temperatures.](image2)

Figure A.32: The strain development in point C for different ambient temperatures.
Figure A.33: The strain development in point C for different wind speeds.

Figure A.34: The strain development in point C for different degrees of restraint in translation.
APPENDIX A. NUMERICAL RESULTS

A.5 Measurement point B

Figure A.35: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point B are presented below.
A.5.1 Temperature development

Figure A.36: The temperature development in point B for different cement contents.

Figure A.37: The temperature development in point B for different ambient temperatures.
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Figure A.38: The temperature development in point B for different wind speeds.

Figure A.39: Combining the factors in point B. Case a) wind speed 1 m/s, cement content 370 [kg/m³] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m³] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m³] and amb. temp 10°C.
A.5.2 Strain development

Figure A.40: The strain development in point B for different cement contents.

Figure A.41: The strain development in point B for different ambient temperatures.
Figure A.42: The strain development in point B for different wind speeds.

Figure A.43: The strain development in point B for different degrees of restraint in translation.
A.6 Measurement point A

Figure A.44: Drawing of the cross section showing the placement of the measurement equipment marked with the points A-G. The results on the temperature- and strain development in point A are presented below.
A.6.1 Temperature development

Figure A.45: The temperature development in point A for different cement contents.

Figure A.46: The temperature development in point A for different ambient temperatures.
**A.6. MEASUREMENT POINT A**

Figure A.47: The temperature development in point A for different wind speeds.

Figure A.48: Combining the factors in point A. Case a) wind speed 1 m/s, cement content 370 [kg/m$^3$] and amb. temp 20°C. Case b) wind speed 1 m/s, cement content 360 [kg/m$^3$] and amb. temp 13°C. Case c) wind speed 12 m/s, cement content 335 [kg/m$^3$] and amb. temp 10°C.
A.6.2 Strain development

Figure A.49: The strain development in point A for different cement contents.

Figure A.50: The strain development in point A for different ambient temperatures.
Figure A.51: The strain development in point A for different wind speeds.

Figure A.52: The strain development in point A for different degrees of restraint in translation.