EXPLOSIVE SPALLING OF CONCRETE IN FIRE

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ABSTRACT

Spalling of concrete at high temperature, mainly in the form of a violent breaking off of concrete layers, was already observed in the early research work on concrete at high temperatures. During the 1970s, research indicated different mechanisms leading to explosive spalling: High internal pore pressure due to the evaporation of moisture as well as thermal stresses caused by temperature gradients or a combination of both were proposed in literature to be the governing mechanisms leading to spalling. It was mentioned that high pore pressure is assumed only as “trigger” leading to explosive spalling due to a combination of pore pressure and thermal stresses.

Recent tests at ETH Zurich on the spalling behavior of high (HPC) and ultra-high performance concrete (UHPC) mixes with a low permeability indicated a trend towards explosive spalling mainly due to high pore pressure. Spalling was noticed even with very low heating rates and negligible temperature gradients. The tested concrete mixes covered a strength grade of up to $f_{c} = 150$ MPa and different amounts of silica fume. Concrete cylinders ($\Phi = 150$ mm, $h = 300$ mm) were heated linearly at different rates to determine critical rates leading to explosive spalling. A plateau phase during heating was noticed which could be identified as a slower increase in core temperature compared to the surface temperature. This plateau phase is caused by the evaporation of moisture inside the concrete and each concrete mix had a specific plateau temperature, which increased with lower permeability of the concrete. A detailed analysis on this plateau phase and the corresponding losses in weight was carried out.

The use of PP fibers is known as one possible measure to minimize the risk of explosive spalling since the melting fibers increase the permeability of the concrete and release high pore pressure. This thesis analyzes the results from tests on different types of PP fibers with changes in length and diameter leading to a beneficial influence on minimizing the risk of spalling. Short and thin fibers showed overall good results in terms of increasing the spalling resistance. However, additional tests showed that these types of fiber have an adverse effect on the workability and mechanical properties of the concrete. In contrast to the use of PP fibers, no beneficial effects were observed with the use of steel fibers to minimize the risk of explosive spalling.

Tests at IBK on the temperature-dependent permeability of concrete showed an increase in permeability for the mixes including PP fibers starting with the melting temperature of the fibers at $T = 170{^\circ}C$. The increase in permeability was significantly slower for those concrete mixes without PP fibers. Furthermore, a significant decrease in permeability was observed for concrete mixes with a high content of silica fume and with no PP fibers added to the mix within a temperature range of $T = 150 - 275{^\circ}C$.

Spalling of concrete at high temperature is usually considered if the pore pressure inside the concrete exceeds the tensile strength at high temperature. This thesis presents a simple mechanical model for the stresses acting close to a concrete pore. The model compares stresses due to a pore pressure and stresses due to an applied tensile stress. It was concluded that both stress peaks are within the same range, independent of the porosity of the concrete or the temperature level.

Several considerations on the pressure development inside the heated concrete were made within the framework of this thesis, based on own test results and an extensive literature review. Three categories for the possible failure of concrete due to high pore pressure could be identified: (1) Concrete will fail by
explosive spalling if the pressure rises according to the saturation vapor pressure curve and exceeds the tensile strength of the concrete. (2) No spalling will occur if the tensile strength has sufficient resistance. (3) For low heating rates and concrete mixes with a higher permeability, the pressure increase might be lower compared to the saturation vapor pressure curve. Spalling might still occur, depending on the tensile strength as resistance.

A hydrothermal model to take into account the pore pressure development inside concrete during heating was developed, based on the fundamentals of thermodynamics and the flow of vapor in porous media. In this model the concrete pores were idealized as hollow spheres embedded in a dense layer of cement paste. Pressure inside these pores will build up according to the temperature and thermodynamics laws, as long as sufficient moisture is available inside the pore. The pressure gradient will lead to moisture migration between the pores or even out of the modeled concrete segment according to the permeability. Pressure will rise inside the pores until they become dry and the pressure decreases or failure due to a high pressure is considered.

Material properties, such as temperature-dependent changes in the porosity, permeability and moisture content, were used as input parameters for the hydrothermal model. They were obtained from tests and could be directly implemented in the model.

Results obtained from the model were compared to experimental data and showed a good overall agreement in terms of critical temperatures and the corresponding pressure at spalling.

The presented hydrothermal model provides a simplified method to estimate the critical pore pressure in concrete at high temperatures leading to explosive spalling, even though the influences due to thermal stresses and cracking on the risk of spalling were neglected in this stage of the modeling. The required material properties for the model can be easily obtained using small-scale tests, even from existing structures.
KURZFASSUNG


Die untersuchten Betonmischungen deckten einen Festigkeitsbereich bis zu $f_c = 150$ MPa sowie verschiedene Mengen Silikastaub ab. Betonzylinder ($\varnothing = 150$ mm, h = 300 mm) wurden linear aufgeheizt, um kritische Aufheizraten zu bestimmen, die zu explosiven Abplatzungen führen. Eine Plateauhase während des Aufheizens wurde durch einen langsamen Anstieg der Kerntemperatur im Vergleich zur Oberflächentemperatur bemerkt. Diese Plateauhase wird durch das Verdampfen von Wasser im Beton verursacht. Bei jeder Betonmischung konnte eine spezifische Plateautemperatur bestimmt werden, die mit abnehmender Permeabilität des Betons zunimmt. Im Weiteren wurde eine detaillierte Analyse über diese Plateauhase mit der zugehörigen Abnahme des Betongewichts durchgeführt.


Explosive Abplatzungen treten bei hohen Temperaturen normalerweise auf, wenn der Porendruck im inneren des Betons die Zugfestigkeit überschreitet. In dieser Arbeit wurden in einem vereinfachten mechanischen Model Spannungen im Beton durch einen hohen Porendruck sowie durch aufgebrachte
Zugkräfte verglichen. Es konnte gezeigt werden, dass beide Spannungsspitzen in der gleichen Größenordnung liegen, unabhängig von der Betonporosität oder dem Temperaturbereich.


Materialeigenschaften, wie temperaturbedingte Änderungen in der Porosität, der Permeabilität oder des Feuchtegehaltes wurden als Eingangsparameter für das Ingenieurmodell verwendet. Diese Parameter wurden experimentell bestimmt und konnten so direkt in das Modell implementiert werden.

Die mit Hilfe des Modells generierten Ergebnisse zeigten eine gute Übereinstimmung mit Versuchsdaten in Bezug auf kritische Temperaturen und zugehörige Porendrücke.

Das Ingenieurmodell bietet eine einfache Möglichkeit, kritische Porendrücke in Beton bei hohen Temperaturen abzuschätzen, die zu explosiven Abplatzungen führen, auch wenn der Einfluss von thermischen Spannungen und Rissbildung unberücksichtigt bleibt. Alle notwendigen Materialparameter können einfach in Kleinversuchen bestimmt werden, auch von existierenden Bauwerken.
1 INTRODUCTION
1.1 Background / Motivation

Concrete has been known as a very durable material for almost 2000 years. The use of cement as hydraulic binder, or “opus caementitium” as it was called at that time in the Roman Empire, allowed the construction of complex shapes like the pantheon’s dome in the city of Rome [46, 87].

While the knowledge of cement and concrete was almost lost during the Middle Ages, the use of concrete and the improvement in compressive strength has increased significantly within the last 150 years. First observations on concrete at high temperatures were then made at the beginning of the 20th century. Among others, several tests on spalling were carried out by Gary [40] from the 1910’s onwards. Damage to the concrete was noticed in general and temperature-dependent deterioration processes during heating in particular. Gary already noticed that high internal stresses caused by water evaporation or temperature gradients are the chief governing factors leading to explosive spalling of concrete.

In favor of emphasis on the need for research on spalling of concrete, Gary cited an article from 1886 in the “German potter and brickmaker journal” [1]. It is mentioned in this article that roofs made of concrete have a tendency to violent explosive spalling during fire. Fire fighters were hindered in their rescuing work as a result of flying and falling debris from the roof. Authorities considered a ban on these rather new and innovative roofs made of concrete.

With ongoing research on explosive spalling several challenges related to the spalling of concrete structures in fire could be solved and even concrete roofing is still used today. However, innovations in concrete technology, including developments in the area of admixtures led to new mixes with increased strength levels of up to f_c = 250 MPa [99]. These high performance concrete mixes achieve this high strength grade due to their “dense” structure. However, this high degree of density together with low permeability are known as one critical factor leading to spalling of concrete at high temperatures. Similar to the concrete roofing of one century ago, fire authorities and design standards require the use of concrete that does not tend to spalling or spalling only occurs within predictable limits [6]. However, the spalling of concrete elements strongly influences structural safety and reliability as it may lead to failure, which is unacceptable.

Even though extended research was carried out on determining critical parameters leading to explosive spalling and providing possible protective measures against it, neither standardized test methods nor engineering models are available to predict the likelihood of explosive spalling. Some ideas and models for explosive spalling have been presented in the past. However, some of these models are not necessarily valid until today.

This leads to the requirement of a new, basic, hydrothermal model for explosive spalling of concrete at high temperatures to determine if a given concrete structure tends to explosive spalling due to high pore pressure or if it can be considered as spalling safe. Basic universal physical models, equations and material properties are essential for such a model.
1.2 Objectives and outline
This thesis presents a basic hydrothermal model for the development of high pore pressures inside concrete with increasing temperatures allowing the prediction of a possible occurrence of explosive spalling. Based on several tests, the presented hydrothermal model provides information on the location and magnitude of pore pressures inside the concrete’s cross-section and its influence on explosive spalling.

Several tests on explosive spalling of high (HPC) and ultra-high performance concrete (UHPC) were performed at ETH Zurich and are presented in detail in the IBK report “Explosive spalling of concrete - Test report” [75]. The main results from these tests are analyzed and discussed in detail in this thesis to study the critical concrete parameters leading to explosive spalling. In addition, material properties are provided for use as input parameter for the hydrothermal model.

A new hydrothermal model is developed, based on governing thermodynamic laws and vapor flow in porous media. This model considers in small time steps the interconnectivity of pores and the moisture movement at different depth in the analyzed concrete cross section. The model is explained in detail with all boundary conditions and verified with tests results. A preview of the general adaptability of the model for existing structures for real fire scenarios is also provided.

The current thesis is divided into five chapters, starting with the introduction in chapter 1, followed by three chapters presenting the main elements of the work.

A detailed literature review on explosive spalling of concrete at high temperatures is given in chapter 2. Apart from an historical overview from early to recent tests on explosive spalling, the main influences and mechanisms causing explosive spalling are summarized. Further, selected methods of preventing explosive spalling of concrete are discussed.

A wide range of tests on the explosive spalling of concrete cylinders was carried out and the results are discussed in detail in chapter 3. The discussion covers the influence of heating rate, different types of fiber (steel- and PP fibers) added to the concrete and tests on the porosity and permeability of concrete specimens at high temperatures. Several of these test results, like the temperature-dependent permeability of the concrete, were compared to existing material models or new models were developed to use all the material properties from these tests for an analytical hydrothermal model for spalling due to high pore pressure.

The hydrothermal model on spalling due to high pore pressure is explained in chapter 4. The governing equations to model the pore pressure in different depth of the modeled concrete’s cross-sections and for different time step are provided. In addition, stop-criteria indicating failure of the modeled concrete are discussed. This is followed by case studies on own tests indicating a general applicability of the model.

The thesis closes with conclusions regarding the proposed model and outlook for further research as given in chapter 5.
1.3 Scope of the thesis

The development of pore pressure inside the concrete can be predicted with the hydrothermal model as described in chapter 4. A high pore pressure might lead to failure due to explosive spalling. This high pressure usually builds up in concrete mixes with low permeability, since the release of vapor is small.

The scope of application of the hydrothermal model is therefore limited to very “dense” concrete mixes with low permeability like high (HPC) and ultra-high performance concrete (UHPC) mixes. Ordinary performance concrete (OPC) usually resists rapid heating rates with only minor spalling or even without spalling. For these types of concrete pore pressure is less important and other mechanisms leading to spalling become more relevant.

This leads to a second limitation of the analytical model on spalling. The presented hydrothermal model only covers failure due to high pore pressure. Additional stresses caused by high temperature gradients or external loads are not yet included in the model, even though these stresses are known to increase the risk of explosive spalling [57].

In addition to neglecting thermal and load-induced stresses, the general application of the model is further limited to the first occurrence of spalling. Explosive spalling usually causes severe cracking within the concrete’s structure. Cracks have a significant influence on consecutive spalling of the structure since they might release high pore pressure. In addition, deeper concrete sections are directly exposed to the heat after spalling. These secondary effects are not yet included in the model.
2 SPALLING OF CONCRETE
2.1 Spalling at high temperatures
2.1.1 Introduction

Research on the spalling of concrete at high temperatures started almost one century ago. During his research in the 1910’s, Gary [40, 41] already mentioned high internal stresses caused by water evaporation or temperature gradients inside the concrete as the main factors leading to explosive spalling. In order to identify the origin of moisture inside the concrete initiating - among other things - explosive spalling, the physicochemical changes for concrete at high temperatures are briefly discussed in the following.

Concrete is a very complex material. Aggregates such as filler are cheap and easily obtainable and have to be “conglutinated” with cement paste. As cement paste, cement and water are mixed. During hardening this mix inside fresh concrete starts creating - among others - calcium silicate hydrate phases (CSH phases), chemically binding the water as silicate hydrate phases. The developments of these CSH phases are the main factor for the increase in compressive strength of concrete during hardening [108].

For complete hydration of the cement paste, in theory a minimum water/cement (w/c) ratio of 0.4 is required. About two-thirds of this water is chemically bound in CSH phases while the rest of the water will remain physically bound in small gel pores within the cement paste. These gel pores have an average pore radius of about \( r = 1 - 10 \) nm. This physically bound water is released from these small pores by drying the concrete at \( T = 105°C \) until constant in mass [30]. Vapor will be generated and evaporate to the outside according to the permeability of the concrete. The less permeable the concrete, the longer the time required for the drying process.

Higher amounts of water exceeding the w/c ratio of 0.4 will remain unhydrated inside larger capillary pores with an average diameter of \( \varnothing = 10 - 10^3 \) nm. This free water will evaporate partially or completely out of the concrete over time, depending on the environmental conditions.

High (HPC) and ultra-high performance concrete (UHPC) mixes usually have a w/c ratio significantly lower than ordinary performance concrete (OPC). The high amount of unhydrated cement added to the concrete mix acts like very dense, high-strength aggregates and increases the compressive strength of the concrete. The w/c ratio is within the range of 0.2 - 0.3 [73] and only very small amounts of water will remain free inside the cement matrix.

At ambient conditions, concrete is usually exposed to maximum temperatures below \( T < 80°C \), which will not causes any significant physicochemical changes to the cement matrix and the concrete aggregates. However, significant changes to the concrete take place as soon as the concrete exceeds temperatures of about \( T = 100°C \). First, the free and the physically bound water is released and starts to evaporate at temperatures of \( T > 100°C \). Higher temperatures cause deterioration processes to the CSH phases and the chemically bound water is released from the CSH phases. A shown in Table 1, this process already starts at temperatures between \( T = 120°C - 150°C \) and has several peaks up to temperatures of \( T = 800°C \) where a complete decomposition of the CSH phases is noticed and all water is evaporated [50, 57]. In addition, aggregates used
inside the concrete may release water at high temperatures. Limestone aggregates mainly consist of calcium carbonate, which decomposes at temperatures of about $T = 700^\circ C$ [57]. Ichikawa [53] presented a figure with types of water inside concrete at different temperature levels. Figure 1 shows these types of water including quantitative volume share and their temperature-dependent increase.

The main physicochemical changes for concrete at high temperatures are summarized in Table 1, with main focus on water related changes. Creep and shrinkage at high temperature are neglected here. The critical point for water at $T = 374^\circ C$ is explained later in chapter 3.4.1.

Table 1: Physicochemical changes for concrete at high temperatures according to [38, 50, 52, 57]

<table>
<thead>
<tr>
<th>Concrete Temperature in °C</th>
<th>Changes in concrete’s performance</th>
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<tbody>
<tr>
<td>20°C - 100°C</td>
<td>Loss of free and physically bound water, concrete starts to “dry” out</td>
</tr>
<tr>
<td>105°C</td>
<td>Hydrothermal reactions start</td>
</tr>
<tr>
<td>120°C - 150°C</td>
<td>Decomposition of calcium silicate hydrate phases (CSH phases)</td>
</tr>
<tr>
<td>150°C</td>
<td>First peak of CSH composition [57]</td>
</tr>
<tr>
<td>270°C</td>
<td>Second peak of CSH composition [50]</td>
</tr>
<tr>
<td>&gt;300°C</td>
<td>Noticeable increase in porosity and micro cracking</td>
</tr>
<tr>
<td>374°C</td>
<td>Critical water point, all physically bound and free water evaporated</td>
</tr>
<tr>
<td>400°C - 600°C</td>
<td>Dissociation of Calciumhydroxide $\text{Ca(OH)}_2 \rightarrow \text{CaO} + \text{H}_2\text{O}$</td>
</tr>
<tr>
<td>535°C</td>
<td>Highest peak of Calciumhydroxide decomposition [50]</td>
</tr>
<tr>
<td>573°C</td>
<td>Transformation of $\alpha$- to $\beta$- quartz causing noticeable expansions (reversible)</td>
</tr>
<tr>
<td>&gt;700°C</td>
<td>Decarbonation of CaCO$_3$ to CaO + CO$_2$ in cement paste and limestone aggregate</td>
</tr>
<tr>
<td>710°C - 720°C</td>
<td>Third peak of decomposition of CSH phases</td>
</tr>
<tr>
<td>&gt;800°C</td>
<td>All chemically bound water fully evaporated</td>
</tr>
<tr>
<td>1200°C</td>
<td>Concrete starts to melt</td>
</tr>
</tbody>
</table>
2.1.2 Definition of spalling

Spalling of concrete is be defined by Khoury as “the violent or non-violent breaking off of layers or pieces of concrete from the surface of a structural element, when it is exposed to high and rapidly rising temperatures as experienced in fires” [57].

This general definition of explosive spalling from Khoury covers the entire range of observations and observations made in tests for over a century for all types of spalling at high temperature (see following chapter 2.1.3) and is still valid today even with the rapid development of new types of concrete and materials.

However, tests on concrete cylinders [74] showed that even if no spalling was observed at high temperatures, significant post-cooling spalling including complete deterioration of the concrete may occur weeks after cooling to ambient temperatures with significant influences on the structural performance. Since this delayed spalling might occur even a long time after the actual exposure to high temperatures, the second part of the definition by Khoury [57] should be extended to “during or after it is exposed to high and rapidly rising temperatures as experienced in fires”.

2.1.3 Types of spalling

In terms of categorizing types of spalling, Gary [40] already noted four main categories of spalling in his tests in 1916. Over the next decades two additional groups were mentioned. Post cooling spalling and sloughing off spalling were noticed the ongoing research on explosive spalling. Today, according to Khoury [57], spalling of concrete at high temperatures may be grouped in the following categories:
- Explosive spalling
- Surface spalling
- Aggregate spalling
- Corner spalling
- Post cooling spalling
- Sloughing off spalling

Brief definitions on these different types of spalling are given in the chapter “Notations and Definitions”.

In the case of fire, usually several of these different types of spalling occur at the same time; depending on the duration the concrete is exposed to high temperatures. It is very difficult to distinguish between some of these types of spalling, explosive and surface spalling in particular.

According to the literature [57] it is noted that explosive, surface and aggregate spalling usually occurs within the first few minutes after the concrete member has been exposed to high temperatures, e.g. according to the ISO fire curve [5]. Corner spalling is usually observed after $t = 30 - 90$ min exposure to high temperatures, since a certain temperature gradient is required. The risk of corner spalling is increased when reinforcement bars close to the concrete edges are heated [94].
In contrast to these four categories of spalling, post cooling and sloughing off spalling are a rather rare occurrence. They usually take place after concrete has been exposed to high temperatures for a very long time or even some time after the concrete cooled again to ambient temperatures [57, 68].

Table 2 summarizes the different forms of spalling and their main characteristics [57]. These six categories are briefly described and discussed in the following with main focus on the explosive spalling of concrete. The main influences on the possible occurrence and extent of concrete spalling at high temperatures are discussed in chapter 2.1.4.

The difference between violent and non-violent spalling is based on the energy release during spalling. The expression “violent spalling” is frequently used in the literature; however it has not been clearly defined so far. Explosive spalling due to the sudden release of high internal stresses is considered as violent, since a high amount of energy is required to cause these required stresses leading to spalling. This high amount of energy is released at once.

In contrast, sloughing-off and post cooling spalling usually start when the concrete begins to weaken or even after cooling. Since concrete parts simply fall off or ripple the structure, the nature of these types of spalling is considered as non-violent due to their rather low energy demand. However, no quantitative criterion is given to distinguish between a violent and non-violent type of spalling.

Table 2: Spalling characteristics by Khoury [57]

<table>
<thead>
<tr>
<th>Spalling</th>
<th>Usual time of occurrence</th>
<th>Nature 1)</th>
<th>Sound 2)</th>
<th>Influence on structure</th>
<th>Governing factors leading to spalling 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Explosive</td>
<td>7-30 min</td>
<td>violent</td>
<td>loud bang</td>
<td>very serious</td>
<td>material, structural / mechanical and temperature related</td>
</tr>
<tr>
<td>Surface</td>
<td>7-30 min</td>
<td>violent</td>
<td>cracking</td>
<td>can be serious</td>
<td>mainly material related</td>
</tr>
<tr>
<td>Aggregate</td>
<td>7-30 min</td>
<td>splitting</td>
<td>popping</td>
<td>superficial</td>
<td>mainly material related</td>
</tr>
<tr>
<td>Corner</td>
<td>30-90 min</td>
<td>non-violent</td>
<td>none</td>
<td>can be serious</td>
<td>mainly structural / mechanical related</td>
</tr>
<tr>
<td>Post-cooling</td>
<td>after cooling</td>
<td>non-violent</td>
<td>none</td>
<td>can be serious</td>
<td>structural / mechanical and material related</td>
</tr>
<tr>
<td>Sloughing-off</td>
<td>when concrete weakens</td>
<td>non-violent</td>
<td>none</td>
<td>can be serious</td>
<td>mainly structural / mechanical related</td>
</tr>
</tbody>
</table>

1) In standard fire (i.e. ISO fire exposure [5])
2) Based on energy release upon spalling (boundaries not defined in the literature)
3) Explained in detail in chapter 2.3

Figure 2 to Figure 4 show examples of spalling as observed during fire exposure of concrete structures.

Figure 2 shows the heated side of a HPC slab after explosive spalling exposed to the ISO fire curve for t = 119 min. Even though this slab was protected with h = 10 mm lining, explosive spalling accompanied by a single loud bang was noticed, leading to a total loss of up to 60 mm concrete [75].
Figure 3 shows a concrete column in which corner spalling was observed. Even though corner spalling is usually considered to be non-violent, this figure shows that it might be critical for slender concrete elements like columns if the longitudinal reinforcement bars close to the heated surface are no longer covered with concrete.

Tests on an unloaded concrete cylinder (ø = 150 mm, l = 300 mm) with an average compressive strength of $f_c = 80$ MPa and exposed to the ISO fire [5] for 120 min remained intact during the test [71]. However, after one week of storage at $T = 20^\circ$C and 40% ambient humidity the concrete showed significant post cooling spalling, leading to a complete deterioration of the specimen with a compressive strength below the lower measuring limit of the testing machine ($f_c < 1.1$ MPa). Figure 4 shows the concrete cylinder before testing, after 120 min ISO fire and after storage for one week. The mix design was not further specified.
2.1.4 Historical overview on spalling of concrete at high temperatures

The phenomenon of concrete spalling at high temperatures has been known since the last century. The first tests focused on a general understanding of spalling and identifying types of spalling including the corresponding processes inside the concrete. During the 1970th, several tests on explosive spalling lead to design criteria of concrete structures which are still applied today.

From the late 1980th onwards, the rapid development in new types of concrete (HPC, UHPC, SSC) strengthened the need of research in the area of explosive spalling and providing measures against it. In addition to tests on large-scale specimens exposed to standardized fire curves, research was focused on individual parameter and phenomenon inside the heated concrete related to spalling within the last decades. This included measuring the pore pressure during heating, MRI scans or load regimes.

A detailed literature study on several publications is given in appendix A. Main findings from researches in the past are summarized in Table 3. This table is without any claim to completeness.

<table>
<thead>
<tr>
<th>Table 3:</th>
<th>Overview on general research on spalling of concrete at high temperatures (without any claim to completeness)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Author</td>
<td>Main area of research</td>
</tr>
<tr>
<td>Gary - 1916 [40, 41]</td>
<td>Fire test on large structures, observation and description of different types of spalling. “Dense” concrete is more subject to spalling.</td>
</tr>
<tr>
<td>Hasenjäger - 1935 [45]</td>
<td>Identification of different mechanisms leading to spalling (pore pressure + thermal stresses). Brief recommendations to improve the spalling resistance (expansion joints, modified reinforcement design).</td>
</tr>
<tr>
<td>Hertz, Jumpannen - from 1984 onwards [47-51, 62]</td>
<td>Research on HPC and UHPC with increased risk of explosive spalling due to low permeability (use of silica fume). Analysis on critical temperatures, heating rates and load levels leading to explosive spalling.</td>
</tr>
<tr>
<td>Nishida - 1995 [100]</td>
<td>Use of PP fibers as protective measure against explosive spalling (see chapter 2.5.3).</td>
</tr>
<tr>
<td>Pimienta et al. - 2011 [96, 102]</td>
<td>In depth tests on pore pressure. Analysis of w/c ratio, storage conditions and heating rate on pore pressure development.</td>
</tr>
<tr>
<td>van der Heijden - 2011 [128]</td>
<td>Visualization of moisture migration inside heated concrete with NMRI scans. Identification of “boiling front” inside concrete at $T &gt;&gt; 100^\circ C$.</td>
</tr>
</tbody>
</table>
2.1.5 Concluding remarks on the literature review of concrete spalling

Based on the historical overview on the spalling of concrete at high temperatures as summarized in Table 3 and explained in detail in appendix A, the following general conclusions can be drawn:

- NMRI scans of heated concrete indicated the movement of a “boiling front” of evaporating water at temperatures significantly higher of $T > 100^\circ C$ $[127, 128]$.

- The use of “dense” concrete mixes with low permeability increases the risk of explosive spalling, mainly due to the development of high pore pressure $[41, 45, 94]$.

- Pore pressure measurements at high temperatures showed that a lower w/c ratio leads to a higher peak pore pressure at a higher temperature (lower permeability). At peak pressure, a plateau phase with constant pressure is observed before the pressure decreases $[96, 97, 102]$.

- Pre-drying the specimen reduces pore pressure due to the lack of free water. Free and physically bound water are mainly responsible for the development of pore pressure $[96]$. Higher heating rates lead to lower pore pressures due to cracking $[96]$.

- A high pore pressure, high thermal stresses or a combination of both are considered as the main causes of explosive spalling in concrete $[94]$.

- Some studies postulate that a high pore pressure due to rapid heating is considered to be only the “trigger” of explosive spalling. Thermal stresses due to temperature gradients seem to be superior $[16, 19, 20]$.

- Tests on the spalling behavior of UHPC concrete specimens with low permeability indicated a shift towards spalling caused by a high pore pressure $[75]$.

- PP fibers added to the concrete were found to be effective in terms of minimizing the risk of explosive spalling (increase in permeability $[133]$).

- Low stresses as a preloading on concrete have no beneficial effect in terms of reducing the risk of explosive spalling $[25]$ or even seem to increase this risk of spalling $[23, 24]$.

- Design criteria to minimize the risk of spalling that were established during the 1970’s by Meyer-Ottens $[94]$ are still part of current design standards (i.e. EN 1992-1-2 $[6]$).

- Standardized and reliable tests to assess the risk of spalling are not available and upscaling the results obtained from tests on small scale specimens is challenging, since todays methods still lack precision and reliability $[50]$.

- Several issues on concrete spalling at high temperatures remain unsolved until today. International research groups (fib, RILEM) focus on different aspects of spalling to extend current knowledge and provide data for a safe concrete design $[56, 57]$.
2.2 Governing factors leading to spalling

2.2.1 Introduction

Several factors have an important influence on the risk and extent of spalling. The majority of these parameters can be directly associated with explosive spalling. However, some are also related to other types and a clear separation between individual parameters is difficult to match. Tests from the past indicate several structural or material related parameters, which are considered to increase the risk of explosive spalling. Overall information on spalling is, e.g., summarized by Hertz [49], Khoury [57, 68] and Kodur [79, 82].

The governing factors leading to the explosive spalling of concrete at high temperatures can be divided into three main categories:
- Material related parameters
- Structural / mechanical parameters
- Heating characteristics

However, some of these parameters would fit into more than one category.

Within the following several of these material related, structural / mechanical and temperature related parameters with noticeable influence on concrete spalling are briefly presented. An in-depth discussion of these parameters is given in appendix B.

2.2.2 Material-related parameters

The research on concrete spalling at high temperatures identifies several material related parameters with a big influence on spalling. Table 4 gives a brief overview on these governing parameters in relation to the concrete mix design or the choice of materials used for the concrete.

The discussion on these parameters is presented in appendix B, chapter B.1.

<table>
<thead>
<tr>
<th>material related parameter</th>
<th>increasing risk of spalling</th>
<th>influence on spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td>silica fume</td>
<td>very high</td>
<td>Silica fume lowers the permeability and increases the possibility of explosive spalling due to the reduced release of high vapor pressure [57, 81].</td>
</tr>
<tr>
<td>limestone filler</td>
<td>high</td>
<td>Lowers permeability, similar behavior compared to silica fume [23, 103].</td>
</tr>
<tr>
<td>permeability</td>
<td>high</td>
<td>Low permeability and insufficient temperature-dependent increase in permeability increases the risk of spalling due to insufficient release of pore pressure [44, 75].</td>
</tr>
<tr>
<td>porous / lightweight aggregates</td>
<td>variable</td>
<td>Higher porosity and permeability enables the release of high pore pressure and decreases the risk of spalling [57, 68]. The higher moisture content of lightweight aggregates promotes the risk of spalling [79].</td>
</tr>
<tr>
<td>quartzite aggregates</td>
<td>high</td>
<td>Can increase the risk of spalling due to a change in the quartzite phase at T = 573°C.</td>
</tr>
</tbody>
</table>
Table 4 (cont.): Governing material related parameters with an influence on spalling

<table>
<thead>
<tr>
<th>material related parameter</th>
<th>increasing risk of spalling</th>
<th>influence on spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td>carbonate aggregates</td>
<td>lowering risk</td>
<td>Remains stable even at very high temperatures, have a very low thermal expansion [68, 82].</td>
</tr>
<tr>
<td>aggregate size</td>
<td>moderate</td>
<td>Larger aggregates increase the risk of explosive spalling [27] due to a poor surface to mass ratio.</td>
</tr>
<tr>
<td>internal cracks</td>
<td>variable</td>
<td>Two opposing effects. Small cracks might promote the release of high pressure and reduce the risk of spalling. However, parallel cracking close to the heated surface (i.e. due to loads) might increase the risk of spalling. [68]</td>
</tr>
<tr>
<td>compressive strength</td>
<td>high</td>
<td>Higher strength grade usually increases risk of explosive spalling, mainly due to the lower w/c ratio and permeability [82].</td>
</tr>
<tr>
<td>moisture content</td>
<td>very high</td>
<td>Higher moisture content (mainly free water) significantly increases the risk of explosive spalling, since more vapor pressure must be released depending on the permeability of the concrete. Critical moisture content is difficult to obtain, in particular for HPC.</td>
</tr>
<tr>
<td>cement content</td>
<td>high</td>
<td>High cement content increases the total amount of water added to the concrete, even with low w/c ratios.</td>
</tr>
<tr>
<td>concrete age</td>
<td>variable</td>
<td>Young concrete has a high amount of free water, which increases the risk of spalling [94]. This effect decreases with HPC and UHPC due to the low permeability.</td>
</tr>
</tbody>
</table>

2.2.3 Structural / mechanical parameters

The main structural / mechanical parameters with a significant influence on spalling are summarized in Table 5. In some cases it is difficult to distinguish between pure material and pure structural / mechanical parameters leading to spalling, since some parameters can be attributed to both categories. For example, the compressive strength of the concrete is one of these parameters, since several individual material related parameters have an influence on the compressive strength of concrete.

A discussion of structural / mechanical parameters is presented in appendix B, chapter B.2.

Table 5: Governing structural / mechanical related parameters with an influence on spalling

<table>
<thead>
<tr>
<th>Structural / mech. parameter</th>
<th>increasing risk of spalling</th>
<th>influence on spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td>tensile strength</td>
<td>lowering risk</td>
<td>A high tensile strength is considered as lowering the risk of explosive spalling since it offers a higher resistance:</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- against spalling due to a high pore pressure [53, 70].</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- of high thermal gradients, stresses and expansion.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- of corner spalling or thermal stresses from two sides [27].</td>
</tr>
<tr>
<td>applied load</td>
<td>high</td>
<td>The risk of spalling increases with applied higher load levels [62, 94]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Preload as low as 5% of the cold strength increases the risk of spalling [23, 24]</td>
</tr>
<tr>
<td></td>
<td></td>
<td>It remains unknown if a low preload minimizes the risk of spalling, since small cracks used for the release of vapor are pressed together ([34] cited in [27]).</td>
</tr>
<tr>
<td>hindered thermal expansion</td>
<td>high</td>
<td>Fixed ends as boundary conditions, eccentric load or bending increases risk [49]</td>
</tr>
<tr>
<td>cross section geometry</td>
<td>high</td>
<td>Round cross section, rounded corners, sufficient reinforcement cover and spacing and modified tie design lowers the likelihood of spalling or increases the remaining load bearing capacity of concrete members after spalling [6, 68, 81, 86, 94]</td>
</tr>
</tbody>
</table>
2.2.4 Heating characteristics

In terms of the parameters that depend on heating characteristics, the heating rate and the hereby caused temperature gradients have a strong influence on explosive spalling. High heating rates and thermal gradients increase spalling.

Table 6 summarizes the governing parameters depending on the heating characteristics that influence spalling in general.

Further discussion of these parameters is presented in appendix B, chapter B.3.

Table 6: The governing parameters depending on the heating characteristics with an influence on spalling

<table>
<thead>
<tr>
<th>Heating characteristic parameter</th>
<th>Increasing risk of spalling</th>
<th>Influence on spalling</th>
</tr>
</thead>
<tbody>
<tr>
<td>high heating rate</td>
<td>very high</td>
<td>Higher heating rates usually lead to explosive spalling with HPC mixes [57, 62, 75].</td>
</tr>
<tr>
<td>temperature gradient</td>
<td>high</td>
<td>Closely related to the heating rate. Higher temperature gradients ($\Delta T &gt; 1.0 \text{ K/mm}$) promote the risk of explosive spalling due to thermal stresses [114].</td>
</tr>
<tr>
<td>absolute temperature</td>
<td>moderate</td>
<td>Explosive spalling might occur with temperatures as low as $T = 300 - 350^\circ\text{C}$. Very high temperatures $T &gt; 1000^\circ\text{C}$ increase the risk of post cooling spalling [27, 75].</td>
</tr>
<tr>
<td>exposure on multiple surface</td>
<td>high</td>
<td>Heat exposure on more than one side increases the risk of corner or explosive spalling due to higher temperature gradients and thermal stresses [94].</td>
</tr>
</tbody>
</table>

2.2.5 Concluding remarks on governing factors with regard to explosive concrete spalling

In terms of the governing parameters with a noticeable influence on the risk of explosive spalling, the following conclusions can be made:

- Silica fume is known as one of the main parameters significantly increasing the risk of explosive spalling, mainly due to reducing the permeability of the concrete [48, 57, 82].

- The choice of aggregates has only a secondary influence on the risk of explosive spalling.

- High heating rates and high temperature gradients significantly increase the likelihood of explosive spalling [49, 57, 68, 114].

- A higher compressive strength and lower w/c ratio is mentioned to increase the risk of spalling, mainly because of the lower permeability [47, 57, 62, 68, 82].

- High moisture content leads to high amounts of vapor to be released, which increases the risk of spalling further. Sufficient maturity lowers the amount of moisture inside the concrete [49, 57, 68, 94].

- Higher tensile strength increases the resistance to spalling [49].
Mechanisms of explosive spalling at high temperature

2.3 Mechanisms of explosive spalling at high temperature

2.3.1 Introduction

It is generally agreed in the literature [27, 57, 68] that explosive spalling can be considered as the most violent form of concrete spalling at high temperatures with significant influence on the structural performance. The major parameters leading to explosive spalling were described and analyzed in the previous chapter 2.1.4. From this analysis it can be concluded that explosive spalling is mainly caused by two phenomena. The first is a rapid build-up of pore pressure. The release of high pore pressure is minimized mainly by a low permeability or high moisture content of the concrete. The second is thermal stresses close to the heated surface due to preload or a high temperature gradient caused by a high heating rate. A combination of both phenomena is also possible. These different mechanisms may occur individually or combined and are discussed below including a literature review.

2.3.2 Pore pressure-induced explosive spalling

Pore pressure-induced spalling was already mentioned by Gary in 1916 [40]. He noticed that - apart from other influences - high vapor pressures inside concrete would lead to spalling and very “dense” concrete showed an increased risk of explosive spalling.

Pore pressure-induced explosive spalling can be briefly explained as follows: Moisture is present inside the pore system of the concrete. With increasing temperatures this moisture expands and with temperatures above \( T > 100^\circ C \) water as liquid and vapor is present inside the pore system. Pressure will build up according to the degree of pore saturation and the temperature. Part of this vapor is released via the pores and voids according to the permeability of the concrete; however, pore pressure will rise as long as sufficient moisture is available. This pressure will induce bursting stresses in the concrete leading to failure.

In the literature, mainly three different models of pore pressure-induced explosive spalling are proposed:
- Shorter and Harmathy [120] proposed the “moisture clog model”
- Meyer-Ottens [94] coins the term “vapor drag forces”
- Akhtaruzzaman [10] analyzed pore pressure as an “idealized spherical pore model”

The three mechanisms are summarized briefly in the following and explained in detail in appendix C including an additional literature review.
Spalling due to formation of moisture clog

Shorter and Harmathy [120] concluded from tests on concrete slabs that spalling might be caused by “moisture clog”. The mechanisms leading to explosive spalling are given as follows:

- At an ambient temperature of $T = 20^\circ$C, moisture is distributed uniformly within the concrete.

- Increasing temperature to $T > 100^\circ$C leads to vaporization and moisture movements. Moisture migrates partly outwards towards the heated surface and partly inwards into deeper concrete sections.

- Moisture condenses again in the deeper cooler sections, leading to drying of the concrete areas close to the heated surface and to a saturated layer with very low permeability and hence making further migration processes nearly impossible.

- The saturated zone at a distance from the dry concrete zone is called “moisture clog” as indicated in Figure 5.

- Two opposing forces are considered to lead to spalling due to the presence of a moisture clog:
  - Movement of clog due to the pressure gradient towards the heated surface ($v_p$)
  - Movement of clog due to the heat flux further inside the concrete ($v_h$)

- Sufficient fractional pore saturation is required for the build-up of pore pressure.

- Moisture is “trapped” between the dry zone and the saturated zone and cannot migrate further inwards due to the low permeability of the saturated zone. Pressure will rise with increasing temperature.

- Spalling due to a high pore pressure occurs if the stresses due to pore pressure exceed the tensile strength of the concrete.

![Figure 5: Moisture clog model for spalling of concrete according to Shorter and Harmathy [120]](image)
**Spalling due to high vapor drag forces**

Meyer-Ottens [94] explained in 1972 the occurrence of explosive spalling by the presence of “vapor drag forces”. His theoretical explanations can be summarized as follows:

- Moisture inside the concrete starts to vaporize at temperatures of $T = 100^\circ C$ with corresponding moisture migration processes.

- A laminar vapor stream causes shear stresses along the pore cells. This friction is called a “drag force”. Spalling occurs if the bursting stresses exceed the tensile strength of the concrete.

- The predicted bursting stresses vary according to the movement of the $T = 100^\circ C$ isotherm, the flow length of the vapor (distance from the heated surface to the moisture clog) and the moisture content.

- Even though well explained in theory, the drag forces were never experimentally verified. It remains doubtful if this mechanism actually occurs inside heated concrete as predicted.

**Idealized spherical pore model**

Akhtaruzzaman analyzed losses in weight during the heating of concrete specimens using a model of “idealized spheres” inside the concrete surrounded by a dense cement gel layer as shown in Figure 6. He presented his results in 1973 [10]. His main findings can be summarized as follows:

- Pore pressure is assumed to develop in the spheres. These are converted to hoop stresses that lead to explosive spalling when exceeding the tensile strength of the cement layer.

- He noticed that specimens heated after water storage are more likely to result in explosive spalling and concluded that pressure will build up in the spheres leading to a high pressure.

- Migration processes are not mentioned, but losses in weight during heating are associated with the release of vapor from the inside of the concrete and lost concrete fragments due to spalling.

![Idealized spherical pore model](image.png)

**Figure 6:** Idealized spherical pore model according to Akhtaruzzaman ([10] cited in [27] and [118])
2.3.3 Thermal stress-induced explosive spalling

Spalling due to thermal stresses was analyzed in detail by Saito in 1965 [109] and further explained by Dougill in 1972 [32]. Their findings can be summarized as follows:

- Thermal stresses will occur inside the concrete due to temperature gradients from the heated surface towards the inner, cooler sections of the concrete. These gradients will increase with rapid heating rates.

- Different strains due to the thermal gradient are deemed to cause tensile and compressive stresses, depending on the thermal and mechanical properties of the concrete.

- Hindered expansion, loads and restraints as well as the heating rate are mentioned as further parameters [109].

- Failure due to spalling is considered to exceed the compressive strength of the concrete close to the heated surface. The compressive stresses due to the thermal gradient also lead to tensile stresses in the cooler sections of the concrete.

- Moisture migration is not considered with spalling due to thermal stresses [68] and spalling of HPC or of ordinary concrete with a high moisture content cannot be explained by thermal stress spalling.

- Explosive spalling only due to thermal stresses is said to be a rather rare occurrence [68].

2.3.4 Combined thermal stress and pore pressure-induced explosive spalling

The experience and analysis from several tests carried out over the last few decades on explosive spalling showed that it is almost impossible to assign the different observed types of failure to just one of the two mechanisms of spalling, either spalling due to high pore pressure or spalling due to thermal stresses.

Starting in the mid-1970's, the idea of a combined thermal stress and pore pressure-induced explosive spalling was first proposed by Zhukov in 1975 [134] (cited in [27] and [68]). Sertmehmetoglu presented his considerations in 1977 [118] and at later stage Connolly in 1995 [27], who developed new ideas on combined pore pressure and thermal stress spalling. The main results from these findings can be summarized as follows:

- Zhukov considered uniform stresses due to thermal expansion and an applied load causing stresses and cracks in longitudinal direction.

- Additional stresses due to a high pore pressure perpendicular to the heated surface will cause concrete parts to spall.

- Based on Zhukov's ideas, Khoury [68] presented a general sketch of combined thermal stress and pore pressure-induced explosive spalling as shown in Figure 7.
Mechanisms of explosive spalling at high temperature

Figure 7: Explosive spalling caused by combined thermal stresses and pore pressure by Khoury [57] based on Zhukov [134]

- Zhukov [134] provides additional equations on predicting bursting stresses and a general function determining the likelihood of spalling, which also includes several concrete related material properties. However, several assumptions have to be made for analysis purposes (see appendix C.2).

- Based on test results, Zhukov [134] presented a spalling envelope to assess the likelihood of spalling, depending on the moisture content and applied stress of the concrete as shown in Figure C3. It is mentioned that moisture content up to 3% will not lead to spalling.

- Bažant [16, 19, 20] noted that high pore pressure can only be assumed as a “trigger”, which then leads to combined spalling due to high pore pressure and thermal stresses.

- Low permeability as observed with HPC and UHPC mixes changes the governing mechanism leading to explosive spalling in the direction of high pore pressure even with very low heating rates with negligible thermal gradients [75].
2.3.5 Concluding remarks on mechanisms of explosive spalling

Based on the literature review on mechanisms of explosive spalling as well as results from own tests [75], the following conclusions can be drawn in terms of mechanisms leading to explosive spalling:

- Most models on spalling due to high pore pressure take moisture migration due to evaporation into account. It is suggested that vaporization of moisture starts at temperature of $T = 100^\circ$C.

- It should be noted that literature reviews [102, 128] and results from own tests [75] indicated that moisture vaporization starts at much higher temperatures of $T >> 100^\circ$C, in particular for UHPC.

- In terms of the moisture clog model by Shorter and Harmathy [120] it remains unanswered why the pressure rises within the “dry zone” without any moisture.

- The idea of spalling due to high vapor drag forces as described by Meyer-Ottens [94] was never verified by tests.

- The idea of “idealized spheres” by Akhtaruzzaman [10] is a simplification and permeability and migration processes are not considered in the model.

- Spalling by one single mechanism (thermal stresses or pore pressure) is a rather rare occurrence. In particular spalling only caused by high thermal stresses is most unlikely for HPC and UHPC.

- The combined mechanism - spalling due to high pore pressure and thermal stresses - is considered important for spalling [27, 57, 68].

- The spalling envelope (Figure C3) on explosive spalling due to combined pore pressure and thermal stresses after Zhukov [134] is not valid for HPC and UHPC. A moisture content of less than 3% showed an increased risk of spalling [75]. According to Zhukov’s model, spalling is unlikely for a moisture content less than $m_w < 3\%$.

- The explanation by Khoury [57] (Figure 7) on combined mechanisms leading to spalling is well explained in theory, but an engineering model based on these explanations is not available.

2.4 Models for explosive concrete spalling due to high pore pressure

2.4.1 Introduction

Research on explosive spalling indicated the two main mechanisms initiating explosive spalling mentioned above. Several studies focused on the general understanding, explanation and modeling of high pressures inside concrete leading to failure.

From the very first tests on concrete at high temperature, it was generally agreed that the vaporization of moisture inside the concrete is the governing factor towards increasing pressure [45]. Somehow this pressure leads then to high bursting stresses.
Starting with Shorter and Harmathy in 1965 [44], several new ideas and proposals on modeling spalling due to pore pressure have been published. The majority of models on spalling as developed in the past are not necessarily valid today for recent structures and concrete mixes. The increase in compressive strength and the use of silica fume leading to a denser and less permeable concrete minimizes the general application of these models that assess the risk of explosive spalling.

However, even though the concrete properties might have changed significantly in recent developments in concrete design, most of the fundamental considerations are still valid today and are presented briefly in the following. An in-depth discussion is provided in appendix E.

### 2.4.2 Overview of selected analytical models for spalling due to high pore pressure

In the following some basic models on explosive spalling due to high pore pressure are briefly described. Table 7 summarizes the main findings. A more detailed description is provided in appendix E.1.

<table>
<thead>
<tr>
<th>Author, Year</th>
<th>Basic description of the model</th>
</tr>
</thead>
</table>
| Shorter and Harmathy, 1965 [120] | - Spalling due to the presence of moisture clog as shown in Figure 5.  
- Moisture migration follows the $T = 100^\circ C$ isotherm  
- Several temperature-dependent concrete parameters were already taken into account. |
| Sertmehemetoglu, 1977 [118] | - Modifications to the moisture clog model.  
- Spalling liability curve to assess the likelihood of spalling, depending on the permeability and the degree of pore saturation. |
| Meyer-Ottens, 1972 [94] | - Vapor drag forced due to friction affecting moisture movement inside the concrete.  
- Moisture migration as vapor movement starts with $T = 100^\circ C$. |
| Connolly, 1995 [27] | - Movement of vapor is proportional to the heating rate, starting at $T = 100^\circ C$.  
- Moisture clog moves according to the pressure gradient between heated surface and cooler inner sections.  
- Peak pressure is achieved when the vaporization front reaches its maximum.  
- Combination of pore pressure, thermal stresses and load must be greater than the tensile strength to cause spalling. |
| Gawin, 1999 [42] | - Thermo-hygro mechanical model (THM), numerically solved  
- Includes nonlinear terms of moisture migration and heat flux, for an in-depth analysis on the risk of spalling.  
- Accuracy of the model strongly depends on the input parameters. |
- Mobility of water in the liquid state is ignored.  
- Pressure development is considered inside the concrete pores as a function of the temperature.  
- Pressure is released in the form of vapor as a function of the temperature-dependent permeability.  
- Failure due to exceeding the tensile strength of the concrete according to EN 1992-1-2 [6].  
- See additional discussion in appendix E.2. |
2.4.3 Concluding remarks on well-known models for spalling due to high pore pressure

Based on the review of existing models on explosive spalling due to high pore pressure the following general conclusions can be made:

- The moisture clog model by Shorter and Harmathy [120] takes several material properties into account. However some of them could not be properly determined at that time. The pressure gradient in the dry and saturated zone was never verified with tests.

- Most of the analytical models consider moisture migration in the form of vapor starting from temperatures of $T = 100^\circ C$ upwards. However, it was noticed in tests that the movement of the boiling front is observed at higher temperatures for HPC and UHPC [127, 128], due to higher pressure inside the concrete.

- The spalling liability curve (Figure E1) by Shorter, Harmathy and Sertmehemetoglu [118, 120] to assess the likelihood of spalling might be applicable for OPC. The model leading to this curve is based on the permeability and degree of pore saturation of concrete. However, for material properties from HPC and UHPC mixes it leads to doubtful results since it overestimates the risk of spalling.

- Connolly’s [27] model is a good approach towards an engineering method to assess the risk of explosive spalling. Pore pressure, thermal and load-induced stresses are taken into account. It considers the movement of the boiling front starting at a temperature of $T = 100^\circ C$.

- Numerical models are very precise in terms of predicting the pressure development. However, all numerical models are only as good as the selected input parameters. Reliable and precise models on concrete properties at high temperature (i.e. permeability) are not widely available.

- The engineering model by Dwaikat [33] provides good results in terms of the general prediction of critical pore pressure leading to explosive spalling. The main governing input parameters are based on well-established models.

- The decrease in pore pressure due to drying of the pores is taken into account in Dwaikat’s model. However, it remains unclear how migration processes of vapor inside the concrete are considered in the model.
2.5 Methods to prevent explosive spalling

2.5.1 Introduction

From the beginning of the research on concrete spalling at high temperatures, possible methods of reducing the risk of spalling were also discussed.

Gary [40] concluded in 1916 that explosive spalling can be avoided if a sufficient amount of water is provided inside the pore system of the concrete. He compared the build-up of a critical pressure with a gasoline engine. Only a stoichiometric rate between fuel and air leads to combustion. An excess supply of fuel will not lead to combustion. He assumed that if the concrete is water saturated a critical pressure will not develop, since the water will not evaporate.

In the following decades, several structural measures were discussed which led to an improved concrete design and general guidelines were presented, e.g. by Kordina and Meyer-Ottens [86]. However, recommendations on how to release high and critical pore pressure was not given at that time. Hertz [47] discussed the high risk potential of dense concrete with a low permeability and simply suggested not to use dense concrete.

Today, with the increased use of silica fume as part of HPC and UHPC mixes and the resulting increased risk of explosive spalling, fibers added to the concrete are mentioned as a protective measure. However, their use is limited to new concrete structures, while existing structures have to be protected in a different way, i.e. with a protective lining. Even though a protective lining can be applied easily in the case of existing structures, their use is limited due to the lack of generally accepted design criteria.

In the next section, design recommendations according to the European design standard EN 1992-1-2 [6] are described. The possible use and effectiveness of steel- and PP fibers are then discussed in addition to general thoughts on the use of a protective lining and changes to the structural design of concrete members.

2.5.2 Protective measures according to EN 1992-1-2

The European design standard EN 1992-1-2 [6] mentions the occurrence of explosive spalling and distinguishes between OPC and HPC. In general, explosive spalling must be avoided or its influence on the structural resistance must be taken into account when verifying the structural fire behavior.

Measures for OPC

EN 1992-1-2 [6] states that no spalling needs to be considered if the moisture content is below 3.0% - 4.0% in mass. Values can be set according to the national annex. If the moisture content exceeds this limit, an in-depth analysis of explosive spalling must be carried out, including the influence of the aggregates, permeability and heating rate. However, no data on these concrete parameters are given.

An in-depth analysis on the risk of explosive spalling is not required for OPC according to EN 1992-1-2, if tabulated data is used for material properties and structural details, i.e. reinforcement layout or cross
section. The occurrence of sloughing off spalling does not have to be analyzed if the spacing of the reinforcement bars is 70 mm or less.

**Measures for HPC against explosive spalling**

The use of HPC with a class in compressive strength of C55/67 to C80/95 is allowed in EN 1992-1-2 [6]. The Eurocode states in particular for these more dense concrete mixes that the risk of spalling is limited if the amount of silica fume does not exceed 6.0% of the total mass of the concrete. However, tests showed [75] that spalling might occur even with a significantly lower content of silica fume. Higher amounts of silica fume, exceeding the given limit of 6.0% in total mass, or if a concrete class exceeds the strength grade of C80/95 to C90/105, spalling might occur at any time at high temperatures. In this case EN 1992-1-2 provides four different methods to prevent concrete from explosive spalling. Some of them may be modified according to the national annex. The four methods are as follows:

**Method A:** Use of a secondary reinforcement mesh with a concrete cover of 15 mm. The reinforcement mesh should have a maximum spacing of 50 mm with reinforcement bars having a minimum diameter of 2.0 mm. No specification regarding the anchoring of this reinforcement is given.

**Method B:** Use of a concrete mix that does not have a tendency to explosive spalling at high temperatures as proved by tests or experience. No specification of testing or experience is mentioned.

**Method C:** Use of a thermal barrier to ensure that no spalling will occur. No specification of the design and verification of the thermal barrier is provided.

**Method D:** Use of 2.0 kg/m³ monofilament polypropylene fibers (PP fibers). No specification of the geometry and chemistry of the fibers is given.

Some of these methods are explained in depth in the following. Focus is put on the use of fibers, covering both steel- and PP fibers added to the concrete (Method D). Further information on the possible use of a protective lining as thermal barrier is also given (Method C).

### 2.5.3 Influence of fibers

To provide adequate permeability in order to release high pore pressures at high temperatures and to ensure an overall dense concrete at ambient conditions, the melting of fibers was considered to be a promising approach to prevent concrete from explosive spalling. In recent decades, different synthetic fibers have been tested in terms of melting characteristics, workability and overall performance with the aim of reducing the risk of spalling.

Table 8 gives a brief overview of fibers available to reduce explosive spalling. A detailed description of the effect of fibers is given in appendix D in chapter D.1. This description includes a detailed analysis of the performance of PP fibers inside the concrete.
Table 8: Use of different types of fiber to minimize the risk of explosive spalling

<table>
<thead>
<tr>
<th>Fiber</th>
<th>Mode of action and effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP</td>
<td>Melting at $T = 170^\circ C$ increases permeability and releases high pore pressure $[100]$. Fibers might have negative influence on workability, in particular very thin fibers $[75]$. &quot;Thumb rule&quot;: 2 - 3 kg/m$^3$ fibers ($l = 6 - 12$ mm $\varnothing = 15$ $\mu$m) seems to be most effective in terms of spalling protection $[67]$.</td>
</tr>
<tr>
<td>Nylon</td>
<td>A rather high melting temperature of $T = 200^\circ C$, which might be too high for some mixes $[88]$.</td>
</tr>
<tr>
<td>PVC</td>
<td>Releases hazardous chlorides, should not be used with concrete.</td>
</tr>
<tr>
<td>PE</td>
<td>Low melting temperature ($T = 90^\circ C$) but high viscosity of molten fibers minimizes the increase in permeability - less applicable $[110]$.</td>
</tr>
<tr>
<td>Steel</td>
<td>Increases ductility of HPC $[35]$ and increases spalling resistance of columns with narrow spacing between the ties $[78, 79]$. However, no noticeable increase in resistance with other structures was observed in tests $[75, 116]$.</td>
</tr>
</tbody>
</table>

2.5.4 Influence of protective coatings

The use of coatings as protective measure is known since the 1970’s. Meyer-Ottens $[94]$ discussed the effect of thermal barriers for the spalling protection of concrete.

Protective coatings usually limit the temperature increase and the maximum temperature at the concrete surface. Their layer thickness has to keep these temperatures below a critical level for spalling. However, critical temperatures leading to spalling are not generally available because they change with each individual concrete mix.

Table 9 summarizes the main coatings that may be used on concrete structures. The mode of action is explained in in appendix D in chapter D.2.

Table 9: Use of protective coating to minimize the risk of explosive spalling

<table>
<thead>
<tr>
<th>Coating</th>
<th>Mode of action and effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boards or coatings</td>
<td>Reduce increase in temperature and reduce spalling. Generally accepted design criteria for HPC and UHPC are not available $[6]$.</td>
</tr>
<tr>
<td>Intumescent coating</td>
<td>Swells at high temperatures and reduces increase in temperature over a certain time. Used mainly on steel structures but rather uncommon on concrete $[9, 107]$. Generally accepted design standards are not available for concrete structures.</td>
</tr>
</tbody>
</table>

2.5.5 Influence of admixtures

Admixtures as chemical agents are easy to use, since they can be directly poured to the fresh concrete when mixing. However, their use is very limited and there is a lack in experience with these agents. A critical review of admixtures is presented in appendix D in chapter D.3.

Table 10 summarizes briefly the mode of action of chemical admixtures added to the fresh concrete.
Table 10: Use of admixtures to minimize the risk of explosive spalling

<table>
<thead>
<tr>
<th>Admixture</th>
<th>Mode of action and effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air entraining admixtures</td>
<td>Increase pore volume and lower the saturation level of the pores. The risk of spalling decreases due to the additional expansion voids [27] and increased permeability [55]. Test on C25/30 concrete mixes with an air void content of 7.7% showed reduced spalling [55]. Silica fume has a similar diameter as air bubbles, which limits the application of these admixtures in the case of HPC and UHPC.</td>
</tr>
</tbody>
</table>

2.5.6 Influence of modified concrete mixes and structural design

Modifications in the concrete mix usually include changes in the concrete aggregates used. A lower thermal expansion is thought to reduce the risk of spalling due to a lower level of internal stresses. However, influences on the overall performance must be considered. Additional filler might be beneficial as well [13].

As structural modifications, mainly adapted reinforcement layouts are presented in the literature leading to a significant increase in spalling resistance [45, 80, 81, 86, 94].

Table 11 gives an overview of possible modifications to the concrete mix and structural design, which are explained in detail in appendix D in chapter D.4.

Table 11: Modified concrete mixes and structural design minimizing the risk of explosive spalling

<table>
<thead>
<tr>
<th>Modification</th>
<th>Mode of action and effectiveness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modifies concrete mix</td>
<td>Modified concrete mix including the use of selected lightweight aggregates and aggregates with low thermal expansion minimizes the risk of spalling [27, 68]. Additional tests might be required and negative influences on other performance criteria must be considered.</td>
</tr>
<tr>
<td>Secondary reinforcement</td>
<td>Secondary reinforcement protecting the main reinforcement bars further inside concrete beams increases the resistance to spalling [94]. Sufficient anchoring of the secondary reinforcement has to be provided</td>
</tr>
<tr>
<td>Less spacing between bars</td>
<td>Reduced spacing and good anchoring of all longitudinal reinforcement by bending the ties towards the inner section of the concrete increases the resistance against spalling of columns [81-83].</td>
</tr>
</tbody>
</table>

2.5.7 Concluding remarks on methods of preventing explosive spalling

The literature study on measures preventing concrete from explosive spalling leads to the following general conclusions:

- EN 1991-1-2 [6] provides only limited information on assessing and minimizing the risk of explosive spalling. Especially in-depth data for HPC and UHPC are not available.

- PP fibers are today considered as the most effective type of fibers increasing the resistance of concrete to explosive spalling [63, 67, 106, 133].

- 2 - 3 kg/m³ PP fibers (l = 6 - 12 mm; ø = 15 μm) seems to be most effective fiber admixture. Thinner fibers decreases the workability of the concrete, while thicker fibers are less effective [67, 75, 106].
- The use of steel fibers increases the spalling resistance of highly reinforced columns [81-83] but not with other structural members [75, 116].

- Thermal barriers reduce the heating rate and absolute temperature. However, critical temperature levels and heating rates are a function of the individual concrete mix [75].

- Design criteria for linings on OPC are available in the design standard DIN 4102-4 [3]. However, design criteria for linings on HPC and UHPC are not yet available.

- It has not yet been finally clarified whether air entrainment admixtures increase the spalling resistance of HPC and UHPC.

- Modifications of a concrete's mix design are effective but challenging, as they seem to be very sensitive in terms of reliably meeting the expected performance.

- Structural modifications, like secondary reinforcement or modified tie layout with adequate detailing improve the spalling resistance and residual resistance of spalled concrete members significantly [81-83, 86, 94].

2.6 Overall conclusions on literature study on concrete spalling at high temperatures

According to the literature review on tests and the analysis of concrete spalling at high temperatures in general and explosive spalling in particular, the following conclusions can be drawn:

- Especially for HPC and UHPC mixes, a high pore pressure inside the concrete during heating seems to be the governing factor leading to explosive spalling. Thermal- and load-induced stresses contribute to the occurrence and extent of explosive spalling. However, spalling was observed on unloaded specimens even at very low heating rates with negligible thermal gradients. These observations strengthen the conclusion that pore pressure-induced spalling is more important in regard to the spalling of dense concrete mixes [12, 63, 64, 75].

- Silica fume and other fillers or low w/c ratios as part of HPC and UHPC mixes are known to be critical factors in the case of explosive spalling. These components are usually associated with a decrease in the permeability of the concrete at high temperatures, minimizing the release of high vapor pressure [47-49, 68, 75, 81, 82].

- In addition to the concrete permeability, experience from tests indicated several additional factors, enhancing the occurrence of the explosive spalling of concrete. Mainly moisture content, compressive strength, w/c ratio and heating characteristics may be mentioned. Even though a ranking of these factors is given in the literature, no clear acceptance criterion is provided [27, 49, 57, 68].

- Generally accepted, standardized tests for a reliable determination of the possible occurrence of the spalling of concrete structures are not yet available. New approaches are presented in the literature but some influence parameters are usually neglected in these tests, which makes an assessment of the test results challenging [50, 51].
Many models of pore pressure development leading to explosive spalling are limited to OPC mixes [27, 94, 118, 120]. They might not be applicable to recent HPC and UHPC mixes with a low permeability.

The hydrothermal model of spalling by Dwaikat [33] considers several temperature-dependent concrete parameters and is based on governing universal equations, e.g. the conservation of mass and the ideal gas law. Pressure is released according to the permeability but moisture migration is not considered.

The European Standards like EN 1992-1-2 [6] suggest the use of a concrete mix that is known by tests or experience not to spall in fire conditions. However, it is difficult to verify this requirement without any engineering methods and standardized tests predicting the possible occurrence of spalling, which are not mentioned in the standard.

The use of PP fibers is generally accepted to increase permeability at high temperatures leading to a reduced risk of spalling due to the relief of high pore pressure. However, apart from the amount of monofilament PP fibers to be added to the concrete, no further specifications in terms of fiber geometry or melting characteristics are mentioned in codes such as EN 1992-1-2 [6]. Even though this protective measure is reported to be the most promising approach towards a spalling-safe concrete, several issues like spalling due to high thermal gradients remain unsolved.

A better knowledge exists regarding a protective lining on structural members made of OPC. The use of these thermal barriers is allowed in codes [3], but there is a still a lack of generally accepted design criteria, in particular for dense concrete mix designs with a higher risk of spalling.
3 EXPLOSIVE SPALLING AND INDICATORS

3.1 Introduction

A testing series on explosive spalling of HPC and UHPC was carried out at ETH Zurich. The test set-up and all detailed results are presented in the test report “Explosive spalling of concrete - Test report” [75]. This IBK report No.: 352 is available for downloading.

These tests indicated critical heating rates and temperature levels leading to explosive spalling of concrete. In addition, material properties were obtained for the tested concrete mixes at high temperatures. In the following, the governing factors influencing explosive spalling are discussed:

- Influence of the concrete mix and heating rate
- Influence of steel- and PP fibers
- Influence permeability and the development of porosity
- Influence of moisture content
- Discussion of pore pressure development.

The present thesis summarizes the main results of tests on concrete cylinders indicating and analyzing critical parameters leading to the explosive spalling of concrete. These results and material properties from the tests, as given in the IBK test report [75], are analyzed and discussed for the development of a basic model of explosive spalling due to high pore pressure at high temperatures as presented in chapter 4.

The IBK test report [75] includes further data on the development of material properties of protective linings on concrete and their possible use on structures. In addition, a description of tests on UHPC slabs (l · b = 1100 · 900 mm²) protected either with PP fibers or a protective lining and exposed to the standardized ISO fire curve [5] is presented. These tests on protective linings are not further analyzed in detail in this thesis.

3.2 Need for tests on the explosive spalling of concrete at high temperatures

A high pore pressure due to the evaporation of moisture is mentioned in the literature as one of the governing factors leading to the explosive spalling of concrete at high temperatures, apart from thermal- and load-induced stresses [27, 48, 49, 57, 68]. With the increasing strength of UHPC mixes and the increased use of silica fume, the risk of explosive spalling seems to shift towards spalling mainly due to a high pore pressure for these “dense” concrete mixes.

The aim of tests carried out at IBK as presented in [75] was to determine the spalling behavior of different HPC and UHPC, mixes which is mainly caused by high pore pressure. This was achieved using three concrete mixes, with different amounts of silica fume for a very “dense” and less permeable concrete. Very low heating rates were used to minimize possible effects like thermal stresses due to possible high temperature gradients. All specimens remained unloaded in all tests at high temperatures in order to neglect the influence of load-induced stresses on the heated concrete cylinders.

In the following all test results are summarized and - if possible - compared to existing models.
3.3 Tests on the explosive spalling of concrete at high temperatures

3.3.1 Scope of tests and tested concrete mixes

The tested concrete mixes presented in this chapter and the general testing procedure is described very briefly in the following. A detailed description of all tests, including individual test results is given in the technical report [75].

Two different concrete categories in the range of $f_c \approx 90 - 160$ MPa were tested:
- M1, M2 and M3 concrete mixes with and without PP fibers ($f_c \approx 90 - 153$ MPa)
- P1 concrete mix with different amounts and content of PP fibers ($f_c \approx 132 - 159$ MPa)

For the M1 - M3 concrete mixes, the main test results are discussed in the following:
- Linear heating of concrete cylinders to analyze the influence of the heating rate on the occurrence of spalling (chapter 3.3.3)
  - No use of PP fibers with tests on linear heating
  - Linear heating rates between $\bar{T} = 0.25 - 8.0$ K/min
  - Use of concrete cylinders ($\phi = 150$ mm, $h = 300$ mm)
  - Analysis of the influence of steel fibers (chapter 3.3.4)
  - Analysis of the use of different amounts of silica fume added to the concrete mix
  - No preloading during heating
- Tests on residual permeability ($T_{\text{max}} = 500$°C) including influence of steel- and PP fibers added to the concrete (chapter 3.3.6)
- Tests on the total porosity and average pore radius at different temperatures (chapter 3.3.7)
- Tests on the moisture content and loss in weight during heating (chapter 3.3.8)
- Additional tests on concrete slabs ($l \cdot w \cdot h = 1100 \cdot 900 \cdot 150$ mm$^3$) made from the M2 concrete mix without steel fibers and exposed to the ISO fire curve [5] (description only in the test report [75])

For the P1 concrete mix, the following main test results are discussed in the following:
- Linear heating including the influence of different amounts and types of PP fibers on the risk of explosive spalling (chapter 3.3.5)
  - Use of concrete cylinders ($\phi = 150$ mm, $h = 300$ mm)
  - No preloading during heating
- Tests on the total porosity and average pore radius at different temperatures (chapter 3.3.7)
3.3.2 Spalling due to linear heating - Tested concrete mixes and critical temperatures

Specimens made from the M1, M2 and M3 concrete mixes were tested in terms of explosive spalling by linear heating. Table 12 summarizes the relevant mechanical properties and mix design characteristics.

Table 12: M1 - M3 concrete without PP fibers - mechanical properties

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>Sub-mix</th>
<th>V1</th>
<th>V2</th>
<th>V1</th>
<th>V2</th>
<th>V1</th>
<th>V2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel fibers in Vol.-%</td>
<td>none</td>
<td>2.5</td>
<td>none</td>
<td>2.5</td>
<td>none</td>
<td>2.5</td>
<td>none</td>
</tr>
<tr>
<td>Cement content in kg/m³</td>
<td>832 kg/m³</td>
<td>580 kg/m³</td>
<td>400 kg/m³</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>water/cement ratio</td>
<td>0.22</td>
<td>0.30</td>
<td>0.33</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica fume content in % of cement content</td>
<td>16.2%</td>
<td>11.0%</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silica fume content in % of tot. mass</td>
<td>5.71%</td>
<td>2.63%</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength in N/mm²</td>
<td>153.1</td>
<td>151.3</td>
<td>114.6</td>
<td>129.9</td>
<td>90.3</td>
<td>96.5</td>
<td></td>
</tr>
<tr>
<td>Tensile strength in N/mm² 1)</td>
<td>7.1</td>
<td>5.2</td>
<td>4.2</td>
<td>5.3</td>
<td>5.6</td>
<td>5.8</td>
<td></td>
</tr>
<tr>
<td>Density in kg/m³</td>
<td>2318</td>
<td>2411</td>
<td>2353</td>
<td>2502</td>
<td>2436</td>
<td>2568</td>
<td></td>
</tr>
<tr>
<td>Young’s modulus N/mm²</td>
<td>47’700</td>
<td>46’600</td>
<td>41’250</td>
<td>43’300</td>
<td>46’550</td>
<td>48’150</td>
<td></td>
</tr>
</tbody>
</table>

1) Tests in direct tension

All tests on spalling were carried out on regular concrete cylinders (φ = 150 mm, h = 300 mm). The concrete was heated linearly with rates between Ṫ = 0.25 - 8.0 K/min measured directly at the surface of the concrete cylinders. The increase in temperature was controlled by two thermocouples placed directly on the surface of the concrete cylinder. In addition, the temperature was monitored from 30 mm depth to the surface and in the center core of the cylinder. During these tests on explosive spalling, all specimens remained unloaded. Figure 8 shows the general test layout with a concrete cylinder already prepared for testing and a cylinder placed inside the electric furnace. The location of the thermocouple is shown in the right part of Figure 8.

![Figure 8](image_url)

Figure 8: left: Concrete cylinder with thermocouples glued to the surface inside electric furnace right: Position of the thermocouples during heating

All specimens were heated to a maximum temperature of T = 500°C or to a temperature where the first occurrence of explosive spalling was noticed. In total 24 tests on concrete cylinders heated with different rates were performed using the electric furnace. The change in the heating rate Ṫ in K/min was the only test parameter. Each test with the corresponding concrete mix, heating rate, the possible occurrence of explosive spalling, and critical temperatures.
Explosive spalling and indicators

spalling and the maximum temperature gradient are summarized in Table 13. Information on “plateau” and “spalling” temperatures as well as the corresponding temperature curve is explained in detail in the following. Pictures of the tested specimens, including fragments of the spalled concrete cylinders are shown in the test report [75].

Table 13: Summary of tests on explosive spalling by linear heating [75]

<table>
<thead>
<tr>
<th>Test</th>
<th>Heating rate</th>
<th>Max. temp. gradient</th>
<th>Spalling</th>
<th>Plateau temperature&lt;sup&gt;4)&lt;/sup&gt;</th>
<th>Spalling temperature&lt;sup&gt;4)&lt;/sup&gt;</th>
<th>Temperature curve&lt;sup&gt;4)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K/min</td>
<td>K/mm</td>
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1) Spalling was accompanied by a single loud bang
2) Spalling was accompanied by a continuous popping sound
3) Temperature measured at surface of the concrete cylinder only (no thermocouples inside concrete)
4) As discussed in chapter 3.3.3 and shown in Figure 9 and Figure 10
3.3.3 Influence of heating rate on explosive spalling

It was observed in these tests that explosive spalling only occurs with the M1 and M2 concrete cylinders, while no spalling was observed in the M3 concrete mix, even though the M3 concrete mix was exposed to the highest heating rates. The tests showed that explosive spalling could be avoided for all three concrete specimens by keeping the heating rate below a critical value, depending on the concrete mix. These results confirm that the use of silica fume as part of the concrete mixes M1 and M2 increases the risk of explosive spalling significantly. The influence of steel fibers on spalling behavior is analyzed in chapter 3.3.4.

Observation of plateau phase with constant temperature during heating

During all tests on the linear heating of concrete cylinders, the temperature development at the surface, at 30 mm depth and at the center core of the concrete specimen was monitored continuously using thermocouples as shown in Figure 8. While the temperature at the surface of the concrete cylinder increased linearly according to the selected heating rate, a “plateau” phase with the core temperature was noticed. A significantly slower increase in temperature was observed. This effect is subsequently called the “moisture-induced plateau”. For the temperatures measured at 30 mm depth, this moisture-induced plateau is less pronounced. Figure 9 (left) shows the temperature development for a typical concrete cylinder heated with $\dot{T} = 8.0 \text{ K/min to } T = 700^\circ\text{C}$. In this case, the moisture-induced plateau was clearly visible for about half an hour at a core temperature of $T = 120^\circ\text{C}$. This effect of delayed increase in temperature is mainly visible for the core temperature of the concrete specimen. At 30 mm depth, the moisture-induced plateau is less pronounced. Due to the direct exposure to high temperatures, the plateau phase is not visible at the surface of the specimen.

![Figure 9](image_url)

**Figure 9:** left: Temperature development of OPC heated with $\dot{T} = 8 \text{ K/min}$
moisture-induced plateau phase at core
right: Definition of temperature during moisture-induced plateau (plateau temperature)
and temperature at onset of spalling (spalling temperature) (typical shape of curve)

Figure 9 (right) shows an idealized temperature distribution at the core of a concrete cylinder during heating illustrating the moisture-induced plateau. The start and end temperatures of the plateau phase as well as the spalling temperature in the core of the specimen are shown here.
The results shown in Figure 9 (right) serve only to illustrate the behavior and are not taken from real tests. The starting point and end point of this plateau phase are not defined by standards. The corresponding start and end temperatures of the plateau phase were measured manually by assuming two lines from $T = 20^\circ$C onwards and after the plateau phase when the core temperature increases again. The slopes of these two lines change according to the heating rate and the measured temperatures inside the concrete. In addition, a line during the plateau phase is drawn. These three lines are shown in Figure 9 (right). The start and end temperatures of the plateau phase are now defined as the intersection of two lines but it was rather difficult to determine the temperatures in some cases.

During the heating of the concrete sample, the formation of the moisture-induced plateau was not always clearly visible. It was noticed that the formation varied according to the heating rate and concrete mix. While - generally speaking - low heating rates cause a clearly pronounced moisture-induced plateau, high heating rates leading to explosive spalling reduce the formation of this plateau phase. It was noticed that the plateau phase is usually not fully developed when spalling occurs. In total, three different temperature developments during the tests were observed. Figure 10 shows these three different temperature developments for selected concrete specimens tested at elevated temperature. The differences may be explained as follows:

Type 1 temperature curve - moisture-induced plateau fully developed

Figure 10 (left) shows the temperature curve measured for the specimen made of M2 concrete mix with steel fibers, heated with $\dot{T} = 1.0 \text{ K/min}$ that did not fail by explosive spalling. In the temperature range of around $T = 245 - 255^\circ$C to $T = 277^\circ$C the formation of the moisture-induced plateau is visible. It seems that because of the low heating rate, the emerging vapor can be released depending on the permeability of the concrete via the existing voids. The pore pressure does not reach a critical value and the vapor can be released without spalling, i.e. the specimen remained intact. All concrete specimens that did not fail by explosive spalling exhibited a similar temperature development and are indicated as “Type 1” temperature curve in Table 13. The start and end points of the temperatures of the moisture-induced plateau are given here. The moisture-induced plateau was observed more clearly for the specimens made of the M3 concrete mix without steel fibers (see Figure 11 left). Further, it was noticed that the temperature of the moisture-induced plateau depends on the concrete mix tested. The lowest temperature was measured for the specimens made of the M3 concrete mix without silica fume, while the highest temperature was observed for the specimens made of the M1 concrete mix with the highest amount of silica fume and thus for a very dense concrete. Comparing these results with the plateau phase of the OPC with a compressive strength of about $f_c = 33 \text{ N/mm}^2$ as displayed in Figure 9 (left), it may be seen that the moisture-induced plateau can be observed at a temperature of about $T = 120^\circ$C, i.e. much lower than the values measured for the concrete specimens M1 and M2. This supports the assumption that peak vapor production occurs at different temperatures well above $T = 100^\circ$C depending on the concrete mix used. This agrees with other observations found in the literature [96, 102, 127, 128], where a boiling front at $T = 200^\circ$C was experimentally analyzed for HPC.
Type 2 temperature curve - moisture-induced plateau partly developed

Figure 10 (middle) shows the temperature curve measured for the specimen made of the M2 concrete mix without steel fibers, heated with $T = 2.5 \text{ K/min}$ that failed by explosive spalling. The temperature measured in the core at $T = 205$ - $215^\circ\text{C}$ increased slowly compared to the surface temperature and may indicate the start of the moisture-induced plateau as explained previously. The specimen however failed by explosive spalling at a core temperature of $T = 277^\circ\text{C}$. The specimens that showed a similar behavior are indicated as “Type 2” temperature curve in Table 13. From the analysis of the fracture pattern it is assumed that explosive spalling was initiated from the core of the specimens. The sizes of the spalled pieces are rather large. The specimen made of the M1 concrete mix heated with $T = 0.75 \text{ K/min}$ was split into clear fragments (Figure 15) as if cut lengthwise. This observation, in addition to the sound upon spalling, strengthens the assumption that failure of the specimen was initiated in deeper sections of the concrete. Spalling was accompanied by a single loud bang. Since the heating rates are usually higher for “Type 2” temperature curves, the rate of water evaporation inside the concrete is increased. The generated vapor cannot be relieved with the present permeability of the concrete. A higher pressure inside the concrete is expected with rising temperatures, even during the low increase in temperature during the plateau phase. Exceeding a critical pressure level will result in explosive spalling.

Type 3 temperature curve - moisture-induced plateau not developed

Figure 10 (right) shows the temperature curve measured for the specimen made of the M1 concrete mix with steel fibers, heated at $T = 1.0 \text{ K/min}$ that failed by explosive spalling. It may be seen that the temperature measured in the core increased almost linearly until spalling, without the formation of a moisture-induced plateau. The specimens that showed a similar behavior are indicated as “Type 3” temperature curve in Table 13. From the analysis of the fracture pattern (Figure 15) it may be assumed that explosive spalling took place layer-by-layer beginning from the surface. The sizes of the spalled pieces are rather small and even decrease with increasing heating rates as shown in Figure 16. Spalling was accompanied by a “popping” sound, which indicated the relief of concrete fragments. Spalling continued until the test was stopped manually. Similar to the observations made for specimens that failed according to the “Type 2” temperature curve, a high rate of water evaporation is expected. However, due to rapid heating, the concrete sections close to the heated surface are affected first and the buildup of high pore pressures in these areas is expected first. The relief in pressure will result in continuous layer-by-layer spalling. The development of high pore pressures exceeds the probable relief of vapor outwards or into deeper sections of the concrete due to the low permeability of the concrete.
**Formation of the moisture-induced plateau due to vaporization of moisture**

The development of the moisture-induced plateau is caused by vaporization of moisture inside the concrete as stated above. This vaporization of free and physically bound water inside the concrete starts at temperatures exceeding $T = 100°C$. The change from the liquid to the gaseous phase of the water is related to an increased uptake of evaporation energy, which is about $J = 2.26 \text{ MJ/kg}$ water during vaporization at $T = 100°C$. This high amount of energy uptake during the vaporization process causes the delay in temperature increase. With increasing temperature, this energy demand decreases as shown in Figure 12.

However, this vaporization takes place at any temperature above $T > 100°C$ and does not explain the occurrence of the moisture-induced plateau at a specific temperature as observed in the tests.
It is obvious that the release of vapor is limited due to the low permeability of HPC and UHPC. At a temperature of \( T = 100^\circ \text{C} \) and only part of the evaporated moisture will migrate outwards in the form of vapor, while the other part remains inside the concrete as water. This slow release of moisture is also noticed during the drying of HPC and UHPC to constant mass at \( T = 105^\circ \text{C} \), which is a time consuming process. Elevated temperatures of about \( T = 170^\circ \text{C} \) accelerate this drying process as already observed by Hertz in 1984 [47].

With increasing temperatures, pressure from evaporating vapor rises inside the concrete, combined with an increase in permeability of the concrete (see chapter 3.3.6). Vapor is released according to the pressure and permeability values. At a specific temperature the evaporation rate of moisture and release of vapor due to the current permeability have equalized and the pressure will not rise.

Assuming an isochoric state of the pores inside the concrete, the temperature will not rise if the pressure remains constant as observed during the plateau phase. This process mainly depends on the heating rate, since an increase in temperature has a direct influence on the vaporization rate. Higher vapor content will exceed the release of vapor and lead again to an increase in pressure.

It should be noted that an increased rate of microcracking can be expected at higher temperatures, due to shrinkage and decomposition of the cement paste [57], hence an isochoric state of the concrete pores is a rather rare occurrence. However, microcracking leads to a sudden release of high pore pressures, which might even lead to a decrease in temperature inside the concrete, while the surface temperature still increases according to the selected heating rate. This was observed with rather high heating rates at which a rapid increase in pore pressure and microcracking is expected. Figure 11 (middle) shows the temperature development for an OPC (\( f_c = 43.3 \text{ MPa} \)) heated with \( \dot{T} = 3.0 \text{ K/min} \). The temporary decrease in core temperature is visible here during the plateau phase at \( T = 200^\circ \text{C} \).

During the plateau phase, the increased production and release of vapor leads to drying-out of the concrete. All free and physically bound water will evaporate at this temperature; hence sufficient moisture for an additional buildup of pressure with rising temperature is not available. The amount of moisture from decomposition of the CSH-phases is usually insufficient to build up vapor pressure; in addition, this additional vapor is released immediately due to the increased permeability. A detailed analysis of the loss of weight during heating is given in chapter 3.3.8.

**Formation of moisture-induce plateau in concrete mix containing PP fibers**

It may be seen that the formation of the moisture-induced plateau is mainly visible for low and moderate heating rates and mainly for temperatures measured in the inner sections of the specimen. The delay in temperature increase is usually overcompensated by rapid heating rates as experienced in real fires.

These observations were made during the heating of the P1 concrete specimen containing PP fibers (\( f_c = 135.2 \text{ MPa} \)). This concrete cylinder was heated with \( \dot{T} = 10 \text{ K/min} \) as shown in Figure 11 (right) and the plateau phase is visible within a temperature range of \( T = 170 - 230^\circ \text{C} \). However, a significant increase in temperature was observed even during this phase.
It is interesting to note that the plateau phase starts at the temperature of $T = 170^\circ C$, the melting temperature of the PP fibers and the temperature at which the permeability of the concrete increases significantly (see chapter 3.3.6). This jump in permeability causes a release in vapor and leads to a slower increase in pressure with the corresponding temperature inside the concrete. This observation confirms the statement that the moisture-induced plateau usually starts when the release of vapor due to sufficient permeability and the rate of vaporization are balanced. In the case of rapid heating, the increased permeability due to the use of PP fibers can counterbalance the high rate of vaporization.

**Brief conclusions on the formation of the moisture-induced plateau:**
- Moisture inside the concrete starts to evaporate at temperatures exceeding $T > 100^\circ C$.
- A mix of water as liquid and saturated vapor is present inside the pores of the concrete.
- Vapor pressure rises inside the pores with rising temperature.
- Only parts of the vapor pressure are released due to the low permeability of the concrete.
- Both pressure and permeability continue to increase with increasing temperature.
- At a specific temperature, the vapor production rate and the release of vapor due to the permeability are balanced.
- Vapor production and the pressure inside the concrete peak at this temperature.
- Temperature and pressure will not rise due to the increased energy uptake during evaporation. The moisture-induced plateau phase starts, which takes place mainly in inner parts of the concrete.
- Temperature increases again once all moisture evaporates and the concrete becomes dry.
- Adding PP fibers to the concrete will lead to the start of the plateau phase at $T \approx 170^\circ C$ due to the increase of permeability (melting of PP fibers).
Observations mentioned in the literature in terms of evaporation at high temperatures

Two references from the literature substantiate the analysis of the temperature plateau phases and pressure development as derived from these tests on concrete cylinders.

Van der Heijden [127, 128] visualized the vaporization of moisture inside concrete at higher temperatures with the help of MRI scans of concrete during heating. A vaporization front was observed in deeper sections of the concrete at temperatures of about $T \approx 200^\circ$C. This confirms that the majority of moisture inside the concrete evaporates at higher temperatures than $T = 100^\circ$C.

Mindeguia and Pereira [96, 102] performed tests on the pore pressure development inside the concrete at high temperatures using pressure transducers. In their tests a plateau phase inside the concrete was also observed where both temperature and pressure did not rise, even with increasing temperatures at the concrete surface.

The evaporation energy decreases with increasing temperature as shown in Figure 12. It can be assumed that the length of the moisture plateau would decrease with higher temperatures due to the smaller energy intake. However, this effect is a rather rare occurrence since the permeability increases and the moisture content decreases with higher temperatures; hence in general less evaporation energy would be required.

Figure 12: Temperature-dependent evaporation energy for the change in phase from water to vapor

Moisture-induced plateau with multiple heating cycles

In order to support the explanation that the plateau phase is caused by the release of vapor pressure at high temperatures and increased uptake of energy during evaporation of the free- and physically-bound water an additional test with two heating cycles was carried out. The moisture-induced plateau phase should be visible only during first heating cycle, since all water would have been vaporized after the first cycle.

A concrete cylinder ($f_c = 42.3$ MPa) was heated linearly at the surface with a rate of $\dot{T} = 3.5$ K/min to a maximum surface temperature of $T = 500^\circ$C. After reaching the peak temperature the specimen was kept inside the furnace and cooled “naturally” to an ambient temperature of $T = 20^\circ$C by switching off the heating. After cooling, a second heating cycle was applied.
Explosive spalling and indicators

Figure 13 shows the temperature development measured during both heating cycles. It is interesting to note that the plateau phase is visible only during the first heating cycle, observed at a core temperature of about $T \approx 190^\circ C$. During the first heating cycle all free and physically-bound water evaporates, which leads to the plateau phase. In addition, a small decrease in temperature during this plateau phase was noticed. This might be caused by local cracking, which increases the pore volume and decreases the pressure respectively.

During the second heating cycle, the temperature increase in the core is almost linear to maximum temperature and no plateau phase is visible. Even though the risk of explosive spalling due to high pore pressure is minimized in the second run, critical thermal stresses might still occur, which then lead to failure.

![Temperature development graph](image)

Figure 13: Temperature development inside OPC cylinder during two heating cycles ($\dot{T} = 3.5 \text{ K/min}$)

Moisture-induced plateau at core temperature of $T = 170^\circ C$ is only visible during first cycle

**Influence of temperature gradients**

By comparing the tests on concrete mixes M1 and M2, it is noted that in terms of the M2 concrete mix, explosive spalling of the specimens generally took place at higher heating rates. Since these heating rates lead to higher temperature gradients inside the concrete, the influence of thermal stresses on the risk of explosive spalling cannot be neglected in these cases.

In the literature, a temperature gradient of up to $\Delta T < 1.0 \text{ K/mm}$ [114] only leads to minor thermal stresses and may be assumed to have insignificant influence on the risk of spalling. Higher temperature gradients cause considerably higher thermal stresses close to the heated concrete surface, which contributes to explosive spalling.

The maximum temperature gradient $\Delta T$ in K/mm measured during the tests and given in Table 13 is simply calculated by subtraction of the core temperature from the surface temperature and dividing the result by the radius of the cylinder ($r = 75 \text{ mm}$). The gradient shown refers to the maximum temperature gradient during the entire heating cycle. This covers both types of test: Tests which were stopped manually since the specimens failed by spalling or tests on concrete cylinders that were heated to a maximum surface temperature of $T = 500^\circ C$.

For all M1 concrete specimens a temperature gradient well below $\Delta T << 1.0 \text{ K/mm}$ was measured, hence thermal stresses can be neglected in these tests. The test on the M1 concrete mix heated with $\dot{T} = 0.75 \text{ K/min}$ (M1 - test 3) exhibited explosive spalling at a surface temperature of $T = 368^\circ C$. The maximum difference in
the surface and core of the specimen was $\Delta T = 64 \text{ K}$, which is equivalent to a temperature gradient of $\Delta T = 0.85 \text{ K/mm}$. For a heating rate of $\dot{T} = 1.0 \text{ K/min (M1 - test 4)}$, spalling was observed at a lower surface temperature of $T = 297^\circ\text{C}$ and without the formation of a moisture-induced plateau. This leads to a decreased temperature difference of $\Delta T = 42 \text{ K}$ with a corresponding gradient of $\Delta T = 0.56 \text{ K/mm}$.

As mentioned above, it can be assumed that the influence of thermal stresses-induced by thermal gradients was low and negligible. Pore pressure can be considered as the main factor leading to explosive spalling.

Further, it is interesting to note that for the M1 concrete specimens that failed by explosive spalling due to rapid heating rates an even lower temperature gradient was observed compared to the specimens that did not fail by linear heating. This is mainly because the specimens failed before the development of the moisture plateau phase was observed (type 3 temperature curve). At the point of failure, the difference between the surface and core temperatures was low.

Figure 14 shows temperatures inside the concrete with increasing distance from the surface measured for a corresponding surface temperature of $T = 150^\circ\text{C}$, 300°C and 500°C for two different heating cycles: the M1 concrete heated with $\dot{T} = 0.25 \text{ K/min}$ and the M3 concrete heated with $\dot{T} = 8.0 \text{ K/min}$.

At a surface temperature of $T = 300^\circ\text{C}$ at which spalling usually takes place, significant changes in the temperature gradient between the different heating rates were observed. For the low heating rate of $\dot{T} = 0.25 \text{ K/min}$, the temperature gradient between the surface and the core is almost linear and with a gradient of $\Delta T = 0.26 \text{ K/mm}$ well below the gradient of $\Delta T = 1.0 \text{ K/mm}$ when thermal stresses must be taken into account [114]. The linearity of the temperature gradient is almost constant for all three temperature levels.

In contrast, the difference for a temperature gradient from the surface to the core increases to $\Delta T = 2.1 \text{ K/mm}$ for the rapid heating of $\dot{T} = 8.00 \text{ K/min}$. The influence of possible thermal stresses must be considered. In addition, the temperature gradient becomes nonlinear.

![Figure 14: Thermal gradient during heating](image-url)
After the analysis of all test data it may be concluded in terms of temperature gradients inside the concrete that for low heating rates of up to $\dot{T} \leq 2.0$ K/min a very low thermal gradient can be observed between the surface and the core for all temperature levels. Further, the additional temperature measured at 30 mm depth demonstrates that the thermal gradient within the specimen’s cross-section is linear for these low heating rates, similar to the one shown in Figure 14. The opposite trend could be observed with higher heating rates starting with $\dot{T} > 2.0$ K/min. In this case an increasing thermal gradient was measured with increasing temperatures and the thermal gradient within the specimen’s cross-section became nonlinear. This trend is even more pronounced with increasing surface temperatures.

Even though a thermal gradient of $\Delta T = 1.0$ K/mm is considered to lead to high thermal stresses inside the concrete, which promotes spalling [114], the measured thermal gradients during heating the M2 concrete specimens and shown in Table 13 do not significantly exceed this limit. For those specimens that failed by explosive spalling and in which a thermal gradient in the range of $\Delta T = 1.5 - 1.8$ K/mm was measured, a high pressure can still be assumed as the main factor leading to explosive spalling. This is substantiated by the visual inspection of the concrete fracture pattern after spalling.

Visual inspection of concrete fracture pattern after spalling

The analysis of the measured temperature and thermal gradients inside the concrete showed that spalling due to high pore pressure is the main factor for the M1 concrete mix and can still be considered as the defining factor for the M2 concrete mix, even though the temperature gradient exceeds the critical value of $\Delta T = 1.0$ K/mm. With these considerations, a possible moisture migration and its corresponding influence on the extent of spalling and the fracture pattern in the specimens are analyzed further.

The visual inspection of the concrete fragments after spalling as shown in Figure 15 and Figure 16 for the M1 and M2 concrete mixes shows that an increase in the heating rate leads to smaller fragments after spalling. A correlation between the heating rate and size and shape of the spalled concrete fragments can be considered, in particular for the M1 concrete mix.

Fracture pattern of M1 concrete specimens after spalling

The M1 concrete specimen containing steel fibers and heated with $\dot{T} = 0.5$ K/min (M1 - test 2) remained intact, while the specimen heated with $\dot{T} = 0.75$ K/min (M1 - test 3) spalled into a few large pieces of concrete, thus indicating that explosive spalling is initiated from the core of the specimen and is accompanied by a single loud bang. The specimen heated with $\dot{T} = 1.0$ K/min (M1 - test 4) exhibited smaller flakes upon spalling. This leads to the assumption that spalling took place by the release of small concrete parts, layer-by-layer beginning from the surface. This consideration is strengthened since a “cracking noise” was noticed during spalling.

The specimen made from the M1 concrete mix without steel fibers and heated with $\dot{T} = 0.5$ K/min spalled into a few medium-sized pieces (M1 - test 6) at a surface temperature of $T = 300^\circ$C. No significant change in the size of the fragments was observed. Due to the lack of steel fibers the more brittle material behavior has an influence on the shape of the spalled concrete fragments. The entire cylinder was destroyed completely.
by spalling and it was accompanied by a single loud bang. It is assumed that spalling was initiated from the core of the specimen. A “Type 3” temperature curve without the development of a plateau phase was noticed during heating, which is usually associated with high heating rates and layer-by-layer spalling. With a maximum temperature gradient of $\Delta T = 0.257 \, \text{K/mm}$ thermal stresses can be neglected.

Tests on permeability (chapter 3.3.6) showed a decrease in permeability for both types of M1 concrete specimen within a temperature range from $T = 150 - 275^\circ\text{C}$. This decrease in permeability was very pronounced for the M1 concrete mix without steel fibers within this temperature range. This leads to the conclusion that only very limited moisture migration takes place inside the concrete and the release of vapor to the outside is restricted as well. This would explain the “Type 3” temperature curve, since vapor is not released that would usually lead to the plateau phase as previously explained.

The temperature difference upon spalling was as little as $\Delta T = 18 \, \text{K}$. This leads to the conclusion that the temperature-dependent pressure inside the concrete is within the same range at any position over the entire cross section. If this pressure exceeds a critical value, the entire specimen tends to exhibit explosive spalling. A starting point of failure due to spalling cannot be identified.

The tests on losses in weight during heating (see chapter 3.3.8) clearly showed that moisture migrates in the form of vapor outside the concrete. However, in the case of concrete specimens that failed by explosive spalling, moisture migration from the outer areas of the concrete to inner sections must be considered too. For the test on the M1 concrete heated with $\dot{T} = 0.75 \, \text{K/min}$ migration processes inside the concrete very probably occurred. It is obvious that moisture evaporates and migrates mostly out of the concrete due to the pressure gradient from the heated areas to ambient conditions outside the concrete. However, moisture will also migrate further inside the concrete into cooler sections, condense there and lead to a higher moisture content in these inner sections. This “moisture clog” close to the heated surface will move further inside the concrete with increasing temperatures [19, 20, 118, 120]. The velocity of this moisture clog depends on the increase in temperature and hence the vaporization rate (boiling front) and the permeability of the concrete, enabling the movement of vapor further inwards. Apart from the presence of sufficient moisture, the migration process strongly depends on the temperature, its corresponding pressure inside the heated concrete and the permeability of the concrete.

Higher moisture contents in different sections of the concrete will lead to a higher vapor content, which has to be released once the boiling front reaches these concrete sections. In addition, once the temperature and pressure inside the concrete promote the development of microcracks, the sudden release of vapor and mainly the remaining moisture might lead to violent spalling.

A rise in heating rates leads to a rapid increase in temperature close to the heated surface. This exposure promotes explosive spalling, characterized by flaking off of the surface, i.e. the concrete specimen spalls layer-by-layer as observed with the M1 concrete specimen heated with $\dot{T} = 1.0 \, \text{K/min}$ (M1 - test 4). The visual observations after the tests confirm the noise observations during the tests, i.e. at high heating rates spalling was accompanied by popping, while a single loud bang characterized spalling at lower rates.

This audible inspection of spalling at high temperatures and visual inspection of the concrete fragments strengthens the assumption that a high pore pressure close to the heated surface leads to spalling. Due to the higher temperature gradient inside the concrete caused by rapid heating, a high temperature-dependent pore pressure will develop mainly in the concrete sections close to the heated surface. Once this concrete
layer fails by explosive spalling, the next concrete layer located further inside the concrete is directly exposed to high temperatures. This also causes a high temperature-dependent pore pressure showing a tendency to explosive spalling again.

Independent of the spalling mechanism, the visual inspection of the spalled concrete fragments showed that the steel fibers slipped within the concrete and fiber ends with a fiber length of about $l = 3$ mm or more are visible in the fracture zone. This effect is analyzed in detail in chapter 3.3.4.

![Figure 15: Fragments after spalling of M1 concrete mix with and without steel fibers and decreasing size of spalled concrete fragments with increasing heating rate](image)

**Fracture pattern of M2 concrete specimens after spalling**

Similar to the M1 concrete mix, the heating rate and the resulting moisture migration have an all-important influence on the occurrence and extent of explosive spalling for the M2 concrete specimens. Figure 16 shows fracture patterns for selected specimens made of the concrete mix M2 with steel fibers. The fracture pattern for the entire testing series is shown in the test report [75]. Similar to the observation made with the M1 concrete specimens it should be noted that the size of the spalled concrete pieces decreased with increasing heating rate confirming the results observed for the specimens made of the M1 concrete mix (Figure 15). Since by increasing heating rates the concrete zones close to the surface are exposed to rapidly increasing temperatures, explosive spalling characterized by flaking off of the surface is more likely, i.e. the concrete specimen spalls layer-by-layer. The visual observations after the tests confirm the noise observations during the tests, i.e. at high heating rates spalling was accompanied by popping, while a single loud bang characterized spalling at lower heating rates. Independent of the spalling mechanism, the visual inspection of the spalled concrete fragments showed that the steel fibers slipped within the concrete and fiber ends with a fiber length of about $l = 2 - 3$ mm are visible in the fracture zone, similar to the observations made with the M1 concrete mix.

![Figure 16: Fragments after spalling of M2 concrete mix with steel fibers with a decreasing size of spalled concrete fragments with an increasing heating rate](image)
For the M3 concrete, no spalling was observed. The M3 concrete mix does not contain silica fume and has a higher permeability (chapter 3.3.6), enabling the release of high pore pressure. All tested concrete specimens showed a clear moisture plateau phase during heating, starting at rather low temperatures due to the increased permeability of the concrete.

Pictures of all concrete specimens, including the fragments of the specimens that failed by explosive spalling are shown in the test report [75].

**Tests on concrete slabs exposed to the ISO fire curve**

Tests on concrete slabs (l·w·h = 1100·900·150 mm³) were carried out in addition in order to study the possible use of protective linings to minimize the risk of explosive spalling. These tests are described in detail in the IBK test report [75].

Concrete slabs made from the M2 concrete mix without steel fibers were exposed to the ISO fire curve [5] for a duration of \( t = 120 \text{ min} \) from the bottom side. Two of these tests are further analyzed since similar observations were made in the case of explosive spalling.

The first test was carried out on an unprotected concrete slab, which was directly exposed to the fire; the second test was covered with a 10 mm protective lining to reduce the heating at the concrete surface.

Explosive spalling was noticed on the unprotected concrete slab (Test 1) starting after a fire exposure time of \( t = 15 \text{ min} \) and a temperature measured at the surface of the slab of \( T = 400°C \). It was accompanied by a loud popping sound as concrete fragments were released layer-by-layer from the concrete surface. A rapid increase in the surface temperature took place after an exposure time of \( t = 15 \text{ min} \). The thermocouples placed at the surface were directly exposed to the fire, since the surrounding concrete acting as a protective cover spalled.

Some moisture in the form of water was noticed during the tests on the “cold” top surface of the specimen along with smaller cracks, which were visible after first spalling was observed. The test was stopped manually.
Explosive spalling and indicators

after a total time of t = 30 min. A maximum spalling depth of up to 63 mm in the concrete cover was measured after the test.

The use of a 10 mm protective lining decreased the temperature rise measured at the interface between the lining and the concrete surface significantly, leading to a surface temperature of $T \approx 407^\circ C$ after a fire exposure time of $t = 119$ min. Similar to the first tests, moisture was noticed on the “cold” surface of the slab, coming out of small cracks inside the concrete on the top surface as well as along the slab edges.

Explosive spalling in the form of a sudden and violent release of concrete was accompanied by a single loud bang after a fire exposure of $t = 119$ min. This spalling leads to a loss of up to 63 mm in concrete cover, similar to that observed in the test on the unprotected concrete slab. The test was stopped after spalling occurred.

**Possible assessment of critical temperatures leading to explosive spalling**

It is difficult to assess possible critical temperatures from these tests leading to explosive spalling, even though both failed by explosive spalling within the range of a surface temperature of $T \approx 400^\circ C$. However, moisture migration must be considered. For the slab with a 10 mm protective layer it can be assumed that the areas close to the surface are getting dry due to the increase in temperature and moisture migration. With rising temperatures, these areas do not contain sufficient moisture to build up a critical pore pressure. However, a moisture-induced plateau for temperatures in deeper sections of the concrete was not observed in these tests. The layer towards the fire exposure is dry before the formation of a moisture plateau. The temperatures in the inner layers (30 mm, 50 mm inside the concrete) are still below the expected temperature range for the moisture-induced plateau phase of this concrete ($T \approx 240 - 250^\circ C$ for M2 concrete).

Even though critical temperatures could not be derived from these tests, a correlation between heating rate and extent of spalling was observed as in the observations made in tests on concrete cylinders:

- High heating rates (Test 1) lead to rapid rising temperatures and pressures inside the concrete close to the heated surface, which will result in layer-by-layer spalling, since the release of vapor is limited due to the low permeability of the concrete.
- A small increase in surface temperature as achieved by using a thin sheet of protective lining (Test 2) leads to spalling initiated in deeper sections of the concrete, due to moisture migration into these sections.

**Influence of cracks on moisture migration**

These observations from the tests on concrete slabs substantiate the assumption that moisture migration inside the concrete has a significant influence on the extent of explosive spalling. Moisture was visible on the cold surface of the concrete slab indicating that both vapor and water migrate within the concrete. Rapid movements of water are most unlikely due to the high viscosity of water compared to vapor [130], which is also mentioned in the literature by Dwaikat [33]. However, a considerable influence of thermal stresses on the spalling behavior of the concrete slabs must be taken into account when analyzing the test results, mainly due to the rapid heating of the concrete slab. These stresses seem to promote the formation of microcracks and increases in moisture migration inside the concrete via these cracks. This was noticed with cracks along
the edges and on the upper side of the slab. A significant flow of water was observed in these cracks. It must be considered that water vaporizes in inner sections of the concrete and migrates, due to the pressure gradient, into cooler sections of the concrete where it condenses again. The additional cracks lead to a flow of water, promoted by the high pore pressure in deeper concrete sections.

This phenomenon is also described in the literature, cf. Meyer-Ottens [94]. He noticed severe cracking and the release of large amount of vapor, in some cases even “fountains” of water on the cold side of the heated slab in some of his tests. Hasenjäger made similar observations during his tests in 1935 [45].

Concluding remarks on observations of the influence of heating rate

The tests by linear heating of the concrete cylinders provide information on the spalling mechanisms in the concrete cylinders. The main findings regarding the influence of the concrete mix and heating rate can be summarized as follows:

- Silica fume was confirmed as a decisive parameter leading to explosive spalling. Spalling was noticed only in concrete specimens containing silica fume (M1 + M2 concrete mix).

- A slower increase in temperature in inner sections of the concrete was noticed. The formation of this plateau phase is caused by the increased energy uptake during the peak evaporation of free and physically bound water. Each concrete mix showed a plateau phase within an individual temperature range.

- The moisture plateau starts when vaporization of moisture and release of moisture depending on the permeability are balanced. This phenomenon is clearly visible in the case of concrete containing PP fibers, since the permeability increases significantly at $T = 170^\circ$C.

- Spalling was only observed during or before the formation of the moisture-induced plateau phase.

- The presence of silica fume shifts the mechanism of spalling into the area of spalling due solely to a high pore pressure caused by evaporating moisture. In particular, for the M1 concrete mix thermal stresses can be neglected with these low heating rates leading to explosive spalling.

- High heating rates lead to spalling layer-by-layer, while low heating rates lead to spalling initiated in the core of the specimen. Moisture migration into and out of the specimen and drying of concrete layers caused these different failure mechanisms.

- Tests on M2 concrete slabs exposed to the ISO fire and partly covered with lining as a thermal barrier exhibited similar spalling mechanisms. Direct exposure (rapid heating) leads to layer-by-layer spalling, protected concrete (slow heating) failed at once by sudden spalling.

- Cracking promotes moisture migration, in particular of water as liquid. This leads to a more rapid release of pore pressure and minimizes the risk of spalling. The migration of water as liquid in uncracked concrete is very low due to the high viscosity.
3.3.4 Influence of steel fibers

Steel fibers should always be added to HPC and UHPC mixes since they increase the ductility of the concrete. Several fundamental tests on UHPC mixes by Fehling [35] clearly indicated the need for steel fibers for a safe and durable concrete design in the cold state.

It is generally agreed that the use of steel fibers increases the tensile strength, which is then assumed to lead to a higher resistance against explosive spalling [57]. Tests on concrete samples (PP fibers free mixes M1 - M3) with 2.5% in volume of typical steel fibers ($\phi = 0.15$ mm, $l = 6$ mm, $f_t = 2400$ N/mm$^2$) showed a diverging picture regarding any possible beneficial effects of steel fibers increasing the tensile strength and reducing the risk of spalling at high temperatures.

The M1 concrete containing steel fibers had a slightly higher resistance with increased heating rates, even though its tensile strength was lower compared to that of the mix without steel fibers. The M2 concrete exhibited an opposite trend. The M2 concrete specimens with steel fibers had a slightly higher tensile strength compared to the fiber-free sub-mix. However, the use of steel fibers could not increase the spalling resistance. Spalling was not observed for the M3 concrete, independent of the use of steel fibers.

Figure 18 shows the typical stress-deformation curves for the concrete mixes M1 used, with and without steel fibers, for direct tension [75]. It can be seen that the presence of steel fibers does not in general increase the strength of the concrete. For specimens containing steel fibers a sudden drop to one third of the maximum strength is observed due to the fracture of the cement matrix after reaching the peak strength. The presence of steel fibers then leads to a post fracture behavior since the deformation can be increased until total failure.

Figure 18: M1 concrete in tension, stress-deformation curve

With respect to the fracture pattern of the spalled concrete pieces it is difficult to ascertain any significant difference in spalling due to the presence of steel fibers. For all tested concrete mixes containing steel fibers where explosive spalling was observed, the influence of the heating rate seems to be decisive regarding the size of the pieces (small or large concrete fragments) as described in chapter 3.3.3.

Figure 19 shows the fracture pattern of an M1 concrete with steel fibers after spalling (M1 - Test 4) compared to tensile tests on an M1 concrete sample. The visual analysis of the spalled concrete fragments showed that
the steel fibers were completely pulled out the cement matrix and the steel fibers did not fail by exceeding their tensile strength. This indicates that the probable benefits of increasing the resistance to explosive spalling by the fiber-bridging of cracks as described by van Mier [129] is negligible.

By comparing the two fracture patterns as shown in Figure 19, it may be seen that the length of the pulled out steel fibers upon spalling exceeds the visible fiber length observed in tensile strength tests. This indicated that the sudden release of energy during spalling at high temperatures causes a bigger slip of the steel fibers in the concrete than observed for tensile testing at a constant deformation rate of 0.005 mm/s. In addition, effects like “tension softening” [129] of the concrete due to the high concrete temperatures, causing changes in the mechanical and bond parameters of both the steel fibers and the cement matrix, must also be taken into account.

Figure 19: Fracture pattern of M1 concrete mix with steel fibers
left: After explosive spalling (Test 4 - heating rate \( \dot{T} = 1.0 \text{ K/min} \))
right: After tensile testing

Mechtcherine [93] could increase the tensile strength of concrete using a similar amount steel fibers of 2.4% in volume. He used long and oriented fibers and could increase the tensile strength by a factor of 5.5 compared to the concrete mix without steel fibers, which indicated a general maximum in practical tests. In these tests, the fracture energy up to a complete failure of the test specimen could be increased.

Figure 20 shows typical stress-deformation curves for concrete in tension with different amounts of steel fibers. It is interesting to note that the maximum strength for failure of the pure cement matrix and hence the fracture energy, does not depend on the presence or amount of steel fibers.
Concluding remarks on the use of steel fibers influencing the spalling behavior of concrete

- Based on the tests at IBK, it can be concluded that the presence of steel fibers added to the fresh concrete does not increase the resistance of explosive spalling of unloaded UHPC cylinders. These observations agree with the findings from Schneider [116].

- It is still unclear if the use of long and orientated steel fibers would increase the spalling resistance due to the higher tensile strength and better performance in terms of fiber bridging, or if the “purely” tensile strength of the cement matrix is essential. However, it remains unclear if a higher tensile strength has any significant effect on the spalling resistance in real fires.

- Considering highly loaded concrete columns, the presence of steel fibers might be beneficial in terms of spalling resistance. Combined with the modified reinforcement layout with less spacing between the ties as suggested by Kodur [80, 81], steel fibers increase the resistance to spalling significantly. Kodur observed that the form of fracture of the concrete between the reinforcement ties after spalling is similar to truncated cones as observed in regular compression tests on concrete specimens.

- It can be concluded that steel fibers as part of a UHPC mix are essential to achieve a sufficient ductility of the concrete for a safe design. In terms of explosive spalling the presence of steel fibers only has minor beneficial effects. However, some exceptions like highly loaded columns exist.
3.3.5 Influence of PP fibers

Tests on the P1 UHPC specimens with different amounts and types of PP fibers clearly indicated a lower risk of explosive spalling with the use of PP fibers. A brief summary on the main mechanical properties of the tested concrete mixes and the used PP fibers is given in Table 14. The entire testing program is described in detail in the technical report [75] and can be summarized as followed:

- Use of P1 concrete mix, SCC UHPC mix
- Compressive strength between $f_c = 132 - 159$ MPa
- Tests on unloaded concrete cylinders ($\phi = 150$ mm, $l = 300$ mm)
- Linear heating to $T_{\text{max}} = 500$°C or until spalling was observed
  - P1-0 mix without PP fibers: $\dot{T} = 1.5 - 3.0$ K/min
  - P1 mixes containing PP fibers: $\dot{T} = 10$ K/min
- Use of different amounts and types of PP fibers:
  - Typ A PP fibers: $l = 20$ mm; $\phi = 32$ $\mu$m
  - Typ A* PP fibers: $l = 6$ mm; $\phi = 32$ $\mu$m
  - Typ B PP fibers: $l = 6$ mm; $\phi = 15.4$ $\mu$m
  - Typ C PP fibers: $l = 6$ mm; $\phi = 18$ $\mu$m

Table 14: Major mechanical properties of P1 concrete mix
Type, amount and geometry of PP fibers used

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>0</th>
<th>2A</th>
<th>3A</th>
<th>3B</th>
<th>2C</th>
<th>3C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of PP fibers</td>
<td>-</td>
<td>A</td>
<td>A</td>
<td>A*</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td>PP fibers in kg/m$^3$</td>
<td>0</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>2.0 Vol.-%</td>
<td>2.0 Vol.-%</td>
<td>2.0 Vol.-%</td>
<td>2.0 Vol.-%</td>
<td>2.0 Vol.-%</td>
<td>2.0 Vol.-%</td>
</tr>
<tr>
<td>Silica content</td>
<td>yes, but unknown</td>
<td>yes, but unknown</td>
<td>yes, but unknown</td>
<td>yes, but unknown</td>
<td>yes, but unknown</td>
<td>yes, but unknown</td>
</tr>
<tr>
<td>Compressive strength $f_c$ in MPa</td>
<td>154.1</td>
<td>132.0</td>
<td>135.1</td>
<td>133.5</td>
<td>159.8</td>
<td>146.2</td>
</tr>
<tr>
<td>Tensile strength $f_t$ in MPa</td>
<td>4.4</td>
<td>7.7</td>
<td>4.8</td>
<td>3.8</td>
<td>2.5</td>
<td>n.a.</td>
</tr>
<tr>
<td>Density $\rho$ in kg/m$^3$</td>
<td>2450</td>
<td>2430</td>
<td>2403</td>
<td>2327</td>
<td>2355</td>
<td>2353</td>
</tr>
</tbody>
</table>

Testing procedure and test results

Eight tests were carried out in total. Table 15 shows the fracture pattern of the tested concrete cylinders after rapid heating, including a brief summary of the PP fiber content.
Explosive spalling and indicators

Table 15: P1 UHPC cylinders with different amounts of PP fibers - fracture pattern after heating

<table>
<thead>
<tr>
<th>Test 1 – P1-0</th>
<th>Test 2 – P1-0</th>
<th>Test 3 – P1-2A</th>
<th>Test 4 – P1-3A</th>
</tr>
</thead>
<tbody>
<tr>
<td>no PP fibers</td>
<td>no PP fibers</td>
<td>2 kg/m³ - Type A</td>
<td>3 kg/m³ - Type A</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test 5 – P1-4A</th>
<th>Test 6 – P1-3B</th>
<th>Test 7 – P1-3B</th>
<th>Test 8 – P1-3C</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 kg/m³ - Type A*</td>
<td>3 kg/m³ - Type B</td>
<td>3 kg/m³ - Type B</td>
<td>3 kg/m³ - Type C</td>
</tr>
</tbody>
</table>

Test 1 + 2 - concrete mix without PP fibers

Violent explosive spalling was noticed on concrete cylinders without PP fibers (tests 1+2) at surface temperatures between \( T = 260 - 320^\circ\mathrm{C} \). The average temperature gradient within the concrete at the onset of spalling was below \( \Delta T < 1.5 \, \text{K/mm} \) for test 1 with a heating rate of \( \dot{T} = 3.0 \, \text{K/min} \) and about \( \Delta T < 0.6 \, \text{K/mm} \) for test 2 with a heating rate of \( \dot{T} = 1.5 \, \text{K/min} \). Thermal stresses can be neglected in most cases with temperature gradients of less than \( \Delta T = 0.6 \, \text{K/mm} \) [114].

The analysis of the spalled specimen as shown in Table 15 indicates an almost identical fracture pattern of test specimen 1 and 2. The total deterioration of these two specimens in addition to the low temperature gradients leads to the conclusion that explosive spalling due to high pore pressure is the main factor in terms of spalling at these tests.

Test 3 - 5 - concrete mix with type A and A* PP fibers (\( \phi = 32 \, \text{µm} \))

Concrete specimens protected with type “A” and “A*” PP fibers (tests 3-5) tend to moderate spalling at the surface, in the form of small “concrete flakes” released from the surface. Corners were not affected by spalling. Since no violent spalling was noticed, it can be assumed that the spalling is related to the choice of sub-optimal PP fibers. The used type “A” and “A*” fibers have a diameter of \( \phi = 32 \, \text{µm} \), which is almost double the diameter compared to the other two types of PP fibers.
Tests on the explosive spalling of concrete at high temperatures

Using thicker PP fibers leads to a decrease in the total number of fibers per concrete unit, which then results in fewer but wider cracking of the concrete during heating as noticed by Kalifa [63]. In addition, a sufficient interconnectivity of the fibers might not be achieved which minimizes the important development of connected microcracks for pressure release [67]. At high temperatures, pore pressure will now migrate partly outwards via the microchannels and it might also cause additional cracking in areas of high pressure, leading to very local spalling as “concrete flakes”.

Test 6 + 7 - concrete mix with type B PP fibers (ø = 15.4 µm)

Spalling was observed with P1-3B concrete mixes, even though they were strengthened with 3 kg/m³ PP fibers (tests 6+7). The fibers used were the thinnest in all tests with a diameter of ø = 15.4 µm and a length of l = 6.0 mm; hence an overall good spalling protection can be expected.

Two tests were carried out and both tests showed local spalling at the surface of the concrete. The most probable explanation for this spalling is an insufficient distribution of the PP fibers in the concrete. During concreting a planetary mixer with high speed was used. However, the distribution of the fiber bundles during mixing is difficult, in particular with the high viscosity of the UHPC mix. In addition, it should be noted that both specimens tested came from the same concrete batch.

It was noticed that only local concrete fragments were released from the concrete during spalling, while the other parts remained undamaged. Cracking after cooling was not observed on the concrete surface. This reinforces the assumption high pore pressure, which could not be released in some areas of the concrete lead to local spalling.

Test 8 - concrete mix with type C PP fibers (ø = 18 µm)

Type “C” PP fiber seems to be the best match in terms of optimal fiber length and diameter, since no spalling or local failure was observed during the test (test 8).

During these tests, a maximum heat gradient in the range of ΔT < 4.5 K/mm was measured, which must be considered as noteworthy thermal stresses. However, no spalling was observed and the high pore pressure was released via microchannels and microcracks from the molten PP fibers. This confirms that a high pore pressure is the main factor regarding spalling in these tests.

The overall good performance of type “C” PP fibers (l = 6 mm; ø = 18 µm) was confirmed by tests on a concrete slab [75] exposed to the ISO fire curve [5]. A concrete slab (l·w·h = 1100·900·150 mm³) made of the P1 concrete mix was tested. The slab was divided into two segments, one segment was protected by adding 2 kg/m³ of the type “C” PP fibers to the fresh concrete, the other segment contained 3 kg/m³ of fibers. No spalling and only minor cracking at the concrete surface was noticed after 2 h of fire exposure [75]. This confirms that both an overall good distribution of fibers during mixing and a fiber geometry enabling the release of high pressure are achieved.
Explosive spalling and indicators

Concluding remarks on the use of PP fibers minimizing the risk of the explosive spalling of concrete

- UHPC mixes without PP fibers have a high risk of explosive spalling at high temperatures and cannot be used for structural concrete members.

- The use of PP fibers added to the concrete minimizes the risk of explosive spalling and reduces the amount of concrete spalled from the specimen (flaking).

- The fiber geometry is superior in view of the actual numbers of fibers added to the concrete. Thinn fibers lead to a higher number of fibers per concrete unit. An higher amount of fibers in kg/m$^3$ added to the concrete does not necessarily increase the spalling resistance.

- Short and thin fibers limit the workability of the fresh concrete and could lead to an inhomogeneous distribution of the fibers. This might cause local spalling at high temperatures.

3.3.6 Influence of permeability

A sufficient permeability of concrete must be provided to ensure the relief of high vapor pressure and to avoid an increased risk of explosive spalling. The permeability is usually very low for HPC and UHPC mixes due to their dense structure and has to be increased at high temperature, usually by the use of PP fibers. As described in chapter 2.5.3, the fibers melt at high temperatures and provide adequate permeability. Schneider [115] and Zeiml [132] analyzed the permeability of concrete at high temperatures and some of Zeiml’s tests covered the use of PP fibers. However, only OPC without any silica fume was tested.

In order to provide additional data on temperature-dependent changes of permeability, tests on the residual permeability of concrete after cooling from high temperatures were carried out at IBK [75]. The same three concrete mixes M1 - M3 as used in the tests on explosive spalling were tested, including sub-mixes to study the influence of steel and PP fibers.

Khoury [67] mentioned that tests on residual properties might lead to doubtful results, in particular if PP fibers are included, i.e. due to the possible re-crystallization of polypropylene. However, tests on the hot permeability of concrete are challenging and have never been carried out with UHPC for temperatures exceeding $T = 250^\circ$C. It is known that the residual permeability based on gas or air viscosity is different from the hot permeability based on vapor viscosity inside concrete, but the tendency of results can lead to a better understanding of explosive spalling due to high pore pressure.

Tests by Bošnjak [22] on the hot permeability of concrete to a maximum temperature of $T = 300^\circ$C included tests on the residual permeability of concrete after cooling to ambient temperature. A slightly higher residual permeability was noticed in these tests.

Test on residual permeability - overview of tested concrete mixes and test set-up

The residual permeability of concrete at $T = 20^\circ$C was tested after cooling from different high temperatures of up to $T = 500^\circ$C. In order to minimize possible negative influences of residual measurements, a sufficient
storage time of t = 4 h at high temperature was ensured to provide sufficient time for vaporization and percolation of the molten PP fibers and minimize a possible re-crystallization of the polypropylene.

As described in the technical report [75], the permeability is measured with a handheld device developed by Torrent and further analyzed in regarding suitability for general application by Jacobs in 2006 [59, 126]. A handheld device with two vacuum chambers is placed on the test specimens. Concrete disks (l = 40 mm, ø = 150 mm) were chosen as test specimen. The permeability is determined by the pressure difference between the two chambers and the flow towards them.

For concrete, the three concrete mixes M1 - M3 were tested, including four sub-mixes (V1 - V4) as summarized in Table 16 with the main mechanical parameters given in Table 17. Some sub-mixes had to be modified in order to achieve an overall good workability. In particular the M3 - V3 and M3 - V4 mixes sometimes exhibited severe segregation and bleeding and had to be modified. All modifications are summarized in the technical report [75].

Table 16: Tests on residual permeability - Tested concrete mixes including sub-mixes

<table>
<thead>
<tr>
<th>Concrete mixes</th>
<th>M1, M2, M3</th>
<th>Sub-mixes</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>no fibers</td>
<td></td>
</tr>
<tr>
<td>V2</td>
<td>2.0% in vol. of steel fibers</td>
<td></td>
</tr>
<tr>
<td>V3</td>
<td>2.0% in vol. of steel fibers + 2.0 kg/m³ type “B” PP fibers (l = 6 mm; ø = 15.4 µm)</td>
<td></td>
</tr>
<tr>
<td>V4</td>
<td>2.0% in vol. of steel fibers + 2.0 kg/m³ type “C” PP fibers (l = 6 mm; ø = 18 µm)</td>
<td></td>
</tr>
<tr>
<td>Temperature levels</td>
<td>20°C, 105°C, 150°C, 175°C, 200°C, 250°C, 300°C, 400°C, 500°C</td>
<td></td>
</tr>
</tbody>
</table>
1) Modifications to the mix design as shown in the technical report [75]

Table 17: Tests on residual permeability - Main mechanical properties

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>Young’s modulus</td>
<td>51'959 N/mm²</td>
<td>48'243 N/mm²</td>
<td>48'037 N/mm²</td>
</tr>
<tr>
<td></td>
<td>fₜ</td>
<td>108.2 MPa</td>
<td>148.7 MPa</td>
<td>147.6 MPa</td>
</tr>
<tr>
<td></td>
<td>initial moisture content</td>
<td>1.22%</td>
<td>1.03%</td>
<td>1.06%</td>
</tr>
<tr>
<td></td>
<td>initial density</td>
<td>2258 kg/m³</td>
<td>2401 kg/m³</td>
<td>2393 kg/m³</td>
</tr>
<tr>
<td>M2</td>
<td>Young’s modulus</td>
<td>43'749 N/mm²</td>
<td>41'478 N/mm²</td>
<td>47'214 N/mm²</td>
</tr>
<tr>
<td></td>
<td>fₜ</td>
<td>91.1 MPa</td>
<td>120.0 MPa</td>
<td>115.2 MPa</td>
</tr>
<tr>
<td></td>
<td>initial moisture content</td>
<td>3.17%</td>
<td>2.31%</td>
<td>2.49%</td>
</tr>
<tr>
<td></td>
<td>initial density</td>
<td>2274 kg/m³</td>
<td>2447 kg/m³</td>
<td>2442 kg/m³</td>
</tr>
<tr>
<td>M3</td>
<td>Young’s modulus</td>
<td>41'783 N/mm²</td>
<td>52'601 N/mm²</td>
<td>45'252 N/mm²</td>
</tr>
<tr>
<td></td>
<td>fₜ</td>
<td>65.7 MPa</td>
<td>93.3 MPa</td>
<td>82.0 MPa</td>
</tr>
<tr>
<td></td>
<td>initial moisture content</td>
<td>2.95%</td>
<td>2.32%</td>
<td>2.45%</td>
</tr>
<tr>
<td></td>
<td>initial density</td>
<td>2343 kg/m³</td>
<td>2549 kg/m³</td>
<td>2491 kg/m³</td>
</tr>
</tbody>
</table>

Analyzing permeability according to Jacobs and Torrent

As described in the technical report [75], to remove all free and physical bound water before testing the concrete samples were dried at T = 105°C until reaching a constant mass. This procedure is essential in terms of reliable results on the residual permeability. Water as liquid in the concrete pore system will block the pores and minimize the permeability at an ambient temperature of T = 20°C. After drying the concrete at
temperatures of $T = 105^\circ\text{C}$, this influence is negligible. Jacobs [59] also analyzed the influence of moisture on the accuracy of the measuring device and clearly indicated a noticeable influence on the test results. It can be concluded from his tests that the drying of the concrete specimens until attaining constant mass is reasonable.

Apart from a negative effect in terms of moisture inside the concrete when testing the residual permeability of concrete as mentioned above, tests on the permeability by outgassing the concrete with a vacuum cell placed on the concrete surface is different compared to tests in which a high pressure inside the concrete is produced [67]. A high pressure produced inside the concrete will cause microcracking and widening of the pores. This will lead to an increased interconnectivity of the pores and hence a higher permeability of the concrete. By contrast, outgassing the concrete under low pressure will close cracks and voids and lead to a decrease in permeability. This effect cannot be compensated with the test set-up used in these tests.

The handheld device used for the tests on permeability as shown in the test report [75] provides an easy tool to estimate the development of the concrete’s permeability. The measuring cell is simply placed on the concrete surface and the test starts by outgassing this cell. As output, the concrete gas permeability $k_a$ in $\text{m}^2$ and the penetration depth $l$ in $\text{m}$ are given.

Torrent developed this handheld device to allow a simple, on-site estimation of the permeability of concrete. His findings were presented in 1995 [126]. The permeability based on the gas viscosity of concrete can be calculated according to the pressure difference as given in equation (3 - 1):

$$k_a = \left(\frac{V_c}{A}\right)^2 \cdot \frac{\eta_a}{2 \cdot \varepsilon \cdot p_a} \cdot \left(\frac{\ln\left(p_a + p_t \cdot p_a - p_0\right)}{p_a - p_0 \cdot p_a - p_0}\right) \cdot \left(\ln l - \ln l_0\right)$$  \[3 - 1\]

with:
- $k_a$: permeability (based on gas viscosity) in $\text{m}^2$
- $V_c$: volume of measuring cell in $\text{m}^3$
- $A$: cross section of measuring cell in $\text{m}^2$
- $\eta_a$: dynamic viscosity of air taken as $17.1 \cdot 10^{-6} \ (\text{N} \cdot \text{s})/\text{m}^2$ at $20^\circ\text{C}$
- $\varepsilon$: porosity of concrete, taken as 0.15
- $p_a$: ambient air pressure taken as 96000 $\text{N}/\text{m}^2$
- $p_0$: pressure in measuring cell at beginning of test after outgassing in $\text{N}/\text{m}^2$
- $p_t$: pressure in measuring cell at time $t$ in $\text{N}/\text{m}^2$
- $l_0$: time at beginning of testing in $\text{s}$
- $t$: time at end of testing in $\text{s}$

**Permeability based on “gas” and “vapor” viscosity and compensation of possible influences**

The permeability as measured with the device depends on the dynamic viscosity of air at $T = 20^\circ\text{C}$ as shown in equation (3 - 1). However, it is obvious that mainly vapor will migrate inside the concrete depending on the permeability. In terms of permeability based on “gas” and “vapor” viscosity, the difference in the dynamic viscosity must be considered which changes for both gases with increasing temperature. Any differences in
Tests on the explosive spalling of concrete at high temperatures

dynamic viscosity become negligible when exceeding a temperature of $T = 374^\circ$C. At this temperature, vapor and air can be considered to perform as an ideal gas with the same properties.

This difference in dynamic viscosity between ambient air and vapor leads to an average error in permeability of about 50%. This sounds significant at first sight but it should be mentioned that the differences between the dynamic viscosity of air and vapor become smaller with increasing temperatures.

The movement of water as a liquid can be ignored, in general, since the dynamic viscosity of water as liquid is significantly lower than that of vapor. In addition, it can be assumed that only a minor amount of water remains inside the concrete from decomposition of the cement paste, since the concrete was pre-dried to a constant mass before testing.

According to equation (3 - 1), the middle term containing parameters on the dynamic viscosity of air ($\eta_a$), the porosity of concrete, which is considered to be $\varepsilon = 15\%$, and ambient air pressure ($p_a = 960$ hPa at an altitude of $h = 460$ m) can be modified according to a specific concrete mix at different temperatures. In equation (3 - 1), the term is constant independent of the concrete mix or temperature level. This equation has to be modified as shown in (3 - 2) to include this temperature-dependent change in the viscosity.

\[
\frac{\eta_a}{2 \cdot \varepsilon \cdot p_a} = \frac{171 \cdot 10^{-6}}{2 \cdot 0.15 \cdot 96000} = 5.9375 \cdot 10^{-10} \text{ [s]} \quad \text{constant term for viscosity of air} \quad (3 \cdot 2)
\]

In order to include the temperature-dependent changes in the viscosity of vapor and the porosity of the concrete the equation is modified. The ambient pressure remains constant, since all tests were carried out at a constant initial air pressure. In terms of viscosity of vapor, data is taken from the literature [113, 130]. Figure 21 left shows the changes of dynamic gas and vapor viscosity with increasing temperature to a temperature of $T = 374^\circ$C. For higher temperatures, vapor is considered as an ideal gas and its specific dynamic viscosity is shown.

In terms of the porosity of concrete, specific parameters were taken from tests as discussed in detail in chapter 3.3.7. The possible influence of spalling on the development of porosity is not taken into account in the analysis of spalling. A linear increase of the porosity during heating is assumed for all concrete mixes within the temperature range of $T = 100 - 500^\circ$C. The development of porosity in concrete is discussed in detail in chapter 3.3.7.

In order to convert the measured residual permeability of concrete at ambient temperatures a correction factor “$b_{v(\eta)}$” is required. Equation (3 - 3) provides an additional conversion factor, which can be used to determine the final temperature-dependent permeability based on vapor viscosity “$k_{v(\eta)}$” as given in equation (3 - 4). Figure 21 right shows this conversion factor “$b_{v(\eta)}$” for different temperatures up to the maximum temperature of $T = 374^\circ$C.

\[
b_{v(\eta)} = \frac{\eta_{v(\eta)} \cdot \varepsilon}{\eta_a \cdot \varepsilon_{a(\eta)}} \quad [-] \quad (3 \cdot 3)
\]

\[
k_{v(\eta)} = k_a \cdot b_{v} \quad [m^2] \quad (3 \cdot 4)
\]
The conversion factor can be estimated for all three concrete mixes M1 - M3, including the use of steel fibers (sub-mixes V1 - V2). However, due to a lack of data on the pore volume, the -V2 factor is also used for the two sub-mixes containing PP fibers (sub-mixes V3 - V4). It is interesting to note that all factors do not differ significantly as shown in Figure 21 right. The conversion factor M3-V2 is slightly higher at high temperatures due to the lower porosity of the concrete of only $\epsilon_{(500°C)} = 11\%$ compared to the other concrete mixes, which are usually in the range of $\epsilon_{(500°C)} = 14 - 16\%$. The compensation factor of vapor “$b_v(T)$” is taken as constant, i.e. $b_v = 1.0$, for temperatures exceeding $T > 374°C$, since vapor is assumed to be an ideal gas.

It should be noted that this conversion does not cause a significant increase in permeability compared to the overall temperature-dependent changes. Neglecting this factor “$b_v(T)$” would lead to a slightly lower permeability. However, the results would be still within the usual spread in results.

Permeability for concrete mixes M1, M2 and M3

The converted permeability based on vapor viscosity for all concrete mixes is shown in Figure 22. The test results clearly indicate significant changes in the permeability of the three concrete mixes with rising temperatures. Apart from the influence of the PP fibers on the initial permeability, the general reactivity in terms of changes in the increasing the permeability with higher temperatures varies according to the tested concrete mixes. In addition, the local minimum in permeability for the M1 concrete mix has to be analyzed further.
Test results - initial permeability

The initial permeability at T = 20°C is not influenced by the presence of PP fibers for the M1 and M2 concrete mixes, even though this might be considered to be the case due to the hydrophobic behavior of polypropylene in concrete. The test exhibited the same initial permeability within the range of \( k_v(T) = 1.3 \times 10^{-18} \text{ m}^2 \) for M1 concrete mix and \( k_v(T) = 4.5 \times 10^{-19} \text{ m}^2 \) for the M2 concrete mix. Even though a lower initial permeability with the M1 concrete compared to the M2 mix is expected, the differences in results between M1 and M2 are still within the usual spread of results and assumed to occur with the used Torrent method. Similar results in terms of presence of PP fibers are mentioned in literature [22, 132]. Zeiml observed a slightly higher initial permeability for those concrete mixes containing PP fibers which are considered to be “practically the same” [132]. Bošnjak [22] noticed no difference in initial permeability between HPC mixes with and without PP fibers. The average initial permeability was \( k_0 = 2.6 \times 10^{-18} \text{ m}^2 \) in her tests.

The M3 concrete mix is different in terms of initial permeability at T = 20°C. The M3-V2 concrete mix containing only steel fibers shows the lowest initial permeability at T = 20°C with \( k_v(T) = 8.9 \times 10^{-19} \text{ m}^2 \) compared to the other M3 concrete sub-mixes. This initial permeability is in the same range compared to the M1 and M2 concrete mixes. Apart from adding an additional superplasticizer, no modifications were made to the M3-V2 concrete mix and an overall good workability was achieved during concreting. This is in clear contrast to the M3-V4 mix, where major modifications to the mix design were made. The limited workability and segregation during compaction is clearly visibly resulting in an increased initial permeability of \( k_v(T) = 4.8 \times 10^{-17} \text{ m}^2 \), as well as in overall poor mechanical properties as shown in Table 17. The M3-V4 concrete mixes show both a lower compressive strength and Young’s modulus compared to the other mixes.

Test results - permeability of concrete at high temperatures

A general rapid increase in permeability for the concrete mixes containing PP fibers (V3, V4) was clearly shown by the test results, in particular if compared to the two sub-mixes without PP fibers (V1, V2). It is noticeable that the increase in permeability is almost identical for both types of PP fibers (V3, V4). A slight plateau phase during the increase of permeability is visible for the M2 and M3 concrete mixes within the temperature range of T = 150°C - 200°C, clearly observed with the type “B” PP fibers (V3). This delayed
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increase in permeability might be caused by a recrystallization of the fibers, since the polypropylene starts melting in this temperature range.

The M1 concrete mix shows no changes in permeability up to a temperature of $T = 175^\circ C$, independent of the sub-mix. In contrast, this “reaction temperature” for which a significant increase in permeability is observed at lower temperatures than with the other concrete mixes. For the M2 and M3 concrete mixes changes in permeability were observed directly after the concrete exceeded temperatures of $T = 105^\circ C$.

For the M3 concrete mix, the poor workability of the concrete containing PP fibers as described in the technical report [75] might be the governing factor influencing the rapid increase in permeability starting with temperatures of 150°C. In addition, high temperature shrinkage of the cement paste and microcracking also promotes the increase in permeability. This can also be seen with the M2 concrete mix. Even though the M1 concrete mix has the highest amount of cement and a high thermal shrinkage leading to an increased permeability can be expected, the pozzolanic reaction with the high silica fume content seems to over-compensate these increases [84, 121].

Decrease in permeability with the M1 concrete mix

Both M1 concrete sub-mixes without any fibers (M1-V1) and with steel fibers (M1-V2) showed a significant decrease in permeability between temperatures of $T = 175^\circ C$ and $T = 275^\circ C$. Within this area an even lower permeability compared to the initial permeability at $T = 20^\circ C$ was noticed.

Schneider [115] analyzed the hot permeability and observed a decrease in permeability for temperatures between $T = 110^\circ C$ to $T = 250^\circ C$ caused by the evaporation of water leading to a congestion of the pore system. A local minimum within this temperature range depends on the strength grade of the concrete and is usually observed at higher temperatures if a concrete with a higher compressive strength is tested. The influence of water saturation of concrete and permeability was analyzed by Jacobs [58] in 1994. Jacobs noted that the permeability decreases in the magnitude of 100 if the concrete gets totally water saturated concrete compare to the dry specimen.

The test specimens in own tests on the residual permeability were dried to constant in mass at $T = 105^\circ C$ prior testing permeability. However, it should be mentioned that the heating of the concrete to higher temperatures leads to a significant additional release of moisture from the cement paste. Figure 24 (see page 83) shows the measured additional losses in mass during heating even after drying the concrete to constant in mass at $T = 105^\circ C$. It is interesting to notice, that the loss in weight of the -V1 and -V2 mixes up to a temperature of $T = 250^\circ C$ is less pronounced compared to that of the -V3 mix containing PP mixes up to a temperature of $T = 250^\circ C$. At a temperature of $T = 200^\circ C$, the additional losses in weight for the -V2 mix are about $\Delta m = 2\%$ in mass, while the -V3 mix showed additional losses of $\Delta m = 4\%$ in mass. At higher temperatures, the difference between the three sub-mixes becomes less significant.

When testing the residual permeability of the -V1 and -V2 mix between temperatures of $T = 175^\circ C$ and $T = 275^\circ C$, a significant higher moisture content is still present inside the concrete. This might be one explanation for the decrease in permeability within this temperature range.
As probable additional explanation on the decrease in permeability, the high amount of silica fume added to the M1 concrete mix may be noted. At high temperatures, an increased pozzolanic reaction can be expected, which causes additional hardening of the concrete [84] and leads to an even denser and less permeable concrete. This effect seems to be even more pronounced for the fiber free M1-V1 concrete mix.

This consideration agrees with the observations from tests with critical heating rates leading to explosive spalling as described in chapter 3.3.3. These tests indicated that the M1 concrete mix without steel fibers (V1) tends to exhibit explosive spalling at a slightly lower heating rate compared to the mix with steel fibers (M1-V2).

In addition, these observations in terms of temperature-related hardening of the M1 concrete mix match other findings for this concrete. It was noticed that the M1-V2 concrete mix shows a decrease in total porosity and average pore diameter during heating as further discussed in chapter 3.3.7. In addition, tests on the hot and residual compressive strength of a similar concrete mix with a high amount of silica fume showed an increase of more than 15% in compressive strength at a temperature of $T = 300^\circ\text{C}$ compared to the initial strength. The residual strength greatly exceeded this gain in strength [75].

It remains unanswered whether the decrease in permeability for the M1-V1 and -V2 concrete mix would also be different if the permeability was tested at high temperatures (hot permeability). Possible temperature-related effects like moisture content, thermal expansion and cracking might have an influence on the test results. A possible re-crystallization of polypropylene is irrelevant, since these sub-mixes where this temporary decrease in permeability was observed contained no PP fibers.

For the M2-V2 concrete mix a single minimum at a temperature of $T = 150^\circ\text{C}$ was observed. This is less pronounced compared to the M1 concrete mix and is still within the expected spread in results during testing.

**Main concluding findings from tests on concrete permeability at high temperatures**

From tests on residual permeability, the following conclusions can be drawn:

- The handheld device seems to provide reasonable data in terms of the permeability of concrete.
- Differences between the permeability based on “gas” and “vapor” viscosity are small and can be adjusted.
- A constant low pressure is used with the Torrent method to analyze the permeability of concrete. Differences when testing the permeability with different high pressures levels might occur.
- PP fibers added to the fresh concrete do not necessarily increase the initial permeability. The observed differences are within a low, uncritical limit. However, a significant increase in initial permeability was noticed if the presence of PP fibers already limits the workability of the fresh concrete (i.e. M3-V4).
- PP fibers significantly increase the permeability at higher temperatures.
- A significant decrease in permeability between $T = 150 - 275^\circ\text{C}$ was observed with the M1 concrete mix. This is explained by a pozzolanic reaction with the high amount of silica fume at higher temperatures.
Permeability of concrete as a function of temperature - comparison with existing models

In order to provide a general model on changes of the permeability with increasing temperature, as a next step the test results are compared to several different models for the permeability of concrete.

Table 18 gives an overview of the available models for investigating the permeability of concrete at high temperatures. All models are explained in detail in appendix G.1 including the corresponding equations and factors.

<table>
<thead>
<tr>
<th>Model</th>
<th>Function</th>
<th>Comparison to test results on permeability at high temperatures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Davie [29]</td>
<td>Constant function including material coefficients depending on concrete mix.</td>
<td>Good match for initial permeability and high temperatures. Insufficient description of increase in permeability between $170^\circ C &lt; T &lt; 400^\circ C$</td>
</tr>
<tr>
<td>Bary (cited in [29, 43])</td>
<td>Constant function including damage factor of concrete.</td>
<td>Not applicable - permeability remains constant at all temperature levels.</td>
</tr>
<tr>
<td>Picandet [104]</td>
<td>Exponential function including constant material and damage factors$^1)$.</td>
<td></td>
</tr>
<tr>
<td>Souley [122]</td>
<td>Similar to model of Picandet. Exponential function including constant damage factor$^1)$.</td>
<td></td>
</tr>
<tr>
<td>Tenchev [125]</td>
<td>Function based on the temperature-dependent porosity of the concrete</td>
<td>Insufficient increase in permeability with higher temperatures, influence of PP fibers neglected.</td>
</tr>
<tr>
<td>Gawin [29, 42, 43]</td>
<td>Temperature-dependent function including several material properties$^1)$.</td>
<td>Overall good results. The decrease in permeability is not described by the model.</td>
</tr>
<tr>
<td>Gawin (modified) [29, 42, 43]</td>
<td>Including an additional damage factor compared to the first model by Gawin</td>
<td>No noticeable difference compared to first model.</td>
</tr>
</tbody>
</table>

1) Function containing coefficients without any physical meaning

Concluding remarks for modeling the permeability of concrete at high temperatures

In terms of analytical models for investigating the permeability of concrete at high temperatures, the following conclusions can be drawn:

- All analytical models for the temperature-dependent permeability of concrete are lacking in precision for HPC and UHPC mixes. The decrease in permeability of the M1 - V1 and -V2 concrete mixes, with significant influence on the spalling behavior of this concrete mix is not accounted for in any model.

- Several of the analytical models consider the deterioration of the concrete with a factor “D”. However, this factor remains constant and does not change with temperature.
- Tenchev’s function [125] based on the porosity seems to be a very promising approach, since the permeability as interconnectivity of the pores increases with a temperature-dependent increase of porosity. However, it is shown that this model clearly underestimates the actual development of permeability.

- In terms of general application and feasibility of the models for the “more dense” HPC and UHPC mixes, the regular Gawin model [29, 42, 43] shows an overall good agreement with the test results, in particular for the M2 and M3 concrete mix. This model for permeability is also used in the hydrothermal model predicting the spalling of concrete as presented by Dwaikat [33].

- The modified Gawin model with the additional term for the deterioration of the concrete has only minor influence on the results and does not improve the general accuracy of the estimated permeability.

- Even though the Gawin model for permeability shows overall good agreement with the test results, the decrease in permeability is not taken into account. This makes the Gawin model inappropriate for hydrothermal models for the development of pore pressure, since the essential decrease in permeability is not predicted.

3.3.7 Influence of porosity

As with the measurements of residual permeability, the porosity of different concrete mixes was analyzed in small concrete pieces by mercury intrusion porosimetry. A detail description on these tests is given in the technical report [75]. The analysis covered the three concrete mixes M1, M2 and M3, including the influence of steel fibers. The tests covered the analysis on the initial porosity at \( T = 20°C \) and after cooling from \( T = 500°C \) with low heating rates that did not caused explosive spalling of the specimen. Further tests were carried out on concrete specimens that failed due to explosive spalling after rapid heating. As with the tests on permeability, all specimens were analyzed in the residual stage after cooling from high temperatures, so that possible influences of cracking or additional deterioration during cooling had to be considered. PP fibers were not added to any concrete mixes.

Table 19 summarizes the test results. The M1 concrete mix without steel fibers was not analyzed. Apart from an analysis of the cumulative pore volume and pore size distribution, the average pore radius and total porosity were determined. Analysis is limited to the three concrete mixes without PP fibers, i.e. M1, M2 and M3 (\( V_1 + V_2 \) sub-mix).

Assessment of average number of pores based on the concept of concrete pores as radial spheres

Based on considerations taken from Sauter [111] in the mid-1920’s, pores inside dense materials may be assumed to be hollow spheres with an average diameter. Sauter considered the particle and pore size distribution, which are nowadays part of the particle size analysis in international standards [123].

Similar considerations were made by Akhtarruzaman, who also considered concrete pores as radial hollow spheres [10] (cited in [27]). He assumes that the pore pressure development in these pores depends on the
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saturation level. This agrees with the considerations of Ichikawa [53], who also noted that the temperature-dependent increase in pressure is caused by vaporization of water inside the pores.

For feasible models of explosive spalling including the increase in pore pressure based on thermodynamic laws, these considerations from the literature lead to the idea that the number of pores can be estimated based on the average pore radius as shown in equation (3 - 5).

By comparing the $T = 20^\circ\text{C}$ and $T = 500^\circ\text{C}$ temperature levels, any changes in the number of pores during heating were analyzed. Table 19 summarizes the main results.

\[ n = \frac{3V_c\varepsilon}{4\pi r^3} \]  (3 - 5)

With:
- $n$: number of pores per concrete volume
- $\varepsilon$: concrete porosity in %
- $V_c$: concrete volume in $\text{m}^3$
- $r$: average pore radius in $\text{m}$

<table>
<thead>
<tr>
<th>silica fume content$^1$</th>
<th>2.5% steel fibers</th>
<th>no fibers</th>
<th>2.5% steel fibers</th>
<th>no fibers</th>
<th>2.5% steel fibers</th>
<th>no fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>total spalling$^3$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20°C$^2$</td>
<td>9.63%</td>
<td>9.03%</td>
<td>10.18%</td>
<td>8.74%</td>
<td>8.11%</td>
<td></td>
</tr>
<tr>
<td>500°C</td>
<td>15.64%</td>
<td>14.98%</td>
<td>15.86%</td>
<td>13.57%</td>
<td>11.28%</td>
<td></td>
</tr>
<tr>
<td>avg. pore radius in nm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20°C$^2$</td>
<td>12.11</td>
<td>27.50</td>
<td>34.25</td>
<td>30.19</td>
<td>36.32</td>
<td></td>
</tr>
<tr>
<td>500°C</td>
<td>8.42</td>
<td>22.78</td>
<td>44.13</td>
<td>29.73</td>
<td>51.92</td>
<td></td>
</tr>
<tr>
<td>avg. number of pores per m$^3$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20°C$^2$</td>
<td>1.29·10$^{22}$</td>
<td>1.04·10$^{21}$</td>
<td>6.05·10$^{20}$</td>
<td>7.58·10$^{20}$</td>
<td>4.04·10$^{20}$</td>
<td></td>
</tr>
<tr>
<td>500°C</td>
<td>6.25·10$^{22}$</td>
<td>1.16·10$^{21}$</td>
<td>4.41·10$^{20}$</td>
<td>7.47·10$^{20}$</td>
<td>1.92·10$^{20}$</td>
<td></td>
</tr>
<tr>
<td>in-/ decrease</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20°C / spalling</td>
<td>+500%</td>
<td>+12%</td>
<td>-2%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>20°C / 500°C</td>
<td>+383%</td>
<td>+192%</td>
<td>+63%</td>
<td>-27%</td>
<td>-52%</td>
<td></td>
</tr>
</tbody>
</table>

1) As a percentage of the total mass of the concrete
2) Dried at $T = 105^\circ\text{C}$
3) Average temperature on spalling: M1 $T \approx 300^\circ\text{C}$; M2 (with fibers) $T \approx 310^\circ\text{C}$; M2 (no fibers) $T \approx 356^\circ\text{C}$
4) No spalling up to $T = 500^\circ\text{C}$ due to low heating rate
5) Determined using mercury intrusion porosimetry

Table 19 shows the results of the mercury intrusion porosimetry. It is noted that the concrete mixes containing steel fibers always show a slightly increased initial total porosity at $T = 20^\circ\text{C}$ due to the intake of additional air during concreting which cannot be released during compaction. The M1 concrete mix has by...
far the lowest workability, which makes high compaction even more challenging. The slightly higher initial porosity is caused most likely by the limited energy of the vibration table during compaction.

**Influence of silica fume on temperature changes in porosity and pore radius**

The difference in initial pore radius between the concrete mixes containing steel fibers and the fiber-free mix is not significant. Considering the porous boundary layer of cement paste and steel fibers [108], micropores are the most probable explanation for the smaller average pore radius.

In contrast to the difference in initial pore radius with regard to steel fibers, the amount of silica fume added to the concrete has a significant influence on the pore radius. The test results clearly showed a shift in initial average pore radius towards finer pores with an increasing amount of silica fume. With a radius of only $r = 12$ nm for the M1 concrete mix, the average pore radius is only about one third of the initial pore radius of the M3 concrete mix. The literature review of tests on concrete with different amounts of silica fume showed a noticeable pore refinement with higher silica fume content, as for tests on concrete with decreasing w/c ratio and higher hydration grades [108]. It is stated that the average pore size decreases on adding silica fume to the concrete, mainly caused by the pozzolanic reaction and cement hydration. Elevated temperature even emphasizes the reactivity of silica fume [35, 84] increasing the compressive strength of the concrete further [75]. In addition, this process leads to both a reduced permeability of cement paste and a smaller average pore radius, mainly within the transition zone between cement paste and aggregate [121].

Regarding a lower permeability, observations were made during the corresponding tests as described in chapter 3.3.6. The permeability tests showed that the pozzolanic reaction takes place between temperatures of $T = 175\,^\circ\text{C} - 275\,^\circ\text{C}$. This is also an reasonable explanation of the significant decrease in the average pore diameter observed at a spalling temperature ($T = 300\,^\circ\text{C}$) for the M1 concrete mix, which also causes the increase in total number of pores.

Concrete with a low w/c ratio and a high content of silica fume as used in these tests with the M1 concrete mix exhibits two different trends at high temperatures, apart from explosive spalling. In addition to the temperature-related reactions in concrete caused by silica fume, high temperatures cause damage to the concrete. Deterioration, decomposition and cracking of the concrete at high temperatures causes a significant increase of total porosity for all concrete mixes as observed at a temperature of $T = 500\,^\circ\text{C}$ when compared to the initial porosity at $T = 20\,^\circ\text{C}$. Microracking during cooling of the concrete mixes containing steel fibers prior testing increases the porosity further compared to the test specimen without steel fibers.

In terms of the M1 concrete mix, the pozzolanic reaction at the spalling temperature of $T = 300\,^\circ\text{C}$ is the main factor in terms of temperature-related changes of the concrete that lead to the significant decrease in both total porosity and average pore radius. At the higher temperature of $T = 500\,^\circ\text{C}$ an increase in total porosity is noticed, whereas the change in average pore diameter remains rather low.

The M2 concrete mix has a roughly 50% silica fume content compared to the M1 concrete mix. The test results show that the temperature-dependent deterioration of the concrete exceeds that of probable pozzolanic reactions, since a continuous increase in porosity is noticed. In addition, higher heating rates compared to those for the M1 concrete mix applied. The significant reduction in average pore diameter for
the M2 concrete mix at $T = 500^\circ C$ needs to be verified again with further tests, in particular the results for the concrete mix containing steel fibers.

**Temperature-dependent changes in total number of pores**

The average number of pores per m$^3$ concrete can be computed according to the test data as shown in Table 19 and equation (3 - 5). It is interesting to note that the average number of pores of the M1 concrete mix increases significantly for all temperature levels, mainly for concrete which failed by spalling. Due to the reduced average pore diameter upon spalling, the total number of pores increased by +500%.

This significant increase in number of pores is in clear contrast to that of the M2 concrete mix. By comparing the average number of pores at the initial state and after spalling temperature, almost no change in the number of pores is observed. At high temperatures of $T = 500^\circ C$, the number of pores increased. Even though an increase of +192% and +63% depending on the presence of steel fibers is computed, these increases are less pronounced compared to those for the M1 concrete mix. The higher total number of pores is caused by two influencing parameters, i.e. an increase in total porosity and a decrease in average pore diameter.

High heating rates as applied to M3 concrete samples lead to an increased porosity and pore widening due to thermal stresses and other temperature-related deterioration processes. It is worth noting that the total number of pores slightly decreases during heating.

**P1 concrete containing PP fibers - temperature related changes in porosity**

In addition to the tests on porosity of the M1 - M3 concrete specimens, the P1 UHPC mix with different amounts of PP fibers were also analyzed. The total number of tests on the P1 UHPC specimens is very limited and some results seem to be incoherent as mentioned in the IBK test report [75]. However, a general trend can be observed.

The analysis included tests at different temperature levels on the main P1 UHPC mix without any PP fibers (P1-0). In addition, the porosity of the P1-2A and P1-3A concrete mix containing 2 - and 3 kg/m$^3$ type “A” PP fibers ($\varnothing = 32 \, \mu m, l = 20 \, mm$) was analyzed. Table 20 summarizes the test program which is also presented in the test report [75]. It should be noted that the type A PP fibers could only minimize the occurrence of explosive spalling, since flaking of concrete fragments at the surface of the test specimens was observed.

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>P1-0</th>
<th>P1-2A</th>
<th>P1-3A</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP fiber content in kg/m$^3$</td>
<td>none</td>
<td>2 kg/m$^3$</td>
<td>3 kg/m$^3$</td>
</tr>
<tr>
<td>Type A PP fiber geometry</td>
<td>-</td>
<td>$l = 20 , mm$, $\varnothing = 32 , \mu m$</td>
<td></td>
</tr>
<tr>
<td>Testing temperature</td>
<td>120°C, 200°C, 275°C, 350°C</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 20: P1 UHPC mix with and without PP fibers Tests on pore volume and average pore radius after the heating cycle
Temperature-dependent changes in total porosity

Figure 23 summarized the results from this testing series. As mentioned in the test report [75], the sudden decrease in total porosity of the P1-0 concrete mix without PP fibers at a temperature of \( T = 350°C \) cannot be explained at the moment. It is assumed that the growth of several cracks and pores divided the concrete into several smaller fragments due to the high temperature. This might lead to a lower total porosity. However, additional tests have to be made first to confirm this supposition. Additional tests should also be made for the P1-3A concrete mix, mainly to verify the high initial porosity at \( T = 1210°C \) of \( \varepsilon = 14\% \).

Even though some results require further verification, some general conclusions on these tests can be drawn. It is evident that the total porosity for the concrete mixes containing PP fibers (concrete mixes P1-2A and P1-3A) does not increase significantly. Even the melting of the PP fibers at \( T = 170°C \) does not lead to a significant increase in porosity. A slight increase in total porosity is noted when the temperature rises from \( T = 275°C \) to \( T = 350°C \), mainly due to the temperature-dependent decomposition of the cement paste.

In contrast, the concrete mix without PP fibers (P1-0) shows a constant increase in porosity with increasing temperatures, since deterioration due to high internal pressures promotes the growth of cracks. As mentioned above, all values for the P1-0 concrete mix at \( T = 350°C \) have to be verified.

Temperature-dependent changes in pore radius

The average pore radius does not change significantly for the P1-2A and P1-3A concrete mixes containing PP fibers. The minor difference in average pore radius cannot be explained. It is interesting to note that the average pore radius did not increase after the melting of the PP fibers at a temperature of \( T = 170°C \), which would have been noticed between the tests at \( T = 120°C \) and \( T = 200°C \). According to the MRI scans by Pistol [106], a growth of microcracks close to the empty pore channels is observed, hence an increase in pore radius is expected. However, with an average pore radius of only \( r \approx 2.0 \text{ nm} \) at \( T = 200°C \) compared to the radius of the type A PP fibers with \( r = 17 \cdot 10^3 \text{ nm} \), this effect was not observed in this analysis.

Similar to the observations made with the M1 concrete mix, a minor decrease in average pore radius is observed. However, for the P1-0 concrete mix, this decrease is less pronounced. The value at a temperature of \( T = 350°C \) needs additional verification.

Comparing the results to the M1 concrete mix it may be stated that the average pore radius is significantly smaller compared to that for the M1 concrete mix. A large amount of reactive fillers like silica fume leading to a very dense structure are the most likely cause. In terms of average number of pores it is noted that the increase in pores of +700% of the fiber free concrete mix is even higher than or the M1 concrete. In contrast, a minor decrease was observed with both mixes containing PP fibers.
Explosive spalling and indicators

Figure 23: Porosity of P1 UHPC mix with 0, 2.0 and 3.0 kg/m$^3$ PP fibers (Type A)
left: Total porosity at high temperatures
middle: Average pore radius
right: Average number of pores

Possible assessment of changes in porosity in terms of risk of spalling

In the analysis of the porosity of concrete at high temperatures it is difficult to make an assessment of the total porosity and the probable increased risk of spalling. The tests showed that even significant increases in total porosity as observed with the M2 concrete specimens could not prevent the concrete from explosive spalling. In contrast, the total porosity of the P1-2A concrete mix remained nearly constant for temperatures up to $T = 350^\circ C$ without spalling. The risk of spalling was minimized by increasing the permeability of the concrete by adding PP fibers. This leads to the conclusion that the interconnectivity of the pores - the permeability - is more relevant in understanding explosive spalling than the total porosity or pore size distribution of the concrete. Khoury [67, 68] also mentioned this. However, a high porosity is regarded in the literature as beneficial in terms of decreasing the risk of spalling, since some authors assume high porosity to act like a “pressure reservoir”, which can buffer high or peak pressures without leading to explosive spalling.

However, these findings regarding the total number of pores per concrete unit are the most important for modeling pore pressure-induced explosive spalling, since moisture inside these pores are a critical factor for the build-up of critical pressures leading to spalling [27, 53].

Concluding remarks on the analysis of the porosity of heated concrete

Based on the discussion of the tests and considerations in terms of pores as hollow spheres, the following conclusions can be drawn:

- The initial porosity at $T = 20^\circ C$ of concrete does not differ significantly between the individual concrete mixes independent of the use of silica fume.

- The use of silica fume (M1) leads to a much smaller average pore radius compared to the mixes with a smaller amount of silica fume (M2) or without any (M3) at all temperature levels. This leads to a much higher number of pores for the concrete containing silica fume (M1).
- Higher temperatures of up to $T = 500^\circ C$ lead to additional deterioration of the concrete, which mainly results in a higher total porosity.

- The P1 concrete mix containing PP fibers showed a significantly smaller increase in total porosity up to a temperature of $T = 350^\circ C$ in contrast to the P1 concrete specimen without PP fibers (P1-0).

### 3.3.8 Influence of moisture content and losses in mass

A high moisture content combined with low permeability of the concrete is known to be a critical factor leading to a high pore pressure with rising temperatures [27, 49, 57, 68, 82, 94].

To analyze the moisture content of the different concrete mixes, the amount of free and physically-bound water was determined first. This initial water starts to evaporate at a temperature of $T = 105^\circ C$. The vapor is then being released according to the permeability of the concrete until the concrete dries out, which is noticed by reaching a constant mass. This drying process can take several days since the permeability of UHPC is very limited. Storage of the specimens at a higher temperature than $T > 105^\circ C$ leads to a more rapid drying but also causes decomposition of chemically-bound water as shown in Table 1.

As already discussed, both forms of water lead to an increase in pore pressure with rising temperatures, since the release of vapor is depends on the permeability of the concrete.

Two different tests were performed to analyze any moisture related losses in mass at high temperatures. The test set-up for both tests is described in detail in the test report [75].

#### Tests on initial moisture content and additional losses at high temperatures to $T = 500^\circ C$

The moisture content at different temperatures was analyzed by drying concrete disks as described in the test report [75], which can be summarized as follows:

- Testing the moisture content by drying the specimens to a constant mass at different temperature levels.
- Maximum temperature of $T = 500^\circ C$.
- Use of concrete disks ($l = 40 \text{ mm}, \varnothing = 150 \text{ mm}$) as test specimens.
- Analysis on M1 - M3 concrete including the sub-mixes V1 - V4 (see Table 16).
**Initial moisture content**

The initial moisture content after drying at $T = 105^\circ C$ is presented in Table 21.

Table 21: Initial moisture content of different concrete mixes as shown in Table 16

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\phi_0$ in % by mass</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M1</td>
<td>1.22%</td>
<td>1.03%</td>
<td>1.06%</td>
<td>0.90%</td>
<td>1.05%</td>
</tr>
<tr>
<td>M2</td>
<td>3.17%</td>
<td>2.31%</td>
<td>2.49%</td>
<td>2.72%</td>
<td>2.67%</td>
</tr>
<tr>
<td>M3</td>
<td>2.95%</td>
<td>2.32%</td>
<td>2.45%</td>
<td>2.59%</td>
<td>2.57%</td>
</tr>
</tbody>
</table>

It was noticed that the M1 concretemix has the lowest initial moisture content with an average value of only 1.05%. For the other two main concrete mixes, significant higher moisture content was determined. It was noticed further that for all three concrete mixes the initial moisture content for the specimens without any fibers (V1) is usually slightly higher than for the mixes containing steel fibers.

Apart from the M2 - V1 mix, all initial moisture contents are well below the “critical value” of 3.0% in mass. This value was postulated by Meyer-Ottens [94] and Zhukov [134] as the lower boundary. Spalling is most likely to occur if the initial moisture content exceeds this limit. The tests, as described in chapter 3.3.3, clearly showed that this limit is not valid for HPC and UHPC mixes, since M1 and M2 concrete specimens tend to exhibit violent spalling. In contrast, the M3 - V1 concrete mix did not fail by explosive spalling, even though the initial moisture content was almost 3.0% in mass.

This leads to the conclusion that it is impossible to make an assessment on the basis of an initial moisture content regarding the possible likelihood of explosive spalling due to high pore pressure. Even very low moisture contents might lead to explosive spalling. These considerations confirm that the release of high pore pressure due to the parameter concrete permeability is more important than the actual quantity of moisture that is present inside the concrete.

**Losses in weight at high temperatures**

Figure 24 shows the expected additional changes in mass due to losses in moisture during heating to a maximum temperature of $T = 500^\circ C$. New concrete disks were used for each individual level at high temperature, which leads to a non-monotonic development of the curve with local peaks. Some results should be confirmed by additional tests (i.e. M1 - $T = 400^\circ C$).
Tests on the explosive spalling of concrete at high temperatures

Even though some results are doubtful, the tests clearly indicated that mainly for the M2 and M3 concrete mixes no significant differences in losses in mass between the four sub-mixes can be noticed.

For the M1 concrete mix at T = 175 - 200°C, the losses in mass were higher for the two mixes containing PP fibers (-V3 and -V4) compared to those for the mixes without PP fibers (-V1 and -V2). Even though this difference is rather low, this is an indication that the concrete mixes without PP fibers did not completely dry to a constant mass and some moisture still remained inside the specimen. As mentioned in chapter 3.3.6, this remaining moisture inside the concrete causes a congestion of the pores and lowers the permeability. At temperatures exceeding T = 300°C this difference becomes small.

In terms of additional losses in mass at high temperatures is can be seen that the M1 concrete shows an overall high total loss in mass of up to additional 6% in mass at T = 500°C, compared to the dry specimen at T = 105°C. For the two other mixes these average additional losses are less pronounced with a maximum additional loss of ≈5.0% for the M2 concrete mix and about ≈2.5% for the M3 concrete mix.

As mentioned in chapter 2, HPC and UHPC mixes as used for these test have an overall low initial moisture content after drying the specimen at T = 105°C to constant mass. However, due to the high cement content of HPC and UHPC mixes, the total amount of water added to the mix is higher than for the OPC mixes, even with the usual low w/c ratios. During the decomposition of the concrete at higher temperatures (Table 1) this water is released from the cement paste and evaporates, leading higher losses in weight at high temperatures.
Models for losses in weight of concrete at high temperatures

To develop a general model for losses in weight during heating based on the initial density of the concrete at \( T = 20^\circ\text{C} \), we developed our own model. This was compared to existing models leading to a comparison of three models:
- Own model as power function on loss of mass at high temperatures.
- Regression on loss of mass provided by EN 1992-1-2 [6].
- Losses in weight as a linear factor according to Bažant’s design recommendation [18].

The models are explained in detail in appendix G.2. Figure 25 shows the average loss in weight for the three concrete mixes from the tests and the corresponding regression to a maximum temperature of \( T = 500^\circ\text{C} \). In addition, these results are compared to those obtained from the EN 1992-1-2 design standard [6] and the analysis proposed by Bažant [18]. It may be concluded that the regression analysis leads to an overall good agreement if compared to the test results, while the models of Bažant and EN 1992-1-2 give inferior results for the individual concrete mixes.

![Figure 25: Temperature-dependent losses in mass to \( T = 500^\circ\text{C} \) Regression analysis according to own analysis, Bažant [18] and EN 1992-1-2 [6] M1, M2 and M3 concrete mixes](image)

Losses in weight during heating with different heating rates

These tests on losses in weight at high temperatures as previously discussed were analyzed under steady-state conditions by drying the concrete to a constant mass. However, these steady-state conditions never occur with rising temperatures as they shown in tests on explosive spalling by linear heating as summarized in Table 13.

In order to quantify moisture evaporation during heating in general and during the plateau phase in particular, the loss in weight during heating was analyzed with some tests.

Losses in weight during heating were measured for some M2 and M3 concrete specimens heated with different heating rates. The test set-up is described in detail in the test report [75]. Figure 26 (left) shows the surface temperature of the tested specimens and the corresponding temperature-dependent losses in
weight for the M2 concrete with steel fibers heated with different rates. All specimens failed by explosive spalling, except for the specimen heated with $\dot{T} = 1.0 \, \text{K/min}$. The specimens that failed by explosive spalling are indicated with a dot in this figure.

In accordance with the results from Khoury [67], a higher heating rate causes fewer losses in weight during heating - simply because of the low permeability of the concrete. It was noticed that a greater amount of moisture is still present inside to concrete for the concrete specimens that failed by explosive spalling. In terms of the M2 concrete mix as shown in Figure 26 (left), the ultimate weight loss at different temperatures by drying concrete samples until a constant mass is reached is shown here as well. This comparison shows that even a low heating rate of $\dot{T} = 1.0 \, \text{K/min}$ could not ensure complete drying of the concrete during heating. However, the slope of the curve for this heating rate shows that the peak in losses in weight during heating is expected between a surface temperature of $T = 200 - 300 ^\circ\text{C}$. The plateau phase was also observed within this temperature range.

Results for the M3-V2 concrete mix with steel fibers are shown in the right part of Figure 26 (right), for which different observations can be made. It is worth noting that the measured loss in weight follows the ultimate weight loss and no significant changes in weight at a specific temperature can be seen. This is because of the significant higher permeability of the M3 concrete mix with increasing temperatures, as they are analyzed in chapter 3.3.6. This accelerates the drying process. It should be mentioned again that the M3 concrete mix does not include silica fume.

For some concrete specimen a loss in weight exceeding the ultimate weight loss is noticed. The difference is caused by the test set-up, which is affected by the high temperatures. In general, the test set up was very sensitive to external influences as seen with the data from the M3 concrete specimens. The wave-like curve results from movements of the test device.

Figure 26: Temperature-dependent loss in weight of concrete with steel fibers for different heating rates compared to ultimate weight loss up to constant mass

left: M2-V2 concrete with steel fibers
right: M3-V2 concrete with steel fibers
Concluding remarks on the analysis of moisture content and loss in weight during heating

The analysis of moisture content and loss in weight during heating leads to the following general conclusions:

- The M1 concrete mix with the lowest w/c ratio and the highest amount of silica fume has the lowest initial moisture content compared to the values for the M2 and M3 concrete mixes.

- The total losses in weight up to a temperature of $T = 500^\circ C$ are by far the highest for the M1 concrete mix due to the decomposition of the cement paste.

- Explanations of differences between the individual sub-mixes (steel and PP fibers) require additional tests.

- The existing models on losses in mass at high temperatures by Bažant [18] and EN 1992-1-2 [6] provide overall good results, but are poor for the individual concrete mixes tested here. The power function calibrated with the test results shows an overall good agreement (see appendix G.2).

- For concrete mixes of low permeability (M2), the losses in weight during heating do not follow the ultimate loss in weight curve. This was noticed even for very low heating rates. Concrete mixes with higher permeability (M3) follow the ultimate loss curve during heating.

- The tests on losses in weight lead to the conclusion that the highest losses in weight in the form of moisture vaporization are observed during the plateau phase. Higher temperatures cause only minor additional losses in weight. These observations confirm the assumption that high pore pressure will not be present after the plateau phase simply due to the lack of moisture inside the concrete.
3.4 Analysis of experimental results in terms of pore pressure

The tests on the linear heating of concrete cylinders as presented in Table 13 indicated critical heating rates that lead to explosive spalling of the specimen, depending on the tested concrete mix. In addition, a corresponding surface temperature at the onset of spalling was also measured in these tests. However, these surface temperatures at spalling varied significantly according to the selected heating rate.

In contrast to the surface temperature, the temperatures during the moisture-induced plateau phase did not change significantly for one concrete mix, whether the concrete specimen failed by explosive spalling or not.

The analysis of the tests showed that temperature in the core of the concrete specimens increases until the moisture-induced plateau phase develops. As mentioned above, this leads to the conclusion that the evaporation of moisture and pressure development inside the concrete reach a peak during this plateau phase and spalling might occur at this stage. However, a possible failure criterion of the concrete has still not been analyzed in detail. According to the literature, concrete will fail by explosive spalling if the pore pressure exceeds the tensile strength of the concrete [57, 68, 82].

In the following, a possible relation between vapor pressure at high temperatures and tensile strength is analyzed further according to basic thermodynamic calculations.

3.4.1 Saturation vapor pressure curve

Saturation vapor pressure

The properties of water and steam at high temperatures were determined and summarized by the “International Association for the Properties of Water and Steam” over the last few decades and are now part of the industrial standard IAPWS-IF97. Tabulated data [112, 130] on the temperature-dependent pressure development for water and vapor is given here.

According to the data, the vapor pressure inside a closed system follows the saturation vapor pressure curve as long as sufficient moisture for a saturated state is available. Data on the required moisture content in the form of water and vapor is given in the tabulated data as well. Additional information on the vapor content is given in chapter 4.2.4.

The saturation vapor pressure curve represents the relation between temperature and vapor pressure for a closed and saturated system. The pressure development only depends on the temperature for a saturated system.

To achieve a pressure according to the saturation vapor pressure curve with increasing temperature as shown in Figure 27, a closed system with a minimum amount of moisture is required. With increasing temperatures, the amount of water decreases (m‘) while the amount of vapor increases (m‘‘) if the system remains closed during heating. At a temperature of T = 374.15°C no difference in density between vapor and water as liquid can be made. The density of both components is identical at this temperature. The temperature of T = 374.15°C marks the critical point of water. For higher temperatures, no differences in viscosity and density
between vapor and water is observed. With a pressure exceeding $p = 2.21 \times 10^7$ Pa, this pressure range can be assumed to be absent in concrete at high temperature.

For an unsaturated state, the pressure inside a closed system will not follow the saturation vapor pressure curve, but will develop pressure according to the ideal gas law. This is explained in detail in chapter 4.2.6 including the governing thermodynamic laws.

This leads to two different scenarios for pressure build-up in closed systems as expected with concrete pores at high temperature:
- Pressure according to the saturation vapor pressure curve for the saturated state.
- Pressure according to the ideal gas law for unsaturated state.

Further explanations and equations are given in chapter 4.2.6.

![Figure 27: Saturation vapor pressure curve according to [130]](image)

Pressure development inside the pore system of heated concrete

It is stated in the literature and verified by tests that the increase in pressure occurs within the pore system of the concrete [10, 96]. However, the pore system of concrete cannot be assumed to be a totally closed system, since permeability influences the moisture content of the pores due to migration. However, the pressure development according to the saturation vapor pressure can be assumed as long as sufficient moisture is available, even with the inflow and outflow of moisture. For comparison purposes, a common pressure cooker can be considered. The pressure inside the cooker follows the saturation vapor pressure curve even if some vapor is released via the safety opening of the cooker. However, once the release of vapor via the safety opening is more than the rate of production of vapor according to the supplied energy, the thermodynamic laws do not apply any longer and the pressure decreases.

Influence of cracks on pressure release

In terms of growth of microcracks inside the concrete, it is worth recalling that boiler explosions as happened during the beginning of the use of steam machines. Once a pressurized boiler opens, i.e. by failure of a welding seam, the vapor is released at once. This causes a sudden decrease in pressure, which will result in
an immediate change of water from liquid to vapor with the corresponding increase in volume since the
temperature is still very high inside the boiler. This case would correspond to a vertical drop in pressure in
Figure 27 from the saturation vapor pressure curve to the gaseous zone.

The change from the liquid to the gaseous phase of water is combined with a significant increase in volume. A small amount of m = 1.0 kg of water will develop about V = 1.7 m³ of vapor at a temperature of T = 100°C.

The increase in vapor volume will cause severe damage to the boiler, which is equivalent to an explosion. Considering wider cracks inside the concrete, similar considerations can be made. The increased release of vapor might cause a sudden change of the remaining liquid moisture to vapor with an increase in volume that leads to sudden very violent explosive spalling of the concrete.

First general conclusions on pressure at high temperatures

In terms of pressure development inside concrete pores at high temperatures the following conclusions can already be drawn:

- Pressure is a function of the concrete temperature only.
- The degree of moisture saturation has no influence on the pressure development as long as sufficient moisture is available. Pressure will follow the saturation vapor pressure curve.
- Moisture migration processes influence the degree of pore saturation.
- Cracking might cause a sudden release of pressure including high release of energy leading to more violent spalling.

3.4.2 Failure due to high pressure

In terms of explosive spalling due to high pore pressure it is mentioned in literature that concrete tends to exhibit explosive spalling once the pressure exceeds the tensile strength of the concrete [27, 33, 49, 57, 68, 79, 81, 94]. This relation can be simplified as given in equation (3 - 6).

\[ p(T') > f_t(T) \quad \text{failure by pore pressure due to exceeding the tensile strength} \quad (3 - 6) \]

With:

- \( p(T) \) temperature-dependent pore pressure
- \( f_t(T) \) temperature-dependent tensile strength of concrete

It should be noted that the expression “failure due to a high pore pressure” is technically not correct. A high pore pressure causes stresses inside the heated concrete and these resulting stresses might exceed the tensile strength as resistance which leads to explosive spalling.
In terms of a possible correlation of high pore pressure and tensile strength as resistance Felicetti [36] analyzed the pore pressure development and splitting tensile strength when peak pressure was observed at different temperature levels and heating rates.

These splitting tests at the “pressurized” stage test are in contrast to several tests on the tensile strength of concrete at high temperatures. The strength is usually determined after a conditioning time at a high temperature, when all pressure is usually released.

Based on concrete mixes with a compressive strength of \( f_c = 40 \) MPa Felicetti concluded from his tests that the high pressure inside the concrete minimizes the tensile strength as a resistance against spalling. This reduced strength was in the range of 0.8 - 1.2 as the pore pressure itself. It is mentioned that these observations are almost independent of the heating rate. These tests are further analyzed in chapter 3.4.4.

Based on his tests, Felicetti concluded further that a high pore pressure presents a major risk regarding explosive spalling. However, it must be mentioned that his tests could not identify a high pore pressure as the only mechanism leading to explosive spalling.

Based on these tests results of Felicetti and the general recommendations given in the literature, a probable scaling factor \( \alpha \) for adjusting the pore pressure must be considered as shown in equation (3 - 7).

\[
\alpha \cdot p(T) > f_t(T) \quad \text{failure by pore pressure due to exceeding the tensile strength} \quad (3 - 7)
\]

With:

- \( p(T) \): temperature-dependent pore pressure
- \( f_t(T) \): temperature-dependent tensile strength of concrete
- \( \alpha \): scaling factor

As mentioned, this scaling factor is usually within the range of zero to \( \alpha = 1.0 \) according to the literature [36]. Dwaikat [33] mentions a factor of \( \alpha = \varepsilon \), where \( \varepsilon \) is the porosity of the concrete, which is usually around \( \varepsilon = 0.1 \).

**Simplified model for pores as hollow spheres inside concrete**

In terms of tensile stresses applied to concrete and a pore pressure at a high temperature, the stress distribution inside the concrete remains unknown; in particular for the stresses inside the dense cement layer close to concrete pores. Some basic mechanical considerations were made first by Ichkawa [54], who assumed 2D pores in a dense cement layer. He focused on stresses due to the pore pressure at the inner layer of the pores according to the pipe formula. However, these results were not compared to the stress peaks caused by applied tensile stresses.

In the following, a stress analysis due to tensile stresses and pore pressure is carried out. In addition, these results are discussed with the pore size distribution from own tests as shown in chapter 3.3.7. The aim is to find out if failure due to a high pore pressure is similar to failure due to tensile stresses.
Zweidler [135] suggests developing a basic 3D model with spherical voids inside, similar to concrete with pores. All voids have the same radius and distance to each other according to the pore size distribution. The aim is to analyze the stress concentration at the void either due to a tensile stress or due to pressure inside the void. Figure 28 shows a general sketch of the model.

Both stress distributions, either due to a tensile stress or due to a pore pressure lead to concentrations close to the voids. Figure 28 indicates the basic assumption that these two peak stresses close to the void are within a close range and depend on a scaling factor $\alpha$ as shown in equation (3 - 7).

**Stress concentration close to voids due to tensile stresses**

Stress concentration factors for voids are useful in evaluating the effects of porosity in materials. Peterson [105] provides stress concentration factors for all possible voids and cavities in homogenous material due to external loads. Firstly, the stress concentration factor for central spherical cavities in a finite-width body in tension was analyzed. This factor takes several cavities in a row into account, similar to the model by Zweidler. Figure 29 sketches the model according to Peterson.

This stress concentration factor is based on the $c/(0.5w)$ ratio, which is 2 for the selected model. It follows that pores in a row have no significant influence on the stress peaks.

Based on the consideration that the pores do not have a significant influence, the stress concentration for "an ellipsoidal cavity of circular cross section in an infinite body in tension" [105] is taken. The ratio of the two radii describing the ellipsoidal cavity is taken as 1 since the cavity is taken as a sphere. In addition, the
ratio of the Young's modulus of the concrete and the Young's modulus of the water/vapor mix inside the sphere is taken as $E_{\text{sphere}} / E_{\text{concrete}} = 0$.

For the P1 concrete mix, this leads to a constant stress concentration factor of $K_t = 2.05$, independent of the temperature level.

These stress concentration factors are valid for a homogenous material with a constant Poisson's ratio of $\nu = 0.3$. It should be noted that the Poisson's ratio has a minor influence on the stress concentration factor. A decrease of the stress concentration factor of 3% can be taken compared to the literature value for a Poisson's ratio of $\nu = 0.2$.

The peak stress at the voids due to an applied tensile stress can be determined using equation (3-8).

\[
\sigma_{\text{max}} = 0.97 \cdot K_t \cdot f_t = 1.99 \cdot f_t
\]  
(3-8)

With:

- $K_t$: stress concentration factor due to tensile stresses with $K_t = 2.05$ for $\nu = 0.3$

**Stress concentration close to voids due to internal pressure**

To determine the peak pressure close to the void due to internal pressure, a thick-walled sphere can be considered.

Stresses in thick-walled shells of hollow spheres depend on the pressure acting inside the sphere and the ratio of the outer and the inner radius [117]. The governing equation for tangential stresses due to an internal pressure over the shell thickness $\sigma_p(x)$ is given in equation (3-9) and the peak pressure at the inside of the shell $\sigma_{\text{max}}$ is given in equation (3-10).

\[
\sigma_p(x) = p \cdot \frac{0.5 \cdot (\frac{ra}{ri})^3 + 1}{(\frac{ra}{ri})^3 - 1}
\]  
(3-9)

\[
\sigma_{\text{max}} = p \cdot K_p = p \cdot \frac{1.5 \cdot (\frac{ra}{ri})^3}{(\frac{ra}{ri})^3 - 1}
\]  
(3-10)

With:

- $p$: internal pore pressure
- $ra$: outer radius of hollow sphere, including shell thickness (based on $ri = 1.0$)
- $ri$: inner radius of hollow sphere (taken as 1.0)
- $K_p$: stress concentration factor due to internal pressure

**Scaling factor $\alpha$**

With these two peak pressures close to the void, the scaling factor $\alpha$ can be determined using equation (3-11). Both peak stresses are the same at failure.

\[
\alpha = \frac{K_p}{K_t}
\]  
(3-11)
Both stress concentration factors as well as the scaling factor are given in Table 22 for the P1 concrete mix including two sub-mixes containing PP fibers. The M1 - M3 concrete mixes were not analyzed, since the porosity was only analyzed at the cold stage after spalling and at $T = 500°C$ for these mixes. The temperature range between $T = 250 - 350°C$ where spalling is usually observed was not analyzed.

Table 22: Stress concentration scaling factor for pore pressure and tensile stresses

<table>
<thead>
<tr>
<th>P1 concrete mix</th>
<th>0 kg/m³ PP fibers</th>
<th>2 kg/m³ PP fibers</th>
<th>3 kg/m³ PP fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Temp in °C</td>
<td>120 200 275 350³</td>
<td>120 200 275 350</td>
<td>120 200 275 350</td>
</tr>
<tr>
<td>avg. pore rad. in nm</td>
<td>2.84 2.31 1.88 4.30</td>
<td>1.88 2.05 2.86 2.84</td>
<td>1.88 1.95 1.88 1.88</td>
</tr>
<tr>
<td>porosity ε in %</td>
<td>7.49 12.68 15.22 10.88</td>
<td>8.87 9.53 8.46 10.89</td>
<td>14.77 10.79 10.06 12.46</td>
</tr>
<tr>
<td>radius $r_s$ in nm ³</td>
<td>1.91 1.60 1.51 1.69</td>
<td>1.81 1.76 1.84 1.69</td>
<td>1.52 1.69 1.73 1.61</td>
</tr>
<tr>
<td>$K_t$</td>
<td>1.99</td>
<td>1.99</td>
<td>1.99</td>
</tr>
<tr>
<td>$K_p$</td>
<td>1.75</td>
<td>1.98</td>
<td>2.11</td>
</tr>
<tr>
<td>scaling factor $\alpha$</td>
<td>0.88</td>
<td>1.00</td>
<td>1.06</td>
</tr>
</tbody>
</table>

1) Radius of modeled hollow sphere based on $r_i = 1.0$ for all samples
2) Type A PP fibers (l = 20 mm; $\varnothing = 32 \mu m$ - see Table 14)
3) Results from mercury intrusion porosimetry lead to doubtful results (see chapter 3.3.7)

All scaling factors are within a close range, even for higher temperatures. It seems that the scaling factor slightly increases with increasing temperatures for the fiber-free mix. However, this trend could not be observed for specimens containing PP fibers.

The scaling factor is near to $\alpha = 1.0 \pm 10\%$ for all temperatures and tested mixes. The governing assumption, that spalling occurs if the stresses at peak pore pressure exceeds the temperature-dependent tensile strength of the concrete, remains reasonable.

**Changes in pore geometry and influence of microcracking and crack growth**

Two questions remain unanswered after this brief analysis: The effect of changes in the geometry of the pores and the influence of microcracking or crack propagation on the peak stress distribution.

**Changes in pore geometry**

Changes in the geometry might influence the stress concentration factor close to the pores. The model as described above is based on circular cavities inside an infinite body in tension [105]. However, the shape of each cavity might vary. The occurrence of hollow spheres as they are presented in Figure 28 and Figure 29 is rather unlikely and the presence of ellipsoidal cavities has to be considered.

The stress concentration factor $K_t$ will increase to $K_t = 3.0$ for rather flat ellipsoidal cavities with rounded corners as they are expected in the concrete microstructure according to the Powers’ model (cited in [129]). It is mentioned in the Power’s model, wherein the pores between the CSH phases are usually rounded with...
a radius of $r = 3$ nm or less. The stress concentration factor due to high pore pressure $K_p$ will vary between $K_t \approx 0 - 3.0$, depending on the rotation of the cavity.

Considering the distribution of the cavities inside the concrete, an overall average scaling factor $\alpha$ within a close range of $\alpha \approx 1.0$ can be expected, as already mentioned in the simplified model.

**Influence of microcracking and crack growth**

The influence of flat cracks, crack propagation or crack widening on the occurrence of explosive spalling remains unknown. It seems very likely that a high pore pressure will lead to significant microcracking, which then lowers the tensile strength as concrete resistance. This would lead to higher risk of explosive spalling.

However, the microcracking and crack widening at temperatures that are usually observed at spalling is very small. Figure 30 shows the cumulative pore volume for different concrete mixes. This analysis shows the cumulative pore volume compared to the pore radius. It is an additional output from the mercury intrusion porosimetry. This testing procedure is explained in detail in the test report, including all test results [75].

![Figure 30: Cumulative pore volume for concrete at different temperatures or after spalling](image)

- left: M1 concrete with steel fibers, without PP fibers
- middle: P1-0 concrete with steel fibers, without PP fibers
- right: P1-3A concrete with steel fibers, with 3 kg/m$^3$ Type A PP fibers

It is interesting to note that temperature- or spalling-dependent changes in the cumulative pore volume mainly leads to changes within the range of a pore radius of $r = 1 - 10$ nm. Even for the P1-3A concrete mix containing PP fibers, a majority of changes took place below a radius of $r = 1 \cdot 10^2$ nm.

In contrast to the increase in cumulative pore volume with small cracks, the melting of the PP fibers only leads to a minor increase in pore volume at a pore radius of $r = 1.6 \cdot 10^4$ nm. This corresponds to the radius of the PP fibers used.

It is mentioned in Van Mier’s book on “Fracture processes of concrete” [129] that changes in material properties on the micro-scale are within the range of $r = 1 - 1 \cdot 10^3$ nm. These changes at the micro level have an influence on the mechanical properties of the concrete. However, test results showed that the majority of changes in the microstructure are within the lower range of this micro-scale with pores and voids with a radius of $r < 10$ nm or less. Based on the test results, the influence of the mechanical properties or the changes
in the release of pore pressure remains unclear. It is unknown if the governing equations of fracture mechanics apply at this level. Van Mier states [129] that an in-depth modeling of the concrete at the micro-level is impossible.

The cumulative pore volume observed with the M1 concrete mix with steel fibers but without PP fibers is of further interest as shown in Figure 30 (left). The difference in cumulative pore volume between the test specimen at the cold stage and the specimen that failed by explosive spalling after heating with $\dot{T} = 1.0 \text{ K/min}$ at a temperature of $T = 297^\circ\text{C}$ is rather low. A higher cumulative pore volume was observed with the specimen heated to $T = 500^\circ\text{C}$ with the lower rate of $\dot{T} = 0.5 \text{ K/min}$, which is in contrast to the other results. Deterioration of the concrete at higher temperatures leads to an increased cumulative pore volume.

It can be concluded that spalling does not lead to a significant increase in microcracking or pore widening, which would have been observed as an increase in cumulative pore volume. It seems that the concrete remains undamaged until peak pressure, independent of whether this peak pressure leads to explosive spalling or not.

Pore pressure measurements presented in the literature by Mindeguia [96, 97] and Felicetti [37] show a rather rapid decrease in pressure after reaching peak pressure. These observations strengthen the argument that microcracking is not expected until peak pore pressure. These pore pressure measurements are discussed in detail in chapter 3.4.4 with Figure 34 and Figure 35 showing the results from the pore pressure measurements.

**Concluding remarks**

In terms of failure criteria, the following conclusion can be drawn:

- The simplification that failure due to high pore pressure and failure due to exceeding the tensile strength lead to same results is reasonable.

- Microcracking that occurs due to high pore pressure during spalling seems to have only a minor influence on the general mechanical properties of concrete. The deterioration processes inside concrete at high temperatures are more important.

- The analysis of stress peaks considered pores in the concrete as hollow spheres of constant radius. Different geometries were not analyzed.

- Differences in stress peaks due to pore pressure and tensile stresses exhibit a 10% spread.

- The spread in test results for the tensile strength of concrete are in the same range. Models of the tensile strength at high temperatures during spalling obtain additional uncertainties.
3.4.3  Effect of pore pressure on spalling and temperature-dependent tensile strength

To determine the possible pore pressure upon spalling, the temperature upon spalling and the corresponding temperature-dependent tensile strength are used as parameters and compared to the pressure according to the saturation vapor pressure curve.

Based on several simplifications, the following points were considered:

- Failure due to a high pore pressure is taken as failure due to exceeding the tensile strength of concrete (pore pressure ≈ tensile strength upon spalling, as discussed in chapter 3.4.2).
- Temperature upon spalling defines the X-axis value on the saturation vapor pressure curve.
- Temperature-dependent tensile strength defines the Y-axis value on the saturation vapor pressure curve as resistance against high pore pressure.
- For the specimens that failed due to spalling, the data point is most probably close to the saturation vapor pressure curve.

Temperature upon spalling (X-axis)

First, the corresponding temperature upon spalling was defined for each test. The average temperature during the plateau phase was taken for those specimens that did not fail by explosive spalling and a “Type 1” temperature curve was determined as well as for the specimens that failed by explosive spalling according to the “Type 2” temperature curve. For the “Type 3” temperature curve, the surface temperature upon spalling was considered, since these specimens usually spalled layer by layer and the buildup of a critical pore pressure can be expected close to the heated surface. In addition, the difference between the core and surface temperature was usually very low in these tests.

Tensile strength as resistance (Y-axis)

As tensile strength, the average results from centric tests on small concrete cylinders (l = 150 mm, φ = 50 mm) as described in the test report [75] were taken. It should be noted that a spread in tensile strength of Δf_t = ±1.0 MPa is within the usual limit for tests in direct tension of small concrete cylinders. As with concrete in compression at high temperatures, the tensile strength of concrete reduces with increasing temperature.

It is mentioned in EN 1992-1-2 [6] that a temperature up to T = 100°C will not cause any significant changes to the tensile strength and the initial tensile strength can be taken. For higher temperatures, the tensile strength of the concrete was reduced according to the concrete mix as shown in appendix G.3. Different models for different concrete mixes are presented here.
**Comparison to saturation vapor pressure curve**

Figure 31 shows the saturation vapor pressure curve in function of the spalling temperature / temperature-dependent tensile strength ratio. The figure contains all test results that were carried out at IBK including additional tests on various ordinary concrete mixes (OPC), which are not described in the test report [75]. The concrete specimens from these additional concrete mixes had a compressive strength of $f_{ck} = 30 - 45$ MPa and all tests were carried out identically in terms of temperature cycle and mechanical properties. These specimens did not fail by explosive spalling.

Each data point in Figure 31 corresponds to one test and is labeled by abbreviations to identify the test. The results from tests on M1 - M3 concrete specimens are summarized in Table 13, while data for tests on the P1- concrete mix is given in Table 14. In addition, three different symbols (● + ○) were chosen according to the different temperature curves “Type 1 - 3” depending on the development of the moisture-induced plateau phase during heating.

The analysis as shown in Figure 31 indicates that the specimens that did not fail by explosive spalling and where a plateau phase during heating in the form of a “Type 1” temperature curve was observed are usually within the “liquid” area of the saturation vapor pressure curve. This is in contrast to the specimens that failed by explosive spalling, independent of whether the plateau phase partly developed (“Type 2” temperature curve) or did not develop at all (“Type 3” temperature curve). These data points derived from the tests are usually located on the gaseous side of the curve.

Two major issues have to be considered upon spread in results and a possible analysis of the test results. The tests in direct tension on concrete in the cold stage are challenging and show a rather large spread in the results. As mentioned above, the tensile strength can differ easily by $\Delta f_t = \pm 1.0$ MPa, which would have a significant influence on the results of the analysis as shown in Figure 31.

In addition, even if some design criteria to determine the tensile strength of concrete at elevated temperatures are given, the temperature-dependent tensile strength was not verified by own tests. Finally, the average plateau temperature was taken as the spalling temperature. It should be mentioned that the
plateau starts already at lower temperatures, which would also imply lower pore pressures. It can be assumed that most test results are usually located to the “left” of the lower spalling temperatures.

**Considerations on the pore pressure development inside heated concrete based on test results**

With an analysis of the results as shown in Figure 31, several factors can be considered to explain the pressure development inside the concrete. During heating it is assumed that the pressure development inside the concrete only depends on the temperature. As long as sufficient moisture is available, the pressure rises according to the saturation vapor pressure curve. This assumption agrees with observations from real tests by Mindeguia in 2011 when measuring the pore pressure in heated concrete [96]. He noticed that the increasing branch of the pore pressure curve follows the saturation vapor pressure curve within close limits. These test results are discussed further in the following.

**Test results on the “liquid” side of the saturation vapor pressure curve**

For the specimens that did not fail by explosive spalling and where a fully developed plateau phase is observed during heating, it can be assumed that the pressure rises mainly according to the saturation vapor pressure curve until the vaporization rate of moisture and the release of this vapor is balanced due to higher permeability and microcracking. Once this state has been reached, the pressure inside the concrete will not rise even though the concrete would resist a much higher pressure. These test results are located on the “liquid” side of the curve, indicating that the temperature-dependent tensile strength acting as a resistance against explosive spalling exceeds the probable pore pressure inside the concrete. The tests on the M3 concrete specimens with steel fibers are located on the “liquid” side of the saturation vapor pressure curve. It is observed that higher heating rates applied during these tests lead to lower plateau temperatures and pore pressures. The data points are shifted further left. This can also be explained with Mindeguia’s tests on pore pressure [96]. It is stated that higher heating rates lead to additional microcracking of the concrete due to stresses-induced upon expanding. This increased microcracking leads then to a higher permeability and releases pressure.

**Test results on the “gaseous” side of the saturation vapor pressure curve**

Some test results are located on the “gaseous” side of the curve, even though the specimen did not fail by explosive spalling and the expected pressure inside the concrete according to the plateau phase would be much higher than the expected tensile strength. These observations can be explained with the spread in results for the tensile strength for most cases, i.e. tests on the M1 concrete mix.

This might also be the case for the specimens that failed by explosive spalling without the formation of a plateau phase (“Type 3” temperature curve). These specimens were exposed to rapidly increasing temperatures. It might be assumed that the tensile strength for these specimens is still higher as suggested by the design recommendation and the data points would shift further up towards the saturation vapor pressure curve.
However, several data points for specimens that failed by spalling and where “Type 2” temperature curves with a partly developed plateau phase were observed further into the “gaseous” area. Mainly the M2 concrete specimens with steel fibers, which failed after heating with $\dot{T} = 2.0 - 2.5 \text{ K/min}$ (M2 - test 4 + 5), exhibited a high plateau temperature upon spalling while the expected resistance in the form of tensile strength is rather low. These results cannot be explained with the usual spread in results and other explanations for these data points must be found.

With the concrete specimens that failed by explosive spalling, it is generally assumed that the pressure inside the concrete rises according to the saturation vapor pressure curve until the pressure exceeds the temperature-dependent tensile strength, which will result in spalling. However, the results from the tests on the M2 concrete specimens have shown that this is not the case, since the expected pressure at the spalling temperature would be much higher than the remaining resistance of the concrete in the form of tensile strength. Considering the M2 - V2 test 4 specimen, with a plateau temperature upon spalling of $T = 312^\circ\text{C}$, a high pore pressure of $p = 9.8 \text{ MPa}$ would be expected if the pressure inside the concrete were to follow the saturation vapor pressure curve. A resistance against this high pressure is rather unlikely.

The most probable explanation is that the pressure rises according to the saturation vapor pressure curve until the increased formation of microcracks starts during the beginning of the plateau phase. This has several overlapping factors in terms of pressure development. Firstly, the increased permeability causes increased moisture migration and concrete areas close to the heated surface tend to dry out. Secondly, the microcracks lead to an increased “interconnectivity” of the pores and a nearly constant pore volume cannot be considered any longer.

An isochoric state inside the concrete pores with a nearly constant volume is essential to achieve pressure according to the saturation vapor pressure curve.

With changes in volume due to microcracking, temperature and significant changes in vapor content, this thermodynamic state cannot be analyzed using conventional methods in a satisfactory way to predict changes in the pore pressure. The pressure inside the concrete might still increase with increasing temperature and as long as moisture is available. However, the increase in temperature is reduced due to the uptake of vaporization energy (moisture-induced plateau). In addition, the pressure increase might be lower than that of the saturation vapor pressure curve with increasing growth of microcracks and moisture migration.

Spalling might still occur as was observed in all tests with a partly developed plateau phase (Type 2 temperature curve) as due to a high pressure and a reduced tensile strength of the concrete.

**Observations on M1 concrete mix**

It is interesting to note that all data points for the M1 concrete mix more or less correspond, independent of the concrete sub-mix or whether the specimen failed by spalling or not. It seems that the pore pressure inside the concrete is close to the saturation vapor pressure curve for all tests, independent of whether the specimen exhibit explosive spalling or not.
Explosive spalling and indicators

As an exception, the M1-test 4 is mentioned. The heating rate of $\dot{T} = 0.75$ K/min caused explosive spalling, initiated in the core of the specimen. Due to the moisture migration during heating, drying of concrete sections within the specimen and the formation of microcracks during the long temperature exposure might lead to a lower increase in pore pressure.

**Observations on OPC mix**

The OPC specimens did not fail by explosive spalling and a plateau phase in the form of a “Type 1” temperature curve was clearly observed. These results show a clear correlation of temperature during this plateau phase with a temperature-dependent tensile strength of the specimen. It seems most probably that the pressure inside the concrete during the plateau phase is close to the pressure according to the saturation vapor pressure curve.

**Observations on M3 concrete mix**

In terms of the M3 concrete mix it is interesting to note that the temperature / tensile strength ratio moves further into the “liquid” zone with increasing heating rates. This phenomenon can be observed for both submixes, independent of the use of steel fibers added to the concrete mix. The M3 concrete specimens were heated with rather high heating rates of up to $\dot{T} = 8.0$ K/min compared to the other tests, which leads to high temperature gradients of up to $\Delta T = 4.4$ K/mm. These high temperature gradients inside the concrete promote thermal stresses and microcracking during heating. As discussed before, the growth of microcracks inside the concrete enable an increased release of vapor due to the higher permeability; hence the temperature during the plateau phase decreases.

**Observations on P1 concrete mix containing PP fibers**

The P1 concrete mix was also analyzed. Four tests are included in Figure 31, with two tests on specimens without any PP fibers added to the concrete, as well as one test each on a concrete specimen with type “A” PP fiber and type “C” PP fiber. Similar to the observations made with the M3 concrete mix, the use of PP fibers increasing the permeability of the concrete shifts the data points further left to the “liquid” area, where spalling is very unlikely. It is interesting to note that both data points for the P1-2A and P1-3A tests containing type A PP fibers are still close to the saturation vapor pressure curve. As discussed in chapter 3.3.5, the type A PP fibers are not optimal in terms of providing sufficient spalling resistance as reflected in these observations.

**General considerations on the pore pressure development inside heated concrete**

Figure 32 concludes these considerations in terms of changes of the permeability, cracking and tensile strength and the expected pressure inside the concrete during heating. The following cases for spalling can be summarized:
Case 1 - Pore pressure is equal to tensile strength upon spalling (spalling occurs)

In theory, pressure will rise according to the saturation vapor pressure curve until spalling occurs by exceeding the tensile strength of the concrete. In this case the data point derived from the tests is located directly on the pressure curve (case 1). This is a rare theoretical occurrence.

Case 2 - Pore pressure lower than tensile strength (no spalling due to higher resistance)

Adding PP fibers to increase the permeability or choosing a mix that is known to increase in permeability at high temperatures (OPC) or applying a rapid heating rate, which causes microcracking of the concrete but does not lead to spalling will decrease the temperature when the vaporization of moisture peaks (plateau phase). However, the tensile strength of the concrete acting as a resistance against explosive spalling is still high and the concrete will not spall due to high pore pressure. Data points from these tests are shifted towards the “liquid” area of the saturation vapor pressure curve (case 2).

Case 3 - Pore pressure lower than saturation vapor pressure (spalling might occur)

Some specimens failed by explosive spalling at a rather high temperature during the plateau phase. The tensile strength of these specimens acting as a resistance is significantly lower at this temperature compared to the expected pressure according to the saturation vapor pressure at this temperature. For example, a specimen is assumed that failed at a temperature of $T = 270^\circ C$, with a corresponding temperature-dependent tensile strength of the sample of $f_{t(270^\circ C)} = 3.5$ MPa (case 3). At this temperature, the pressure inside the concrete is considered to be much higher with $p_{(270^\circ C)} = 5.5$ MPa, which clearly cannot be the case. It is considered that microcracking starts at lower temperatures and increases the permeability and vapor migration. This higher permeability leads to a release of pressure during heating. In addition, the interconnectivity of the pores is increased. An isochoric state cannot be assumed any more. Pressure inside the concrete will not follow the saturation vapor pressure curve but will increase much slower. This low increase in pressure might still cause explosive spalling during heating (case 3). However, the specimen might also remain intact since the tensile strength also varies and might be higher than the pressure. This might be the case for concrete for which the plateau phase is observed at very high temperatures ($T > 300^\circ C$) without spalling.

The possible cases of spalling are summarized in Figure 32. The data points are for display purposes only and are not taken from real tests.
Explosive spalling and indicators

General conclusions in terms of development of high pore pressure

The analysis of temperature upon spalling, tensile strength and possible pore pressure leads to the following general conclusions:

- The temperature-induced plateau indicates the peak pressure inside the concrete during heating. Pressure decreases when temperature increases again since no moisture is available.

- The pressure development inside the concrete follows the saturation vapor pressure curve as maximum pressure. A higher permeability leads to lower pressure development.

- The saturation pressure according to the temperature during the moisture-induced plateau and the reduced tensile strength indicate a result from tests that can be categorized further.

- Three categories of test results could be derived if compared to the saturation vapor pressure curve:
  - Case 1: Pore pressure = tensile strength upon spalling. Pore pressure according to the saturation vapor pressure curve.
  - Case 2: Pore pressure < tensile strength. No spalling due to high resistance, pore pressure according to the saturation vapor pressure curve
  - Case 3: Pore pressure < saturation pressure. Spalling might occur if pore pressure > tensile strength.
3.4.4 Comparison with tests reported in the literature on pore pressure

The previous chapter 3.4.3 explained the correlation between the apparent tensile strength of concrete and the pore pressure that is expected at spalling. However, own tests on the pore pressure inside concrete during heating were not made.

To compare the considerations from own tests with data provided in the literature, some test results for pore pressure measurements are further analyzed. It should be mentioned that pore pressure measurements are rather difficult [67], since the pressure from several pores is measured simultaneously due to the size of the pressure sensor. The influence of different thermal expansion coefficients of vapor inside the pores and the hydraulic oil or the cement paste and the hydraulic pipes have never been discussed in detail.

It should be noted that none of the tested concrete mixes included silica fume. This leads to a significant higher permeability of the concrete at high temperatures as observed with the M3 concrete mix. The expected pore pressure is significantly lower in these cases compared to the proposed pore pressure for own tests as shown in Figure 31. In addition, high heating rates were applied to test specimens presented in the literature leading to significant thermal stresses, which might become the governing factor in terms of explosive spalling. The main results are summarized in Table 23.

Fundamental tests on pore pressure by Sermethmengolu in 1977

The first ideas regarding the pore pressure development in concrete were already mentioned by Sermethmengolu in 1977 [118]. He performed fundamental tests on the pore pressure development of concrete at high temperatures. As concluding remarks on his work, he noticed that the maximum pressure measured in his tests seems to be proportional to the depth of the specimen. In addition, he noticed that higher pore pressures are usually obtained in deeper sections of the concrete with increasing distance from the heat-exposed surface. The possibility of a relationship between pressure and the saturation vapor pressure curve was not mentioned at that time.

In these tests, a maximum pore pressure of up to \( p = 2.1 \) MPa was measured at a depth of 100 mm from the fire exposed surface. It is interesting to note that the pressure measured between 75 mm and 100 mm in depth did not rise, in contrast to the rapid increase in pressure between 60 mm and 75 mm of nearly \( \Delta p = 1.5 \) MPa. The specimen was heated using an open gas flame and the temperature during the peak pressure at 75 mm depth was about \( T = 250^\circ\text{C} \). Figure 33 shows the peak pore pressure measurements for different depths of the concrete.

It was mentioned that both drying of the heated surface and moisture migration into deeper sections of the concrete lead to a higher pore pressure in these areas once sufficient moisture content is present in these sections. In addition, the concrete surface is subjected to rapid drying, since the exposure to the gas flame leads to increased cracking and higher permeability.
Explosive spalling and indicators

Figure 33: Peak pore pressure measurements with increasing depth according to Sertmehtmengolu [118]

Pore pressure measurements by Jansson in 2006 on ISO fire exposed concrete

Jansson made several tests in 2006 [60] on concrete slabs ($f_c = 40$ MPa) exposed to the ISO fire curve [5]. He measured the pore pressure with oil-filled tubes at different depths in the concrete. As peak pressure he measured $p = 1.6$ MPa at 10 mm depth to the heated surface. The pressure decreased significantly directly after this peak, combined with a sudden increase in temperature at 10 mm depth. It can be assumed that all moisture evaporated at this depth or migrated due to increased cracking caused by thermal stresses. At peak pressure, a temperature within the range of $T \approx 250^\circ C$ was measured, which would correspond to a vapor pressure of $p \approx 4.0$ MPa. This high pore pressure was clearly not measured. This leads to the conclusion that the high heating rate caused cracking of the concrete and released the high pressure. The tensile strength of the concrete mix is not mentioned; hence no final assessment on the type of failure can be made. It can however be assumed that the tensile strength of the used concrete with $f_t \gg 1.6$ MPa stressing the supposition that high thermal stresses have a large influence on spalling in these tests.

Pore pressure measurements at various depths by Bangi and Horiguchi in 2010 (HPC and PP fibers)

Bangi and Horiguchi presented their results on pore pressure measurements inside concrete in 2010 [15]. They tested HPC with a compressive strength between $f_c = 84 - 105$ MPa. Specimens were exposed to low- ($T = 5.0$ K/min), moderate ($T = 10.0$ K/min) and high heating rates (ISO fire curve with $T_{max} = 800^\circ C$), while the pressure is measured at 10 mm, 30 mm and 50 mm from the heated surface. PP fibers were added to some concrete specimens.

The heating rate was determined as a main factor having an influence on the pore pressure development inside the concrete, which is similar to Sermethmengolu’s findings. Bangi observed a pressure of $p = 2.0$ MPa at 30 mm depth upon initial spalling of the concrete. However, the pore pressure continued to rise to $p = 2.17$ MPa at 50 mm depth. The temperatures corresponding to these pressures are not mentioned.

Microcracking at the concrete surface was noticed in all tests, independent of the heating rate. This leads to a low increase in pore pressure close to the heated surface. A rise of only $p = 0.30 - 0.35$ MPa was measured.
In terms of the test specimen with PP fibers added to the concrete, the highest pore pressure was noticed as expected in deeper sections (50 mm) of the concrete due to the increased moisture migration. A maximum pressure of up to \( p = 2.0 \) MPa was measured for the high heating rate. PP fibers with a rather high average diameter of \( \varnothing = 310 \) \( \mu \)m were used and it was stated that the chosen PP fibers are not optimal in terms of spalling protection. The maximum pressure for specimens heated with the slow heating rate was significantly lower with \( p = 1.0 \) MPa, which is caused by two effects: The lower heating rate causes a slower increase in pore pressure with the time and the increased permeability of the concrete due to the PP fibers enables an extensive reduction of high pressure with the time. The pores get dry more quickly, which limits the pressure development. In both cases the pressure development was far below the saturation vapor pressure curve and a peak pressure was observed at a concrete temperature of about \( T = 300^\circ\text{C} \). This is similar to a case 3 development of pressure.

Temperature pressure curves are only given for the concrete mixes containing PP fibers. These results are summarized in Table 23.

Data on the tensile strength of the concrete is not given. However, it can be expected that the tensile strength is higher than \( f_t = 2.0 \) MPa with the tested HPC mix.

### Table 23: Results provided in literature from pore pressure measurement

<table>
<thead>
<tr>
<th>Source</th>
<th>Source</th>
<th>p in MPa</th>
<th>Height / distance in mm</th>
<th>Heating</th>
<th>Temp. at pressure</th>
<th>( f_c ) and ( f_t ) at ( T = 20^\circ\text{C} ) in MPa</th>
<th>Vapor saturation pressure ( p ) in MPa ( ^{a1} )</th>
<th>Spalling and data point according to Figure 32</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jansson [60]</td>
<td>1.6</td>
<td>200 / 10</td>
<td>ISO</td>
<td>( = 250^\circ\text{C} )</td>
<td>( f_c = 39.4 ) ( f_t = N/A )</td>
<td>( = 4.0 )</td>
<td>spalling / 3 ( ^{2} )</td>
<td></td>
</tr>
<tr>
<td>Sertmehetoglu [118]</td>
<td>2.1</td>
<td>100 / 75</td>
<td>open gas flame</td>
<td>( = 250^\circ\text{C} )</td>
<td>( f_c = 46 ) ( f_t = 2.9 )</td>
<td>( = 4.0 )</td>
<td>spalling / 3</td>
<td></td>
</tr>
<tr>
<td>Bangi [15]</td>
<td>0.9</td>
<td>100 / 50</td>
<td>5.0 K/min</td>
<td>( = 250^\circ\text{C} )</td>
<td>( f_c = 105 ) ( f_t = N/A )</td>
<td>( = 4.0 )</td>
<td>no spalling / 3 ( ^{2} )</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>100 / 50</td>
<td>ISO &lt; 800°C</td>
<td>( = 300^\circ\text{C} )</td>
<td>( f_c = 105 ) ( f_t = N/A )</td>
<td>( = 8.6 )</td>
<td>no spalling / 3 ( ^{2} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) Height of tested specimen and distance of pore pressure measurement from heated surface
2) Based on assumption that \( f_t > 2.0 \) MPa
3) Concrete mix included PP fibers
4) Expected saturation pressure according to the measured temperature at peak pressure

### Pore pressure measurements on slabs with varying concrete strength by Mindeguia from 2010

Mindeguia [96, 97] analyzed the pore pressure inside concrete at high temperatures in own tests on concrete slabs of l·w·h = 300·300·120 mm\(^3\) exposed to heat from one surface. The pressure and corresponding temperature were measured between 10 mm and 50 mm depth in steps of 10 mm from the heated surface.

Two concrete mixes are of main interest, the B450 and B500 mixes with an average compressive strength of \( f_c = 62 - 76 \) MPa. The mix design included neither the use of PP fibers nor of silica fume. Data on the tensile strength is not available. The concrete specimens were heated to \( T = 600^\circ\text{C} \) in about \( t = 5 \) min. The temperature was then kept constant. During the test, only minor aggregate spalling was observed but noticeable cracking.
The permeability of the individual concrete mixes was also analyzed at the cold stage and is given as $k = 1.2 \times 10^{-18}$ m$^2$ and $k = 1.6 \times 10^{-20}$ m$^2$. Changes in the permeability during heating were not analyzed. However, it should be mentioned that the initial permeability of these concrete mixes is rather low compared to the initial permeability of the concrete mixes M1 - M3 as discussed in chapter 3.3.6.

Similar to the findings from Sertmehmetoglu [118] and Bangi [15] a higher pore pressure in deeper sections of the concrete was usually measured. The pressure close to the heated surface increase first which is followed by a decrease due to drying of the areas close to the heated surface, moisture migration in deeper sections and an increase in permeability due to the higher temperatures or cracking.

This effect of sudden cracking and release of pore pressure was noticed with the B450 concrete mix at 10 mm depth from the heated surface. A peak pressure of $p = 1$ MPa was observed. The pressure development followed the saturation vapor pressure curve. At a temperature of $T = 170\,^\circ C$ a significant decrease in both temperature and pressure was noticed. Similar to own tests, the moisture vaporizes at this temperature, which causes a decrease in temperature due to the increases energy uptake. In addition, at the saturated stage pressure and temperature are linked. Once the pressure is released to about $p = 0.45$ MPa, the pressure at 10 mm depth decreases continuously. During this descending branch of the pressure curve the pressure does not follow the saturation vapor pressure curve, since insufficient moisture is available.

The sudden release in pressure is related to cracking of the concrete in this area. Drying of the pores will lead to a smooth transition to the descending branch of the pressure curve without decreases in temperature, since no additional evaporation energy is required.

In addition, cracking at the surface with the B450 concrete mix was noticed at a rather high temperature explaining, which lead to the high pressure close to the heated surface. Usually cracking is expected to occur earlier as observed with the B500 concrete specimen, for which the pressure at a depth of 10 mm is far lower compared to the tested B450 concrete specimen.

![Figure 34: Pressure development inside concrete at different depths according to Mindeguia [97]](image)

The results as displayed in Figure 34 show a higher pore pressure than the saturation vapor pressure curve. These higher pore pressures were usually noticed in deeper sections of the concrete specimen (50 mm). In general it might be the case that local temperature peaks cause a higher pressure locally, but a significant higher pressure for a longer temperature period is difficult to explain. It was mentioned that dry air in the
pore might cause a higher pressure compared to the saturation pressure. A higher degree of pore saturation due to moisture migration leads to a smaller volume for air inside the pore, which causes increases in the partial air pressure at high temperatures. It is stated that the partial pressure of dry air inside a pore must be considered. However, even for temperatures below $T < 100^\circ$C a pore pressure of already $p = 0.3$ MPa was noticed. At this temperature moisture migration is very limited, in particular for water, and a constant degree of saturation in all pores can be assumed. In this case, the high pressure exceeding the pressure of the saturation vapor pressure curve is observed at all depths during heating. However, this peak pressure at low temperatures was only measured at 50 mm depth.

The permeability based on gas viscosity for concrete is known to be higher than the permeability based on vapor viscosity as shown in chapter 3.3.6, which would lead to a rather rapid release of high pore pressure. Further tests in this area are required in order to explain the possible occurrence of the high pressures. Even though the very advanced test-set up complies with the latest technological developments, it should be discussed in detail in order to exclude possible sources of error. Cracks that promote the interconnectivity of the pores between two pressure transducers should also be taken into account.

**Concluding remarks from tests on pore pressure measurement**

Based on the tests performed by Mindeguia, the findings may be summarized as follows:

- Pressure inside the concrete rises to close to the saturation vapor pressure curve.

- Higher pressure compared to the saturation vapor pressure curve is explained by partial air pressure. However, possible influences of the measuring devices must be discussed in detail prior to a final assessment.

- A lower w/c ratio leads to an overall higher pressure at peak temperature, since these concrete mixes are much “denser” than common concrete mixes with a high permeability.

- Cracking leads to a sudden release of pressure combined with a decrease in temperature, while drying of the pores leads to a smooth transition towards the descending branch of the pore pressure curve.

- The peak in pressure was usually observed at the “plateau temperature”, hence during the evaporation of the moisture. However, the increase in pore pressure is lower compared to the saturation vapor pressure curve in some tests since insufficient moisture leads to a non-saturated state inside the pores and the pressure development follows the ideal gas law.
Combined pore pressure measurement and splitting tensile strength at high temperature by Felicetti

Apart from the pore pressure during heating, the splitting tensile strength at high temperature was also measured by Felicetti et al. on concrete samples and the results were presented in 2012 and 2013 [36, 37]. In these tests an OPC mix with an average compressive strength of $f_c \approx 40$ MPa was chosen. For this mix spalling is very unlikely to occur, even at high heating rates. The initial tensile strength of the concrete was determined as $f_t = 2.79$ MPa.

These tests are closely related to the tests by Mindeguia [97]. A concrete web with a thickness of $h = 100$ mm heated from both sides was chosen as test specimen. Pressure and temperature are measured in the middle of the web. In contrast to previous tests, different heating rates were chosen with $\dot{T} = 1.0, 2.0, 10$ and $120$ K/min.

The pressure was measured with the different heating rates. The splitting tensile strength of the concrete was determined when the peak pressure was reached and decreased again. The end of the plateau phase for temperature also indicated a peak pressure.

Figure 35 shows an excerpt of the measurements on the maximum pressure development for three different heating rates between $\dot{T} = 1.0, 2.0$ and $10$ K/min. It is interesting to note that higher heating rates lead to higher pore pressures inside the concrete. For very low heating rates the pressure development does not follow the pressure according to the saturation vapor pressure curve. The increase of vapor pressure due to rising temperatures is lower than the release of vapor pressure due to the permeability of the concrete. As a simplification it can be said that low heating rates lead to lower pressures due to the more pronounced drying.

For higher heating rates, higher pore pressures were measured, since the increase in vapor pressure is more pronounced than the release of pressure. However, the difference in pore pressure between $p = 1.1$ MPa ($\dot{T} = 2.0$ K/min) and $p = 1.3$ MPa ($\dot{T} = 10$ K/min) is not that significant.

In contrast to these findings, a low pressure which does not exceed $p = 0.49$ MPa and is far below the saturation vapor pressure curve was noticed for the specimen heated with $\dot{T} = 120$ K/min. Thermal stresses due to the rapid heating rate lead to microcracking, which causes an increased pressure release. The pressure development for this test is not shown. Similar observations were already made by Mindeguia [95] who noticed a high pore pressure of $p = 3.7$ MPa inside a concrete cylinder with $f_c = 60$ MPa with low heating rates, which decreased to $p = 0.4$ MPa for an ISO fire exposure.

Some tests with PP fibers added to the concrete mix were also performed. In these tests a very low peak pore pressure of $p \approx 0.2$ MPa was measured even at the high temperature of $T = 220^\circ$C, showing an overall good increase in the permeability of the concrete to release high pore pressure.

In agreement to the findings by Mindeguia [97] a partial pressure higher than the saturation pressure is observed for temperatures to $T = 100^\circ$C for the specimen heated with $\dot{T} = 120$ K/min.

A detailed chart on the measured pore pressure and corresponding splitting tensile strength is presented in the paper by Felicetti [37]. All specimens did not fail by explosive spalling and have a significantly higher tensile strength as resistance compared to the measured pore pressure. These test results would all be
located on the “liquid” side of the saturation vapor pressure curve and all test points would be categorized as “case 2” according to Figure 32.

**Figure 35:** Pore pressure inside heated OPC without fibers measured by Felicetti [37]

**Concluding remarks on pore pressure measurements in the literature compared to own tests**

In terms of own tests on spalling of the M1 - M3 concrete cylinders and the analysis of pore pressure / tensile strength as shown in Figure 31 and summarized in Figure 32, the results presented in the literature further strengthen the explanations of failure of the tested specimens. The following general conclusions can be drawn:

- Tests from the literature show that peak pore pressure increases with decreasing permeability of the concrete. In addition it was confirmed that water inside concrete mainly evaporates at a temperature higher than $T = 100^\circ$C. This evaporation is observed by the plateau phase for temperature.

- The peak in pore pressure is usually observed at the end of the plateau phase when most of the water evaporated.

- The pore pressure inside the concrete rises with increasing temperatures according to the saturation vapor pressure curve. However, rapid heating causes microcracking close to the heated surface, which releases the high pressure.

- If the specimen fails by spalling, a “case 1” pressure development can be assumed. However in most cases the specimen does not fail since the tensile strength as resistance is higher. This would lead to a “case 2” data point as shown in Figure 32.

- Apart from the release of high pore pressure, microcracking that does not lead to spalling close to the heated surface or the temperature-dependent higher permeability promotes the migration of moisture into deeper sections of the concrete where a higher peak pressure is measured. This can lead to spalling initiated in inner sections of the concrete at high temperatures even after a longer time of temperature exposure as it was observed with the M1 - test 3 ($\dot{T} = 0.75$ K/min) specimen.

- Very low heating rates lead to both an increase in pressure below the expected pressure according to the saturation vapor pressure curve and a final lower pore pressure compared to rapid heating. This is caused...
since the release in vapor pressure according to the permeability is higher compared to the temperature-dependent pressure development.

Further, slow heating rates shift the peak in pressure towards higher temperatures. This pressure development would correspond to “case 3” as shown in Figure 32. Spalling might happen if the pressure exceeds the tensile strength. However, pressure might also decrease, since the pores dry without spalling.
3.5 General conclusions from tests on spalling and pore pressure considerations

Several tests were performed on concrete cylinders ($\varnothing = 150$ mm, $h = 300$ mm) to analyze the risk of explosive spalling at high temperatures. Three main mixes M1 - M3 were chosen, covering different grades in compressive strength between $f_c = 90 - 150$ MPa and different amounts of silica fume between 6% and 0% of the total concrete mass. From these tests on linear heating, the following main conclusions can be drawn:

- The use of silica fume increases the risk of explosive spalling.

- Spalling was not observed for the specimens made of the M3 concrete mix without silica fume even though the highest heating rates were applied in these tests.

- By decreasing the heating rate it was possible to avoid explosive spalling even for the test specimens made with the largest amounts of silica fume and lowest permeability in all tests.

- From the temperature developments measured for the specimens that did not fail due to explosive spalling, a moisture-induced plateau caused by evaporating water was observed at different temperatures, depending on the concrete mix tested. The highest temperature at the plateau phase was observed for the specimens made of the M1 concrete mix with the highest amount of silica fume and thus with a very dense structure.

- It can be concluded that the pressure inside the concrete pores usually follows the saturation vapor pressure curve. The pressure development is lower for lower heating rates and concrete mixes with a higher permeability. The increase of vapor pressure due to rising temperatures is overcompensated by the release of vapor pressure due to the permeability of the concrete.

- Spalling is usually expected within the range of the plateau temperature, since the highest pressure is present inside the concrete. The pressure will decrease again with higher temperatures, since the pores are getting dry and no vapor pressure can develop.

- Pore pressure spalling was determined to be the most important factor concerning explosive spalling. Possible influences of thermal stresses can be neglected for the M1 concrete mix due to the low heating rates. In terms of the M3 concrete mix, the higher heating rates showed a noticeable influence on the release of pore pressure due to microcracking of the concrete. This leads to the conclusion that a critical pressure level leading to spalling will hardly be reached inside the ordinary performance concrete (OPC) during heating. The higher increase in permeability and cracking due to thermal stresses will lead to an increased release of high pore pressure.

- No beneficial effects in terms of spalling resistance could be observed in the tests by adding steel fibers to the concrete.

- The tensile strength of concrete at high temperature is of major importance since a high tensile strength increases the resistance to explosive spalling due to high pore pressure. Test results are scarcely available and current design standards do not cover the recent developments in new HPC and UHPC mixes including high amounts of silica fume, fibers or other admixtures. New promising models on the splitting tensile strength of concrete at high temperatures by Khaliq [65] cover these recent developments.
- In terms of the use of PP fibers it was confirmed that mainly the geometry and overall fiber distribution inside the concrete are the governing factors minimizing the risk of spalling. The total amount of fibers in kg/m$^3$ is of secondary importance.

- Tests have shown that the porosity changes significantly with temperature. For the specimens that failed by explosive spalling almost no change in total pore volume was noticed, while a significant increase for high temperatures of $T = 500°C$ was found. In addition, the average pore radius decreases for the M1 and M2 concrete mix containing silica fume, which leads to a significantly higher number of pores with higher temperatures. However, it remains unknown if this effect has a significant impact on the likelihood of spalling.

- The temperature-dependent increase in porosity is almost unaffected by adding PP fibers to the fresh concrete.

- In terms of permeability it was observed that the permeability decreases within a temperature range between $175°C < T < 275°C$ for the concrete mixes containing a high amount of silica fume. No existing models of the temperature-dependent permeability agree with these observations.

- It was interesting to note that the use of PP fibers has no significant influence on the initial permeability at the cold stage, unless they influence the general workability of the concrete during mixing and casting.
4 HYDROTHERMAL MODEL FOR EXPLOSIVE SPALLING

4.1 Main requirements

4.1.1 Introduction

With the ongoing research on explosive spalling of concrete at high temperatures, several models to describe spalling due to high pore pressures have been presented in recent decades. Some of the earliest fundamental models proposed were described briefly in chapter 2.4.

The tests on concrete cylinders as described in the IBK test report [75] and analyzed in chapter 3 indicated spalling due to high pore pressure as one of the main mechanisms leading to failure of the tested concrete specimen, in particular for the M1 concrete mix with a very low permeability. In contrast to several existing tests documented in the literature, a very low heating rate was applied during these tests. This test set-up allowed additional knowledge to be gained on vaporization and moisture migration during heating. Different failure mechanisms could be identified using this test layout. In addition, all governing parameters with influence on the risk of spalling were determined by tests and compared to existing studies.

The aim of this chapter is the development of a hydrothermal model on explosive spalling that could be used as a preliminary but adequate description of the observed test results based on the following assumptions and simplifications:

- The model is based on the material properties gained from own tests [75]:
- The governing equations of moisture migration in porous media are taken into account.
- Thermodynamic laws in terms of evaporation and pressure development apply.
- A high pore pressure is the only mechanism leading to explosive spalling.
- Thermal- or load-induced stresses as well as cracking are neglected.

4.1.2 Main requirements and lessons learned from tests

Several requirements of an analytical model for explosive spalling can be derived from the tests as performed and described in chapter 3. A good model must meet the following observations from the tests and take them into account:

- Explosive spalling is a function of the concrete’s heating rate.
- The different failure modes observed in the small-scale tests (type of temperature curve, spalling from inside / layer by layer) are a function of the concrete mixes and heating rates.
- The permeability of concrete was indicated as one key factor influencing moisture migration and the release of vapor pressure. Migration processes are mainly based on the permeability of the concrete. The direction of these migration processes depends on pressure gradients inside the concrete (flow of vapor).
- Peak vaporization was noticed at temperatures higher than \( T > 100^\circ C \) and the movement of the vaporization front does not follow the \( T = 100^\circ C \) isotherm.
- Failure by exceeding the tensile strength of the concrete due to a high pore pressure is assumed to be the main failure criterion in tests on specimens with a very low permeability (M1 concrete mix). The expected stresses due to pore pressure at failure are close to the temperature-dependent tensile strength of these specimens.
- Possible beneficial effects of steel fibers in terms of an increased resistance against explosive spalling were not observed and can be neglected.
- The development of pore pressure follows the saturation vapor pressure curve. It might be lower with low heating rates, a high permeability or low moisture content. However, higher pressures than those taken from the saturation vapor pressure curve are assumed not to be possible.
- The analytical model must include boundary conditions to allow or prevent moisture migration to the outside of the specimen as losses in moisture, if only parts of a concrete member are modeled.

4.1.3 General concept of an analytical model for the explosive spalling of concrete

Analogy to pressure cooker

A simplified method explaining explosive spalling is the comparison of concrete with a household pressure cooker.

The water inside a closed pressure cooker is liquid at an ambient temperature of $T = 20^\circ\text{C}$. The steel shell of the cooker is subjected to the hydrostatic pressure for the given the water level inside the pot and the partial pressure (air pressure) above the water. With temperatures exceeding $T > 100^\circ\text{C}$, most of the water inside the cooker remains liquid. The layer of air above the water becomes saturated in the form of vapor. With temperatures exceeding $T > 100^\circ\text{C}$, the pressure inside the cooker will rise according to the saturation vapor pressure curve. The saturation pressure is much more important a factor compared to the hydrostatic pressure from water.

Pressure inside the cooker will rise according to the saturation vapor pressure curve with increasing temperatures. However, for safety reasons, a safety valve is embedded in the cooker to release pressure above a certain level. This safety valve will limit the pressure to a constant maximum level and a corresponding boiling temperature according to the saturation vapor pressure curve.

In terms of modeling pore pressure in heated concrete, the concept of the pressure cooker is adopted. Similar to the cooker, water is embedded inside a closed system, which is either the steel shell for the cooker or a dense cement layer enclosing a pore.

The main difference between the pressure cooker and concrete is the reduction of high pressure. While the pressure cooker releases vapor and reduces the high pressure via the safety valve, the concrete is considered as a porous media with a given temperature-dependent permeability, which enables a continuous release of vapor. An additional possibility reducing high pressure in HPC and UHPC mixes are PP fibers added to the concrete mix, which increase the permeability and hence the reduction of pressure.
General concept of analytical model

Based on the analogy to the pressure cooker, the first basic and general ideas for an analytical model of explosive spalling were presented in 2011 [70]. Similar to the model by Akhtaruzzaman [10], a pore is considered to be a hollow sphere embedded in the dense concrete matrix. This pore is partly filled with moisture and the pressure inside the pore will rise with increasing temperatures according to the saturation vapor pressure curve.

This proposed basic model neglects the interconnectivity of pores as well as any migration processes. Failure due to high pressures will always occur at a specific temperature and pressure. However, this is in clear contrast to the observations from tests, where specimens that did not spall were observed after slow heating.

As a first thought it was assumed that a sufficient interconnectivity of the pores is achieved by capillaries that enable migration processes [70]. However, the analysis on the porosity and average pore diameter (see chapter 3.3.7) did not show that the interconnectivity of the pores is increased with increasing temperature, in particular with the decreasing average pore radius noticed in the tests.

The reduction of high pore pressure is mainly based on the permeability of the concrete, which changes at higher temperatures, as concluded from the tests and discussed in detail in chapter 3.3.6. The reduction of pressure and the migration of vapor inside the concrete at high temperatures depend on the flow of vapor inside concrete; hence the concrete is taken as a porous medium with a given permeability.

Concrete modeled as hollow spheres in porous media

Concrete pores considered as hollow spheres with a specific degree of saturation are still assumed to be present inside the concrete. However, in contrast to the first model, these pores are not embedded in dense concrete but inside a porous medium with a given permeability. Figure 36 illustrates this revised model to estimate the pressure development inside concrete at high temperatures.

Combining the idea of pores as hollow spheres and the surrounding concrete as porous media into one model has several advantages. The pressure development inside these pores still follows the governing thermodynamic laws, since the spheres are considered to be closed systems. Further, moisture migration leading to a different degree of pore saturation is also enabled between the pores.

The analytical model can be split into two main parts: a “thermodynamic part” to estimate the pressure development and a “permeability part” covering migration processes due to pressure gradients in porous media.
General procedure for the model on pore pressure development

The general principle of the model is simplified as follows:
- Pressure rises inside the pores according to the temperature and thermodynamic laws. As long as sufficient moisture is available, pressure increases with temperature.
- Pressure gradients between the pores lead to migration vapor processes (movement of water as liquid is neglected due to the high viscosity).
- Migration processes include losses in vapor to the outside.
- Cooler sections are subjected to an increased amount of moisture due to vapor migration towards these areas with lower pressure.
- Once a pore becomes dry, the pressure decreases, even though the temperature in this pore still increases.
- This enables moisture migration again towards areas of higher temperature, due to the pressure gradient in the direction of the dry areas.
- Failure occurs if the stresses due to pressure exceed the tensile strength of the concrete.

Governing concrete parameters to be included in the model

The model needs to include at least the following governing parameters which were analyzed in tests:

- The heating rate is essential since it influences the vaporization front within the cross section and the production of vapor. In addition, higher heating rates lead to higher temperatures and pressure gradients.
- The permeability is another key issue, since the release of pressure and moisture migration mainly depends on the permeability.
- Initial moisture content in combination with the porosity of the concrete is essential since the temperature-dependent pressure development according to thermodynamic laws is based on the degree of pore saturation.
- The tensile strength of the concrete as resistance against spalling.
4.2 Analytical model for explosive spalling

4.2.1 Introduction

Analytical models of explosive spalling were presented and discussed in chapter 2.4. The presented models consider a high pore pressure as the governing factor leading to explosive spalling, while thermal stresses were usually neglected. However, most models were calibrated with results from standardized fire tests on concrete specimens in which thermal stresses have a significant influence on the likelihood and extent of explosive spalling.

The tests on HPC and UHPC cylinders as described in the technical report [75] and analyzed in chapter 3.3.3 showed different fracture patterns, which are related to the development of pore pressure inside the specimen located at different depths from the heated concrete surface. The new analytical model of explosive spalling must be able to predict the observations from tests in terms of fracture patterns obtained upon spalling. The aim is the development of the analytical model is to assess the likelihood and extent of the explosive spalling of concrete at high temperatures due to high pore pressure.

In the following, all required material properties for this analytical model are given. Material properties are mostly derived from own tests [75].

The model indicates if a concrete exposed to high temperatures tends to explosive spalling in general and gives details on the probable time span when spalling is expected. The depth in the concrete at which spalling is most likely to be expected is also indicated.

4.2.2 General concept of the analytical model

Several observations and requirements from tests lead to an engineer-related model for explosive spalling. First, the concrete is divides into small segments from the fire-exposed surface to the cooler inner parts as a solid homogeneous material with a specific amount of pores, distributed homogeneously over each segment. It is assumed that these pores are filled with water and water vapor to a homogenous level. Water vapor can migrate between each of these segments according to the concrete’s permeability and the pressure difference between each segment.

In order order to provide a hydrothermal model, a one-dimensional model as presented in Figure 37 is chosen. This simplified model is based on the concept that a concrete specimen is heated from one side (left), while the temperature of the opposite side of the specimen (right) remains at \( T = 20^\circ C \). Temperature is computed for every segment and each time step. In addition, the modeled concrete section is divided into several smaller segments with a width \( \Delta l_{\text{segment}} \) and a corresponding cross section of \( \Delta A_{\text{segment}} \) of each segment. It is assumed that each segment has the same material properties and same initial conditions, mainly in terms of porosity and moisture content. The total length of the concrete section varies according to specific requirements.
Hydrothermal model for explosive spalling

Figure 37: General layout of one dimensional model on spalling due to high pore pressure

The overall concept of an analytical model on explosive spalling including the main computing steps is shown in Figure 38 as a simplified flowchart. The analytical part of the model is divided into three main parts (Temperature-dependent input parameters / Thermodynamic pressure development / Vapor flow in porous media).

The temperature in each segment and the temperature-dependent input parameter are computed first for each time step. The second part covers the development of pore pressures according to the temperature, moisture content and porosity of each concrete segment based on the governing thermodynamic laws. The third part of the model considers the moisture migration due to pressure gradients and the permeability of the concrete. Several additional checks on intermediate data were made, which are explained in detail in the following chapters.
Figure 38: Simplified flowchart of analytical model

All steps shown in Figure 38 are run for each concrete segment and time step. The main required input parameters are:
- Width and size of cross section of the concrete segment ($\Delta l_{\text{segment}}$ and $\Delta A_{\text{segment}}$)
- Boundary conditions (i.e. $Q_{\text{in}}$ or $Q_{\text{out}} = 0$ m$^3$/s) and initial temperature (i.e. $T_0 = 20^\circ$C)
- Temperature in each segment for each time step ($T_{j(t)}$)
- Permeability based on vapor viscosity of the concrete as a function of temperature ($k_{v(T)}$)
- Porosity of concrete as function of temperature ($\varepsilon_{(T)}$)
- Initial moisture content ($m_{\text{w,t=0}}$) and losses in weight due to dehydration at high temperature ($\Delta m_{\text{w(T)}}$)

Based on this flowchart, a detailed stepwise explanation of how to determine the pore pressure in each concrete segment is given in appendix G. All required equations are given in this appendix.
4.2.3 Simplifications and assumptions

In order to develop a basic analytical approach for modeling the explosive spalling of concrete due to high pore pressure, the following assumptions and simplifications are made:

- Thermal- and load-induced stresses during heating are neglected. The model focuses only on spalling due to high pore pressure.

- The analytical model is limited to the first occurrence of explosive spalling. Spalling leads to an increased crack growth and the propagation of cracks. The changes of several boundary conditions are not taken into account, like the exposure to a higher temperature in deeper sections of the concrete’s cross section after spalling.

- As an initial condition \( t = 0 \), it is assumed that the temperature inside the concrete is constant over the entire cross section and well below a temperature of \( T << 100^\circ\text{C} \). \( (T_0 = 20^\circ\text{C}) \)

- A monotonous increase in temperature in all sections of the modeled concrete section is assumed.

- The model consists of a chain of horizontal one-dimensional concrete segment with several pores as shown in Figure 37. These sections are assumed to remain constant in volume and dimension.

- The modeled concrete assumes to be a homogeneous porous medium, which allows the transport of moisture in the form of vapor depending on the permeability of the concrete. The movement of water as liquid inside the concrete is ignored, because of the very high viscosity of water compared to vapor.

- Boundary conditions in terms of possible release or restrained migration of vapor via both free edges can be adjusted to specific needs.

- Cracks leading to an increased reduction of pressure and migration of water as liquid are not considered.

- The thermodynamic laws are considered in this system.

- Vapor is considered to be an ideal gas.

- Each segment is considered to be “thermally thin”; hence local temperature differences inside the segments can be neglected. Temperature inside each segment is constant within one time step and depends on the time only.

Explanations on chosen simplifications

The movement of water as liquid is ignored due to the high viscosity compared to the viscosity of vapor. Even though this difference decreases significantly with temperatures exceeding \( T = 350^\circ\text{C} \), this simplification has no influence on the results of the model. At concrete temperatures of \( T \approx 350^\circ\text{C} \) or more, the saturation pressure of water / vapor inside the concrete would be as high as \( p = 16.5 \text{ MPa} \), which would exceed the tensile strength of concrete by far and spalling will occur.

The calculation of the loss in mass of concrete during heating it is assumed that the cement is fully hydrated in the cold state. With a w/c ratio of 0.3 or lower, as used for the concrete mixes in all tests (chapter 3), this
is not the case. However, when the concrete is exposed to temperatures of T = 150°C and more, it is assumed that the cement paste immediately starts to decompose and no delayed hydration of the cement takes place. This simplification is used in other models, e.g. Dwaikat [33], and accords with the simplifications for the dehydration of concrete by Bažant and Kaplan [17].

The analytical model is based on time steps. It is considered that the migration of vapor according to the permeability and changes in the pore pressure only occur within the time step. An isothermal and isochoric state is assumed in the different concrete pores at for each time step and all thermodynamic laws apply. This is justified for small time steps as they are used in the model.

### 4.2.4 Discretization of the model

The general layout of the analytical model is shown again in Figure 39 including the model’s process with all sequences is shown in the flowchart (Figure 38). The modeled part of a concrete structure (section) is divided into several segments with the same temperature-dependent material properties. These segments “j” are numbered, starting at the segment directly exposed to the heat with j = 1.

The length of each segment $\Delta_{\text{segment}}$ is chosen according to specific needs. However, a constant length along the modeled concrete segment is recommended and was found to be favorable in terms of the precision of the modeled results.

![General layout of the 1-D model](image)

**Figure 39:** General layout of the 1-D model

**Discretization for different geometries**

The concept of a 1-D model as shown in Figure 39 allows to model most common structures like concrete slabs or columns. Moisture migration is only possible in one direction between the “outside” and “inside” of the modeled segment as shown in Figure 39. Moisture migration perpendicular to this direction is not considered.

Moisture migration is related to temperature-dependent pressure gradients; hence no migration will occur along isotherms. This would be the case for a slab exposed on two sides or a round column exposed on all sides. Moisture migration is not expected along the isotherms due to the lack of temperature and pressure gradients. However, temperature gradients between the isotherms lead to migration processes as shown in general model in Figure 39.
Figure 40 (left) shows the discretization of a slab exposed on one side. In this case an entire section exposed to heat on one side stretching to the cold surface can be modeled as a 1-D model. The different isotherms from the heated surface to the cold surface are parallel to each other. Boundary conditions are not required, since vapor migration is possible to both sides, even to the outside.

In terms of the cylinder exposed from all sides as shown in Figure 40 (middle), one segment from the heat-exposed surface to the center (due to radial symmetry) can be modeled, since this section follows the maximum temperature gradient.

Boundary conditions must be defined according to the modeled segment. At the inner segment in the center core, moisture migration further inside is not possible due to the temperature and pressure axes of symmetry.

Structures heated from three sides such as beams (Figure 40 (right)) are difficult to model, since temperature gradients in two directions exist and no clear axis of temperature symmetry can be found. A segment from one corner and tilted by 45°C might be chosen for modeling. However, boundary conditions in the inner section are difficult to define. In this case 1-D modeling is not possible.

A modeled section in the center-line of the beam does not cover the corners with peak temperatures and temperature gradients where the occurrence of spalling is most likely due to the temperature exposure from two sides.

**Adjustments for models with change in the size of the segment’s cross section**

In most cases, a constant length of each segment of $\Delta l_{\text{segment}}$ and cross section $\Delta A_{\text{segment}}$ depending on the geometry of the modeled concrete segment can usually be chosen. This applies for concrete slabs or columns as discussed above. In these cases, the structure can be modeled as an ordinary 1-D structure.

Very uncommon structures (i.e. spheres or thick shells) would require additional adjustments to the size of the section’s cross section $\Delta A_{\text{segment}}$ of each segment. Figure 41 shows these adjustments for a sphere.
In terms of cylinders, usually the influence of changes in the segment’s cross section does not have to be considered. It was observed in tests that this influence is small if a sufficient number of segments with a small length $\Delta l_{\text{segment}}$ of each segment are chosen.

**Figure 41:** Discretization of a concrete sphere with changes in the segment’s cross section $\Delta A_{\text{segment}}$

\[ \Delta A_{\text{segment}} \text{ j=1} \gg \Delta A_{\text{segment}} \text{ j=2} \]

**Chosen dimensions of segments**

Based on several models, the following dimensions for $\Delta A_{\text{segment}}$ and $\Delta l_{\text{segment}}$ were found to be reasonable:

Concrete cylinders with $\varphi = 150$ mm:
- Modeling half the diameter with boundary condition at the inner section (no inward flow), $Q_{\text{in}} = 0 \text{ m}^3/\text{s}$
- $\Delta A_{\text{segment}} = 1.0 \text{ m}^2$ (remains constant)
- $\Delta l_{\text{segment}} = 2.0 \text{ mm}$
- Total of 37 segments

Concrete slabs with $h = 100$ mm, exposed on one side:
- Modeling the entire height of the slab without boundary conditions
- $\Delta A_{\text{segment}} = 1.0 \text{ m}^2$
- $\Delta l_{\text{segment}} = 2.5 \text{ mm}$
- Total of 40 segments

**Length of time steps**

In terms of time steps, the sequences as shown in Figure 38 are repeated for each time step “$t_i$”. In order to achieve reliable results and model a realistic interconnectivity of the segments small time steps are required. Large time steps lead to unreliable results, since large amounts of moisture migrate between time steps, which might exceed the actual moisture content. This must be considered when modeling higher heating rates, since these require smaller time steps. However, small time steps will lead to a large amount of data. To start with time steps of $\Delta t_i = 1 \text{ s}$ are chosen.

**4.2.5 Concrete temperature**

The temperature of the concrete exposed to higher temperatures can be computed using the thermal conduction laws by Fourier as shown in equation (4 - 1).
\[ c(T) \cdot \rho(T) \cdot \frac{\partial T}{\partial t} = \lambda(T) \cdot \left( \frac{\partial^2 T}{\partial z^2} \right) \] 

with:
- \( T \): temperature in K
- \( c(T) \): temperature-dependent specific heat capacity in J/(kg·K)
- \( \rho(T) \): temperature-dependent density in kg/m\(^3\)
- \( \lambda(T) \): temperature-dependent thermal conductivity \( \lambda \) in W/(m·K)
- \( t \): time in s
- \( z \): position coordinate inside the concrete (concrete depth) in m

Data for temperature-dependent specific heat, density and thermal conductivity can be found in EN 1992-1-2 [6]. Figure 42 summarizes these values. These material parameters lead overall to good results when compared to temperatures from real fire tests. For HPC and UHPC the upper limit for thermal conductivity \( \lambda \) should be used for a better temperature prediction [72].

The concrete’s specific heat \( c_p \) exhibits a peak between the temperatures of 115°C and 200°C that is caused by evaporating water within this temperature range. This peak is more or less pronounced depending on the moisture content of the concrete. In terms of the heat analysis, this value was chosen according to the initial moisture content of the concrete as shown in Table 21. Figure 42 (left) shows the specific heat \( c_p \) of the concrete with a moisture content of 1.5% leading to a peak of \( c_p = 1470 \) J/(kg·K).

![Figure 42: Temperature-dependent material properties for HPC and UHPC according to EN 1992-1-2](image)

<table>
<thead>
<tr>
<th>Temperature in °C</th>
<th>Specific heat in J/(kg·K)</th>
<th>Density in kg/m(^3)</th>
<th>Thermal conductivity in W/(m·K)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2000</td>
<td>1000</td>
<td>0.0</td>
</tr>
<tr>
<td>300</td>
<td>1400</td>
<td>500</td>
<td>0.4</td>
</tr>
<tr>
<td>600</td>
<td>1000</td>
<td>250</td>
<td>0.8</td>
</tr>
<tr>
<td>900</td>
<td>800</td>
<td>100</td>
<td>1.2</td>
</tr>
<tr>
<td>1200</td>
<td>600</td>
<td>50</td>
<td>1.6</td>
</tr>
</tbody>
</table>

In terms of modeling, the temperatures at different depths in the concrete can be estimated numerically with the help of the Finite-Element Method. The Finite Element (FE) program ABAQUS was used to determine the temperature \( T_{ij} \) in each segment \( j \) starting with \( j = 1 \) at the heat exposed surface of the modeled concrete specimen at each time step \( t \). As initial temperature \( T = 20°C \) at \( t = 0 \) was chosen.

This leads to tabulated data for the temperature for any position and any time of the modeled segments \( T_{ij} \).
In the following, explanations of the analytical model, mainly temperature-dependent functions, are given. However, temperature T, time step t, and location j are linked.

4.2.6 Pore pressure development due to high temperatures

Pressure development according to the saturation vapor pressure curve

The tests on concrete cylinders as described in chapter 3.3.3 and discussed in terms of pressure buildup in chapter 3.3.8 showed a correlation between the expected pore pressure inside the concrete upon spalling and the saturation vapor pressure curve. In addition, the literature review on tests investigating pore pressure inside the concrete [96] have shown that the increase in pore pressure with increasing temperature is generally in good agreement with the saturation vapor pressure curve.

The pressure development in a closed system according to the saturation vapor pressure curve is a temperature-dependent function only. Pores inside the concrete are considered as closed systems in which the thermodynamic laws apply. However, saturation pressure is only established if a sufficient amount of water is available. Both water as vapor and water as condensate must be available within the concrete pores at all temperature levels up to \( T = 374.15^\circ C \). Regarding the thermodynamics, three different stages of a “gas-steam-mixture” are considered:

- Unsaturated vapor stage: In this case, the water evaporates completely before it actually starts boiling. The pressure inside the pores follows the temperature-dependent pressure development for an ideal gas in an isochoric state. With increasing temperatures, an increase in pressure can be observed. However, this pressure is far below the saturation pressure curve. This stage can only be found in almost dry pores where only vapor is present, independent of the temperature.

- Saturated vapor stage: Moisture is available in the concrete pores as water and as vapor at all temperatures. The amount of water vapor and condensed water are balanced according to the thermodynamic laws. The pressure development follows the saturation pressure curve as shown in Figure 43.

- Oversaturated vapor stage: This stage is similar to the saturated vapor stage with the only difference that more water than vapor is available in the pores.

As previously mentioned, the vapor saturation pressure curve is defined up to a maximum temperature of \( T = 374.15^\circ C \) and a corresponding pressure of \( p = 22.1 \) MPa. Exceeding this temperature will cause “overheated steam” and is associated with a high expenditure of energy. In terms of pore pressures in concrete it is most unlikely that pressure is as high as the maximum pressure of \( p = 22.1 \) MPa, since spalling will occur well in advance. In addition, most pores are already dry at this temperature.
Hydrothermal model for explosive spalling

Figure 43: Saturation vapor pressure curve

Specific pore volume of concrete

The interaction of water and water vapor inside the concrete pores can be described according to the thermodynamic laws. These physical state variables of water depend on the temperature, the specific volume of the concrete pores and the total mass of moisture inside the pore system.

According to thermodynamics, the amount of water and water vapor can be estimated first by determining the specific volume \( v \) in m\(^3\)/kg of the system as expressed in equation (4 - 2). The total pore volume and mass of moisture refers to a single concrete segment with the dimensions of \( \Delta l_{\text{segment}} \) and \( \Delta A_{\text{segment}} \). The specific pore volume of the concrete pores changes with increasing temperatures as discussed in chapter 3.3.7. It should be noted that the specific pore volume is independent of the average pore radius or the number of pores per segment. The equation can be reduced to the following:

\[
v(T) = \frac{\varepsilon(T) \cdot \Delta l_{\text{slice}} \cdot \Delta A_{\text{slice}}}{m_w(T)} = \frac{\varepsilon(T) \cdot (\Delta l_{\text{slice}} \cdot \Delta A_{\text{slice}})}{m_w(T)} \quad \text{[m}^3\text{kg}^{-1}] \quad (4 - 2)
\]

With:

- \( \varepsilon(T) \) porosity of concrete as a function of temperature
- \( m_w(T) \) temperature-dependent total mass of the moisture (liquid and vapor) inside concrete in kg

Porosity as a temperature-dependent function

The porosity \( \varepsilon(T) \) of the concrete as required in equation (4 - 2) is one governing parameter influencing the model. The porosity was analyzed experimentally at different temperature levels and the major results were discussed in chapter 3.3.7. Table 19 of chapter 3.3.7 summarizes the test results.

It is noted in chapter 4.2.3 on assumptions and simplifications that the proposed model of explosive spalling does not take cracking into account and is only applicable until first spalling occurs. Due to this limitation, the analyzed porosity after spalling as shown in Table 19 is not taken into account in the model. The temperature-dependent porosity \( \varepsilon(T) \) of the concrete can be estimated by linear interpolation between the two measured test results as shown in equation (4 - 3) for a temperature range up to \( T = 500°C \).
Analytical model for explosive spalling

\[ \epsilon(T) = \begin{cases} 
\epsilon(20^\circ C) & \text{for } T \leq 100^\circ C \\
\frac{\epsilon(600^\circ C) - \epsilon(20^\circ C)}{400} \cdot (T - 100) + \epsilon(20^\circ C) & \text{for } T > 100^\circ C 
\end{cases} \]

**Degree of water / vapor saturation**

As a next step, the vapor content \( x \) within the limits of \( 0 \leq x < 1 \) can be estimated as shown in equation (4 - 4). This value for \( x \) is defined for a range between \( x = 0 \) if a system is completely saturated with water as liquid and \( x = 1 \) if only vapor is available. The pressure according to the saturation vapor pressure curve is achieved if the vapor content is within this limit. By rearranging equation (4 - 4), the mass of vapor inside the pores can be calculated.

\[ x = \frac{v' - v'}{v'' - v'} = \frac{m''}{m' + m''} = \frac{m''}{m_w} \]  
\[ m'' = x \cdot m_w \]  

With:

- \( x \) vapor content
- \( v \) specific pore volume in m\(^3\)/kg
- \( v' \) specific volume of the water in m\(^3\)/kg (tabulated data)
- \( v'' \) specific volume of vapor in m\(^3\)/kg (tabulated data)
- \( m' \) mass of water (liquid) in kg
- \( m'' \) mass of vapor in kg
- \( m_w \) mass of total moisture

The vapor content \( x \) depends on the ambient temperature. Both parameters for the specific volume of water and vapor change with temperature. Values are provided in tabulated charts [113, 130] in steps of \( \Delta T = 1 \) K according to the industrial standard IAPWS-IF97.

Figure 44 shows the development of the specific volume of water \( v' \) and water vapor \( v'' \) at different temperatures. In addition the density of vapor \( \rho'' \) is also shown.
The vapor content $x$ is defined for the range of $0 < x < 1$. However, during modeling values outside this range were observed. If the vapor content exceeds $x > 1$, the vapor is not saturated and the pressure does not follow the saturation vapor pressure curve. For values of $x < 0$, the pore would prevent more moisture than their actual capacity. Obviously, this cannot be the case. This might happen if a lot of moisture migrates into one cell. In this case, the moisture transport is limited. This boundary condition is explained in the permeability part of the model.

**Pressure at temperatures of $T < 100^\circ C$**

It should be mentioned that at temperatures of $T < 100^\circ C$, the pressure according to the saturation vapor pressure curve is lower than the ambient air pressure of $p \approx 101325 \text{ Pa}$. In this case, the air pressure can be considered inside the concrete pores. This ambient air pressure is also present if the pores are drying out and no moisture remains inside the system ($m_w = 0$).

**Pressure development for unsaturated vapor stage**

One additional state has to be considered. It was mentioned that the concrete pores dry out with increasing temperatures, depending on the permeability of the concrete. In this case, the moisture content inside the concrete is below the required minimum to reach the values of pore pressure according to the saturation vapor pressure curve. In this unsaturated vapor stage and at temperatures exceeding $T > 100^\circ C$, only vapor will remain inside the pores. This is usually observed if the coefficient $x$ for vapor content exceeds $x > 1$. In this case, the pressure can be computed according to the ideal gas laws according to equation (4 - 6), since vapor inside concrete can be assumed to behave as an ideal gas [92]. For a single time step, an isochoric state with a constant pore volume can be considered.

$$p(T) = \frac{R_s T}{v} \left[ \frac{N}{m^2} \right]$$

(4 - 6)

With:
- $p$ pore pressure in N/m$^2$
- $R_s$ specific gas constant for vapor with $R_s = 462 \text{ J/(kg·K)}$
- $T$ absolute temperature in K
- $v$ specific pore volume in m$^3$/kg

**Summarized pressure development for heated concrete according to vapor content**

With these considerations, the pore pressure inside the concrete can be computed according to the degree of saturation of concrete pores and the temperature and is defined as in equation (4 - 7).

$$p(T) = \begin{cases} 
101325 \left[ \frac{N}{m^2} \right] & \text{for } T \leq 100^\circ C \text{ or } m_w = 0 \\
\text{saturation vapor pressure} & \text{for } T > 100^\circ C \text{ and } 0 \leq x \leq 1 \\
\frac{R_s T}{v} \left[ \frac{N}{m^2} \right] & \text{for } T > 100^\circ C \text{ and } x > 1
\end{cases}$$

(4 - 7)
The pore pressure development determined by equation (4 - 7) must be carried out for each time step \( t \) and for each segment \( j \) of the model as explained in chapter 4.2.4. This enables the prediction of the pore pressure at each individual depth of the modeled concrete segment and indicates the position of failure according to the criterion described in chapter 4.2.9.

### 4.2.7 Moisture content at high temperatures

High pore pressure is caused by evaporation of moisture inside the pores of the concrete. Apart from the initial moisture content inside the concrete pores, the release of physically- and chemically bound water causes constant migration of moisture towards the concrete pores, even at high temperatures. This promotes the buildup of pore pressure. The dehydration and decomposition of concrete at high temperatures was discussed in Table 1.

#### Initial moisture content and density

The initial moisture content was analyzed by drying concrete specimens at \( T = 105^\circ\text{C} \) to a constant mass as described in the test report [75]. Table 24 summarized the results from tests on the density of the concrete and initial moisture content.

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>V4</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>2280</td>
<td>2422</td>
<td>2414</td>
<td>2425</td>
<td></td>
</tr>
<tr>
<td>M2</td>
<td>2332</td>
<td>2494</td>
<td>2492</td>
<td>2476</td>
<td></td>
</tr>
<tr>
<td>M3</td>
<td>2400</td>
<td>2593</td>
<td>2541</td>
<td>2531</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Initial moisture content ( \phi_0 ) in % by mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>1.22% 1.03% 1.06% 0.90% 1.05%</td>
</tr>
<tr>
<td>M2</td>
<td>3.17% 2.31% 2.49% 2.72% 2.67%</td>
</tr>
<tr>
<td>M3</td>
<td>2.95% 2.32% 2.45% 2.59% 2.57%</td>
</tr>
</tbody>
</table>

With the given moisture content and density, the initial moisture content \( m_{w(t=0)} \) in kg in each concrete segment can now be determined using equation (4 - 8). This term is then used to determine the initial specific volume of the concrete pores as shown in equation (4 - 9) and based on equation (4 - 2).

\[
m_{w(t=0)} = \phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice} \quad [\text{kg}] \tag{4-8}
\]

\[
v(t=0) = \frac{\varepsilon_0}{\phi_0 \cdot \rho_0} \quad [\text{m}^3/\text{kg}] \tag{4-9}
\]

With:

- \( \phi_0 \) initial moisture content in %
- \( \rho_0 \) density of water as liquid at \( T = 20^\circ\text{C} \) taken as 1000 kg/m\(^3\)
This initial moisture content \( m_{w(t=0)} \) does not change significantly up to a temperature of \( T = 100^\circ C \). Higher temperatures of \( T > 100^\circ C \) lead to release of additional physical and chemical bound water. These releases of moisture were determined by analyzing the losses in weight during heating.

**Moisture content as function of temperature**

To determine temperature-dependent changes of the moisture content \( m_w(T) \) and the corresponding influence on the saturation level of the pores, losses in weight were analyzed experimentally for all concrete mixes and are presented in the test report [75] and discussed in chapter 3.3.8.

As shown in appendix G.2, the regression curve with the two coefficients “a” and “b” shows an overall good agreement (equation (F - 9)) with the test results, while design recommendations according to EN 1991-1-2 [6] or Bažant [18] do not in general lead to satisfactory results. Equation (F - 9) can be rearranged to predict the temperature-dependent moisture content inside the concrete according to the porosity. Since these regression curves cover only the additional losses in weight after the concrete was dried to a constant mass at a temperature of \( T = 105^\circ C \), the initial moisture content has to be added leading to equation (4 - 10), which covers all temperature ranges. With this moisture content and the temperature-dependent porosity as given in equation (4 - 3), the specific volume of the concrete can be calculated, including the vapor content and hence the pore pressure. This pressure is essential to predict additional moisture migration due to pressure gradients and permeability that will lead to changes of the moisture content and hence the pressure. This is explained in the next chapter 4.2.8.

\[
m_w(T) = \begin{cases} \\
\phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice} & \text{if } T \leq 105^\circ C \\
(\phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice}) + a \cdot (T - 105)^{b} \cdot \epsilon(T) & \text{if } T > 105^\circ C
\end{cases}
\]  

(4 - 10)

With:

- \( a; b \): parameters determining the losses in mass at high temperature (Table F2)

**Increase in moisture between time steps**

For a given concrete mix and initial moisture content, any changes in the moisture content is a function of the temperature only. The available moisture content can be predicted with equation (4 - 3). The analytical model requires the difference in increase in moisture per time step \( \Delta t_i \), which can be computed as in equation (4 - 11). For the initial time step \( t_i=0 \), the initial moisture content as shown in equation (4 - 8) is taken.

\[
\Delta m_{w(T)t_i} = m_{w(T)t_i} - m_{w(T)t_{i-1}}
\]  

(4 - 11)

**4.2.8 Moisture transport in porous media**

The first part of the model covers the pressure development inside the concrete pores with rising temperature, depending on the saturation of each pore. This pressure development refers to the “thermodynamic part” as shown in the general flowchart in Figure 38.
The “permeability part” in this figure covers the moisture migration between the different segments of the model depending on the pressure, moisture content and permeability of each segment and the selected boundary conditions.

**Volumetric flow rate of vapor in porous media**

As a first step, the volumetric flow rate $Q$ in $m^3/s$ has to be determined. This flow rate is mainly based on the pressure gradient between two modeled segments; hence moisture migration might occur in both directions from the segment: Outwards to the heated surface and inwards to cooler sections.

Based on the migration processes in porous media, the volumetric flow of vapor can be defined as a function of temperature as given in equation (4 - 12):

$$Q(T) = \frac{k(T) \cdot \Delta p(T) \cdot \Delta A_{slice}}{\eta_v(T) \cdot \Delta A_{slice}} \left[ \frac{m^3}{s} \right]$$

with:
- $Q$: volumetric flow rate of vapor in $m^3/s$
- $k$: permeability based on vapor viscosity of concrete
- $\Delta p$: pressure gradient in MPa
- $\eta_v(T)$: dynamic viscosity of vapor in $(N \cdot s)/m^2$

**Pressure gradient between two segments**

The pressure in each modeled segment $j$ depends on the temperature and moisture content as given in equation (4 - 7). To predict the volumetric flow of vapor in both directions, the pressure gradient from one segment $j$ to its neighboring segments $j-1$ and $j+1$ must be determined as sketched in Figure 45. This leads to equation (4 - 13) for the pressure gradient to estimate an outward flow of vapor and equation (4 - 14) for the inward flow, respectively.

$$\Delta p_j(\text{out}) = \begin{cases} (p_j - p_{j-1}) & \text{if } p_j > p_{j-1} \\ 0 & \text{if } p_j < p_{j-1} \end{cases}$$

$$\Delta p_j(\text{in}) = \begin{cases} (p_j - p_{j+1}) & \text{if } p_j > p_{j+1} \\ 0 & \text{if } p_j < p_{j+1} \end{cases}$$

Figure 45: Pressure and pressure gradient in different segments
In terms of estimating the outward pressure gradient for the first segment $j = 1$, the ambient air pressure can be taken from the foregoing segment leading to $p_{j-1} = 101325 \, \text{Pa}$. The same applies for the inward pressure gradient for the last segment.

Both equations (4 - 13) and (4 - 14) consider a positive pressure gradient between the modeled segment $j$ and the surrounding segments $j-1$ and $j+1$. This leads to a flow of vapor only towards the segment with the lower pressure.

**Volumetric vapor flow according to pressure gradient**

These considerations based on pressure gradients lead to equation (4 - 15) and (4 - 16) for the volumetric vapor flow in both directions. In terms of modeling, this volumetric flow must be calculated for each segment $j$ and each time step $t_i$ with the corresponding temperature $T_{ji}$.

\[
Q_{j \text{ (out)}}(T) = \frac{k(T) \cdot \Delta p_{j \text{ (out)}(T)} \cdot \Delta A_{\text{slice}}}{\eta v(T) \cdot \Delta l_{\text{slice}}} \, [m^3/s] \hspace{1cm} (4 - 15)
\]

\[
Q_{j \text{ (in)}}(T) = \frac{k(T) \cdot \Delta p_{j \text{ (in)}(T)} \cdot \Delta A_{\text{slice}}}{\eta v(T) \cdot \Delta l_{\text{slice}}} \, [m^3/s] \hspace{1cm} (4 - 16)
\]

**Permeability of concrete**

To estimate the volumetric flow of vapor, the permeability as a function of temperature is required. The analysis of the permeability of concrete specimens as presented in chapter 3.3.6 showed that well-known models for the temperature-dependent development of permeability do not predict the observed test results well. In particular, the M1 UHPC mix without PP fibers showed a significant decrease in permeability in the $T = 150 - 275^\circ C$ temperature range. Since spalling was usually observed in this temperature range, this decrease in permeability must be taken into account for the analytical model on explosive spalling. The permeability of the concrete is considered a key parameter for the reliability of the presented analytical model.

In order to provide a value for the permeability of the concrete for all temperature steps the measured test data were taken. Missing values between the experimentally determined data points are interpolated linearly. Even though a linear interpolation shows the disadvantage of an unsteady development of the results since it might lead to sudden peaks in the moisture migration, this approach takes essential material properties into account.

**Mass of migrated vapor**

For the next step, the mass of transported vapor between the segments can be estimated. Only vapor will migrate between the segments. For the very first time step $t_{i=1}$, no migration process is considered. As with the equations for the volumetric flow of vapor, the mass of in- and outflowing vapor must be considered, as given in equation (4 - 17) and (4 - 18).
Analytical model for explosive spalling

\[ \Delta m_{v(out)}(T) = Q_{j(out)}(T) \cdot \Delta t_i \cdot \rho_v(T) \ [kg] \] (4-17)

\[ \Delta m_{v(in)}(T) = Q_{j(in)}(T) \cdot \Delta t_i \cdot \rho_v(T) \ [kg] \] (4-18)

with:

\( \Delta m_{v(out)} \) mass of outflowing vapor from segment \( j \) to segment \( j-1 \) in kg

\( \Delta m_{v(in)} \) mass of inflowing vapor from segment \( j \) to segment \( j+1 \) in kg

\( \Delta t_i \) time step = \( t_i - t_{i-1} \)

\( \rho_v \) vapor density in kg/m³

New actual moisture content in segment for time step \( t+1 \)

With the temperature-dependent change in moisture content as given in equation (4-11) and the additional migration process between modeled segments, the new actual moisture content \( m_{w_{t+1}} \) for each time step and a specific segment \( j \) can be calculated using equation (4-19). For improved readability, the temperature dependency index \( (T) \) was ignored in this equation. However, the functions still depend on any changes in temperature.

\[ m_{w_j(t+1)} = m_{w_j(t)} + \Delta m_{w_j(t)} - \Delta m_{v(out)_j} - \Delta m_{v(in)_j} + \Delta m_{v(out)_{j+1}} + \Delta m_{v(in)_{j-1}} \ [kg] \] (4-19)

with:

\( m_{w_j(t)} \) mass of actual moisture content in segment \( j \) at time \( t \) in kg

\( m_{w_j(t)} \) mass of moisture content in segment \( j \) at time \( t \) in kg

\( \Delta m_{w_j(t)} \) mass of additional moisture due to drying in segment \( j \) due to time step \( \Delta t \) in kg

\( \Delta m_{v(out)_j} \) mass of outflowing vapor from segment \( j \) to segment \( j-1 \) due to time step \( \Delta t \) in kg

\( \Delta m_{v(in)_j} \) mass of inflowing vapor from segment \( j \) to segment \( j+1 \) due to time step \( \Delta t \) in kg

\( \Delta m_{v(out)_{j+1}} \) mass of outflowing vapor to segment \( j \) from segment \( j+1 \) due to time step \( \Delta t \) in kg

\( \Delta m_{v(in)_{j-1}} \) mass of inflowing vapor to segment \( j \) from segment \( j-1 \) due to time step \( \Delta t \) in kg

Boundary conditions for segments exceeding the physical possible moisture content

The equation for the mass of the actual moisture content as shown in equation (4-19) covers all migration processes due to the pressure gradients in the direction towards a lower pressure.

This value of actual moisture content \( m_{w_{t+1}} \) is used to estimate the specific volume \( v_{t+1} \) according to equation (4-2) for the next time step \( t_{t+1} \). New calculations of the pressure development and moisture migration are then performed after each time step until failure or stop criteria are fulfilled. This general scheme is shown in Figure 38.

The pressure development for different temperature ranges inside the different segments of the model can be calculated. Apart from the temperature in each modeled segment, the pressure depends on the saturation level of the pores in each segment. For any vapor content inside the pores of \( x > 0 \), the pore pressure can be estimated according to equation (4-7). However, the analysis showed that some heating rates might lead to a “negative” vapor content of \( x < 0 \). It should be mentioned that the pore is saturated with water, if the vapor...
content is \( x = 0 \). A lower value would indicate that a pore would be filled to overflowing with water. This cannot be the case.

This phenomenon might be observed with any heating rate but is more likely to occur with low heating rates and low temperature gradients between the modeled concrete segments. Due to the tabulated data of the specific volume, local peaks in the excess moisture content might occur. All vapor properties are given only in temperature steps of \( \Delta T = 1^\circ\text{C} \), which might lead to poor results, in particular for small changes of the temperature between two time steps.

When modeling, this excess moisture content that leads to a negative vapor content is mainly caused by a high amount of moisture migration from one segment to another, due to high pressure gradients in just one time step.

It is difficult to judge whether this effect can be considered as a possible failure criterion. The possible occurrence of a moisture clog in cases where a high degree of pore saturation is observed and the impermeability of the concrete leads to spalling must be taken into account.

Once the values of vapor content become negative in a time step, it must be considered that the temperature increases in the next time step and hence the specific volume of the pore. This will shift the vapor content within the standard range between \( 0 < x < 1 \) and migration processes are enabled again.

Pressure development is not defined for negative values of \( x \). In the case in which the vapor content in one segment of the model becomes negative at a certain time step, the moisture migration towards this segment is limited. The excess moisture content \( m_{\text{excess}} \) is estimated that remains in the surrounding “donor” cells in order to achieve a vapor content of \( x \geq 0 \). Figure 46 shows the general scheme of \( m_{\text{excess}} \).

![Diagram of moisture content and pressure development](image)

**Figure 46:** Occurrence of “excess water” leading to a moisture overflow of a cell

This excess moisture content \( m_{\text{excess}} \) can be determined with an additional subroutine as given by equation (4 - 20).
Analytical model for explosive spalling

\[\text{m}_{\text{excess}} = \begin{cases} \frac{1}{\psi(T)} - \frac{1}{\psi'(T)} & \text{for } x < 0 \text{ according to (4 - 4)} \\ 0 & \text{for } x \geq 0 \end{cases} [kg] \quad (4 - 20)\]

with:

- \(\psi(T)\): temperature-dependent specific pore volume in m\(^3\)/kg
- \(\psi'(T)\): temperature-dependent specific pore volume of water (liquid) in m\(^3\)/kg
- \(\rho_w(T)\): temperature-dependent water density in kg/m\(^3\)
- \(\varepsilon(T)\): temperature-dependent porosity

The inverse value of \(\psi'\) represents the maximum mass of water at different temperatures per m\(^3\) if water is available \((x = 0)\). For negative values of \(x < 0\) according to equation (4 - 4), the inverse value of the specific volume \(\psi\) according to equation (4 - 2) represents the actual moisture content. The excess moisture content \(m_{\text{excess}}\) can now be estimated by subtracting the maximum possible water content in the pore volume from this value.

The excess moisture remains in the “donor” cell, which usually has a higher pressure compared to the “receiving” cell, where a moisture clog is observed.

**Boundary condition for dry pores**

Another boundary condition reflects the drying of the pore in a single concrete segment. If the actual moisture content in a pore decreases to \(m_w = 0\) due to migration, it is assumed that the pore remains dry, even though additional moisture is present in the next time step due to the dehydration of the cement paste leading to additional moisture as given in equation (4 - 10) or additional moisture migration into this segment.

In reality, this amount of moisture is very small and does not usually lead to a critical pore pressure, since the permeability increases leading to a rapid release of this additional moisture. However, due to the calculation of pore pressure in time steps, this small amount of moisture inside the pore system leads to a significant sudden jump in pore pressure in this time step. This high pore pressure then causes an increased migration process to the pores in the surrounding concrete segments in the next time step and the pressure plunges again within the next time step to the value of the ambient pore pressure. It was noticed in the analysis that this procedure is repeated every \(t = 4 - 6\) s. The choice of smaller time steps of \(\Delta t < 1\) s did not eliminate this effect completely.
4.2.9 Failure mechanism

Failure due to exceeding the tensile strength

The pore pressure in all segments and time steps can be predicted with the given equations as described previously.

In terms of failure, it is mentioned in the literature that spalling due to high pore pressure occurs if the pressure inside the concrete exceeds the tensile strength of the concrete [27, 33]. The failure mechanism due to high pore pressure and the analogy between tensile stresses and pore pressure were discussed in chapter 3.4.2.

As tensile strength of the concrete, the reduced strength is chosen according to the design criteria as summarized in Table G3.

Failure due to the formation of a moisture clog

It is usually considered in the literature that the formation of a moisture clog leads to spalling. Migration processes of vapor inside the concrete are minimized, since all pores are congested with water [26]. In addition, the migration of water at high temperatures is considerably lower and can be neglected in most cases in which the release of vapor is decreased. A high pore pressure will develop, mainly because of the high degree of pore saturation and the formation of the clog close to the heated surface. In addition, the expansion of water causes cracking that promotes spalling, as already discussed by Shorter and Harmathy in 1965 [120].

The probable formation of a saturated zone or oversaturated zone, in which the vapor content tends to negative values, has to be analyzed from case to case. In terms of the analytical model it is difficult to judge if this saturated zone leads to spalling due to high pore pressure or if a higher pore pressure is observed at higher temperatures and in other sections of the modeled concrete cross section.
4.3 Validation of the model

4.3.1 Model for validation and additional case studies

With the help of the analytical model and the specific material properties, some of the tests were checked against the theoretical pressure development inside the modeled concrete segment and the expected resistance in terms of tensile strength.

It is known that the expected pressures predicted by the model are limited to certain range of values compared to the tests, in particular for the M2 and M3 concrete mixes, since effects like microcracking of the surface are not included in the model.

Two different models were investigated:
- Concrete cylinder with $\varnothing = 150$ mm, heat exposed from all sides
- Concrete slab with $h = 100$ mm, heat exposed from one side

**Modeled concrete cylinders**

Figure 47 shows the modeled section of the tested concrete cylinder with a diameter of $\varnothing = 150$ mm. Due to the geometry of the cylinder, only one part of the cylinder from the surface to the center was modeled with a length of $l = 75$ mm. Based on the assumptions of a 1-D model, the increase in segment size due to the increasing circumference / radius ratio of the modeled concrete cylinder is not taken into account at this stage of the analysis.

As boundary condition, the inward flow of vapor from the segment located in the middle of the modeled segment was set to $Q_{in} = 0.0$ m$^3$/s. In theory, both segments in the middle of the specimen will have the same moisture content, temperature and pressure; hence a migration between these two segments is unlikely to occur.

![Diagram of modeled concrete section from regular test cylinder with $\varnothing = 150$ mm](image)

**Figure 47:** Modeled concrete section from regular test cylinder with $\varnothing = 150$ mm
Modeled concrete slabs

Figure 48 shows the modeled section of the concrete slab with \( h = 100 \text{ mm} \) and exposed to high temperatures on one side. A total of 40 segments were chosen with a constant segment length of \( \Delta l_{\text{segment}} = 2.5 \text{ mm} \). No boundary conditions apply to the surfaces, since vapor migration outside the modeled segments is not restricted. This agrees with common observations in fire tests on concrete slabs where steam is released via the cold surface.

![Diagram showing modeled concrete section with 40 segments and boundary conditions](image)

Figure 48: Modeled concrete section from slab with \( h = 100 \text{ mm} \)

The influence of a protective lining on the pressure development was analyzed in some models. An insulation cover of \( h = 10 \text{ mm} \) and \( h = 20 \text{ mm} \) was modeled. This insulation layer decreases the temperatures at the concrete surface and causes lower thermal gradients inside the concrete. The thermal properties of the lining were analyzed and the main results are summarized in the technical report [75].

**Heating rate and temperature regime:**

The temperature for each segment and each time step were determined using the finite element method as discussed in chapter 4.2.5. The following heating and temperature scenarios were included with the model:

- **Model on concrete cylinders:**
  - Linear heating with different rates between \( \dot{T} = 0.25 \text{ - } 8.0 \text{ K/min} \)
  - Increase in temperature until most outer segment reach \( T = 500\text{°C} \)

- **Model for concrete slabs:**
  - Heating according to the ISO fire curve
  - Fire endurance time of \( t = 120 \text{ min} \) for models on unprotected concrete slabs
  - Fire endurance time of \( t = 120 \text{ min} \) and \( t = 240 \text{ min} \) for models on protected concrete slabs
4.3.2 Comparison with own tests on concrete cylinders

Table 25 summarizes the main results from modeling concrete cylinders with different material properties and exposed to linearly increasing temperature. These models are compared to results from tests as discussed in chapter 3.3. Material properties are taken from the test report [75].

An in-depth discussion on these test results is given in appendix H.1.

Table 25: Validation of the analytical model for pore pressures - comparison with test results

<table>
<thead>
<tr>
<th>Model</th>
<th>Concrete mixture</th>
<th>Heating rate in K/min</th>
<th>Spalling observed</th>
<th>Plateau temp in °C</th>
<th>expected pressure in MPa</th>
<th>Spalling expected</th>
<th>Peak pressure in MPa / Temp in °C</th>
<th>distance to surface in mm</th>
<th>reduced f_t in N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>M1 - V2</td>
<td>0.5</td>
<td>no</td>
<td>260-270</td>
<td>5.11</td>
<td>no</td>
<td>2.41 MPa / 222°C</td>
<td>58</td>
<td>4.68</td>
</tr>
<tr>
<td>2</td>
<td>M1 - V2</td>
<td>0.75</td>
<td>yes¹</td>
<td>260-270</td>
<td>5.11</td>
<td>no</td>
<td>2.06 MPa / 214°C</td>
<td>74</td>
<td>4.68</td>
</tr>
<tr>
<td>3</td>
<td>M1 - V2</td>
<td>1.0</td>
<td>yes²</td>
<td>-</td>
<td>-</td>
<td>no</td>
<td>2.36 MPa / 219°C</td>
<td>74</td>
<td>4.68</td>
</tr>
<tr>
<td>4</td>
<td>M1 - V1</td>
<td>0.25</td>
<td>no</td>
<td>255-265</td>
<td>4.69</td>
<td>no</td>
<td>2.79 MPa / 226°C</td>
<td>74</td>
<td>5.1</td>
</tr>
<tr>
<td>5</td>
<td>M1 - V1</td>
<td>0.5</td>
<td>yes²</td>
<td>-</td>
<td>-</td>
<td>no</td>
<td>3.26 MPa / 234°C</td>
<td>56</td>
<td>5.1</td>
</tr>
<tr>
<td>6</td>
<td>M2 - V2</td>
<td>1.0</td>
<td>no</td>
<td>245-255</td>
<td>3.97</td>
<td>m_excess</td>
<td>0.74 MPa / 167°C</td>
<td>6</td>
<td>4.77</td>
</tr>
<tr>
<td>7</td>
<td>M2 - V2</td>
<td>2.0</td>
<td>yes²</td>
<td>250-260</td>
<td>4.33</td>
<td>m_excess</td>
<td>1.31 MPa / 192°C</td>
<td>4</td>
<td>4.77</td>
</tr>
<tr>
<td>8</td>
<td>M2 - V2</td>
<td>3.0</td>
<td>yes²</td>
<td>-</td>
<td>-</td>
<td>m_excess</td>
<td>1.52 MPa / 199°C</td>
<td>8</td>
<td>4.77</td>
</tr>
<tr>
<td>9</td>
<td>M2 - V1</td>
<td>1.0</td>
<td>no</td>
<td>220-225</td>
<td>2.55</td>
<td>m_excess</td>
<td>0.30 MPa / 135°C</td>
<td>0</td>
<td>3.45</td>
</tr>
<tr>
<td>10</td>
<td>M2 - V1</td>
<td>2.0</td>
<td>yes²</td>
<td>195-205</td>
<td>1.55</td>
<td>m_excess</td>
<td>0.32 MPa / 136°C</td>
<td>0</td>
<td>3.44</td>
</tr>
<tr>
<td>11</td>
<td>M2 - V1</td>
<td>4.0</td>
<td>yes²</td>
<td>-</td>
<td>-</td>
<td>m_excess</td>
<td>0.36 MPa / 140°C</td>
<td>0</td>
<td>3.42</td>
</tr>
<tr>
<td>12</td>
<td>M3 - V2</td>
<td>8.0</td>
<td>no</td>
<td>205-210</td>
<td>1.73</td>
<td>no</td>
<td>1.40 MPa / 195°C</td>
<td>6</td>
<td>5.2</td>
</tr>
<tr>
<td>13</td>
<td>M3 - V1</td>
<td>8.0</td>
<td>no</td>
<td>135-145</td>
<td>0.36</td>
<td>m_excess</td>
<td>0.35 MPa / 139°C</td>
<td>0</td>
<td>4.6</td>
</tr>
</tbody>
</table>

1) Spalling from inside
2) Layer-by-layer spalling
3) Temperature during moisture-induced plateau phase according to Table 13
4) According to the saturation vapor pressure curve at spalling/plateau temperature
5) Spalling (high pore pressure) or abortion of the model due to the occurrence of moisture clog (m_excess)
6) Location of peak pressure at a distance to heated surface
7) Based on the temperature at peak pressure
8) High pore pressure is expected where spalling cannot be excluded in general improved models on the tensile strength at high temperatures are required
Conclusions from comparison with own tests on concrete cylinders

The test results can summarized as follows:

- Spalling due to high pore pressure does not occur with the modeled pressure, which is in contrast to the tests. The lack of models on the tensile strength of concrete at high temperatures makes a final judgment impossible.

- The modeled peak pressure and corresponding temperature for the M1 concrete mix is lower compared to the measured plateau temperature but still within a close range. Even though the modeled results indicate no failure, spalling cannot be excluded in general due to the high pressure.

- In terms of the model based on the M2 concrete properties, no useful results for the pore pressure could be obtained. After a short time, the modeled peak pressure directly at the heated surface indicated errors in the model during iteration (presence of $m_{\text{excess}}$).

- The M2 concrete mix has high initial moisture content and a low permeability, making vapor migration almost impossible. Cracking is not included in the model, which would enable an increased moisture transport.

- The occurrence of $m_{\text{excess}}$ moisture indicating possible failure due to the probable formation of a moisture clog was noticed with the M2 concrete.

- The modeled pressure and corresponding temperature for the M3 concrete mix gives overall a good agreement with the test results.

- The M3 concrete properties show a high increase in permeability with rising temperatures. The expected peak pressure close to the heated surface is reasonable, since moisture mainly migrates outside from the specimen towards the heated surface.

- The peak pore pressure is observed close to the inner segment for the M1 concrete mix, while the outer segments are subjected to a peak pressure with the M3 concrete mix. This is caused by the moisture migration for the different permeability and confirms similar observations from tests. No useful results were achieved with the M2 concrete mix.

- Higher heating rates lead to more useful results, due to the better use of tabulated data. Thermodynamic data is only provided in temperature steps of $\Delta T = 1.0^\circ C$. The model could lead to doubtful results if several iteration steps are within one of the tabulated temperature steps.

- The model shows significant drawbacks in terms of concrete mixes with high initial moisture content. However, the occurrence of $m_{\text{excess}}$ indicates possible failure due to spalling.


**Validation of the model**

4.3.3  Additional models for pore pressure in heated concrete cylinders and slabs

Table 26 summarizes the results from the additional models on pore pressure in cylinders and slabs. These models were carried out to judge on the general assignability of the model to higher heating rates like the ISO-fire curve. In addition, influences of PP fibers and protective lining on the pressure development are analyzed too.

Results from models 23 - 26 are listed twice according to the different exposure time. An in depth discussion on these test results is given in appendix H.2.

<table>
<thead>
<tr>
<th>Model</th>
<th>Concrete mix</th>
<th>Heating rate in K/min</th>
<th>exposure time</th>
<th>modeled peak pressure in MPa</th>
<th>Temp at peak pressure in °C</th>
<th>distance to surface in mm</th>
<th>reduced f_i in N/mm²</th>
<th>Spalling expected</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>M1 - V1</td>
<td>8.0 K/min</td>
<td></td>
<td>4.66</td>
<td>252</td>
<td>10</td>
<td>4.9</td>
<td>-</td>
</tr>
<tr>
<td>15</td>
<td>M1 - V2</td>
<td>8.0 K/min</td>
<td></td>
<td>4.89</td>
<td>253</td>
<td>14</td>
<td>4.7</td>
<td>pressure</td>
</tr>
<tr>
<td>16</td>
<td>M1 - V3 ³)</td>
<td>8.0 K/min</td>
<td></td>
<td>1.40</td>
<td>193</td>
<td>10</td>
<td>3.9</td>
<td>-</td>
</tr>
<tr>
<td>17</td>
<td>M1 - V4 ³)</td>
<td>8.0 K/min</td>
<td></td>
<td>1.09</td>
<td>191</td>
<td>22</td>
<td>3.9</td>
<td>-</td>
</tr>
<tr>
<td>18</td>
<td>M1 - V1</td>
<td>ISO-fire [5]</td>
<td>120 min</td>
<td>5.17</td>
<td>256</td>
<td>10</td>
<td>4.9</td>
<td>pressure</td>
</tr>
<tr>
<td>19</td>
<td>M1 - V2</td>
<td>ISO-fire [5]</td>
<td></td>
<td>4.61</td>
<td>259</td>
<td>96</td>
<td>4.7</td>
<td>-</td>
</tr>
<tr>
<td>20</td>
<td>M1 - V4 ³)</td>
<td>ISO-fire [5]</td>
<td></td>
<td>2.90</td>
<td>232</td>
<td>4</td>
<td>3.9</td>
<td>-</td>
</tr>
<tr>
<td>21</td>
<td>M2 - V4 ³)</td>
<td>ISO-fire [5]</td>
<td></td>
<td>5.33</td>
<td>268</td>
<td>0</td>
<td>3.9 m_excess</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>M3 - V2</td>
<td>ISO-fire [5]</td>
<td></td>
<td>8.09</td>
<td>286</td>
<td>4</td>
<td>3.9 m_excess</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>M1 - V1</td>
<td>ISO + 10mm lining</td>
<td>120 min</td>
<td>4.12</td>
<td>245</td>
<td>32.5</td>
<td>4.9</td>
<td>-</td>
</tr>
<tr>
<td>24</td>
<td>M1 - V1</td>
<td>ISO + 20mm lining</td>
<td></td>
<td>2.10</td>
<td>212</td>
<td>10</td>
<td>5.4</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>M2 - V1</td>
<td>ISO-fire [5]</td>
<td>120 min</td>
<td>6.41</td>
<td>281</td>
<td>0</td>
<td>2.7 m_excess</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>M2 - V1</td>
<td>ISO + 10mm lining</td>
<td>120 min</td>
<td>1.03</td>
<td>181</td>
<td>0</td>
<td>4.7 m_excess</td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>M1 - V1</td>
<td>ISO + 10mm lining</td>
<td>240 min</td>
<td>4.12</td>
<td>245</td>
<td>32.5</td>
<td>4.9</td>
<td>-</td>
</tr>
<tr>
<td>24</td>
<td>M1 - V1</td>
<td>ISO + 20mm lining</td>
<td></td>
<td>2.10</td>
<td>212</td>
<td>10</td>
<td>5.4</td>
<td>-</td>
</tr>
<tr>
<td>25</td>
<td>M2 - V1</td>
<td>ISO-fire [5]</td>
<td></td>
<td>6.41</td>
<td>281</td>
<td>0</td>
<td>2.7 m_excess</td>
<td></td>
</tr>
<tr>
<td>26</td>
<td>M2 - V1</td>
<td>ISO + 10mm lining</td>
<td></td>
<td>1.03</td>
<td>181</td>
<td>0</td>
<td>4.7 m_excess</td>
<td></td>
</tr>
</tbody>
</table>

1) Similar to test 1 on concrete slabs as presented in the technical report [75]
2) ISO-fire + 10 mm protective lining similar to test 2 on concrete slabs in the technical report [75]
3) Location of peak pressure in distance to heated surface
4) Based on the temperature at peak pressure
5) Concrete protected with PP fibers (see Table 16)
6) Spalling (high pore pressure) or abortion of the model due to the occurrence of moisture clog (m_excess)

**Conclusions from additional models for pore pressure**

These additional models cannot be checked against results from tests apart from models 25 and 26 on M2 concrete slabs. However, the results from the model give an overall impression of the general usefulness of the model for practical purposes. The main findings can be concluded as follows:

- Overall the model gave reasonable results if the concrete is exposed to higher heating rates (i.e. ISO fire exposure [5]). It should be noted that high heating rates lead to thermal stresses and cracking, with a major influence in the reduction of high pore pressures. These influences are not included in the model.
Hydrothermal model for explosive spalling

- Material properties from mixes containing PP fibers (models 16+17; 20+21) show a decrease in maximum pore pressure due to the higher permeability of the concrete. However, with a maximum pressure of $p = 2.9$ MPa (model 20) spalling cannot be excluded in general. The modeled data for pore pressure should be checked and validated with due care.

- A high moisture content combined with a low permeability as observed with the M2 concrete mix (model 21; 25+26) leads to unreliable results in terms of pore pressure development. An oversaturated state of the pore was observed even in the segments close to the heated surface, which are usually the first ones to dry due to migration into deeper sections and to the outside.

- Model 22 (M3 - V3) indicates another drawback of the model. A peak pore pressure of $p = 8.09$ MPa was modeled. However, spalling would occur long before this high pressure can build up inside the concrete. Another failure criterion, that pore pressure cannot exceed the tensile strength must be included in further models.

- Modeling the slab with protective lining (model 23+24; 26) confirmed the beneficial effect of thermal barriers against spalling. It was observed that a thick layer of lining minimizes the peak pressure. In addition, this pressure is assumed closer to the heated surface.

- Model 23 with 10 mm protection layer showed a shift of the peak pressure further inside the concrete. In the worst case, this high pressure could lead to spalling initiated in this deeper section leading to a loss of large areas of concrete cover.

- The results show that the model can be used for practical purposes to assess the probable pore pressure development in heated concrete.
4.4 Influence of input parameters on the accuracy of the model

The aim of the presented analytical model is to provide a simplified tool for estimating the likelihood of explosive spalling due to high pore pressure. The model further provides information on the possible formation of a moisture clog and shows critical temperatures and heating rates where spalling is likely to occur.

The model is based on several input parameters, which were all derived in tests. In several case studies, some of these parameters are considered to have superior influence on the accuracy of the model in general and on the development of higher pore pressures in particular.

As discussed in chapter 4.2 some of these input parameter can be taken as a constant value or can be described with simplified models, while other parameters must be included in the model with high precision. The permeability of the concrete is first to mention, since no existing model for temperature-dependent changes in permeability is able to match the results from own tests.

Table 27 gives a brief overview on the parameters that have more or less an impact on the accuracy of the model.

<table>
<thead>
<tr>
<th>Table 27: Governing parameters influencing the pressure development</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major influence</td>
</tr>
<tr>
<td>temperature gradient - heating rate</td>
</tr>
<tr>
<td>initial moisture content</td>
</tr>
<tr>
<td>permeability at high temperature</td>
</tr>
<tr>
<td>tensile strength</td>
</tr>
<tr>
<td>Secondary influence</td>
</tr>
<tr>
<td>porosity and increase in porosity</td>
</tr>
<tr>
<td>moisture due to decomposition of concrete</td>
</tr>
</tbody>
</table>

Influence of heating rate on modeled pressure development

The heating rate and corresponding temperature gradient inside the concrete have a significant influence on the pressure development. Temperature profiles can be easily estimated using numerical methods including the finite element method. Even though the moisture plateau phase is barely noticeable with this method, numerical analysis provides data of high precision, which can be directly used as input parameters.

Thermal barriers like protective linings or coatings are known to minimize temperature gradients or keep surface temperatures below a critical level. The material properties of these barriers are usually available. With this information, similar numerical studies can be performed, in order to analyze the temperature distribution inside a concrete member at different depths.

Influence of moisture content and permeability and possible small-scale testing

The initial moisture content and the permeability are two additional material properties that have a major influence on the accuracy of the model. It was shown that existing models for the permeability at high temperatures scarcely match the test data. However, both material properties can be easily achieved with sufficient precision in small-scale tests. Similar to own tests, the residual “gas” permeability of concrete can be estimated using a handheld device. To minimize the influence of free water inside to concrete pores, all
specimens have to be dried to constant mass before testing, which provides data on the initial moisture content.

As test specimens, concrete disks drilled concrete cylinders from existing structures or from additional test specimens during the concreting of new structural members can be used. The thickness of each concrete disk should be at least five times the maximum aggregate size. Cracking due to different thermal expansion along the aggregates might lead to a higher permeability.

When concreting additional test specimens, it should be noted that the main fresh concrete properties are the same when concreting additional small-scale specimens for testing and concreting large structural members. The air content for example could be higher in small-scale specimens, in particular if SCC is used, due to the lower self-weight of the specimen. Higher air content might lead to a higher permeability. The results from the analytical model could then lead to incorrect results and involve uncertainties.

**Influence of tensile strength as spalling resistance and possible small-scale testing**

The tensile strength is a key parameter of the model, since the resistance of the concrete to explosive spalling mainly depends on the temperature-dependent tensile strength of concrete. Several models as discussed in chapter 3.4.3 provide additional data on temperature-dependent losses of concrete in tensile strength at high temperatures, based on the initial cold tensile strength. However, in some cases it is difficult to judge if a specific concrete fits into just one category.

The analysis from own tests clearly showed that testing concrete in direct tension is challenging in the cold stage and was never performed at high temperatures. Determining the tensile splitting strength seems to be a promising approach that leads to more reliable results. These tests also provided the opportunity to perform first qualifying tests at high temperatures, since these tests are much easier to perform. Testing up to a temperature of $T = 300^\circ\text{C}$ is sufficient, since spalling will usually occur below this temperature. Several temperature steps are more favorable than data on permeability at very high temperatures.

**Parameters of less importance - additional moisture during decomposition and porosity**

In terms of additional moisture during decomposition, existing models might be taken. Existing models do not differ significantly from tests as observed with the methods given in EN 1992-1-2 [6]. However, data on additional losses in weight during heating can be gained easily during testing of the permeability at high temperatures, since the test specimen is heated anyway.

For the porosity, a linear distribution from $\varepsilon = 10\%$ at $T = 20^\circ\text{C}$ to $\varepsilon = 15\%$ at $T = 500^\circ\text{C}$ can be assumed for all types of concrete. Any changes to the total porosity have only a minor influence on the accuracy of the model and the pressure development in general.
4.5 Conclusions from analytical model for explosive spalling due to high pore pressure

In the past, several models of the risk of explosive spalling due to high pore pressure were developed by different authors [27, 33, 42, 94, 118, 120]. However, some of these models do not cover new types of concrete. For new, dense concrete mixes with low permeability, the following assumptions and considerations are taken into account in the development of new models.

- As a simplified analytical model on spalling due to pore pressure is hardly available. A new, basic model on the estimation of pore pressure development in concrete at high temperatures was established, based on material properties from own tests.

- Dividing the model into a “thermodynamic part” for the rise in pore pressure and a “permeability part” covering the migration processes in porous media seems to be a promising approach for the prediction of high pore pressure by an analysis in time steps.

- To model the pore pressure development in a concrete cross section, the geometry is divided into several small segments. Interconnectivity and moisture migration between the segments is enabled depending on the permeability of the concrete. Time steps of \(\Delta t = 1\) s and a segment length of \(\Delta l_{\text{segment}} = 2.0\) mm were considered in the verification examples as a compromise between precision and computing time using the model.

- The verification of the model by means of tests on spalling showed good results of the analytical model. However, the precision of the model strongly depends on the input parameters, which were all determined in tests. Tests on the hot permeability are not available. The permeability and initial moisture content were found to have strong influence on the reliability of the results.

- The model seems to give good results also for higher heating rates, even though microcracking and the corresponding influences on the permeability are neglected. Critical peak pressures and the corresponding temperatures were predicted within a close range of the temperatures observed in tests during the plateau phase. The peak pressure and the corresponding temperatures are essential for the design of thermal barriers as measures to avoid explosive spalling.

- Modeling concrete mixes with high initial moisture contents leads to a stop of the iteration of the model since the pores are becoming “overfilled” with water. In these cases the influences of cracking leading to increased migration of water and vapor must be discussed with modifications to the model.

- PP fibers decrease the peak pressure due to the higher permeability upon melting. However, some peak pressures are still within critical levels where in general spalling cannot be neglected. The permeability as input parameter for the model is based on the residual permeability based on a concrete mix with a content of 2.0 kg/m\(^3\) PP fibers. Additional tests with higher amounts of PP fibers are required and tests on the hot permeability would increase the reliability of the model.

- The permeability might be “adjusted” with an empirical parameter as a simplification, in order to achieve overall a better agreement between the test results and the model. However, including any empirical parameter without a physical basis might lead to doubtful results and limit the general adaptivity of the model. The model is only be “adjusted” on the basis of reliable test results.
5 CONCLUSIONS AND OUTLOOK

5.1 Summary of most important findings

Within the research project on the explosive spalling of concrete at high temperatures, two main mechanisms leading to spalling were presented which are discussed in the literature. Spalling is expected due to a high pore pressure, high thermal stresses or a combination of these two factors.

It is mentioned in the literature [16, 19, 20] that a high pore pressure is considered to be only the “trigger” for the explosive spalling of concrete leading to spalling due to the combination of high pore pressure and high thermal stresses. However, concrete mixes with a low permeability as observed with common HPC and UHPC mixes shifts the governing mechanisms of explosive spalling in the direction of high pore pressure. Spalling is likely to occur due to a high pore pressure if the corresponding stresses exceed the tensile strength of the concrete.

Several tests [75] on spalling by the liner heating of different HPC and UHPC mixes were discussed in detail. The main findings from these testing series can be concluded as follows:

- Tests on linear heating of concrete cylinders showed different fracture pattern of the spalled concrete parts. Lower heating rates lead to spalling initiated from sections further inside the specimen, higher heating rates lead to smaller concrete fragments after spalling.

- Moisture migration into the cooler inner sections of the concrete lead to high pore pressure in these areas. Outer parts of the specimen are becoming dry and the pressure decreases due to a lack of moisture and vapor.

- During heating, a slower increase in core temperature compared to the surface temperature was noticed as a plateau phase due to the evaporation of moisture. If spalling was observed, it occurred at temperatures before or during this plateau phase. No spalling was noticed with rising temperatures after the plateau phase, since the peak pressure occurs during this plateau phase when the moisture could evaporate.

- Each concrete mix has a specific plateau temperature depending on the permeability of the concrete. A “dense” concrete with a lower permeability shifts this plateau phase to into the higher temperature range. Adding sufficient PP fibers to the mix decreases the plateau temperature to $T = 170^\circ$C, i.e. the melting temperature of the fibers.
Conclusions and outlook

High temperatures lead to high pore pressures inside the concrete. It is mentioned in the literature [57, 68, 82] that spalling occurs if the stresses due to pore pressure exceed the tensile strength of the concrete. Based on simplifications, the stress concentration inside the concrete was discussed leading to the following conclusions:

- Pores inside the concrete are modeled by assuming hollow spheres as voids with a constant radius and distance between each other based on results from tests on mercury intrusion porosimetry.

- The pore pressure and tensile stresses cause stress concentrations close to the voids. The ratio of these stress concentration factors and according to the model is usually close to $\alpha \approx 1.0$.

- The general failure criterion that spalling occurs if the pore pressure exceeds the temperature-dependent tensile strength is reasonable.

Based on the test results and supported by the literature review [37, 97, 102], the pore pressure development inside the heated concrete was discussed, providing the following conclusions:

- Pressure increases with increasing temperatures, as long as sufficient moisture is available (saturated state), usually according to the saturation vapor pressure curve.

- Peak pore pressure is noticed at the end of the plateau phase, before the concrete dries and the pressure decreases.

- The pore pressure, based on saturation pressure during the plateau temperature phase and the corresponding temperature-dependent tensile strength as resistance were compared with the saturation vapor pressure curve. This led to the following cases for spalling due to a high pore pressure:
  - Case 1: High concrete temperatures lead to an increase in pore pressure according to the saturation vapor pressure curve. Stresses due to the high pressure exceeding the tensile strength of the concrete will lead to explosive spalling.
  - Case 2: The pressure increases according to the saturation vapor pressure curve, with a peak pressure during the plateau phase. No spalling will occur if the tensile strength as resistance is higher than the stresses due to pore pressure.
  - Case 3: The release of pressure due to a high permeability or microcracking of the concrete leads to a slower increase in pressure compared to the saturation vapor pressure curve. The peak pressure is observed at the plateau temperature before the concrete dries. Spalling will occur if the stresses due to pore pressure exceed the tensile strength. The pressure might also decrease again without spalling if the permeability is high or with a sufficiently high tensile strength as resistance.
The use of PP fibers minimizes the risk of explosive spalling, since they increase the permeability of the concrete and allows the relief of pore pressure. The results from tests can be concluded as follows:

- The fiber length and diameter have a significant influence on explosive spalling, irrespective of the total amount of PP fibers added to the fresh concrete.
- A sufficient length of the fibers is essential to ensure an interconnection of the concrete pores with fibers. Thinner PP fibers have a higher specific surface per kg fibers, which leads to an improved formation of micro channels for pressure relief.
- The workability (segregation, bleeding) is influenced with the amount and type of the used PP fibers (surface area, coating).
- The use of steel fibers has no significant influence on the spalling behavior.

An analytical model was developed based on this knowledge on explosive spalling due to high pore pressure. The aim was to model pore pressure development inside concrete with increasing temperatures and taking moisture migration into account. The concept of the model can be concluded as follows:

- The model is based on the analogy of permeable concrete with pores, similar to a pressure cooker with a safety valve.
- Concrete specimens are modelled as a one-dimensional strip divided into segments. All segments have the same initial temperature-dependent material parameter.
- The pore pressure development due to temperature and moisture migration in each segment and time step is modelled in three separate steps.
  - In the first step, the temperature and temperature-dependent material properties (i.e. porosity, permeability and additional moisture due to decomposition) are calculated for each segment and time step.
  - In the second step, the pressure according to the degree of pore saturation and thermodynamic laws is calculated.
  - As third step, moisture migration due to pressure gradients and vapor flow inside porous media is estimated based on the permeability of the concrete. This moisture migration leads to changes in the pore pressure according to the level of pore saturation and temperature. The possible occurrence of a moisture clog can be indicated.
- Explosive spalling is expected if the stresses due to the modeled pore pressure exceeds the temperature-dependent tensile strength of the concrete acting as a resistance or a possible moisture clog is indicated by “oversaturated” pores.
- The development of critical pressure inside the cross section of a concrete member determined with the current model is based on a few essential material parameters, mainly the permeability based on vapor viscosity of the concrete at high temperature, the initial moisture content and the tensile strength. Most
of the input parameters can be analyzed with known tests, either from existing structures or from additional specimens taken during construction. Standardized tests on the hot permeability have to be developed.

Some tests were checked against output from the model. These findings, including additional models, can be summarized as follows:

- The model shows overall a good agreement with the test results.

- Material properties of concrete with a high initial moisture content and a low permeability lead to spurious results with the model. Moisture migration due to pressure gradients will lead to a completely saturated state of the concrete pores. The required thermodynamic laws do not apply any more.

- The use of PP fibers decreases the peak pressure inside the concrete. However, the analysis still indicated high pressure within the concrete where spalling cannot be excluded in general. The residual permeability could only be measured which may not reflect the increase of hot permeability at peak temperature of concrete with PP fibers.

- Regarding a protective lining as thermal barrier, it was noticed that thin layers of lining only delay the occurrence of explosive spalling. Spalling initiated in deeper sections might occur at a later stage in the worst case.

- Sufficient protection in the form of a lining leads to a constant drying of the concrete without spalling and reduces the peak pore pressure. Protective layers as thermal barriers are considered with the model by using a modified (lower) temperature distribution inside the modeled concrete section as input parameter, leading to a reduced peak pore pressure.
5.2 Outlook and recommendations for future investigations

Within this work, several material properties were analyzed experimentally with regard to explosive spalling. However, some information is still not available for the development of a more sophisticated model of spalling of concrete with a low permeability.

In terms of material properties, the main focus should be put on the tests to determine the tensile strength and permeability of concrete at high temperatures. The tensile strength of concrete at high temperatures is considered as the governing failure criteria in terms of resistance to explosive spalling due to a high pore pressure. Some approaches with new models to determine the temperature-dependent tensile strength of concrete seem promising. However, data and models for UHPC are not available; hence additional tests are required, mainly on UHPC specimens, including the influence of the presence of steel and PP fibers. Tests on the permeability of concrete at high temperatures are not available and need to be developed, in particular for concrete with PP fibers.

Several revealing tests on the pore pressure measurements are documented in the literature. However, none of these concrete mixes included silica fume, which is known to decrease the permeability and lead to a higher pore pressure. It would be interesting to analyze the actual increase in peak pressure with these concrete mixes, to confirm whether the pressure development follows the saturation vapor pressure curve even for these less permeable concrete mixes. In addition, the high peak pressure levels as calculated in the analytical model were never verified with measured pore pressure in tests.

In terms of permeability, a generalized model of the permeability of concrete at high temperatures is not available at the moment. The existing models do not produce the measured test result and neglect several important findings, such as the decrease in permeability for temperatures in the range $175^\circ C < T < 275^\circ C$. These effects should be further analyzed and discussed in detail.

The permeability is required as an essential input parameter for the model and must be analyzed by means of an additional testing series at high temperatures. However, no reliable test method exists to measure the permeability of concrete at high temperature. The influence of higher amounts of PP fibers including different geometries of the fibers should be included in future tests as well as the influence of microcracking on the permeability of the concrete. This could lead to more reliable analytical results from the model of concrete with a high amount of PP fibers and high initial moisture content. The effect of water migration as a liquid should be analyzed and discussed again.

Moisture migration and the presence of a moisture clog inside the concrete are well explained in theory. However, only very few tests on the visualization of the moisture clog exist. Moisture migration inside the concrete at high temperatures should be experimentally verified.

The additional knowledge from the above mentioned tests could be used to improve the use and reliability of the developed model on pore pressure development and the prediction of explosive spalling in fire.
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## NOTATION AND DEFINITIONS

**Lower case letters**

- **a**: factor derived by integration to determine the vapor drag forces by Meyer-Ottens [94]
- **b**: width
- **c**: reinforcement cover
- **h**: height
- **l**: length

\[
\begin{align*}
  m_w & \quad \text{mass of moisture (in any combination of water as liquid and vapor as gas)} \\
  m_v & \quad \text{mass of vapor (gaseous phase)} \\
  m_l & \quad \text{mass of water (liquid phase)} \\
  m_{10} & \quad \text{mass of water loss due to dehyration} \\
  p & \quad \text{pressure} \\
  q & \quad \text{heat flux} \\
  r & \quad \text{radius} \\
  r_i & \quad \text{inner radius} \\
  r_o & \quad \text{outer radius} \\
  s & \quad \text{spacing of reinforcement bars} \\
  t & \quad \text{time} \\
  t_i & \quad \text{time step} \\
  b_{v(T)} & \quad \text{permeability correction factor according to dynamic gas and vapor viscosity} \\
  k_a & \quad \text{permeability based on gas viscosity} \\
  k_v(T) & \quad \text{temperature-dependent permeability based on vapor viscosity} \\
  k_v & \quad \text{intrinsic permeability based on vapor viscosity} \\
  k_c & \quad \text{reduction factor for tensile strength of concrete at high temperatures} \\
  v & \quad \text{velocity of movement inside concrete} \\
  v_h & \quad \text{velocity of the heat flux} \\
  v_p & \quad \text{velocity of the moisture clog according to the pressure gradient} \\
  v_{100\degree C} & \quad \text{velocity of the T = 100\degree C isotherm} \\
  v_{\text{vapor}} & \quad \text{velocity of the vaporization front} \\
  v_{\text{clog}} & \quad \text{velocity of the moisture clog}
\end{align*}
\]
Notation and Definitions

**Upper case letters**

- $F_{\text{tie}}$: longitudinal force in reinforcement ties
- $F_{\text{bond}}$: force between ties and the concrete interface
- $K_p$: stress concentration factor due to internal pressure
- $K_t$: stress concentration factor due to tensile stresses
- $T$: temperature in °C
- $\dot{T}$: heating rate in K/min
- $\Delta T$: temperature gradient, temperature difference
- $J$: energy in J/kg
- $V$: volume

**Thermodynamic terms**

- $m$: mass of moisture
- $m'$: mass of water (liquid phase)
- $m''$: mass of vapor (gaseous phase)
- $v$: specific volume of a closed system in m$^3$/kg
- $v'$: specific volume of water (liquid) in a closed system in m$^3$/kg
- $v''$: specific volume of vapor (gaseous) in a closed system in m$^3$/kg
- $x$: vapor content

**Subscript letters**

- $i$: time step
- $j$: segment number
- $t$: temperature
- $l$: load
- $p$: pressure

**Greek letters**

- $\alpha$: correlation factor between pore pressure and tensile strength
- $\varepsilon$: porosity of concrete in %
- $\lambda$: thermal conductivity
- $\eta_a$: dynamic viscosity of air
- $\eta_v$: dynamic viscosity of vapor
- $\sigma$: stress in N/mm$^2$
- $\rho$: density in kg/m$^3$
- $1/\phi$: curvature
- $\phi$: moisture content in %
- $\nu$: Poisson’s ratio
**Notation and Definitions**

**Definitions and abbreviations**

**Spalling:** The violent or non-violent breaking off of layers or fragments of concrete from the surface of a structural element during or after it is exposed to high and rapidly rising temperatures as experienced in fires. [57, 68]

**Violent spalling:** Spalling due to a high release of energy as usually observed with explosive spalling, usually with a big influence on the concrete structure. Violent spalling covers several types of spalling (explosive, surface). The amount of energy upon spalling is not defined.

**Explosive spalling:** Violent breaking off of concrete fragments at high temperatures, usually caused by:
- Insufficient release of high pore pressure [10, 94, 120]
- High thermal stresses [32, 109]
- Combination of both [27, 118, 134]

**Surface spalling:** Violent breaking off of concrete layers at high temperatures [57].

**Aggregate spalling:** Splitting of aggregates due to their decomposition or changes at high temperatures [57].

**Corner spalling:** Removal of concrete cover from corners at high temperature due to the temperature impact from two sides [57].

**Post cooling spalling:** Non-violent breaking off of concrete fragments during cooling from high temperatures [57].

**Sloughing-off:** Non-violent breaking off of concrete fragments after longer exposure to high temperatures, when concrete loses its strength [57].

**Moisture clog:** Formation of a saturated layer of moisture close to the heated surface. Spalling might occur due to the impermeability of the layer leading to the buildup of high pore pressure [120].

**Saturation vapor pressure:** Pressure curve representing the relation between temperature and vapor pressure for a closed and fully saturated system. Tabulated data are part of the industrial standard IAPWS-IF97 [112, 130].

**OPC**
Ordinary performance concrete ($f_c < 55$ MPa [4])

**HPC**
High performance concrete ($55$ MPa $< f_c < 100$ MPa [4])

**UHPC**
Ultra-high performance concrete ($f_c > 100$ MPa)

**SCC**
Self-compacting concrete
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APPENDIX A - HISTORICAL OVERVIEW ON SPALLING OF CONCRETE

A summary of the main findings from the literature from the last hundred years or so on the spalling of concrete at high temperatures is given in chapter 2.1.4 in Table 3. The following chapter presents an in-depth summary of the corresponding literature sources including further details on several scopes of research on the spalling of concrete.

First observations by Gary in the 1910’s:

The first tests on spalling were carried out and presented by Gary in 1916 [40, 41]. His work included tests on entire concrete structures. He tested concrete slabs, beams, walls, and columns with changes in the section’s geometry as well as various concrete mixes. He concluded that the presence of concrete usually increases the fire resistance of structures, since the concrete provides an adequate thermal barrier and limits the direct fire exposure of the reinforcement in most cases.

In his tests he observed the effect of concrete aggregates on the spalling phenomenon. It was observed that aggregates with a high content of chemically bound water and a high coefficient of thermal expansion increase the risk of spalling. In addition, the influence of the geometry and the cross section of the concrete member on the likelihood of spalling was observed in the case of two-sided heat exposure leading to corner spalling in concrete staircases and columns.

In addition to these tests, Gary noticed spalling in concrete with an average compressive strength of about $f_c = 21$ MPa. This concrete mix was called “granite” concrete and was found to be a very dense concrete in terms of permeability, even though the phenomenon of the concrete’s permeability was little known at that time. This occurrence of spalling was considered to be related to the relief of high internal pressures caused by water vaporization. Gary noticed that the exhaust of vapor depends on the size of the concrete’s pores and the degree of water saturation of these pores. A high pressure inside the concrete will only build up if a sufficient amount of water vapor is available. The fragment size of the average concrete part detached from the surface was about $100 \cdot 100$ mm$^2$ in length and was observed in particular in the case of highly loaded concrete sections.

In addition, a more violent, explosive spalling of concrete walls was noticed. Concrete fragments with a size of about $1.0$ m$^2$ were blasted from a concrete structure [40]. This violent type of spalling was considered as critical since it minimizes the fire resistance of the structure. However, this phenomenon of violent explosive spalling could not be explained at that time.

As a final conclusion to his work, Gary [40, 41] noted that the risk of explosive spalling can simply be minimized by providing a sufficient water and permeability, either by a modified choice of aggregates or treatment of the surface of the concrete.

First basic design criteria to minimize the risk of spalling by Hasenjäger

In 1935 Hasenjäger [45] summarized the main findings from the past and his own work. In addition to the findings concluded by Gary, Hasenjäger noticed that there are more governing factors leading to explosive
spalling of concrete in addition to the release of water vapor. According to their work, the spalling of concrete is also caused by rapid heating leading to high thermal stresses, that exceed the tensile strength of the concrete. In addition, changes in volume of the aggregates and high pore pressure caused by water vapor from the cement paste or the aggregates are mentioned as governing factors leading to spalling. His experimental findings in terms of explosive spalling could be split into two main categories, spalling by thermal stresses due to restrained dilatation and spalling due to high pore pressure.

The risk of spalling can be minimized by using concrete with low thermal expansion, overall good heat transfer inside the concrete, elasticity and high tensile strength. In terms of aggregates, only aggregates with minor temperature-dependent changes in volume should be used. In addition, Hasenjäger give advice for practical application of his findings. Expansion joints, sufficient reinforcement cover and a modified structural design are - among others - some requirements that increase the fire resistance of concrete structures used as design rules up till now [6].

**Detailed tests on spalling and general design criteria by Meyer-Ottens**

A major milestone in the general assessment of concrete spalling was presented by Meyer-Ottens [94] in 1972. He summarized all major findings from the past and presented the results of an extensive testing series on the spalling of concrete structures in fire. His work was one of the first within the new collaborative research center “SFB 148” at Braunschweig University, on the “fire behavior of concrete structures” [85]. In his extended work, he analyzed the influence of different types of aggregate, degree of reinforcement, thermal stresses or moisture content on the risk of concrete spalling. Further, he provided several design recommendations for concrete structures in order to minimize the risk of spalling by a modified design layout or rearrangement of the reinforcement. Parts of this work later led to the practitioner’s book on fire resistance of concrete “Concrete Fire Safety Handbook” by Kordina and Meyer-Ottens [86].

Meyer-Ottens analyzed the occurrence of spalling close to reinforcement bars in detail and explained this with the presence of large internal stresses due to different mechanical and thermal properties of the reinforcement bars, the surrounding concrete and different thermal expansion properties of the two components. He noticed that the use of rather large reinforcement bars (i.e. $\varnothing = 26$ mm) with little spacing led to an increased risk of spalling. Cracks due to the tensile stresses inside the concrete might overlap from bar to bar and promote spalling.

Additional tests based on concrete with an average compressive strength in the range of $f_{c} = 40 - 50$ MPa showed that no spalling will occur, if the moisture content of the concrete is below $m_{w} = 0.5\%$ in mass. However, higher moisture contents do not necessarily lead to spalling. For the concrete specimens that failed by explosive spalling, critical temperatures in inner sections of the concrete within the range of $T = 100 - 150^\circ C$ were measured in these experimental studies by Meyer-Ottens. These temperatures were linked to the evaporation of water in these inner sections of the concrete. In addition, based on own tests and a literature review, Meyer-Ottens concluded that migration processes of moisture lead to spalling close to the heated surface. Considering a concrete slab exposed to high temperatures on one side, water vapor and water will migrate within the concrete. Parts of the moisture will migrate towards the heated surface, mostly in the form of vapor due to the high temperatures. Water and vapor was also noticed on the cold side of the heated concrete. Large amounts of vapor and even “fountains” of water were observed on the cold
side of the heated slab in some tests. Meyer-Ottens considered both the permeability and pore size distribution of the concrete as major parameters influencing the risk and extent of spalling.

For a better explanation of his observations, Meyer-Ottens further analyzed the pore size distribution of concrete. He considered that all pores show a different filling level of water inside the pore at the cold stage. This water is assumed to migrate in the form of water within the large pores and cracks, while vapor migrates between the small pores with increasing temperatures, leading to different degrees of moisture saturation. However, a critical degree of saturation of the concrete pores could not be found in his tests. It should be noticed that in general even though the migration processes could be explained in theory, the migration process could not be verified by tests. Meyer-Ottens did not discuss the difference in the dynamic viscosity of water as liquid and vapor as gas, which decreases the movement of water inside uncracked concrete to a minimum [33].

Meyer-Ottens concluded that an internal friction of the water vapor along the concrete pores caused by moisture migration leads to high shear stresses within the pores. The total sum of these shear stresses then exceeds the tensile strength of the concrete, resulting in spalling. This migration process of vapor increases with increasing depth from the heat exposed side of the concrete. This leads to a higher tensile strength in deeper sections which peaks at about $h = 60 - 70$ mm from the surface. Rapid heating and decreasing height of the concrete’s cross section promotes the development of critical stresses and enhances the risk of spalling.

In order to explain explosive spalling, Meyer-Ottens concluded that tensile stresses within the concrete caused by moisture migration are the primary factor causing concrete spalling. No spalling will occur without evaporating water. Further, temperature-dependent stresses and the resulting cracks contribute to the risk of spalling. However, these stresses and cracks usually do not lead to explosive spalling themselves. Spalling caused by aggregates play only a minor role. In addition to the occurrence of concrete spalling, temperature-dependent deterioration of concrete can always be observed at high temperatures and is independent of the occurrence of spalling.

The analysis of Meyer-Ottens clearly indicated two different mechanisms leading to violent, explosive spalling of concrete at high temperatures: (1) spalling caused by high thermal stresses and (2) spalling caused by the presence and migration of moisture. The term of high pore pressure was not mentioned at that time.

**Basic analysis on high pore pressure development leading to spalling by Sertmehmetoglu**

Sertmehmetoglu discussed in 1977 [118] the mechanisms of concrete spalling. He performed tests to investigate the pore pressure of concrete at high temperatures. A concrete with an average strength in the range of $f_c = 38 - 45$ MPa was chosen and pressure transducers were attached to the specimen to measure the pressure at the unexposed side. However, the pressures measured during these tests were around $p = 2.1$ N/mm², which is usually insufficient for spalling to occur since the tensile strength of the concrete is about $f_t = 3.0$ N/mm² including temperature-dependent losses in strength. However, it was noticed by visual inspection that the presence of cracks inside the concrete parallel to the heated surface on loaded specimens seems to increase the likelihood of spalling due to high pore pressure.
Appendix A ‐ Historical overview on spalling of concrete

Sertmehmetoglu agreed with Meyer-Ottens’ findings that spalling only occurs if a sufficient amount of water is available. Sertmehmetoglu then carried out case studies using an analytical model of spalling based on moisture migration due to Shorter and Harmathy [120], using material properties that he obtained from own tests. As concluding remarks, he clearly stated that “spalling occurs due to the generation of pore pressure” [118] that with a low permeability of the concrete might lead to an increased risk of spalling. The model by Shorter and Harmathy [120] presents a prediction of “moisture clog spalling” of concrete at high temperatures. During heating concrete on one side, moisture migration to the heated surface as well as migration further inwards to cooler concrete sections is considered to take place. This leads to a dry zone close to the heated surface followed by a saturated zone, since vapor condenses in cooler sections leading overall to a higher moisture content. This high moisture content will build up a critical pressure with increasing temperatures leading to spalling at the interface between the dry and saturated concrete zones. These general ideas in terms of moisture migration within heated concrete are discussed in detail in chapter 4.

Theoretical predictions of pore pressure buildup by Bažant

Systematic theoretical predictions together with the numerical modeling of the drying of concrete at high temperatures to investigate the corresponding buildup of pore pressure were presented by Bažant in the late 1970’s [19, 20]. He presented a numerical model based on the permeability, water content and various thermal properties of the concrete. The model predicted the time- and temperature-dependent pore pressure development and moisture clog movement for rapidly heated concrete at different distances from the heated concrete surface. Furthermore, different concrete boundary conditions like a sealed concrete surface due to a steel liner are included.

Development of HPC and use of silica fume by Hertz and Jumpannen

Hertz was one of the first researchers studying the influence of silica fume added to the concrete for an increased compressive strength. In his 1984 [47] report it is stated that spalling was noticed on concrete cylinders even with low heating rates of \( \dot{T} = 1.0 \text{ K/min} \), at around \( T = 300°C \) furnace temperature. Hertz observed that concrete containing silica fume is very dense in terms of permeability and minimizing the release of vapor. This could be shown when drying concrete samples at \( T = 105°C \). A loss in mass of 1.2% was measured after a period of \( t = 7 \text{ d} \). Higher temperatures of about \( T = 300°C \) caused a rapid loss in weight of 3.0% after a short period of time of only \( t = 2 \text{ h} \) due to the increased permeability and pressure inside the concrete at the higher temperature level. Additional tests on concrete cylinders with steel fibers showed a slight improvement in spalling resistance. The spalling now occurred at temperatures of about \( T = 400°C \). The heating rate remained constant with \( \dot{T} = 1.0 \text{ K/min} \). However, Hertz suggested that silica fume should not be used as concrete admixture until the problem of spalling is can be properly described.

Jumpannen [62] tested in 1989 the mechanical properties and the spalling behavior of HPC with a compressive strength of up to \( f_c = 110 \text{ MPa} \) and UHPC of \( f_c = 240 \text{ MPa} \). Tests on unloaded HPC cylinders (\( l = 40 \text{ mm}, \varnothing = 8 \text{ mm} \)) did not lead to spalling, even with high heating rates of \( \dot{T} = 32 \text{ K/min} \). However, an
increased specimen size to prisms of \( l \cdot w \cdot h = 100 \cdot 100 \cdot 400 \text{ mm}^3 \) and applied load always lead to violent spalling. Test on unloaded UHPC prisms \( l \cdot w \cdot h = 40 \cdot 40 \cdot 160 \text{ mm}^3 \) showed violent explosive spalling after 7 min when exposed to the standard ISO fire [5]. A surface temperature of \( T = 310^\circ C \) was measured upon spalling. The heating rate was reduced systematically; however spalling always took place at surface temperatures of around \( T = 310^\circ C \). Only very low heating rates of \( \dot{T} = 1.0 \text{ K/min} \) inhibit spalling. The tests showed a combination of critical heating rates and critical temperatures leading to spalling. Similar observations were already made by Sullivan [124] in the 1970’s.

**General fire performance of HPC and mechanical properties at high temperatures by Anderberg**

Anderberg [12] started - among others - with research on the general fire performance of HPC in 1997. The scope of the work included an in-depth analysis of the spalling of concrete including an assessment of the tendency to spalling as well as thermal and mechanical properties of concrete at high temperatures. In addition, it was planned to develop structural models of the mechanical behavior and structural design models for practical use. The general trend in research shifted from tests on large specimens to smaller tests in which the influence of individual parameters on the risk and extent of spalling can be analyzed [27]. Further the possibility of repairing concrete after temperature-related damage was considered in a general way, including protection against spalling and repair of structures after spalling.

**Studies on PP fibers as protective measures against spalling by Nishida**

During the 1990’s, tests were focused on the rapid development of new HPC and UHPC mixes. The analysis focused on the use of silica fume, for which an increased risk of spalling was noticed [31, 48]. In addition, measures for spalling protection were analyzed. It was determined in several studies that a high pore pressure is the governing factor increasing the risk of spalling. In addition, the latest developments in concrete showed a significant decrease in the permeability of concrete [115], hence measures to relieve the high pore pressure were developed. The use of polypropylene fibers (PP fibers) to prevent concrete from spalling seemed to be a promising approach. These ideas and test results were presented by Nishida in 1995 [100]. The functionality of PP fibers in concrete at high temperatures are discussed in chapter 2.5.3.

**Retrofitting of thermically-damaged and spalled concrete by Bamonte**

The 1996 Channel Tunnel fire [69] showed the increased demand for improved repair technologies and a general spalling-safe concrete design. Several additional tunnel fires in the last 15 years like the fire in the Mont-Blanc-Tunnel between France and Italy and the Tauern Tunnel in Austria, both in 1999, or the Gotthard Tunnel in Switzerland in 2001. These tunnel fires resulted in severe damage to the tunnel linings. As a consequence, research was focused on concrete designed for the rehabilitation and retrofitting of tunnel linings damaged by high temperatures. This development including studies on the thermo-mechanical properties of concrete for retrofitting was carried out by Bamonte and presented in 2006 [14]. In his research, Bamonte defined several requirements of the new concrete mix to improve both the application of the concrete to temperature-damaged surfaces as well as the overall long-term durability. Among others, a
sufficient resistance of the concrete against chemical attacks must be provided as well as adequate mechanical properties and an overall good workability. Additional focus was put on the thermal resistance of retrofitting concrete at high temperatures. Aggregates with a low tendency to spalling were used and the adding of PP fibers to the concrete seemed to be very helpful to prevent concrete from spalling. It was concluded that the concrete mixes developed seem to be very promising in terms of retrofitting concrete structures. However, the significant losses in strength and of Young’s modulus at high temperatures were recognized as the main challenges in these tests.

Development of SCC mixes

At the turn of the last century further improvements in concrete technology were made by introducing self-compacting concrete (SCC). The increased workability of SCC enables a wide range of applications, from precast elements to exposed concrete, for which an improved surface finishing is required. Regarding an economic use of SCC, the reduced labor costs are mentioned first. Today, SCC covers a wide range of strength grades up to a compressive strength of \( f_c = 150 \) MPa [75].

Fire resistance of SCC by Persson and Boström (Sweden)

Persson analyzed the fire resistance of SCC in general and summarized his findings in 2004 [103]. He analyzed several SCC mixes with a range in compressive strength between \( f_c = 41 \) - 88 MPa. Tests were carried out on prestressed concrete columns of \( l \times w \times h = 0.2 \times 0.2 \times 2.0 \) m, loaded to 30% of the maximum strength in the cold state. The test specimen were exposed to both the ISO fire curve [5] with a maximum temperature of \( T = 1049^\circ C \) after 120 min and the more rapidly increasing HC fire curve [8] with \( T = 1100^\circ C \). In addition to these tests, mechanical properties were determined on concrete cylinders at high temperatures. PP fibers were added to some of the test specimens in order to minimize the risk of spalling. The use of silica fume, however, is not mentioned limestone but instead powder was used as filler. The tests results in terms of spalling were compared to ordinary vibrated concrete specimen, cast and cured similarly to the SCC specimens.

It was concluded that the SCC specimens showed an increased tendency towards spalling compared to the ordinary vibrated concrete specimens, which could not be explained at the time. The use of PP fibers was found to limit the amount of spalling. In addition, it was mentioned that both the cement/powder and w/c ratio strongly influence the risk of spalling and they should be kept high for a spalling-safe concrete design. A difference in spalling related to the different fire exposures is not mentioned.

Boström studied twelve different qualities of SCC in terms of spalling and presented his results in 2006 [23]. SCC within the range of \( f_c = 34 \) - 52 MPa was tested. To prevent the specimen from spalling, PP fibers with different geometries and filler made from polypropylene (PP filler) were added to the fresh concrete. In most cases small concrete slabs of \( l \times w \times h = 600 \times 500 \times 200 \) mm were cast, in addition to some larger slabs \( l \times w \times h = 1800 \times 1200 \times 200 \) mm) and beams \( l \times w \times h = 3600 \times 600 \times 200 \) mm). All specimens were loaded with a compressive stress between \( \sigma = 0 \) - 6.3 MPa during testing and exposed on one side to the ISO fire curve [5].
Appendix A - Historical overview on spalling of concrete

The dead load of the test specimen has to be considered in all cases. It was noticed that PP fibers are required to prevent the concrete from spalling, since all fiber-free specimens failed by spalling [23]. A clear indication for choosing the right amount and geometry of fibers could not be found, since all types of fibers worked well with this concrete. It should be noted that the compressive strength of the concrete is rather low and no silica fume was added to the concrete. By contrast, even high amounts of 10 kg/m³ PP filler added to the concrete could not prevent the specimens from spalling.

For the small slabs a slight increase in the extent of spalling was noticed with higher applied loads on the specimen, which might be caused by additional cracking of the test specimen. However, this increase became very small for test specimens after additional storage of nine months. For the larger slabs and beams it was noticed that it is rather difficult to obtain a possible correlation of spalling depth, applied load and the size of the tested specimen. Boström concluded that probable influences and different boundary conditions must be considered [23].

In addition to the first testing series, Boström [24] analyzed the risk of SCC mixes and discussed the influence of the specimen size and applied loads on the extent of spalling in 2007. He analyzed - among other mixes - a SCC mix including four sub-mixes which can be summarized as follows:

- Range of compressive strength between $f_c = 37.0 - 78.3$ MPa
- Use of limestone as filler
- Use of PP fibers for spalling protection with a diameter of $\phi = 18 \mu$m (dosage not mentioned)
- Pre-stressing or preloading the specimens with
  - Tests of five different geometries with different preloads ($\sigma_0$)
    - Large slabs with $l \cdot w \cdot h = 1.8 \cdot 1.2 \cdot 0.4$ m to small ones with $l \cdot w \cdot h = 0.6 \cdot 0.5 \cdot 0.2$ m ($\sigma_0 = 8.8$ MPa)
    - Beams with $l \cdot w \cdot h = 3.6 \cdot 0.6 \cdot 0.2$ m ($\sigma_0 = 7.7$ MPa)
    - Two types of concrete cylinder ($\phi = 150$ mm, $h = 300 - 450$ mm) ($\sigma_0 = 2.5$ MPa)
- Temperature exposure after 3 months of storage according to the ISO fire curve [5] and modified temperature curve ($T = 1300°C$ after a total exposure time of $t = 180$ min)

The mechanical load ($\sigma_0$) was applied at an ambient temperature of $T = 20°C$ to some of the large slabs and beams by preloading the concrete in the longitudinal direction. Internal prestressing wires were installed in the large slabs, while the small slabs were prestressed externally using a steel frame.

**Observations by Boström - Influence of preload and specimen size on the extent of explosive spalling**

As concluding remarks on the tests, it should be noted that these specimens were loaded in direct compression, which is in contrast to the other tests by Boström, in which the concrete specimens were tested in bending. Spalling was noticed mainly on the test specimens without any PP fibers. With those concrete slabs where spalling was noticed that the spalling formed a shape that was called an “inverted pillow”. This shape is caused by the boundary conditions of the specimen, since the edges are not directly exposed to the fire and their lower temperature leads to lower pressures inside the concrete. This phenomenon was observed on both types of slab.
Some of Boström’s main observations can be summarized as follows:
- Spalling was noticed mainly on the test specimens without any PP fibers
- The level of preload strongly influences the extent of spalling.
- The applied low compression forces limit the growth and expansion of cracks and minimize the migration of vapor towards the concrete surface and lead to critical pore pressures.
- Boström concludes that spalling increases with an increased level of applied stresses.

In other tests by Boström, an increased amount of spalling was noticed in tests on a small slab, while spalling came to a stop in these tests after some time in the tests on large slabs. Boström stated that the most probable explanation of these observations might be the loss in prestress due to high temperatures at the prestressing wires placed inside the large slab. After some time, the concrete as a thermal barrier spalled leading to direct heat exposure of the wires. At this point, the large slabs can be considered as non-prestressed with a decreased risk of spalling. The prestress of the small slabs remained constant throughout the test, since these specimens were prestressed externally with a steel frame.

It may be concluded in general from the tests by Boström that it is difficult to estimate the average and maximum spalling depths for cylinders and cubes, since these small specimens are exposed to heat from all sides. However, a general analysis, if a specific concrete mix tends to produce spalling at high temperatures, can be carried out easily with cylinders. In terms of additional external loads applied to the specimen during testing Boström concluded in his research that loads have a significant influence on spalling and should be applied to the test specimen to achieve overall reliable results. However, the use of prestressed cables is not recommended since the prestressing forces might be affected by the heat. As a final conclusion for choosing the optimum specimen size, it was mentioned that having slabs exposed on one side to high temperatures seems to be the most promising approach. Slabs are easy to produce and external loads leading to bending moments can be applied during the tests, depending on the boundary conditions. Losses in weight during testing and the spalling depth can easily be measured. However, it is not known if these losses in weight are related to spalling or dehydration of the concrete.

Towards achieving a simplified, standardized test method on small-scale samples by Hertz

Large-scale tests on structural concrete members, like beams, slabs or columns, exposed to standardized temperature curves and including external loads are necessary as qualifying tests if a specific concrete structure including all boundary conditions tends to spall at high temperatures. However, these tests are very cost-intensive and require much laboratory work.

In recent years, tests on small-scale concrete specimens were performed to assess the general risk of spalling. These test results can be used for additional in-depth analysis of concrete spalling and micro-structural modeling. However it is not clear if these tests are sufficient to estimate the spalling behavior of real structures that are exposed to rapidly increasing temperatures.

Several ideas on new testing methods using smaller concrete samples and low heating rates were presented in the last few years. The aim was to provide ideas for the development of a standardized test method for
small concrete samples. However, a general standardized test method for concrete spalling using small-scale tests specimens is not available yet.

In 2005, Hertz [51] developed a simplified test method for determining the possibility whether a concrete mix tends to explosive spalling at high temperatures. He suggested concrete cylinders (φ = 150 mm, l = 300 mm) as regular test specimens. The cylinder is inserted into a steel tube covered with neoprene to ensure a sealed skin surface of the cylinder, while the top and bottom ends remain uncovered. The specimen is then placed in front of a furnace with heat exposure of the uncovered top of the cylinder. High heating rates of about 20 K/min to surface temperatures of T = 800°C should be applied. The benefit of this layout is that the effect of restrained thermal expansion is taken into account during the test. In addition, the moisture migration is limited to the longitudinal direction of the specimen. Failure of the concrete by spalling can usually be detected by a cracking sound. The moisture content of the specimen after testing is then compared with its initial state. This comparison indicates if the concrete tends to exhibit spalling with its initial moisture content. The tests are then repeated with different moisture contents to determine a critical level. Hertz concluded that the results from these tests agree with his experience gained from other more complex tests in terms of spalling. His test method is cheap and easy to apply.

**Research on individual parameters and phenomena influencing the risk of explosive spalling**

In addition to the need for simplified tests methods, the improvement in concrete technology in recent years focused research on individual parameters and their corresponding influence on the risk and extent of concrete spalling at high temperatures. The main individual parameters analyzed include:

- Pore pressure measurements
- NMRI scans on moisture migration inside heated concrete

**Pore pressure measurements**

Apart from tests on new concrete mixes and different specimen sizes for standardized tests, new devices led to improved techniques for measuring the influence of specific concrete parameters on the risk and intensity of spalling. For example, small pressure transducers were developed. With the help of these improved measuring devices, the pore pressure inside concrete at high temperatures could be determined. Several tests were made in recent years in France and Japan.

**Tests on pore pressure by Bangi on HPC including PP fibers**

Bangi [15] presented in 2010 his findings regarding pore pressure measurements in HPC with an average compressive strength of f_c = 85 - 105 MPa. The use of silica fume is not mentioned in his report. He installed thermocouples and pressure transducers inside concrete cylinders (φ = 175 mm, l = 100 mm) and heated them towards the top surface only with different heating rates between Ṭ = 5.0 - 10.0 K/min and according to the ISO fire curve. The pressure was measured at different depths towards the heated surface. It was
noticed that for slowly heated concrete, the pressure increases to levels of \( p = 4.0 \text{ N/mm}^2 \) close to the heated surface and \( p = 2.0 \text{ N/mm}^2 \) in inner sections of the concrete. For high heating rates, the pressure remained at a constant high level of \( p = 4.0 \text{ N/mm}^2 \). In terms of duration of high pressures during heating it was observed that the duration of the peak pressure is very short for the area close to the heated surface and increased in length for inner sections. However, the rapid increase and decrease in pressure becomes less pronounced for inner sections.

Bangi concluded that a high heating rate clearly leads to a higher pore pressure in deeper sections of the concrete, while the pressure close to the surface is almost independent in terms of the pressure built up.

Tests were repeated with concrete specimens containing PP fibers [15]. The specimens were heated to \( T = 600^\circ\text{C} \) after 100 min. It was noticed that the pore pressure did not exceed \( p = 1.0 \text{ N/mm}^2 \) with the use of thin PP fibers with an average diameter of \( \phi = 18 \mu\text{m} \). The use of thick PP fibers (\( \phi = 310 \mu\text{m} \)) leads to a lower pore pressure towards the heated surface (\( p = 1.0 \text{ N/mm}^2 \)) but a significant increase in pressure in inner sections close to \( p = 2.0 \text{ N/mm}^2 \) was observed.

**Tests on pore pressure by Pimienta including different storage conditions**

Pimienta et al. improved the techniques for pore pressure measurement and presented them in 2011 [96, 102], by coupling pore pressure measurements with acoustic emission detection. OPC specimens with average dimensions of \( l \cdot w \cdot h = 600 \cdot 600 \cdot 300 \text{ mm}^3 \) and an average compressive strength of \( f_c = 46 \text{ MPa} \) were tested, including the influence of reinforcement, since half of the specimens remained without any reinforcement. Pressure transducers were placed at different depths between 10 - 50 mm from the heated surface of each test specimen and four acoustic emission detection sensors were attached to the cold surface of the slab. The prepared tests specimens were then exposed on one side to different fire scenarios. In some tests the ISO fire curve [5] and in other tests the Hydrocarbon (HC) fire curve [8] were applied, leading to a total of four different test combinations in terms of reinforcement.

From these test results, some general conclusions can be drawn. Spalling was observed mainly on the test specimens exposed to the HC fire curve. In addition, the average crack width of the specimens without reinforcement was significantly higher compared to the slabs without reinforcement. In terms of pressure development, it was interesting to note that the maximum pressure inside the concrete slab heated with the ISO fire curve was almost twice as high (\( p = 0.8 \text{ N/mm}^2 \)) compared to the specimen heated with the HC curve (\( p < 0.4 \text{ N/mm}^2 \)). It was stated that these observations are in clear contrast to the common understanding of spalling due to high pore pressures, since it was always assumed that higher pore pressure leads to an increased extent of spalling. However, severe deterioration of the concrete close to the heated surface should also be considered, since the HC fire curve exceeds \( T = 1000^\circ\text{C} \) after less than 10 min exposure time. This rapid heating causes high thermal stresses close to the heated surface and promotes crack growth and the increase in permeability, which then causes a decrease in pore pressure.

Further tests were conducted on the influence of water content on the pressure development. Two different mixes with an average compressive strength of \( f_c = 37 \text{ MPa} \) and \( f_c = 61 \text{ MPa} \) were used. Specimens of size \( l \cdot w \cdot h = 300 \cdot 300 \cdot 120 \text{ mm}^3 \) were stored at ambient temperature, pre-dried at \( T = 80^\circ\text{C} \) or stored in water
until a constant mass was reached. After curing, the concrete was exposed on one side to a radiant heater with a constant temperature of \( T = 600^\circ\text{C} \), while measuring the pore pressure at different depths.

**Influence of w/c ratio, storage conditions and heating rate on pore pressure inside concrete**

It was noticed [96, 102] that an increase in compressive strength caused by a lower w/c ratio increases the pore pressure to a higher level at a higher temperature. In addition, plateaus in the pressure development appear for those concrete samples containing free water (storage at ambient air and in water). During these plateau phases no increase in pressure was measured. In general, the pressure measurements indicated that free water inside the concrete pores must be considered as an important factor leading to high pore pressure. A lower peak pressure and no plateau phase were observed when testing the pre-dried concrete.

Interesting observations [96, 102] were made in pressure measurements for the specimens after water treatment and storage at ambient temperature. Here, a lower pressure for the specimen after water treatment was noticed. It was considered that the high amount of water inside the concrete causes several cracks and promotes the deterioration of the concrete upon expanding. The increased growth of cracks leads then to a higher permeability. It should be noted that these observations are in contrast to earlier findings. Meyer-Ottens [94] clearly indicated a high moisture content as a critical parameter increasing the risk of explosive spalling due to high pore pressure.

In addition, similar to the previous testing series on pore pressures by Pimienta, it was confirmed that higher heating rates lead to a lower pore pressure inside the concrete. This is mainly caused by cracking, which leads to an increased permeability of the concrete.

**Visualization of moisture migration inside heated concrete with NMRI scans**

Nowadays, one of the main challenges is a sophisticated, in-depth analysis of moisture migration inside concrete at high temperatures. Even though well described in theory, moisture migration and vapor movements inside the heated concrete could not be visualized in detail until today. An early attempt was made by Jansson, where he split concrete cubes after fire exposure from one side [61]. A saturated zone was noticed by a change in color of the concrete.

A new and promising approach was pioneered at TU Delft by van der Heijden. With the help of nuclear magnetic resonance imaging (NMRI), it was possible to visualize a saturated moisture layer close to the heated surface. His results were presented in 2011 and 2012 [127, 128].

The aim was to determine and analyze moisture migration inside the concrete during heating and scanning the specimen. As concrete specimens, a cylinder (\( \phi = 80 \text{ mm}, l = 100 \text{ mm} \) drilled from larger concrete slabs was placed inside the NMRI scanner. The average compressive strength of the concrete cylinder was about \( f_c = 40 \text{ MPa} \). As a heat source halogen lamps were used to heat the concrete to temperatures of \( T = 400^\circ\text{C} - 500^\circ\text{C} \). The heat flux was given as \( q = 12 \text{ kW/m}^2 \). The thermal radiation was focused on one cross section, while the rest of the specimen was covered with insulation material. For the necessary furnace...
Appendix A - Historical overview on spalling of concrete

protection higher temperatures cannot be reached, however the drying of the concrete is almost entirely covered and several tests have shown that critical temperatures leading to explosive spalling are below \( T = 500^\circ\text{C} \).

The test specimens by van der Heijden were dried to a constant mass and then conditioned for three months before testing, covering levels between 50\% and 97\% ambient humidity. A very precise treatment is essential, since the results from the scans strongly depend on the moisture level and density of the concrete.

The analysis [127, 128] showed that the temperatures at the “boiling front”, the area where the moisture changes from the liquid to the gaseous phase, is significantly higher than \( T = 100^\circ\text{C} \), i.e. at about \( T = 200^\circ\text{C} \), with a corresponding vapor pressure of \( p = 1.4 \text{ N/mm}^2 \). In addition to these observations, a first rough verification of the presence of a moisture peak is given. It was stated that additional temperature measurements using regular thermocouples are mandatory for calibration purposes and corrections of the measured data.

**International research groups researching the spalling of concrete in fire**

Today, there are several international research groups that pool results from a wide area and different approaches in research with the aim of providing the best solution for engineers and the industry as a whole. Two of them are - among others - the RILEM technical committee 227-HPB on “Physical properties and behavior of High-Performance Concrete at high temperature” or the fib task group 4.3 on “Fire design of concrete structures”. Here, an in-depth analysis of concrete at high temperature is provided, covering a wide range of structural- and material-related influences on spalling as well as measures to prevent it.

One of the challenges for the future is the assessment of existing structures in terms of the possible occurrence of spalling at high temperatures, in addition to the general development of spalling-safe concrete designs.
APPENDIX B - MAIN PARAMETERS LEADING TO SPALLING

APPENDIX B.1 - Concrete mix design / material-related parameters

Silica fume:

In terms of concrete strength, Khoury coined the phrase that “ironically, poor quality concrete is superior to good concrete in spalling” [57]. The development of HPC and UHPC improved significantly with the use of silica fume as part of a concrete mix. Silica fume consist mainly of amorphous silicon dioxide ($\text{SiO}_2$). With an average diameter of less than $d = 1.0 \, \mu \text{m}$, the silica particles fill voids and gaps inside the cement matrix leading to a much denser and harder structure that increases the durability and compressive strength. In addition, silica fume in concrete causes a pozzolanic reaction. The calcium hydroxide from the hydration of the cement reacts with the silica particles and leads to an additional production of CSH phases. However, at ambient temperature conditions, the pozzolanic reaction leads to an increase in compressive strength of only 20%. The majority of the gain in strength comes from an improved bonding of the cement matrix and the aggregates. Higher temperatures are known to increase the pozzolanic reaction of silica fume, which increases the compressive strength significantly [84]. Silica fume added to the concrete improves the microstructure of the concrete [108]. This is noticed by the fracture pattern of loaded HPC and UHPC specimens with high amounts of silica fume. In contrast to OPC specimens, aggregates are usually experience cracking during failure, but are still embedded in the cement matrix.

Silica fume as part of the concrete mix design is known as one of the main parameters decreasing the concrete’s permeability and leading to an increased risk of explosive spalling, mainly due to its small particle size [57, 81]. It is generally agreed that a more dense concrete with low permeability shows an increased likelihood of explosive spalling, since moisture migration mainly in the form of vapor inside the concrete is limited and temperature-dependent high pore pressures cannot be released with the present low permeability. Tests by Schneider in 1989 [115] on OPC samples showed that the permeability of the concrete increases with increasing temperatures, which should contribute to the relief of critical pore pressures. However, recent tests have shown [75] that this usual increase in permeability with high temperatures is significantly delayed by the presence of high amounts of silica fume added to the concrete leading to a much lower permeability even at high temperatures. Furthermore, a temporary decrease in permeability at around $T = 200^\circ \text{C}$ was observed.

The issue of high amounts of silica fume in HPC and UHPC and the related risk of spalling is also considered in the European standard EN 1992-1-2 [6]. In this case, explosive spalling might occur anytime when concrete with high silica fume content is directly exposed to fire. A critical level of 6.0% in mass of the total concrete weight is mentioned. By exceeding these levels, additional protection measures have to be applied to the concrete.

Concrete permeability:

In terms of permeability it is mentioned by Harmathy that spalling is unlikely to occur if the permeability of the concrete exceeds $k = 5.0\cdot 10^{-15} \, \text{m}^2$ [44]. However, recent tests have shown [75] that no spalling was noticed, even with a permeability of the concrete below this value, hence it is difficult to assess the risk of spalling by permeability only.
Apart from silica fume added to the concrete, the limestone fillers frequently used for SCC might decrease the permeability as well and enhance the risk of explosive spalling [23, 103].

During the beginning of research work on explosive spalling, the knowledge of the permeability of concrete at high temperatures was limited. Gary [40] never used the expression of permeability, but he already noticed that vapor transportation processes inside the heated concrete are responsible for spalling. He assumed that the connectivity and distribution of concrete pores are essential for the relief of pore pressure.

**Choice of aggregates:**

Porous aggregates are less dense than ordinary aggregates and they increase the concrete’s permeability. This enables vapor transportation and reduces the risk of explosive spalling. In general, it was noticed that the choice of aggregates and their influence on the risk of spalling can be considered as an additional factor, since all types of spalling except for sloughing off spalling are influenced by the properties of the chosen aggregates [57]. Other sources state that some open-pore lightweight aggregates contain a significantly higher amount of moisture [98], which is released with increasing temperatures and might promote spalling. In addition, the amount of chemically-bound water can be up to 10% compared to common concrete aggregates [98]. Even though these aggregates have a high permeability, it is not known if the permeability of the concrete is sufficient to enable a safe relief of vapor pressure without spalling, due to the high amount of moisture.

In general, most of the aggregates remain stable and do not tend to large increases in volume or decomposition for temperatures of about $T = 500°C$, which is beneficial in terms of minimizing the risk of spalling. However, River Thames gravel is an exception, since it remains stable up to temperatures of about $T = 350°C$ only [57]. Higher temperatures cause a complete breakup of these aggregates, which promotes spalling.

Quartzite aggregates can be considered as stable up to a temperature of $T = 573°C$. At this temperature, quartz changes from the $\alpha$-quartz phase to the $\beta$-quartz phase, which causes a significant increase in volume of about 5.7% [57]. This transformation is rather quick, leading to an increased crack growth in the concrete. Hertz [49] notes that the cracking caused by quartzite transformation contributes to the general deterioration of the concrete, but would not lead to spalling. In addition, most of the chemically bound water in the cement paste has already vaporized at this temperature and it can be considered that explosive spalling is hardly affected.

Kodur [82] stated that the use of carbonate aggregates like limestone provides a higher resistance to spalling and gives the higher heat capacity of limestone aggregates as the explanation. Khoury [68] agrees and mentions the thermal expansion coefficients of different types of aggregates, which are significantly lower for limestone compared to quartzite aggregates. Since the expansion of the aggregates near the heated surface is restrained by the cooler inner sections, low thermal expansion will lead to a lower stress levels and reduce the tendency to spalling at high thermal stresses. These thermal stresses are even more pronounced with a mix of different types of concrete, due to the differences in thermal expansion or if the aggregates are subject to physical changes, leading to a large expansion and large changes in quartzite.
Creep and shrinkage of the cement paste at high temperatures as part of thermal strains counteract the thermal elongation of the aggregates [57]. This difference leads to a loss in bond between the aggregates and the cement paste, which is more pronounced during cooling leading to post-cooling spalling. In addition, the reversible expansion of the quartzite aggregates during cooling and the associated additional change in volume can lead to a complete loss in bond between the cement paste and the aggregates [74], as observed by MRI scanning. This deterioration further promotes the risk of post-cooling spalling, even after a longer time.

It is agreed that the use of lightweight aggregate usually decreases the risk of explosive spalling [57]. Khoury stated [68] that for areas with low humidity lightweight aggregates are “generally recommended” to prevent concrete from spalling. Lightweight aggregates usually have a high permeability, which is beneficial for migration processes. For HPC and UHPC, lightweight aggregates are usually not used, since the compressive strength of these aggregates is rather low.

Tests have shown that the use of lightweight aggregates might lead to an increased amount of explosive spalling as observed by Kodur [79]. It was noted that lightweight aggregates often show a higher amount of free water, which vaporizes with rising temperatures and leads to a higher pore pressure. In addition, some lightweight aggregates like foamed clay decompose and release chemically bound water, which promotes the buildup of these high pore pressures. It should be mentioned that some new lightweight concrete mixes also include high amounts of silica fume of up to 10% of the added cement [39]. This might increase the risk of explosive spalling significantly. It remains unknown if the negative effect of a high amount of silica fume overcompensates the beneficial effects of lightweight aggregates.

In addition to the influences leading to explosive spalling, some aggregates tend to promote aggregate spalling starting at rather low temperatures. Hertz [49], Meyer-Ottens [94] and Connolly [27] mention flint and sandstone as critical aggregates in regard to single aggregate spalling. Sandstone tends to split at the rather low temperatures of \( T = 260\,^\circ C \), flint even from \( T = 150\,^\circ C \). Critical splitting temperatures for feldspar and gneisses are at about \( T = 390\,^\circ C \). It is agreed in general that these splitting forces only lead to superficial and local spalling close to the heated surface and can be considered as non-critical [27, 49].

In terms of maximum aggregate size, Connelly [27] mentions that an increase in aggregate size increases the risk of explosive spalling. In [27] it is noted that the cement paste / aggregate ratio changes with fewer, but larger aggregates particles. Larger aggregates have a smaller surface area per unit mass than smaller aggregates. This decrease in total surface area between larger aggregates and the cement paste might lead locally to a lower permeability of the concrete and promote spalling, since the migration route of vapor along the aggregates is longer. These considerations were strengthened in tests using fewer but larger aggregate particles inside the concrete. In this case, the time until explosive spalling observed was significantly reduced. An exception is very small aggregates in form of powder. These aggregates lead to a decrease in permeability of the concrete.

The European design standard EN 1992-1-2 [6] does not give a recommendation on the type of aggregates or the maximum size of the aggregates used with regard to the spalling of concrete.

Internal cracking of concrete at high temperatures has two opposite effects on explosive spalling [68]. On the one hand, small cracks and voids might relieve critical pore pressure and minimize the risk of explosive spalling while on the other hand parallel cracking close to the heated surface leads to an increase in pore
volume. This causes a sudden vaporization of moisture accompanied by loss in pressure in these areas, leading to violent explosive spalling of the concrete in areas close to the heated surface.

As main sources of microcracking, Meyer-Ottens and Khoury mention cracking caused by high-temperature drying and shrinkage of the cement matrix as well as external loads [68, 94]. A different thermal expansion of cement paste, aggregates and reinforcement steel is also mentioned.

**Compressive strength of concrete:**

The influence of the compressive strength of concrete on the risk of explosive spalling has been analyzed in the past. Even though a higher risk of spalling with increasing strength was found [82], tests on HPC slabs ($f_c = 117$ MPa) with quite a high moisture content do not necessarily lead to explosive spalling. Based on tests presented in the literature, Kodur [82] and Khoury [68] notice that a higher concrete strength usually leads to an increased risk of spalling, mainly because of the lower permeability of concrete mixes with a higher strength grade. However, it was noticed that it is difficult to give a specific strength grade leading to spalling. Based on a review by Kodur [82], concrete exceeding a compressive strength of $f_c = 70$ MPa is more likely to tend to spalling.

**Cement content:**

High cement content is usually required to achieve strength grades as observed with HPC mixes. As mentioned, this high cement content, which can exceed 1000 kg/m$^3$, leads to a very dense and less permeable cement matrix. In addition, the vapor from the chemically bound water during decomposition of the cement matrix must be relieved. It is interesting to note that HPC and UHPC mixes might have an increased amount of water per m$^3$ of concrete compared to OPC mixes, even though the w/c ratio of HPC and UHPC mixes can be low at around w/c = 0.2. This is mainly due to the high amount of cement compared to OPC with a cement content of about only 400 kg/m$^3$ and a w/c ratio of about 0.4. A possible influence of the type of cement on the risk of spalling is not discussed in detail. Tests on concrete cylinders indicated that a high slag content might reduce the risk of post cooling spalling [74]. However, the effects of different storage conditions after cooling were not analyzed in these tests.

Apart from the chemically bound water inside the concrete, which is released during decomposition, the free water inside the pore system will build up critical pressures with increasing temperatures. It is agreed that this initial moisture content is essential in terms of critical parameter causing explosive spalling. Moisture as water vapor will partly migrate outwards or further inwards the heated concrete according to the given permeability via the existing channels and voids. Depending on the amount of water vapor and temperature, moisture gradients inside the concrete and local peaks of moisture must be considered, building up high pore pressures and contributing to explosive spalling. This phenomenon of the “moisture clog” is discussed in detail at later stage in chapter 2.3.2. In contrast to concrete with humidity, “dry” concrete will not tend to explosive spalling [49] due to the lack of building up critical pore pressure. Similar tests were carried out by Connolly [27] on OPC showed no spalling after the concrete specimen were dried to a constant in mass.
In tests by Connolly [27] it could be shown that only concrete with a sufficient humidity tend to explosive spalling at high temperatures. It is mentioned that OPC will not tend to spalling with a moisture content of $m_w = 3\% - 4\%$ in mass or less [49]. Similar observations can be found by other tests from Shorter and Harmathy [49, 119]. In terms of HPC and UHPC it must be noticed that the level of moisture content can be significantly lower, since tests on UHPC with an average moisture content of $m_w = 2.1\%$ in mass showed severe explosive spalling.

In other publications, the term of relative humidity (RH) as function of the degree of water saturation is often mentioned in context of moisture content of concrete. Depending of the concrete, a fully saturated ordinary concrete (RH = 100%) with a w/c ratio of w/c = 0.4 might have a moisture content of $m_w = 6\%$ of the concrete’s initial mass or more. This value decreases rapidly with decreasing w/c ratio because of the decreasing porosity.

Regarding a critical RH level, Kodur [82] notes that a level of 80% or higher will most probably lead to explosive spalling. It is further noted that an “acceptable RH level” of 75% or lower reduces the risk of spalling. However, due to the low permeability of HPC and UHPC it might take a long time to achieve this humidity when the concrete is exposed to ambient environmental conditions.

Closely related to the moisture content of concrete leading to explosive spalling, the moisture absorption of cement paste and aggregates after cooling from high temperatures must also be considered for post-cooling spalling [57]. Calcareous aggregates tend to an increased uptake of moisture from ambient air at $T = 20\,^\circ C$ after cooling from high temperatures of $T = 700\,^\circ C$ and more. This uptake causes an increase in volume of the aggregates and promotes the risk of post-cooling spalling.

In addition, the cement paste itself also tends to an increase in volume due to moisture uptake. However, an increase in bond between the cement paste and the aggregates or a general improvement of mechanical properties could not be observed in tests [74].

**Concrete age:**

The age of a concrete structure is closely related to the concrete’s moisture content. It is noted that the risk of spalling decreases with higher concrete ages, mainly because of the lower moisture content in older concrete [57]. A survey of several tests on OPC after different storage times showed no clear relation between the age of the concrete and the possibility of spalling in fire [68]. As general advice it was mentioned that tests on the risk of spalling of concrete should not be performed on young concrete. This advice is not given in the European standard on fire resistance tests [101]. The standard EN 1363-1 suggests that a sufficient conditioning of the specimen before testing should be ensured to achieve overall a constant moisture content of the concrete. A minimum storage at $T = 23\,^\circ C$ and 50% rel. humidity for 28 d is recommended. It should be mentioned that environmental conditions in tunnels usually decrease the “aging effect” in concrete, due to the humid conditions on both sides of the concrete. Tests on tunnel segments should be performed with all due care to achieve realistic maturity conditions.

It remains unanswered if these recommendations also apply for HPC and UHPC. For these concrete mixes losses in moisture are usually less significant compared to OPC, mainly due to the dense concrete matrix and low permeability. Tests on UHPC cylinders have not shown any significant losses in weight even after years.
of storage (< 1.0%). It can be concluded that for HPC and UHPC the age of the concrete when exposed to fire plays only a secondary role in terms of explosive spalling.

APPENDIX B.2 - Structural / mechanical parameters

The tensile strength of concrete is one of the most controversial parameters with a significant influence on explosive spalling. Several models explaining explosive spalling of concrete consider the temperature-dependent tensile strength of concrete as a failure criterion in terms of explosive spalling due to high pore pressure.

In several research publications it is generally agreed that moisture inside the concrete will expand and evaporate to steam during heating, causing high pore pressures inside the concrete. Briefly explained, it is assumed that once this pressure exceeds the tensile strength of the concrete, the pressure is released by explosive spalling [70]. Three main explanations on explosive spalling due to high pore pressures were given in the past [57] and is explained in chapter 2.3.2.

Apart from high pressures exceeding the tensile strength of concrete, rapid heating can lead to high tensile stresses within the concrete as already noticed by Hasenjäger in 1935 [45]. First, compressive stresses will rise due to the thermal expansion, which might be promoted due to restraints of the concrete. These compressive forces at the heated surface lead to tensile stresses in deeper, cooler sections that might exceed the tensile strength of the concrete and lead to explosive spalling. This phenomenon of “thermal stress spalling” is explained in detail in chapter 2.3.3.

In addition to the risk of explosive spalling Connelly [27] proposes that the tensile strength of concrete influences the risk of corner spalling. A low initial tensile strength or additional losses in tensile strength due to high temperatures do not show sufficient resistance towards thermal stresses caused by heat gradients from two different sides.

Applied high concrete stresses, hindered thermal expansion, restraints or limited dilatation in a concrete member are the governing factors for the risk of violent spalling.

Several tests on the influence of preloading and different boundary conditions on concrete members exposed to fire have been reported [62, 94] and show that the risk of concrete spalling increases with the applied load.

Based on Meyer-Ottens’ [94] development, Sertmehmetoglu [118] improved the design diagram for concrete to assess the likelihood of spalling of concrete with a compressive strength of $f_c = 45$ MPa and a moisture content of 2.0% or more exposed to fire from two sides. As input parameters, the thickness of the concrete member and the stress applied to the member were taken. It was noticed that the relationship between applied compressive stresses exceeding $\sigma = 2.0$ MPa and the concrete element thickness required to ensure no damage by spalling is almost linear. The diagram is shown in Figure B1.
Figure B1: Spalling of concrete after heating from two sides after Sertmehmetoglu [118]
Specimen thickness versus applied compressive stress

Recent data on the influence of applied stresses covering HPC and UHPC mixes are not available. Boström [23, 24] tested SSC with a compressive strength of up to $f_c = 78.3$ MPa and noticed that preloads of about 5% or less of the initial cold strength can lead to explosive spalling. It is interesting to note from his tests on SCC that applying a small load to the concrete will cause a significant increase in spalling, amounting to three times that in unloaded concrete samples.

Hertz [49] mentions fixed ends as boundary conditions for a concrete structure promoting the risk of spalling due to hindered thermal expansion. In addition, he suggested that eccentric loads or bending are critical regarding the risk of spalling due to additional stress peaks.

Recent tests on HPC columns by Kodur [82] showed that a higher degree of restraint increases forces inside the column. However, it is interesting to note that these generated forces are almost negligible if compared to the overall column strength. It is mentioned that these restraint forces can be higher for beams, but adequate test data is meagre.

Even though stresses due to external loads are considered as critical regarding the risk of spalling, it is still unclear why very low compressive stresses and moderate heating rates might be beneficial regarding explosive spalling. It can be assumed that small cracks and voids are pressed together leading to a stiff but permeable concrete. High pore pressure might be relieved without spalling [34] (cited in [27]).

As mentioned previously, concrete columns of rectangular cross section have a higher risk of spalling, mainly corner spalling, due to temperature gradients from two sides compared to columns of round cross section. The influence of the geometry of the concrete member is clearly shown [27].

For concrete slabs and beams, experience from past tests gives an inconsistent picture of the influence of the section size on the risk of spalling. It is agreed that several other influences are more important regarding the risk of spalling [68] such as 3- or 4-sided fire exposure, moisture content, heat gradient, restraints or reinforcement layouts, to name a few, while the section size and shape is only of secondary influence as long as there are no changes in the cross section of the concrete. Changes in the section’s geometry can result in high thermal stresses during heating which promotes explosive spalling.
In terms of minimum section geometries for structural concrete members like slabs and beams in order to prevent these parts from spalling, tests from Meyer-Ottens [94] provide useful design criteria. Some of these criteria, like minimum concrete cover or spacing of reinforcement are now included in the European design standard EN 1992-1-2 [6] to ensure an adequate resistance of concrete structures in fire. However, these design criteria should not necessarily be associated with spalling.

To minimize the risk of corner spalling and other types of violent spalling, Kordina [86] published several design criteria. Among other things, he modified the section shape of concrete members by rounding curves and a rearrangement of the reinforcement to minimize high thermal gradients and stress peaks close to the reinforcement. In his work Meyer-Ottens observed explosive spalling close to the heated reinforcement bars due to different amounts of thermal expansion of the concrete and the reinforcing bars. It was noticed that the occurrence of spalling for tests on slabs with the reinforcement placed in the inner section of the concrete showed less spalling, even if exposed to higher temperatures. These observations lead to minimum reinforcement cover and spacing distance between reinforcement bars, in order to minimize spalling.

In contrast to these reinforcement layout criteria for slabs and beams, concrete columns with conventional tie configurations as reinforcement show an increased risk of explosive spalling leading to a total deterioration of the column. The suggested layout in terms of reinforcement cover seems to be inadequate in most cases. Improved structural details, like lateral reinforcement in columns, was analyzed in detail by Kodur [81] to increase the fire resistance and minimize the risk of explosive spalling. By a modified tie design and less spacing with about 75% of the usual spacing, the fire resistance of HPC columns could be increased significantly. The idea was to bend the ends of the ties further inwards inside the concrete column with an angle of 135° compared to the usual layout, in which 90° ties are used. Even though spalling was observed during the test and the column was loaded with more than 50% of its design load, a fire resistance time of 4 h or more could be achieved. The effect of the modified reinforcement layout improving the fire resistance of columns, even if spalling occurs, and as a protective measure is discussed in detail in chapter 2.5.6.

Finally, according to Hasenjäger and Khoury [45, 68] sufficient shear strength of the concrete is required to prevent a structure from sloughing-off spalling. After longer exposure to high temperatures, when the concrete starts to weaken, insufficient shear strength of the concrete member can lead to a detachment of larger concrete fragments, which may then have a significant influence on the structural performance. However, adequate shear strength can be provided with an adequate layout of reinforcement.

**APPENDIX B.3 - Parameters depending on the heating characteristics**

In regard to temperature-dependent parameters leading to explosive spalling, the heating rate should be mentioned first. OPC can easily withstand heating rates of $\dot{T} = 20$ K/min or higher. Tests by Jumpannen have shown [62] that heating rates of up to $\dot{T} = 32$ K/min are possible in the best case with only minor aggregate spalling. However, these tests also indicated the influence of preloading and the specimen geometry on the spalling behavior, which can decrease the spalling resistance significantly.

With the use of HPC or UHPC and the increased amount of silica fume in these concrete mixes, it is observed that significantly lower heating rates of just $\dot{T} = 1.0$ K/min can lead to explosive spalling [75]. Tests on concrete
specimens heated with even lower heating rates of only $\dot{T} = 0.5$ K/min tend to produce explosive spalling at surface temperatures of $T = 300 - 350^\circ\text{C}$ [49, 68], if no protective measures against spalling are applied.

Regarding high temperatures leading to explosive spalling, concrete temperatures exceeding $T = 500^\circ\text{C}$ as observed in fires do not necessarily lead to explosive spalling. Connolly [27] concludes from the literature that spalling usually takes place within a temperature range of $T = 350^\circ\text{C}$ to $715^\circ\text{C}$. However, most tests on heated concrete showed that explosive spalling is not necessarily to be expected after the concrete exceeded temperatures of about $T = 500^\circ\text{C}$ [27, 75].

Other types of spalling, mainly post-cooling spalling and sloughing-off spalling can be linked to the maximum temperature and the corresponding time during heating. High temperatures lead to a rapid deterioration of the concrete and cause significant decomposition of the cement matrix.

In terms of thermal gradients between the fire exposed surface and cooler concrete sections a critical value of $\Delta T = 1.0$ K/mm is mentioned by Schneider [114], where the heating rate and thermal stresses have a noticeable influence on explosive spalling. In addition, asymmetric heating of a concrete cross section as observed with slabs or tunnel linings causes high thermal gradients and hence could lead to explosive spalling due to thermal stresses and moisture gradients [49].

In terms of the heat exposure of different surfaces, it is obvious that the exposure of more than one surface increases the risk of spalling [94].
APPENDIX C - MECHANISMS LEADING TO EXPLOSIVE SPALLING

APPENDIX C.1 - Pore pressure-induced explosive spalling

First considerations by Shorter and Harmathy

Shorter and Harmathy [120] noted from tests on concrete slabs that spalling might be caused by “moisture clog” spalling. At an ambient state, an almost constant distribution of moisture can be assumed in the concrete. During heating a concrete slab from one side, the increase in temperature in a specific layer of concrete will cause vaporization of moisture. This vapor then migrates partly outwards towards the heated surface, while the rest of the moisture migrates further inwards in the concrete to cooler sections, condense there and leads to a higher moisture content in this zone. With further heating, two separate zones of moisture saturation develop. The parts of the concrete close to the heated surface are drying-out while a “saturated zone” of moisture develops with some distance from the dry zone. This saturated zone is called “moisture clog”.

The moisture clog model is shown on Figure 5 on page 28. In Shorter and Harmathy’s model, it is explained how the “moisture clog” moves according to the rising concrete temperature. It is mentioned that the clog moves by vaporizing moisture into cooler concrete sections once the saturated zone reaches a temperature of $T = 100^\circ C$. However, this movement is limited once the pores in inner sections of the concrete are saturated. This leads to opposing forces of temperature and pressure gradients, which then defines the direction of vapor movement. Heating rate and permeability of the concrete are stated to be governing factors influencing the magnitude of each force.

As a first requirement for a development of critical pressures, Shorter and Harmathy mention that the rate of moisture clog movement according to the pressure gradient $v_p$ (movement outwards to the heated surface) must be smaller than the rate of moisture clog movement according to the heat flux $v_h$ (movement inwards to the cold surface). Second, sufficient fractional pore saturation is required for spalling. Finally, spalling will occur if the pressure in the interfacial layer between the two moving fronts exceeds the tensile strength of concrete. These bursting stresses are a result of static pore pressure at the interfacial layer. The analytical model to describe the likelihood of explosive spalling due to high pore pressure was explained further and analyzed in detail in chapter 2.4.2, starting on page 33.

Some issues with the moisture clog model due to Shorter and Harmathy are shown in Figure 5 and remain unsolved. The increase in pressure from ambient air pressure $P_{\text{atm}}$ to the maximum pressure in the dry zone is rather doubtful as well as the sudden release in the saturated zone, in particular if here the temperature is considered to be constant.

The presence of a moisture clog was analyzed in detail in the past by Chen in 2009 and Kalifa in 2001 [26, 64]. Both authors agree on the existence of the moisture clog as a saturated layer close to the heated surface with an almost dry concrete layer in between. Chen noticed that the gas permeability is limited in this area making any migration processes almost impossible. The concrete pores are saturated with water in the liquid form creating a barrier inside the concrete.
Both authors agree that the formation of a moisture clog has significant influence on the likelihood of spalling. Kalifa performed some tests on the pore pressure inside the concrete during heating. He noticed that the peak pressure is usually followed by a decrease in pressure due to the drying of the concrete. These observations confirm the movement of the moisture clog with increasing temperatures.

The clog is also considered to move into deeper sections of the concrete at higher temperatures. Different parameters like heating rate, permeability or vaporization rate of moisture have an influence on the movement.

**Spalling due to high vapor drag forces by Meyer-Ottens**

Based on his tests and ideas from Waubke, who analyzed transport processes in concrete [131], Meyer-Ottens [94] developed the idea of “vapor drag forces” acting inside the concrete leading to explosive spalling. With temperatures increasing above $T = 100^\circ\text{C}$, the evaporation of moisture will lead to a laminar vapor stream towards the heated concrete surface. This stream causes shear stresses along the pore cells of the concrete by friction action that is called a “drag force”. The total sum of these stresses over the affected concrete cross section leads to high tensile stresses. Explosive spalling will occur when the concrete’s tensile strength is exceeded.

Meyer-Ottens considered the pore system inside the concrete as a set of bundled tubes, in which a laminar vapor stream is possible. The velocity of the vapor movement exhibits a parabolic shape, with a lower speed towards the cell wall, based on the laws derived by Hagen-Poiseulle. By integrating over the diameter of each tube, the average velocity of the vapor stream can be estimated. Assuming constant thermal and mechanical properties of vapor during the movement, the maximum stresses inside the concrete can be computed. These stresses due to vapor movement or “bursting stresses” are a function of the moisture content of the concrete, the velocity of the $T = 100^\circ\text{C}$ isotherm and the distance from the heated surface to the moisture clog, which is then multiplied with a constant factor $a$ by integration. Spalling will occur if the bursting stresses exceed the tensile strength of the concrete. The correlation can be described as shown in equation (C - 1).

$$\sigma_{f(100^\circ\text{C})} = a \cdot m \cdot v_{100} \cdot l < f_t$$

(C - 1)

With:

- $\sigma_{f(100^\circ\text{C})}$: bursting stresses due to the movement of the $T = 100^\circ\text{C}$ isotherm in N/mm$^2$
- $a$: factor from integration ($= 7.16$)
- $m$: moisture content in %
- $v_{100}$: velocity of the $100^\circ\text{C}$ isotherm inside the concrete in m/s
- $l$: flow length, i.e. distance from the heated surface to the moisture clog in mm
- $f_t$: tensile strength of concrete in N/mm$^2$

The factor of $a = 7.16$ from integration was found to be the maximum within the range of $T = 100^\circ\text{C} - 105^\circ\text{C}$, while higher temperatures do not necessarily lead to an increase in bursting stresses. For the $T = 150^\circ\text{C}$ isotherm, the integration factor is given by 1.81, leading to much lower bursting stresses [94].

Meyer-Ottens [94] noticed that the bursting stresses increase rapidly with increasing depth and reach their maximum peak at about $l = 60 - 70$ mm from the heated concrete surface before they decrease to zero stress.
at a depth of about \( l = 120 \) mm. With increasing thickness of the concrete specimen, the peak in bursting stresses is delayed in time. In addition, the increase and decrease in stresses is less steep compared to specimens of less height.

Meyer-Ottens [94] presented the development of critical bursting stresses for a concrete with an average moisture content of \( m_w = 3\% \) and heated on two sides, as shown in Figure C1. Different thicknesses are included, but the distances from the heated surface to the moisture are not shown.

Meyer-Ottens [94] presented the development of critical bursting stresses for a concrete with an average moisture content of \( m_w = 3\% \) and heated on two sides, as shown in Figure C1. Different thicknesses are included, but the distances from the heated surface to the moisture are not shown.

**Figure C1:** Bursting stresses predicted in heated concrete according to Meyer-Ottens [94] for different widths of the concrete cross section between \( w = 50 \) mm - 200 mm

**Idealized spherical pore model by Akhtaruzzaman**

In 1973 Akhtaruzzaman [10] analyzed the loss in weight of concrete specimens during heating and measured the corresponding concrete and furnace temperature. He noticed that concrete specimens heated after water storage are more likely to exhibit explosive spalling.

In terms of spalling, Akhtaruzzaman assumed that water is sealed inside the pores and surrounded by a dense cement gel layer that are assumed to be “idealized spheres” as shown in Figure 6 on page 29. The pressure development inside the pores is considered to follow the vapor saturation pressure curve. Migration processes are not mentioned or considered. However, he observed losses in weight that are associated with the release of vapor from the inside the concrete and losses of concrete fragments due to spalling.

Akhtaruzzaman [10] noticed explosive spalling within a temperature range of \( T = 165 - 198^\circ C \), which corresponds to a vapor pressure of \( p = 0.6 - 1.4 \) N/mm\(^2\). Several of his observations in terms of explosive spalling could be characterized by the pressure buildup in these “idealized spherical pores”. He assumes that these pore pressures cause hoop stresses inside the pores. Spalling will occur if these hoop stresses exceed the tensile strength of the concrete. An analysis of the hoop stresses using a cylindrical analysis based on the expected vapor pressure was carried out. This analysis shows that the expected stresses exceed the tensile strength of the concrete, if temperature-dependent losses in strength are considered, leading to explosive spalling.
It should be mentioned that an adequate knowledge of the pore size distribution, total porosity and different saturation values of the concrete pores were not known at that time. Connolly [27] noted that the idealized sphere model can only be applied to very young concrete, since the concrete’s moisture content decreases with time and can be considered as insufficient to cause explosive spalling due to high pore pressure with increased maturity.

APPENDIX C.2 - Thermal stress-induced explosive spalling

First considerations by Saito

Based on Hasenjäger’s work [45], Saito [109] in 1965 presented his ideas on thermal stress-induced explosive spalling. According to his work, spalling can be considered as concrete compression failure at the heated surface. Saito noticed that temperature gradients within the cross section of the heated concrete lead to thermal stresses.

Further, Saito assumed that every single point within the concrete is subjected to different strains during heating, mainly longitudinal thermal strain ($\varepsilon_{\text{thermal}}$), curvature ($\frac{1}{\rho}$) and strain due to thermal stresses ($\varepsilon_{\text{s,thermal}}$). Considering hindered thermal expansion and restraints within the concrete’s cross section, these strains lead to both tensile and compressive stresses, depending mainly on the thermal and mechanical properties of the concrete and the time-dependent heating rate.

The resulting thermal strains inside heated concrete based on a typical temperature distribution for a concrete specimen with a height of $h = 200$ mm according to Saito’s theory are shown in Figure C2 according to Connolly [27].

![Figure C2: Resulting thermal strains in heated concrete according to Saito’s model [109] and computed by Connolly [27]](image)

It can be stated that compressive stresses will occur close to the heated surface, while tensile stresses dominate within the cooler sections of the concrete. Globally, the different stresses are counterbalanced. Spalling due to thermal stresses will occur if the compressive stresses close to the heated concrete surface exceed the compressive stress of the concrete. Temperature-dependent losses in compressive strength have
a favorable effect on the increased risk of spalling due to thermal stresses, since they reduce the resistance to spalling.

In order to predict the maximum compressive stress acting inside a concrete cross section, the development of temperature gradients inside the concrete was studied in detail. Connolly [27] concluded that the maximum stress would occur close to the heated surface within the first $t = 30$ min of fire due to the highest temperature gradient. This gradient becomes smaller if the specimen is exposed to high temperatures for a longer time. This prediction in terms of time until explosive spalling occurs agrees well with the general observations in tests and is mentioned as well in the literature [57].

Saito was able to explain his observations on the influence of additional loads or prestresses increasing the risk of explosive spalling. External loads lead to an initial high compressive stress of the concrete, which further increases due to thermal stresses. This increases the risk of explosive spalling.

However, several questions remain unanswered with this model of thermally stress-induced explosive spalling and it is be doubtful if the model assumptions provided by Saito are still valid today. The increased risk of spalling with the presence of higher moisture content is simply explained by increased temperature gradients during heating. In terms of failure criteria it is questionable if the presence of thermal stresses exceed the compressive strength of UHPC with a cold strength of $f_c = 150$ MPa or more. In addition, tests on the development of the hot compressive strength indicated a significant increase in compressive strength of UHPC up to almost 150% - 200% of the initial cold strength at $T = 300 - 500^\circ$C. Violent explosive spalling was usually observed in this temperature range [75]. The effect of tensile stresses in the cooler regions of the concrete and their influence on spalling are only briefly discussed.

**Further explanations by Dougill**

Dougill [32] extended Saito’s model in 1972 and tried to enhance his theory by improving possible failure modes. Dougill’s main concern was that the influence of the cross section of the concrete, mainly the thickness of the specimen on the risk of spalling, could not be properly explained with Saito’s model.

Dougill assumed that concrete at high temperatures is similar to concrete in a very stiff testing machine for tensile splitting tests where the applied stresses vary according to the height of the concrete specimen. In a tensile splitting test, outer regions of the specimen are in compression, inner parts in tension, which is identical to the stress distribution obtained from Saito’s model. However, the main difference is that the stress zones change with duration of exposure to high temperature and geometry of the specimen.

Considering different strains within the concrete during heating, namely the longitudinal thermal strain ($\varepsilon_{l,\text{thermal}}$) and the curvature ($1/\phi$), the resulting stresses and the temperature-dependent Young’s modulus, Dougill developed stress-strain relationships in the form of a stiffness matrix depending on the stress level of the concrete. In addition, he took into account the existence of strain softening and differences in slope of the descending branch of the stress-strain curve after peak load. Furthermore, the analogy with the stiffness of the testing machine was modified by assuming an increase in stiffness of the testing machine with increased in specimen thickness.
The model proposed by Dougill was able to explain the occurrence of the spalling of concrete due to thermal stresses. Once the stress caused by high temperatures reaches the maximum compressive strength of the concrete, spalling may occur, depending on the thickness of the specimen. Thicker test specimens are less subject to explosive spalling, which was explained using the analogy of a stiffer testing machine. Once the peak compressive strength of the concrete is reached and the stress-strain relations follow the descending branch, the additional energy from the deformation of the testing frame is less compared to a soft frame, which would directly lead to failure of the concrete sample.

In addition, Dougill could explain the influence of high heating rates on the spalling behavior. Considering the strain-softening of concrete, it can be noted that rapidly heated concrete has a steeper branch of the descending stress-strain curve, increasing the risk of spalling. His main conclusion, which is in contrast to Saito’s work, is that spalling is seen to be a form of instability.

**Concluding remarks on spalling due to thermal stresses**

It can be noted that both models clearly showed an influence of thermal stresses on the general risk of spalling. The influence of external loads was taken into account in the models. However, it is doubtful if all boundary conditions and failure criteria are still valid for today’s concrete, mainly with the high compressive strength of UHPC and different moisture contents. Even though Dougill could explain the increased risk of spalling of HPC and UHPC due to their brittle material behavior and the rapid descending branch of the stress strain curve, Connolly [27] noted that lightweight concrete with similar material properties has almost no risk of explosive spalling.

As concluding remarks on spalling due to high thermal stresses, Khoury [57] mentions that single thermal stress spalling itself is a rather rare occurrence and only highly loaded structures get into those areas where critical stresses due to load and temperature develop, which could then lead to thermal stress spalling.

**APPENDIX C.3 - Combined thermal stress and pore pressure-induced explosive spalling**

Starting in the mid 1970’s, models with a combined pore pressure and thermal stress spalling were developed. One of the first models was described by Zhukov in 1975 [134] (cited in [27] and [68]), which was followed by Sertmehtemoglu in 1977 [118] and at later stage by Connolly in 1995 [27], who developed new ideas on combined pore pressure and thermal stress spalling.

The combination of thermal stress and pore pressure-induced explosive spalling was analyzed in detail by Zhukov [134] in 1975. He considered uniform stresses due to restrained thermal expansion (σ_t) and stresses due to applied load (σ_L) acting onto the concrete with a stress vector parallel to the heated concrete surface. These stresses in the longitudinal direction cause cracks, mainly in thermally affected concrete areas.
Perpendicular to these stresses, additional stresses due to high pore pressure ($\sigma_p$) that act in the direction of the heated surface.

This model was later modified and presented by Khoury [57]. He also mentioned the different stresses acting in different layers of the concrete. Figure 7 shows stresses acting inside heated concrete explaining the general idea of combined thermal stress and pore pressure-induced explosive spalling as presented by Khoury [57].

Zhukov used elastic theory based on Hooke’s law to assess the total thermal stresses and corresponding energy required for spalling, leading to an equation for bursting failure. Additional stresses from external loads can be superimposed with this bursting energy. The development of high pore pressures was taken into account by a model based on vapor drag forces, similar to the findings of Meyer-Ottens [94].

The equation for bursting failure developed by Zhukov [134] is shown in equation (C - 2):

$$\sigma_{\text{rupture}}^2 = \sigma_p^2 + \nu \cdot \sigma_p \cdot (\sigma_l + 2\sigma_t)$$  \hspace{1cm} (C - 2)

With:
- $\sigma_{\text{rupture}}$: bursting stresses
- $\sigma_p$: pore pressure
- $\sigma_l$: load-induced stresses
- $\sigma_t$: thermal stresses
- $\nu$: Poisson’s ratio

Based on 35 tests on concrete slabs, Zhukov verified his model as mentioned in [27]. Additional data, further references or details on his tests are not known. According to his model, Zhukov predicted critical bursting pressures of up to $\sigma_{\text{rupture}} = 15$ N/mm$^2$ for a very dense concrete with a moisture content of 8%. This bursting stress decreases to $\sigma_{\text{rupture}} = 1.6$ N/mm$^2$ when analyzing a lightweight concrete with the same moisture content. These results show the influence of the concrete pore system and the limitation in using models based on vapor drag forces for estimating the pore pressure.

In order to improve his model, Zhukov [134] (cited in [27]) presented some equations to estimate an empirical spalling parameter $K_{\text{spall}}$ as a function of several thermal and mechanical properties of the concrete. Zhukov already noticed that high moisture content plays a major role in terms of the development of critical pore pressures. In order to provide a specific release of these high pore pressures, he extended his model by considering a structural parameter P. This parameter is similar to the permeability.
The likelihood of spalling increases with increasing values of $K_{\text{spall}}$. This general function with its input parameters is given in equation (C-3).

$$K_{\text{spall}} = f \left( \frac{\alpha \cdot E \cdot \Delta T \cdot P \cdot A \cdot m}{K_{\text{stress}} \cdot k} \right)$$

(C - 3)

With:
- $K_{\text{spall}}$: tendency of concrete to explosive spalling
- $\alpha$: coefficient of thermal expansion
- $E$: Young’s Modulus
- $\Delta T$: temperature difference between heated concrete surface and failure zone
- $P$: “structural parameter” to be considered as permeability
- $A$: area of heated concrete
- $m$: moisture content
- $K_{\text{stress}}$: ratio of imposed thermal stresses, internal pressure and stress intensity
- $k$: thermal conductivity

Even though this general function covers several material properties, an in-depth analysis of the risk of spalling cannot be performed. Mainly the distance to the failure zone and hence the required temperature gradient cannot be determined without additional models. Several parameters like the temperature gradient, the “structural parameter” or stress-ratio ($K_{\text{stress}}$) have to be calibrated with additional tests or reasonable assumptions have to be made, limiting the general reliability of the model.

As a final result, Zhukov could draw a spalling envelope for a typical concrete in which the relationship between moisture content and applied stress is as shown in Figure C3 for an ordinary concrete with granite aggregates. As a lower boundary the minimum moisture content of 3% is given. Spalling is most unlikely for lower moisture contents. He already noticed that siliceous aggregates lead to a lower boundary for a critical moisture content of 2%.

![Figure C3: Spalling zone for applied stresses and moisture content of typical concrete after Zhukov as presented in [27]](image.png)
Concluding remarks on combined thermal stress and pore pressure-induced explosive spalling

Regarding the adaptability of Zhukov’s model, it is noted that his model is limited to the area of OPC. The model does not cover the area of HPC and UHPC, since spalling could occur at any moisture content, even below 3%, as seen in several tests [75]. However, even with a moisture content below 3% brittle spalling failure will occur after $t = 5 - 10$ min of heating, which is considered to be rather early in the case of fire.

Bažant analyzed the build up of critical pore pressure in concrete at high temperatures in detail [16, 19, 20]. It is interesting to note that he considers high thermal stresses, which occur in rapidly heated concrete, are the governing factor for violent explosive spalling. As one of his conclusions, he noted that a high pore pressure can only be assumed as a “trigger”, which then leads to combined thermal stress and pore pressure spalling. He stated that the extent of explosive spalling depends mainly on the energy that is released by thermal stress spalling. He suggested that an in-depth analysis on the amount of energy released by thermal stress spalling must be carried out according to fracture mechanics.

However, recent tests [75] on unloaded UHPC cylinders have shown that even very low heating rates of just $\dot{T} = 0.5$ K/min tend to produce violent explosive spalling, even though no significant heat gradient was observed. This investigation supports the assumption that the rapid development in dense concrete with a high amount of silica fume and low permeability shifts the priority in explosive spalling clearly in the direction of a critical pore pressure.
APPENDIX D - METHODS OF PREVENTING EXPLOSIVE SPALLING

APPENDIX D.1 - Influence of fibers

Polypropylene (PP) fibers

The use of PP fibers to prevent concrete from spalling was first mentioned and discussed by Nishida et al in 1995 [100]. Concrete cylinders and columns containing PP fibers and a compressive strength between 42 MPa and 102 MPa were tested in regard to spalling at high temperatures together with possible influences on mechanical properties. The amount of PP fibers added to the fresh concrete varied between 0, 1.5- and 3.0 kg/m$^3$. As PP fibers, a short fiber with an average length of 19 mm was used; the fiber’s diameter was not mentioned.

The tests showed that the presence of PP fibers clearly reduces the risk of explosive spalling, while no significant changes in the concrete’s mechanical properties were noticed. Even tests on columns with a preload of $\sigma = 0.25 \cdot f_c$ showed less spalling compared to fiber-free test specimens.

The use of PP fibers as a protective measure has become a part within the European design standard EN 1992-1-2 [6]. There, the use of PP fibers to minimize the risk of explosive spalling of HPC is recommended. It is suggested that monofilament PP fibers with a minimum dosage of 2.0 kg/m$^3$ are added to the fresh concrete. According to some national annexes to the Eurocode, a higher amount of PP fibers of up to 4.0 kg/m$^3$ is recommended [7], depending on the concrete’s w/c ratio. However, additional advice on the fiber’s geometry is not provided in this standard.

During the development of PP fibers as a protective measure in the 1990’s and providing advice for practical design, in addition the effectiveness of PP fibers in concrete at high temperatures was analyzed. At this early stage of the use of PP fibers it was assumed that the hydrophobic polypropylene melts at temperatures of about $T = 170^\circ$C and the additional channels simply increase the concrete’s permeability. The extended pore volume enables a safe release of vapor pressure without explosive spalling.

Mode of action - Formation of Interfacial Transition Zones (ITZ theory)

Kalifa et al. analyzed in detail the pore pressure in concrete at high temperature [64] and the concrete’s microstructure containing PP fibers [63]. He noticed that even with a melting temperature of polypropylene at $T = 170^\circ$C, the molten fibers are still present inside the concrete since the actual decomposition of polypropylene starts at higher temperatures. Microcracking was detected for temperatures exceeding $T = 300^\circ$C in ongoing tests in contrast to lower temperatures of $T = 200^\circ$C, which seems to have no influence on the crack population. Furthermore, it was noticed that a higher fiber content led to an increased crack density, while a lower fiber content resulted in fewer, but wider cracks within the matrix.

Kalifa [63, 64] concluded that molten polypropylene is absorbed by the cement matrix surrounding the fibers. Since the matrix is a very porous medium, the liquid PP fibers can penetrate partially or totally into the surrounding cement. In addition, he also created the term of ITZ theory (Interfacial Transition Zones). Since it was noticed in his tests that the total pore volume hardly increases between temperatures of $T = 105^\circ$C...
and $T = 400^\circ C$, transition zones between the fibers and the cement matrix are developed, increasing the permeability of the concrete. Even though these zones are rather small, in particular for HPC and UHPC, the molten PP fibers can percolate into the cement matrix. The development of permeability is linked with the formation of these zones and strongly depends on whether the matrix is cracked or not.

**Mode of action - Pressure-induced Tangential Space (PITS theory)**

Reflecting on the ITZ theory, Khoury [67] doubts that the molten PP fibers will percolate into the cement matrix. It is not sure if the provided transport mechanisms for the molten PP fibers based on Fick’s diffusion law are reasonable. In extensive research on the mechanisms on pressure relief, Khoury stated that several “potential mechanisms” could provide sufficient pressure release such as discontinuous reservoirs and continuous channels inside the concrete. Both mechanisms are a direct or indirect result of adding PP fibers to the concrete and their decomposition at high temperatures.

Regarding decomposition of PP fibers, Khoury argued that the rather high viscosity of the molten fibers and the molecular size makes it almost impossible for the PP fibers to penetrate into the surrounding cement matrix. He created the term of PITS theory (Pressure-induced Tangential Space). Here, each PP fiber is compacted in the radial dimension due to high vapor pressure acting on the fiber. The vapor pressure can then bypass the fiber via tangential voids that develop, since the fibers are released from the concrete.

Khoury [67] noticed that mixing PP fibers to fresh concrete increases the air content by about 1% - 2%, which can be considered as “discontinuous reservoirs”. These additional voids and reservoirs can also be used to accommodate high pore pressure and supports the release of pressure via the concrete’s pore and channel system.

Microcracking plays a major role in terms of pressure release inside concrete with PP fibers. At an ambient temperature of $T = 20^\circ C$, the thermal expansion coefficient of polypropylene is roughly a factor 10 times higher that the cement matrix that would cause significant cracking during heating even before reaching the PP fibers melting temperature of about $T = 170^\circ C$. However, the rapidly decreasing Young’s modulus with increasing temperatures reduced probable critical stresses.

**Fiber geometry**

Khoury also stated that the effectiveness of spalling prevention strongly depends on the PP fibers themselves. The use of monofilament fibers seems to be promising.

Regarding the individual fiber length it is mentioned that long fibers have a negative influence on the concrete’s workability and limit the general dispersion of fibers in the concrete. In addition, the risk of insufficient interconnectivity of rather short fibers has to be taken into account too. The ratio between the fiber length and the corresponding diameter of a cement paste sphere / PP fiber should not be less than 1.0 to achieve a sufficient interconnectivity between the fibers in a perfectly dispersed system as shown in Figure D1 by Khoury. He considers that the cement paste has a share of about 30% of the total concrete volume. For practical design, a fiber length between 6 - 12 mm would be a good compromise.
Figure D1: Interconnectivity of the cement paste sphere and PP fibers [67]

Considering the total length of all PP fibers and the number of fibers, Khoury concluded that for the suggested fiber length of 6 - 12 mm, the individual fiber should be small in diameter, since this increases the cumulative surface area and the total number of fibers, leading to an overall good dispersion of the fibers and sufficient formation of microchannels for pressure release.

It has been not answered until today how the melt flow rate of PP fibers (MFR in g/(10 min)) influences the growth of microcracks and supports the formation of links between the microchannels of the molten PP fibers. However, a higher MFR is known to prevent concrete from spalling [76], and this would support the PITS theory, since a rapid melting and high fluidity of the molten fibers would promote the bypass of vapor between the fiber and the concrete.

**Melting characteristics of PP fibers in concrete by CT scans**

Pistol [106] compared existing theories on the mechanisms of PP fibers in HPC and UHPC, since some of the proposed theories are not explained in detail or lead to controversial results. He analyzed the cracking behavior of concrete cylinders during heating by acoustic emission analysis and by scanning cylinders after cooling from high temperatures using CT scans and the Scanning Electron Microscopy (SEM). These scans on concrete specimens were carried out at ambient temperatures and after cooling from $T = 200^\circ$C. This temperature is slightly above the melting temperature of the PP fibers. Additional tests were carried out after cooling from $T = 250^\circ$C and $T = 300^\circ$C. This temperature is clearly above the melting temperature of the PP fibers.

It is mentioned that the concrete specimens are heated with low rates of about 1.0 K/min to minimize the influence of probable thermal gradients. However, the conditioning time at high temperature or the cooling rates are not mentioned. The scans were carried out at residual stage after cooling to $T = 20^\circ$C.

Regarding residual tests Khoury [67] stated that the cooling phase can have an influence on the composition of the cement matrix and the PP fibers may not totally decompose after an insufficient conditioning time at high temperature. The polypropylene will recrystallize at temperatures below $T = 170^\circ$C, depending on the state of decomposition.

The results of Pistol lead to an in-depth visualization of the melting characteristics of PP fibers in concrete and the influence on the surrounding cement matrix. Table D1 summarizes the main findings from the CT and SEM analyses as presented in [106]. Since empty microchannels were noticed on tests after cooling from $T = 250^\circ$C, sufficient conditioning at high temperatures was provided.

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<table>
<thead>
<tr>
<th>PP-fiber</th>
<th>Cement paste sphere</th>
<th>Interconnection</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;1</td>
<td>No interconnection</td>
<td></td>
</tr>
<tr>
<td>=1</td>
<td></td>
<td>Interconnection</td>
</tr>
<tr>
<td>&gt;1</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table D1: CT and SEM analyses on the melting characteristics of PP fibers in concrete [106]

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Main findings</th>
</tr>
</thead>
</table>
| 20°C        | · Ambient state, all PP fibers remain inside the concrete  
             · All fibers are unmolten within the cement matrix  
             · No cracks are visible |
| 200°C       | · Temperature exceeded the melting temperature of PP fibers (170°C)  
             · Fibers start to decompose and microchannels start to build up  
             · Fibers shrink significantly but are still visible inside the microchannels  
             · Cracking close to the PP fibers |
| 250°C       | · PP fibers are fully decomposed  
             · “Empty” microchannels due to full decomposition of the fibers  
             · Microcracks start to build up along the edges and walls of the microchannels  
             · Links between the microchannels via cracks |
| 300°C       | · Links as microcracks between microchannels are fully developed |

The test were revealing in terms of analyzing the influence of PP fibers in concrete at high temperatures with regard to cracking. Pistol concluded that the analysis of concrete specimens indicated the development of microchannels due to the decomposition of PP fibers. In addition, significant growth of microcracks was noticed, linking the microchannels. It was concluded that the microchannels can be regarded as a discontinuity inside the concrete, which initiates the growth of cracks. This leads to two essential effects: relief of residual stresses and constraints (mechanical effect) and increased vapor transportation (permeability effect).

**Hazardous effect of PP fibers**

It is mentioned in the literature that PP fibers of 100% new polypropylene as used in structures release harmful gases during decomposition, like carbon monoxide, smoke and organic irritants. Within the international NewCon project, Khoury [66] analyzed the possible influence of toxic gases during the decomposition of PP fibers.

A tunnel fire and concrete with an average fiber content of 2.0 kg/m³ was considered in a quick sample calculation. During a fire exposure time of \( t = 120 \) min, the decomposition of PP fibers would release as much harmful gases as the burning of about 0.02 kg/min regular gasoline per meter of tunnel segment. In contrast, to reach the required temperatures for a decomposition of the fibers inside the concrete an equivalent amount of gasoline of 80 - 100 kg/min per meter of tunnel segment is required. Even though it could be assumed that 10 m of tunnel segment will be affected by a fire exposure of 1 m of tunnel and the decomposition of polypropylene is twice as fast as the burning of the gasoline, the contribution of the decomposition of PP fibers to a toxic environment in tunnel fire is well below 1%.

Khoury concluded that the role of PP fibers in terms of a higher risk of human fatalities in tunnel fires is negligible. Fatalities will mainly result from smoke and toxins from the fire and the very high temperatures that usually occur in tunnel fires.
Nylon (NY) fibers

Apart from the use of PP fibers, other thermoplastic materials have been checked for their possible use in concrete to minimize the risk of spalling at high temperature. Recent tests by Lee et al. [88] showed an overall good spalling protection of HPC by adding a mix of PP fibers and nylon fibers (NY fibers). As with polypropylene, nylon is a hydrophilic thermoplastic material, which softens and melts at high temperatures. The advantage of NY fibers is the increased workability of the fresh concrete and overall better mechanical properties compared to concrete protected with PP fibers. With a diameter of $\varnothing = 12 \, \mu m$ and a length of $l = 10 \, mm$ it is possible to produce very thin and short NY fibers, leading to a significantly higher number of fibers per kg of fibers and hence increasing the concrete’s permeability.

It should be noted that the diameter of new PP fibers, specially designed for an increased spalling resistance of HPC and UHPC, are within the same range of 15 - 18 $\mu m$ [75].

One main disadvantage of NY fibers is the high melting temperature of $T = 200^\circ C$, which is in close proximity to critical temperatures inside the concrete where explosive spalling was noticed [75].

The tests performed by Lee et al. [88] showed that a fiber mix of PP and NY fibers significantly improves the spalling resistance of concrete. The workability and hardened concrete properties do not differ significantly from the fiber-free concrete, and the risk of explosive spalling could be minimized even with very low dosages of PP fibers.

Polyethylene (PE) fibers

Polyethylene fibers (PE fibers) have a very low melting point of only $T = 90^\circ C$. It was thought that critical pressures can be relieved very early since the additional relief channels are available before critical pressures start to build up when the concrete exceeds temperatures of $T = 100^\circ C$. However, significant spalling was observed in tests with dosages of up to 2.0 kg/m$^3$ PE fibers [110]. It was mentioned that the high viscosity of the PE fibers after melting and the rather large diameter of 64 $\mu m$ of these fibers are the main factors leading to the increased extent of explosive spalling.

Polyvinyl chloride (PVC) fibers

Fibers made from polyvinyl chloride (PVC) might be used to relieve critical pore pressure in concrete, since PVC is an amorphous thermoplastic that directly decomposes at temperatures of about $T = 170^\circ C$, without prior melting. This would increase the direct release of vapor via additional channels. However, PVC will release harmful chloride gases at high temperatures, leading to a health risk and causing further deterioration of the concrete’s reinforcement.
Steel fibers

The use of steel fiber added to the fresh concrete is known to increase not only the ductility of the hardened concrete, but also increase the concrete’s tensile strength. It can be considered that a higher tensile strength improves the resistance to internal pressures and decreased the probable risk of explosive spalling.

Tests on loaded HPC columns with steel fibers performed by Kodur [78, 79] showed overall a significantly higher spalling resistance compared to steel fiber-free specimens. Adding 1.7% in volume (45 kg/m³) of steel fibers to the fresh concrete increased the fire resistance of a concrete column. Similar observations were made by Ali in 2008 [11], who observed significantly less spalling of loaded concrete columns containing a combination of steel and PP fibers added to the concrete.

In contrast to these observations, no improvements in terms of fire resistance due to the presence of steel fibers were observed in tests by Schneider [116] on loaded tunnel UHPC segments. The resistance gained was just a few minutes, which can be neglected in a real fire occurrence. Other tests on unloaded UHPC concrete cylinders with 2.5% in volume of steel fibers confirmed these observations [75]. No increase in spalling resistance was noticed. It should not be forgotten that the cement matrix and steel fibers have different thermal expansion characteristics; hence the steel fibers might contribute to the microcracking close to the surrounding cement matrix.

APPENDIX D.2 - Influence of protective coating

Meyer-Ottens [94] mentioned the use of a lining as a protective coating on concrete to provide a thermal barrier and reduce the risk of explosive spalling. He already noticed that both a sufficient thermal protection and resistance to a detachment of the lining must be provided. Apart from sprayed or a hand applied lining, the protection can also be achieved with the use of gypsum boards.

To provide sufficient durability and resistance to detachment of the lining, the concrete requires additional treatment like roughening the surface, applying an additional reinforcement mesh or lining base before spraying the lining onto the surface of the concrete.

Linings have a significantly lower thermal conductivity compared to concrete. Meyer-Ottens noticed that insufficient reinforcement cover, which he indicated as one source of spalling in fire, can be compensated by using a lining. Usually, 10 mm missing concrete cover can be compensated with 4 - 5 mm of lining.

The use of linings as a protective coating for concrete in terms of a thermal barrier has become common practice, because of these fundamental principles of Meyer-Ottens. Cement or gypsum-based linings are the most common types used on concrete structures. At high temperatures, materials inside the lining like vermiculite, perlite or simply gypsum release high amounts of chemically bound water, which causes a significantly lower increase in temperature at the surface of the protected ground material.

Nowadays, protective linings are usually used to increase the fire resistance of existing structures. Several slabs and beams built in the mid-1960’s and 1970’s do not provide a sufficient reinforcement cover for an
adequate fire resistance as required by the authorities. This lack in reinforcement cover can be compensated by the application of a protective lining to the concrete surface to increase fire resistance. Dimensioning charts [3] are available for different types of lining on OPC.

In these cases, protective linings act as a thermal barrier, to ensure that reinforcement temperatures do not exceed \( T = 500^\circ C \) for a specified time. In terms of the spalling of HPC and UHPC, EN 1992-1-2 [6] allows the use of linings. Here, linings are mentioned to be one method of minimizing the risk of spalling. However, no design criteria regarding the cover thickness or the lining itself are provided. It is just mentioned that coatings shall be used “where it has been proved that spalling at fire conditions do not occur” [6].

Several tests on the general application of protective linings to HPC and UHPC were carried out at ETH Zurich. The main results are summarized in a technical report [75]. Even though no design criteria have been given so far, the general application could be approved. As one main result, it was concluded that insufficient protection might be as critical as or even worse than no protection of HPC or UHPC. One test on a concrete slab exposed to the ISO fire [5] and protected with a thin layer of lining exhibited explosive spalling of similar intensity compared to an unprotected concrete slab. The protected slab spalled at once, while a continuous layer-by-layer spalling was observed with the unprotected concrete slab. However, the time until spalling for the protected specimen was four times greater compared to the unprotected one.

**Intumescent coatings**

Another method of creating thermal barriers is the use of intumescent coatings. These rather new materials are usually used on steel structures, since the thin layer of coating does not interfere with the architectonic requirements for steel structures. At high temperatures, intumescent coatings tend to develop a large increase in volume and provide a sufficient thermal barrier.

Basic dimensioning tools for the practicable application of intumescent coatings to steel were developed at ETH Zurich [107] and are now part of the European standards [9]. Regarding intumescent coatings on concrete structures, no sufficient design criteria are available.

**APPENDIX D.3 - Influence of admixtures**

Another possibility of minimizing the risk of explosive spalling is the use of air-entrained agents, first used in the mid 1990’s by Connolly. These liquid agents can be poured into the concrete during mixing and cause small air-voids inside the hardened concrete with an average size of the additional pores of \( \phi < 0.3 \text{ mm} \) at a spacing of \( l = 0.2 \text{ mm} \). These voids act like release areas for the high pressure, minimizing the risk of explosive spalling.

Connolly [27] observed in his analysis that air entrainment reduces the pore saturation within the concrete and hence lowers the risk of explosive spalling.

Recent tests on C25/30 concrete specimens [55] with an air content of up to 7% showed similar results. Slabs were exposed to the ISO fire curve and the amount of spalled particles was compared to a concrete mix
without additional air entrainment. While none or only minor spalling was noticed on the slabs with higher air content, significant spalling was noticed on common mixes without air entrainment. It should be noted that the permeability of the concrete was not measured and the strength class of the tested specimens is rather low.

Nowadays the use of fine aggregates, like quartzite filler and silica fume for UHPC, reduces the scope of application of air-entrained agents. Fillers usually have a similar diameter compared to the air voids. Further, they fill these voids, minimize the additional entrained air, increase the degree of pore saturation but have no further beneficial effects on the spalling resistance. Apart from the rather high costs for the liquid agent, additional pores increase the concrete’s permeability even at ambient temperatures. In areas with high requirements necessitating a very dense concrete like ground water protection areas or tunnel linings, the use of air-entrained agents is debatable. In addition to the losses in durability, Khoury [67] noticed that an increase in air bubbles by 1% decreases the compressive strength of about 5.5%.

APPENDIX D.4 - Influence of modified concrete mixes and structural design

It was noticed that explosive spalling usually occurs up to concrete temperatures of about $T = 500^\circ C$ [75]. At higher temperatures, spalling of the concrete’s aggregates might still occur. As mentioned in chapter 2.2, the use of basalt as concrete aggregates minimizes the risk of aggregate spalling, since it remains stable up to temperatures of $T = 900^\circ C$. Lightweight aggregates can even withstand temperatures of $T = 1000^\circ C$ without any changes in the chemical or physical composition. However, large expansions of the aggregates are known once they exceed these maximum temperature levels [27, 68].

In terms of a safe and economic design of concrete it is difficult to modify the concrete mix in order to decrease the risk of spalling. Some aggregates have limited availability or do not match the required design criteria in terms of strength and durability.

In contrast to changes in the concrete mix, Hasenjäger [45] already concluded from his tests in 1935 that the risk and extent of spalling can be minimized by simple structural design modifications like the provision of dilatation joints and placing additional stirrups in areas of high shear.

EN 1992-1-2 [6] suggests the use of a secondary reinforcement mesh between the main reinforcement and the fire exposed concrete surface which is called method “A” as good way to minimize the risk of explosive spalling. Again, this idea is based on Meyer-Otten’s work [94] from 1972. He developed an improved reinforcement layout for beams; where the main reinforcement bars are located further inwards in the concrete cross section. He suggested a minimum spacing of three times the diameter of reinforcement bars with a limited diameter and a concrete cover that might be higher compared to the required cold design. An additional secondary reinforcement mesh in between the reinforcement bars and the concrete surface reduces the formation of cracks close to the heated surface and the reinforcement bars leading to spalling. Meyer-Otten’s modified layout for concrete beams is shown in Figure D2.
In addition to the modified reinforcement layout, Meyer-Ottens presented several additional design modifications to minimize the risk of spalling. Some are related to minimizing high thermal gradients as observed at concrete corners. He developed new design criteria for the cross section of concrete segments. These included the minimizing of sharp edges and corners by making them round as well as providing minimum dimensions for concrete slabs and beams. His suggested dimensions might be higher than the required cold design for the ultimate limit state.

Apart from concrete slabs, columns show an increased risk of spalling, in particular if they are loaded during heating. Kodur [81-83] analyzed the spalling behavior of HPC columns in detail and developed a modified design layout with less spacing between the ties. Reinforcement with a diameter of \( \varnothing = 8.0 \) mm was used and a spacing of the ties between \( s = 406 \) mm and \( s = 203 \) mm. This corresponds to a minimum spacing of about only 50% compared to the original design. At a second test series, spacing was further reduced to \( s = 145 \) mm in the central regions and \( s = 75 \) mm close to the ends.

The ties were bent back towards the inner section of the column at an angle of 135°; hence the steel ends are embedded inside the concrete. In some columns additional cross-ties were used [81]. The reinforcement cover is given as \( c = 40 \) mm. The general layout is shown in Figure D3. This improved detailing of the ties corresponds with the ideas of Meyer-Ottens, who stated that thin reinforcement bars with adequate spacing should be used and an increased concrete cover is more favorable regarding a spalling-safe concrete design.
Several columns were loaded centrically with their maximum design load and exposed to the standardized ASTM time-temperature curves [2]. The test by Kodur [81, 83] showed that the loaded columns could resist the fire exposure for the very long time of over 6 h. After the first test series, the improved bending of the ties was found to increase the fire resistance compared to a conventional design. Common 90° tie bending might widen or even open up at the overlapping of the ties which leads to failure of the column.

The further improved tie design in combination with less spacing showed that not only the fire resistance was higher but also the extent of spalling was reduced. The closer spacing seems to minimize the extent of spalling. With the $T = 135^\circ C$ tie layout, the ends of the ties are still embedded in the concrete, keeping the longitudinal reinforcement in place, minimizing a possible eccentricity and buckling. This approach of a modified tie layout by Kodur is “based on seismic design principles” [77]. Spalling is acceptable with this model and still might occur. However, the extent of explosive spalling, minimizing the concrete cross section is reduced and limited to areas between the ties. The risk of failure of heat-exposed columns is then based on the longitudinal forces inside the ties $F_{tie}$ and the corresponding resisting bond strength $F_{bond}$ between the reinforcement ties and the concrete. Failure is expected if the ties start to open since the longitudinal stresses inside the ties exceed the bond strength as resistance as shown in equation (D-1).

\[ F_{tie} > F_{bond} \]  

failure due to opening of the reinforcement ties  

(D-1)

With:
- $F_{tie}$: Longitudinal force in reinforcement ties due to stress components
- $F_{bond}$: Force according to bond strength between ties and the concrete interface

Figure D4 shows a free body diagram including the reinforcement layout and the corresponding forces according to the model by Kodur [77].

![Figure D4: Lateral and longitudinal reinforcement including forces in ties and bond strength [77]](image-url)
The forces acting inside the ties $F_{tie}$ can be expressed as a function based on the cross section of the tie and the stress component of the inner section of the concrete core $\sigma_{core}$. This term can be expressed as in equation (D - 2) and (D - 3).

$$F_{tie} = 2 \cdot A_{tie} \cdot \sin \left( \frac{\pi}{4} \right) \cdot \sigma_{core}$$  \hspace{1cm} (D - 2)

$$\sigma_{core} = \sigma_{Pv} + \sigma_{thermal} + \sigma_{mechanical}$$  \hspace{1cm} (D - 3)

With:
- $A_{tie}$: cross section of reinforcement ties
- $\sigma_{core}$: stresses inside the inner section of the concrete
- $\sigma_{Pv}$: stresses due to temperature-dependent pore pressures
- $\sigma_{thermal}$: thermal stresses due to temperature gradients
- $\sigma_{mechanical}$: load-induced mechanical stresses

The stresses due to temperature or mechanical loads can be predicted with existing models. The stresses due to the pore pressure $\sigma_{Pv}$ can be predicted according to the hydrothermal model of Dwaikat and Kodur [33] as discussed in appendix F.2.

It is interesting to note that the concrete between the ties showed the usual shape of truncated pyramids as known from regular compression tests on concrete cylinders [77, 81]. It might be concluded that failure of the column is also caused by loss in compressive strength of the concrete.

Lindbauer tested concrete slabs with a very dense reinforcement mesh [90] using rebars with a diameter of $\phi = 14$ mm and a spacing of $s = 100$ mm. An additional reinforcement mesh with $\phi = 5$ mm and a spacing of $s = 100$ mm was also applied to some tests. Similar to Kodur’s findings, it was observed that spalling ceased after reaching the additional dense reinforcement mesh. The main reinforcement was still covered by concrete and remained undamaged.
APPENDIX E - MODELS FOR EXPLOSIVE SPALLING DUE TO HIGH PORE PRESSURE

APPENDIX E.1 - First models for spalling due to high pore pressure

Moisture clog model by Shorter and Harmathy

As briefly explained in chapter 2.4.2, one of the first models for explosive spalling due to high pore pressure was developed by Shorter and Harmathy \[120\] in the mid-1960’s. The presence of a moisture clog was already recognized at this early stage of modeling and research on spalling.

As shown in Figure 5, the moisture inside the concrete is assumed to migrate further inwards, following the \( T = 100°C \) isotherm. It is considered that the water vaporizes at this temperature and leads to a congested zone close to the heated surface.

Shorter and Harmathy considered two opposing movements inside the concrete. First the movement of the moisture clog according to the heat flux \( v_h \) that moves from the heated surface further into the concrete. In addition the moisture clog movement according to the pressure gradient \( v_p \) is mentioned, which moves towards the surface due to the pressure gradient inside the concrete.

As an initial condition, pressure will build up if \( v_h > v_p \). In addition, failure due to spalling will most likely happen if the pressure gradient inside the concrete due to the opposing movements exceeds the tensile strength of the concrete.

A minimum degree of pore saturation \( \phi/\varepsilon \) is required to achieve sufficient pore pressure. This minimum degree of saturation is a function of permeability, porosity as well as several geometry parameters related to the modeled cross section.

Shorter and Harmathy provide a simple but sophisticated model for predicting possible high pore pressure inside concrete. The model is based on several material-related parameters that can be determined in small-scale tests for new concrete mixes. However, in terms of the movement of the vaporization front \( v_h \) it was noticed in own tests \[75\] that each concrete mix has an individual vaporization temperature. This temperature is mainly based on the permeability of the concrete and other governing parameters as discussed in chapter 3.3.6. This makes a clear statement on vaporization temperature and the related energy demand almost impossible. These uncertainties limit the general application of this model nowadays.

In addition, it was noticed that the increase in temperature during the vaporization process of moisture is limited due to the additional energy uptake, which would also lead to a delay in moisture migration. However, this effect is not taken into account in the model.

Modifications of the moisture clog model by Sertmehemetoglu

Sertmehemetoglu provided modifications to the model in 1977 \[118\]. He developed a term for the movement of the moisture from the clog via the dry concrete to the heated surface. The velocity \( v_i \) is similar to the volumetric flow of gas in porous media. The permeability, viscosity as well as the difference in pressure
between the clog and ambient air is taken into account in this modification. This leads to a new failure
criterion for concrete given in equation (E-1).

\[(\alpha - 1) \cdot v_h - v_p > v_s\]  \hspace{1cm} \text{(E-1)}

With:
- \(\alpha\): degree of pore saturation
- \(v_h\): velocity of moisture clog according to the heat flux moving away from the surface
- \(v_p\): velocity of moisture clog according to pressure gradient towards the heated surface
- \(v_s\): velocity of moisture migration from the clog to the surface

**Spalling liability curve by Sertmehemetoglu**

In addition, Sertmehemetoglu rearranged some equations in order to develop a spalling liability curve for
concrete. This is mainly based on the different movements of the clog and concrete-related material
properties like permeability \(k\) or the degree of pore saturation \(\phi/\varepsilon\). The equations of this model are not
explained in detail. Additional data and discussion is provided in the thesis by Connolly in 1995 [27].

Figure E1 shows this spalling liability curve adapted to the test results from own tests as discussed in chapter 3
and compared with the results presented in the literature. It is interesting to note, that the curve is clearly
shifted towards a lower permeability compared to the results presented in the literature. This is mainly due
to the basic consideration of the model, which states that vaporization starts at \(T = 100^\circ\text{C}\) with rather low
pressure gradients.

Considering the permeability of the M3 concrete, which is about \(k_v = 10^{-16}\text{ m}^2\) at the end of the plateau phase
where the highest pressure and lowest degree of saturation are expected, the model predicts that spalling
will probably occur. Tests have shown that this is not the case.

Even though this model is considered to be invalid today for HPC and UHPC due to the movement of the
vaporization front, which is assumed to be at \(T = 100^\circ\text{C}\), it takes several material-related parameters into
account and provides several hints on additional research. For example, the movement of moisture in porous
media according to pressure gradients might be considered.

![Figure E1: Spalling liability curve based on moisture clog model](image)

- left: based on material properties by Shorter, Harmathy and Sertmehemetoglu [118, 120]
- right: own material properties [75]
Appendix E - Models for explosive spalling due to high pore pressure

Vapor drag forces by Meyer-Ottens

In 1972 Meyer-Ottens [94] developed the idea of “vapor drag forces” as discussed in chapter 2.3.2. These drag forces are assumed to lead to explosive spalling by exceeding the tensile strength of the concrete.

With increasing temperatures above T = 100°C, the evaporation of moisture will lead to a laminar vapor stream towards the heated concrete surface. This stream causes shear stresses to develop along the pore cells of the concrete by friction, called “drag forces”. The total sum of these stresses over the affected concrete cross section leads to high tensile stresses, which will cause explosive spalling if these stresses exceed the tensile strength of the concrete. It remains unclear if sufficient moisture is present inside the concrete, leading to a continuous vapor stream, which then causes the proposed shear stresses.

In terms of the general adaptivity of the model by Meyer-Ottens it must be mentioned that as with the model by Shorter and Harmath it considers the vaporization of moisture at a temperature of T = 100°C, which is not valid for HPC and UHPC mixes. In addition, any material-related parameters influencing the moisture migration, like permeability or viscosity of vapor, are not included in his model.

Connolly [27] mentioned in addition that the velocity of the T = 100°C isotherm clearly depends on the heating rate of the concrete and its moisture content. Due to the higher peak in specific heat between T = 100°C and T = 200°C, concrete with increased moisture content will have a slower velocity of the T = 100°C isotherm. According to the theory of Meyer-Ottens, a dry concrete with a rapid movement of the T = 100°C isotherm will show an increased bursting stress and a tendency towards explosive spalling compared to a saturated concrete. This is clearly not the case, since own tests by Meyer-Ottens and others gave different results.

Model for pore pressure development by Connolly

Connolly presented a model for predicting pore pressures inside heated concrete in 1995 [27]. His model considers the movement of the vapor \( v_{vapor} \), which is proportional to the heating rate of the concrete. It is mentioned that the vaporization front inside the heated concrete leads to a saturated layer, the moisture clog at a certain distance from the heated surface. The velocity of the vapor \( v_{vapor} \) is based on the movement of the T = 100°C isotherm which is not necessarily valid for HPC and UHPC.

Based on literature studies, Connolly considers additional movement of the moisture clog \( v_{clog} \) due to the high pressure gradient further inwards into cooler sections of the concrete. The velocity of the moisture clog \( v_{clog} \) is a function of the pressure gradient and mainly the viscosity of water at high temperatures. This movement is rather low if compared to the flow of vapor inside the concrete.

The peak pressure inside the concrete is reached when the vaporization front reaches its maximum.

It should be noted that the use of HPC and UHPC mixes was uncommon at that time and only OPC mixes were used in his tests. Connolly validated his model using concrete specimens with a w/c ratio = 0.6, which is more than double that of the the concrete mixes used in own tests. This leads to a higher permeability of the concrete and decreases the vaporization temperature in the region of T = 100°C.
The pressure is transformed to predict stresses acting inside the concrete due to high pore pressure. These stresses due to pore pressure $\sigma_p$ are one of three main influences leading to spalling. Stresses due to temperature $\sigma_t$ and external loads $\sigma_l$ are also included. A combination of these three components divided by the tensile strength of the concrete must be smaller than 1 to minimize the risk of spalling. The model of Connolly is one of the first models to superimpose these stresses in an analytical model.

Based on several results from own tests and additional probabilistic considerations a critical ratio for the possible occurrence of explosive spalling is mentioned.

**Numerical thermo-hygro mechanical models**

Some numerical models on the risk of spalling due to high pore pressure are presented in the literature. These thermo-hygro mechanical (THM) models usually take the ongoing damage of concrete into account and include several overlapping failure mechanisms.

The “Numerical analysis of hygro-thermal behaviour and damage of concrete at high temperature” by Gawin [42] provides an in-depth analysis of the risk of spalling. The model includes nonlinear terms for liquid / vapor transport and dry air migration inside the concrete. The terms are coupled with mechanical properties.

The model covers heat and mass transfer inside concrete at high temperatures. All material properties were included as governing equations, including models of the vapor diffusivity, porosity and permeability of concrete at high temperatures. As output, the gas pressure inside a cross section and the damage parameter distribution is provided to assess the risk of spalling.

The above models, representing several numerical models provided in the past, are very precise in terms of predicting the pressure development. However, they all have one drawback in common: All models are only as good as the input parameters required for modeling. Some of these - like the permeability of concrete at high temperatures - are not well established.

**APPENDIX E.2 - Hydrothermal model for predicting fire-induced spalling by Dwaikat**

In order to establish a model for practical purposes and to predict the risk of explosive spalling due to high pore pressure with sufficient precision, Dwaikat and Kodur presented a hydrothermal model in 2009 [33].

The aim was develop a hydrothermal model for pore pressure calculations, taking fundamental thermodynamic and mechanical properties into account. Similar to own considerations, the model is discretized in small segments and time steps to estimate the time-dependent pressure development in different sections of the modeled concrete specimen. Thermal properties for the concrete were taken from standard manuals [89]. The temperature for each section and time step was modeled numerically.

Several assumptions and simplifications were made. Among others the influence of thermal stresses is neglected, since the general area of application is mainly limited to dense HPC and UHPC mixes. The mobility of water as liquid is also ignored, due to its high viscosity.
Calculation of the pore pressure development

The calculation of the pressure development is based on the assumption that the conservation of mass for water and vapor is given as shown in equation (E - 2). In addition, vapor is assumed to be an ideal gas and the pressure development according to the ideal gas law applies as shown in equation (E - 3). Finally, the total sum of all volume fractions is equal to 1 per volume unit as shown in equation (E - 4).

\[
m_v = m_l(t=0) - m_l + m_D
\]  
(E - 2)

\[
p_{\text{vapor}} \cdot V_{\text{vapor}} = \frac{m_{\text{vapor}}}{M_{\text{vapor}}} \cdot R_m \cdot T
\]  
(E - 3)

\[
V_{\text{vapor}} + V_l + (V_{S0} - V_D) = 1
\]  
(E - 4)

with:
- \( m_v \) mass of vapor in kg
- \( m_{l(t=0)} \) initial mass of water (liquid) at time t=0
- \( m_l \) mass of water (liquid)
- \( m_D \) mass of dehydrated water
- \( M_{\text{vapor}} \) molar mass in kg/mol
- \( R_m \) molar gas constant for vapor (≈ 18.1 g/mol)
- \( V_{\text{vapor}} \) volume of vapor in m\(^3\)
- \( V_l \) volume of water (liquid) in m\(^3\)
- \( V_{S0} \) volume of initial moisture content in m\(^3\)
- \( V_D \) volume of dehydrated water (liquid) m\(^3\)

With these three governing equations, differential equations for the pore pressure development were established, in order to predict the pressure development in different sections and at all time steps of the modeled section.

The initial pore pressure \( p_{\text{vapor}}(t=0) \) is required as a boundary condition for solving the differential equation. Here, the initial saturation pressure at the ambient temperature was taken, multiplied with the percentage of the relative humidity of the concrete pores in the ambient state. Higher temperatures lead to changes in the relative humidity of the concrete pores. These changes are taken into account.

Temperature-dependent permeability of concrete

In addition to the rise in pore pressure, the release of vapor is also included in Darcy’s law for the mass flux due to pressure gradients inside the concrete. The temperature-dependent permeability of the concrete is required as material property.

Finally, it is considered that spalling will occur if the pore pressure exceeds the temperature-dependent tensile strength of the concrete. This failure criterion is shown in equation (E - 5).
\[ \varepsilon \cdot p_{vapor} > f_t(T) \quad (E-5) \]

with:

\[ \varepsilon \] concrete porosity
\[ f_t(T) \] temperature-dependent tensile strength of concrete in MPa

In terms of permeability, Dwaikat considers the temperature-dependent model on permeability according to Gawin. This model is discussed in detail in chapter 3.3.6 and shown in equation (E-6). Even though the model by Gawin did not match the actual permeability determined in own tests, it provides overall a good prediction.

\[ k_{v(T)} = k_{v(0)} \cdot \left(10^{A_T(T-T_0)}\right) \cdot \left(\frac{P_R}{P_{Atm}}\right)^{A_p} \quad (E-6) \]

For the parameter \( A_T \) a more conservative value of \( A_T = 0.0025 \) is assumed and \( A_p \) is chosen as \( A_p = 0.368 \). The intrinsic permeability was determined by calibrating the model and was chosen as \( k_{v(0)} = 1 \cdot 10^{-18} \text{ m}^2 \).

**Dehydration at high temperatures and failure criteria**

Decomposition and dehydration of the cement paste and the additional mass of dehydrated water \( m_D \) is considered too. This additional moisture is estimated with the approach by Bažant [18] as given in equation (F-14) and discussed in chapter 3.3.8.

As failure criterion, a high pore pressure, exceeding the tensile strength of the concrete is proposed. The temperature-dependent decrease is estimated in a similar way to the EN 1992-1-2 design standard [6]. An additional requirement was added, namely the tensile strength decreases linearly from \( T = 100\text{°C} \) to \( T = 550\text{°C} \) to 10% of the initial tensile strength and then decreases to a total loss in tensile strength at \( T = 1200\text{°C} \).

**Case studies for verification by Dwaikat**

With the help of the model, several case studies were performed in which a concrete slab exposed on one side was modeled. It was noted that the pressure rises with time to a peak pressure and decreases slowly afterwards due to drying of the pores.

As in the review of pore pressure measurements discussed in chapter 3.4.4, the results from the model showed an increased pore pressure with increasing distance from the heated surface.

As output, a curve indicating losses in the slab thickness could be derived with its slope depending on the input parameters. The same observations as in own tests were made: a higher tensile strength, higher permeability or lower heating rate decreases the amount of spalled concrete with the time. The relative humidity of the pores was observed to have only a minor influence of the extent of spalling. A slightly higher spalling rate was observed for higher degrees of relative humidity.

Figure E2 shows the expected pore pressure development and predicted losses in the slab thickness compared to an experimental result.
In terms of the study of different heating rates, it should be noted that usually temperatures according to standardized fire curves were taken for the analysis. The lowest heating rate still had an increase in temperature of $\dot{T} = 15$ K/min to $T = 900^\circ$C. These high heating rates are in contrast to the heating rates used in own tests.

**Additional case studies based on Dwaikat’s model by Lu**

Several case studies by Lu [91] on this hydrothermal model showed that the pressure development at different depths of the modeled concrete specimen did not vary much with heating rates between $\dot{T} = 0.5 - 5$ K/min and similar peak pressures are noticed. The predicted pore pressure is shown in Figure E3. This is in clear contrast to observations in own tests, where even these low heating rates have a significant influence on the possibility of spalling.

**Figure E2:** Hydrothermal model on pore pressure by Dwaikat and Kodur [33]
left: pore pressure development at different depths of the modeled section
right: predicted reduced thickness of concrete slab compared to test results

**Figure E3:** Pore pressure development at 30 mm depth from the heated surface with different heating rates
General conclusions on hydrothermal model predicting spalling by Dwaikat

The model by Dwaikat on pore pressure development at high temperature leading to spalling can be summarized as follows:

- It can be concluded that the model by Dwaikat provides excellent results in terms of the general prediction of critical pore pressure leading to explosive spalling. All governing input parameters are based on well-established models.

- The fact that thermal stresses and their influence are neglected is only a minor drawback, since pore pressures are assumed to be the most critical in terms of mechanisms leading to explosive spalling, in particular for HPC and UHPC.

- A rather high pore pressure of up to \( p = 4.0 \) MPa can be expected with this model. The corresponding temperature at peak pressure is not mentioned. In addition, it is not known if the pore pressure development follows the pressure according to the saturation vapor pressure curve during the increasing branch of the pressure curve.

- With the help of the model, the ongoing spalling of concrete members with time can be predicted. However, it remains unclear how sudden increases in temperature due to spalling are taken into account in this model and if the model is still generally valid after the first occurrence of spalling.

- Migration processes, mainly in form of vapor migration, are known to occur inside heated concrete, with a big influence on the risk of explosive spalling. The decrease in pore pressure due to drying of the pores is taken into account in the model. However, it remains unclear how any migration processes for vapor inside the concrete are included in the model.

- In terms of the failure criterion, it is difficult to see why the expected pore pressure is multiplied by the porosity of the concrete and hence reduced to 10 - 15% of the initial pore pressure. With an expected pore pressure of about \( p = 4.0 \) MPa at an expected temperature of \( T = 250^\circ \text{C} \) then multiplied with the porosity, spalling is most unlikely since the tensile strength is usually higher than the reduced pressure.
APPENDIX F - MODELS FOR CONCRETE AT HIGH TEMPERATURES

Appendix F.1 - Models for the permeability of concrete at high temperatures

Appendix F.1.1 - Permeability as a function of temperature - Model by Davie

Davie [29] compares the results of several tests on the permeability of concrete at high temperatures and presented his results in 2012. In terms of the intrinsic permeability of concrete at T = 20°C, he summarized several results presented in the literature and noticed that the average intrinsic permeability for HPC varies from \( k_v = 1.0 \cdot 10^{-22} - 1.0 \cdot 10^{-17} \text{ m}^2 \), which increases to a higher permeability of \( k_v = 5.0 \cdot 10^{-16} \text{ m}^2 \) for OPC. It is not mentioned if any PP fibers were included in some of the tested concrete mixes or if the use of PP fibers have any influence on the intrinsic permeability in general. As conclusion for this testing overview, it can be stated that the intrinsic permeability reported in the literature corresponds with the experience made in own tests. With regard to PP fibers, it was noticed that the tested M1 and M2 concrete mixes and the corresponding -V3 and -V4 sub-mixes containing PP fibers have no significant influence on the intrinsic permeability of the concrete. The M3 concrete mix was different, mainly due to the poor workability.

It is confirmed that mixes containing PP fibers show a rapid increase in permeability with rising temperatures and are less important in terms of analytical modeling. For these concrete mixes containing PP fibers, the permeability may be assumed to be a constant function based on the intrinsic permeability, independent on the temperature. This idea of a constant function was also mentioned by Davie [29]. However, the test results show that a change in porosity at the melting temperature of the PP fibers at \( T = 170°C \) should be considered. A function as shown in equation (F - 1), which is constant within certain temperature ranges, could be used for concrete mixes containing PP fibers. The material coefficient \( C \) is chosen according to the specific concrete mix to achieve a good overall agreement between the measured permeability and analytical results. For the M1 - M3 -V3 concrete mixes, this coefficient was chosen as \( C = 4.0, 4.2 \) and 3.9. The results from this model are shown in Figure F1.

\[
k_v(T) = \begin{cases} 
  k_v(0) & \text{for } T < 170°C \\
  k_v(0) \cdot (10^C) & \text{for } T > 170°C 
\end{cases} \tag{F - 1}
\]

With:

- \( C \) material coefficient depending on concrete mix

It is obvious that this approach only leads to satisfying results for a steep increase in permeability starting at a specific temperature as observed with the concrete sub-mixes -V3 and -V4 containing PP fibers. In terms of concrete mixes without PP fibers this model is far too vague.

Within the last decade, several approaches for implementing the temperature-dependent changes in analytical models were made. Some of them were presented by Bary, Technev, Gawin, Picandet or Souley ([42, 104, 122, 125] cited in [29]). The aim is to check if any of these models would show agreement with the results of own tests.
The different analytical models were studied for all three types of concrete M1 - M3. Here, the -V2 sub-mixes were used, since a concrete mix with steel fibers is most common for practical design. For each concrete mix the intrinsic permeability $k_{v0}$ was taken as reference value.

### Appendix F.1.2 - Permeability as a function of temperature - Models by Bary, Picandet and Souley

Regarding analytical models, Bary (cited in [29, 43]) suggested a potential function, based on the intrinsic permeability of the concrete and a parameter for the damage of the concrete $D$ which varies as a constant value from $D = 0 - 1$ as shown in equation (F - 2). This leads to a constant value for the permeability, which is independent of the temperature itself. The parameter was mentioned by Davie in 2010 [28] and is defined as a parameter measuring the reduction of the resisting area of concrete caused by cracking and the growth of cracks. The deterioration factor $D$ was found to be $D = 0.29$ for the M1-V2 concrete, $D = 0.65$ and $D = 0.46$ for the M2 and M3 concrete mixes, respectively. However, a general agreement with the measured data is not observed. It can be shown that any temperature-dependent changes in permeability are not taken into account with this model. The permeability is simply shifted to a higher constant value with an increasing deterioration factor $D$. Figure F1 shows the results for this model.

$$k_{v(T)} = k_{v(0)} \cdot (10^{4D})$$

(F - 2)

With:

- $D$: damage factor of concrete ($D = 0$ undamaged - $D = 1$ fully damaged)

The idea of a damage factor is also mentioned by Picandet (equation (F - 3)) and Souley (equation (F - 4)), who established similar models and added additional material coefficients. These additional coefficients $\alpha$, $\gamma$, $C$ and $D_0$ have no physical meaning and are used for curve-fitting purposes. However, similar to Bary’s approach, the permeability of the concrete remains a constant function and does not change with increasing
Appendix F - Models for concrete at high temperatures

temperature. The required material coefficients are independent of temperature. The results are not presented.

\[
k_v(T) = k_v(0) \cdot e^{((\alpha \cdot D)^\gamma)} \quad (F - 3)
\]

Souley

\[
k_v(T) = \begin{cases} 
    k_v(0) \cdot (10^{C(D-D_0)}) & \text{for } D > D_0 \\
    k_v(0) & \text{for } D \leq D_0
\end{cases}
\]

With:

\(\alpha, \gamma, C, D_0\) constant material coefficient

\(D\) damage factor of concrete

Appendix F.1.3 - Permeability as a function of temperature - Model by Tenchev

The model presented by Tenchev is different, since it only depends on the concrete porosity, which is also a temperature-dependent function. At first sight this seems to be a promising approach since the permeability of the concrete - the interconnectivity of pores - increases with the concrete porosity. The concrete porosity was measured at an initial temperature of \(T = 20^\circ\text{C}\) and after heating to \(T = 500^\circ\text{C}\) with heating rates that are known to be below critical rates causing explosive spalling, as presented in chapter 3.3.7. With these input parameters, the probable permeability for the three concrete mixes were estimated according to equation (F - 5). The results are shown in Figure F2. The concrete porosity increases with increasing temperatures, which leads to higher values for the permeability. However, this polynomial term has only a minor influence on the increase of the permeability and the actual results are clearly underestimated.

\[
k_v(T) = k_v(0) \cdot \left(\frac{\varepsilon(T)}{\varepsilon_0}\right)^2 \quad (F - 5)
\]

With:

\(\varepsilon(T)\) temperature-dependent porosity of the concrete

\(\varepsilon_0\) initial porosity at \(T = 20^\circ\text{C}\)

Appendix F.1.4 - Permeability as a function of temperature - Model by Gawin

Gawin [29, 42, 43] developed a model based on the concrete temperature and a modified model where the damage of the concrete is also taken into account. The general slope of the curve for temperature-dependent permeability is adjusted using several material coefficients. As with the other models, these coefficients have no physical meaning and are only used for curve fitting. The first model is given in equation (F - 6).

The influence of different gas pressure levels is also considered in this model through the coefficient \(P_0\). This factor is usually determined when testing the permeability of concrete by injecting gas under high pressure into the concrete. It is mentioned in the literature that the term \(P_0/P_{\text{Atm}}\) usually varies between 1 and 20 [29, 33]. Since own tests were made at low pressures, the term is taken as 1; hence \(A_p\) has no influence.

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An additional term for the damage of the concrete is included in the modified model as shown in equation (F - 7). Based on the work of Bary, this damage factor may vary between \( D = 0 \) for undamaged concrete and \( D = 1 \) for totally damaged concrete.

\[
Gawin \quad k_v(T) = k_v(0) \cdot \left( 10^{A_T(T-T_0)} \right) \cdot \left( \frac{P_G}{P_{Atm}} \right)^{Ap} 
\]  

(Gawin)

\[
Gawin \text{ (modified)} \quad k_v(T) = k_v(0) \cdot \left( 10^{A_T(T-T_0)} \right) \cdot \left( \frac{P_G}{P_{Atm}} \right)^{Ap} \cdot 10^{A_D D} 
\]  

(Gawin modified)

With:
- \( A_T \): material coefficient
- \( A_p \): material coefficient
- \( A_D \): material coefficient
- \( D \): damage factor
- \( T \): temperature
- \( T_0 \): initial temperature of \( T = 0^\circ C \)
- \( P_G \): gas pressure inside concrete
- \( P_{Atm} \): atmospheric pressure (=100 hPa)

Both models are studied for all three concrete mixes M1 - M3 including the fiber free sub-mix -V1 and the sub-mix -V2 containing steel fibers. Table F1 summarizes all coefficients for the two models. A significant difference between the two models is not evident. Sub-mixes V3 and V4 were not analyzed. No reasonable results could be achieved with the models for permeability due to the rapid increase in permeability after melting of the PP fibers.

Table F1: Temperature-dependent development of permeability according to Gawin

<table>
<thead>
<tr>
<th>Model</th>
<th>proposed values [29]</th>
<th>M1 V1)</th>
<th>V2)</th>
<th>M2 V1)</th>
<th>V2)</th>
<th>M3 V1)</th>
<th>V2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gawin (eq. (F - 6))</td>
<td>( A_T ) 0.005</td>
<td>0.005</td>
<td>0.0025</td>
<td>0.012</td>
<td>0.009</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>( A_p ) 0.368</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gawin modified (eq. (F - 7))</td>
<td>( A_T ) 0.0025</td>
<td>0.005</td>
<td>0.0025</td>
<td>0.12</td>
<td>0.01</td>
<td>0.008</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>( A_p ) 0.368</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>( A_D ) 2</td>
<td>0.01</td>
<td>0.01</td>
<td>0.005</td>
<td>0.1</td>
<td>0.75</td>
<td>0.01</td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>0.3</td>
<td>0.3</td>
<td>0.5</td>
<td>0.1</td>
<td>0.9</td>
<td>0.46</td>
</tr>
</tbody>
</table>

1) Concrete mix without any fibers
2) Concrete mix with 2.5% in volume of steel fibers (no PP fibers)

Figure F2 shows the results for the -V2 concrete for both models of Gawin. The results for the M1 concrete mix are shown in Figure F3. Both models show in general adequate results in terms of temperature-dependent development of the permeability. However, the decrease in permeability in the temperature range \( T = 100 - 250^\circ C \) for the M1 and M2 concrete mixes are not reproduced correctly by both models. It is noted that no significant increase in accuracy of the model is achieved by adding the term for concrete damage.
Appendix F - Models for concrete at high temperatures

Figure F2: Analytical models for permeability of concrete at high temperatures
-V2 concrete mix with steel fibers
left: permeability according to Tenchev
middle: permeability according to Gawin
right: permeability according to Gawin (modified model)

Figure F3: Analytical models for permeability of concrete at high temperatures
-V1 concrete mix without any fibers
left: permeability according to Gawin
right: permeability according to Gawin (modified model)
Appendix F.2 - Models for concrete losses in mass at high temperatures

Appendix F.2.1 - General model for losses in mass as a power function

In order to develop a general model for losses in mass during heating based on the initial density of the concrete at $T = 20°C$, a power function was chosen. This depends on two coefficients “$a$” and “$b$” without any physical meaning. Up to a temperature of $T = 100°C$, no losses in weight are considered. For higher temperatures, the initial moisture content $\phi_0$ as loss in mass must be considered as a constant term independent of the temperature in addition to regular temperature-dependent losses in mass.

Based on the initial density of the concrete $\rho(20°C)$ the temperature-dependent reduction factor can be estimated according to equation (F - 8) and (F - 9) for the three different concrete mixes:

$$\frac{\rho(T)}{\rho(20°C)} = 1$$ for $T < 100°C$  \hspace{1cm} (F - 8)

$$\frac{\rho(T)}{\rho(20°C)} = 1 - \left( (a \cdot (T - 100)^b) + \phi_0 \right)$$ for $100°C \leq T \leq 500°C$  \hspace{1cm} (F - 9)

with:

- $\rho(T)$ temperature-dependent density in kg/m$^3$
- $\rho(20°C)$ initial density in kg/m$^3$
- $a$ coefficient $a$
- $b$ coefficient $b$
- $\phi_0$ initial moisture content in %

The coefficients “$a$” and “$b$” were derived from the tests as given in Table F2. They have no physical meaning. The regression curves compared to the three concrete mixes are shown in Figure F4.

Table F2: Material coefficients to determine temperature-dependent losses in mass

<table>
<thead>
<tr>
<th>Coefficient</th>
<th>M1</th>
<th>M2</th>
<th>M3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$4.39 \cdot 10^{-3}$</td>
<td>$2.98 \cdot 10^{-3}$</td>
<td>$2.8 \cdot 10^{-4}$</td>
</tr>
<tr>
<td>B</td>
<td>0.445</td>
<td>0.461</td>
<td>0.717</td>
</tr>
</tbody>
</table>

Figure F4: Temperature-dependent losses in mass, average test results and regression curve
Appendix F.2.2 - Regression for losses in mass provided by EN 1992-1-2


Comparing the own regression to the EN 1992-1-2 criteria, differences can be observed. Up to a concrete temperature of 115°C, EN 1992-1-2 assumes a constant mass. For concrete temperatures in the range $115°C < T \leq 200°C$, the losses in mass can be taken as given in equation (F - 11) and for the range $200°C < T \leq 400°C$ in equation (F - 12). Higher temperatures are covered by equation (F - 13).

$$\frac{\rho(T)}{\rho(20°C)} = 1 \quad \text{for } T \leq 150°C \quad \text{(F - 10)}$$

$$\frac{\rho(T)}{\rho(20°C)} = \left(1 - 0.02 \cdot \frac{(T-115)}{85}\right) \quad \text{for } 115°C < T \leq 200°C \quad \text{(F - 11)}$$

$$\frac{\rho(T)}{\rho(20°C)} = \left(0.98 - 0.03 \cdot \frac{(T-200)}{200}\right) \quad \text{for } 200°C < T \leq 400°C \quad \text{(F - 12)}$$

$$\frac{\rho(T)}{\rho(20°C)} = \left(0.95 - 0.073 \cdot \frac{(T-400)}{800}\right) \quad \text{for } 400°C < T \leq 1200°C \quad \text{(F - 13)}$$

Comparing these recommendations to the test results shows that the EN 1992-1-2 standard shows overall good agreement with the losses observed for the M1 and M2 concrete mixes. However, the decrease in mass is clearly overestimated by the standard if compared to the M3 concrete. With the help of regression analysis, the temperature-dependent losses in mass could be determined. This covers the decrease in mass for temperature exceeding $T = 105°C$, in addition to the loss of free water after drying the concrete to a constant mass. The initial moisture content is not included in this regression analysis. The comparison with the test results is shown in Figure 25.

Appendix F.2.3 - Losses in mass as a linear factor according to Bažant

Bažant and Kaplan [18] presented a similar simplified approach in their 1996 book on material properties and mathematical models for concrete at high temperatures. Here, the losses in mass are assumed as a linear factor between temperature of $100°C < T < 700°C$. According to Bažant, the losses in mass can be expressed as in equation (F - 14).

$$\frac{\rho(T)}{\rho(20°C)} = \begin{cases} 
1 & \text{T} \leq 100°C \\
1 - 0.04 \cdot \frac{T-100}{100} & \text{100°C} < T \leq 700°C \\
0.24 & \text{T} \geq 700°C 
\end{cases} \quad \text{(F - 14)}$$
Appendix F.3 - Models for the tensile strength of concrete at high temperatures

For temperatures exceeding $T = 100^\circ\text{C}$ it should be noted that there is a paucity of tests and design standards on the tensile strength of concrete at high temperatures. In particular, criteria for HPC and UHPC concrete mixes containing a large amount of silica fume and steel and PP fibers are generally not available. According to the EN 1992-1-2 standard for concrete at high temperature [6], the tensile strength of concrete can be reduced linearly from an initial tensile strength at a temperature of $T = 100^\circ\text{C}$ to a complete loss in tensile strength ($f_t = 0$ MPa) at a temperature of $T = 600^\circ\text{C}$. This guidance is valid for OPC with a maximum compressive strength of $f_c = 55$ MPa.

Behnood and Ghandehari [21] presented in 2009 their analysis on the compressive and splitting tensile strengths of concrete at high temperatures including the influence of PP fibers and silica fume. According to their findings, the splitting tensile strength is more affected by high temperatures than the compressive strength.

The most significant changes in strength were observed in the temperature range between $300^\circ\text{C}$ and $600^\circ\text{C}$. At this maximum temperature, the tensile splitting strength was about 20 - 23% of the initial cold strength, depending on the amount of fibers used. Adding $2.0$ kg/m$^3$ PP fibers to the fresh concrete seems to increase the residual properties of the concrete. Higher or lower dosages have the opposite effect. It was mentioned that deterioration of the concrete or voids due to other dosages has negative influences. Design criteria and recommendations on their findings were not presented.

Apart from the design criteria according to EN 1992-1-2 [6] and the general findings by Behnood [21], tests of major importance for different types of HPC were provided by Khaliq in 2011 [65]. He performed tests on the tensile splitting strength of concrete cylinders ($l = 150$ mm, $\phi = 50$ mm) up to temperatures of $T = 800^\circ\text{C}$ and provided the data in form of general design criteria. His tests covered in addition the influence of PP and steel fibers on the tensile splitting strength of concrete.

Both references state that the tensile strength $f_t$ of concrete is reduced linearly with increasing temperature exceeding $T = 100^\circ\text{C}$ by a factor $k_c(T)$ as given in equation (F - 15). This factor $k_c(T)$ varies according to the concrete mix.

$$f_t(T) = k_c(T) \cdot f_t$$ \hspace{1cm} (F - 15)

with:

- $k_c(T)$ \hspace{0.5cm} temperature-dependent coefficient
- $f_t$ \hspace{0.5cm} tensile strength of concrete at ambient temperature of $20^\circ\text{C}$

In addition, it is agreed that the tensile strength remains constant for temperatures below $T = 100^\circ\text{C}$, which leads to the following expression for all types of concrete:

$$k_c(T) = 1.0 \hspace{1cm} \text{for } 20^\circ\text{C} \leq T \leq 100^\circ\text{C}$$ \hspace{1cm} (F - 16)
Regarding different concrete mixes, the factor $k_c(T)$ can be calculated according to equations (F - 17) to (F - 27).

**OPC ($f_c < 55$ MPa) [6]:**

$$k_c(T) = 1.0 - 1.0 \cdot \frac{(T-100)}{500} \quad \text{for} \quad 100°C < T \leq 600°C$$ (F - 17)

**HPC without steel fibers [65]:**

$$k_c(T) = 0.97 - 0.0011 \cdot T \quad \text{for} \quad 100°C < T \leq 800°C$$ (F - 18)

**HPC containing 0.54% in volume of steel fibers [65]:**

$$k_c(T) = 0.9 \quad \text{for} \quad 100°C < T \leq 300°C$$ (F - 19)

$$k_c(T) = 1.42 - 0.0018 \cdot T > 0 \quad \text{for} \quad 400°C < T \leq 800°C$$ (F - 20)

**HPC containing 1.0 kg/m$^3$ of PP fibers [65]:**

$$k_c(T) = 1 - 0.002 \cdot T \quad \text{for} \quad 100°C < T \leq 200°C$$ (F - 21)

$$k_c(T) = 0.82 - 0.001 \cdot T \quad \text{for} \quad 300°C < T \leq 800°C$$ (F - 22)

**HPC containing both, 1.0 kg/m$^3$ of PP fibers and 0.54% in volume of steel fibers [65]:**

$$k_c(T) = 0.78 \quad \text{for} \quad 100°C < T \leq 300°C$$ (F - 23)

$$k_c(T) = 1.28 - 0.0016 \cdot T \quad \text{for} \quad 400°C < T \leq 800°C$$ (F - 24)

**Self-compacting concrete (SSC) without fibers [65]:**

$$k_c(T) = 1 - 0.00033 \cdot T \quad \text{for} \quad 100°C < T \leq 500°C$$ (F - 25)

$$k_c(T) = 2.28 - 0.0029 \cdot T \quad \text{for} \quad 500°C < T \leq 800°C$$ (F - 26)

**HPC with fly ash up to 65 kg/m$^3$ [65]:**

$$k_c(T) = 1.05 - 0.0013 \cdot T \quad \text{for} \quad 100°C < T \leq 800°C$$ (F - 27)

Missing values can be interpolated linearly. It should be noted that data and design criteria for UHPC mixes with a high amount of silica fume added to the mix are not available. In addition, subsequent hardening of the concrete at higher temperatures [75] as observed with the P1 UHPC mix at a temperature of $T = 300 - 500°C$ is not taken into account; hence some of the concrete samples might show a higher tensile strength at a high temperature.
Table F3 summarizes the chosen concrete model for the different tested concrete mixes.

Table F3: Concrete mixes and chosen model for prediction of the tensile strength at high temperatures

<table>
<thead>
<tr>
<th>Concrete mix</th>
<th>Model on temperature-dependent tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1 - V1</td>
<td>(F - 18)</td>
</tr>
<tr>
<td>M1 - V2</td>
<td>(F - 19) (F - 20)</td>
</tr>
<tr>
<td>M2 - V1</td>
<td>(F - 18)</td>
</tr>
<tr>
<td>M2 - V2</td>
<td>(F - 19) (F - 20)</td>
</tr>
<tr>
<td>M3 - V1</td>
<td>(F - 18)</td>
</tr>
<tr>
<td>M3 - V2</td>
<td>(F - 19) (F - 20)</td>
</tr>
<tr>
<td>P1 UHPC</td>
<td>(F - 23) (F - 24)</td>
</tr>
<tr>
<td>OPC</td>
<td>EN 1992-1-2</td>
</tr>
</tbody>
</table>
APPENDIX G - MODEL FOR THE PORE PRESSURE DEVELOPMENT

Based on the explanations from chapter 4.2, the following appendix shows the general procedure for the analytical model for explosive spalling. The pore pressure in each segment at each time step during temperature exposure is provided as the result.

1. Discretization of the modeled cross section and initial data

<table>
<thead>
<tr>
<th>Required data:</th>
<th>Geometry of the modeled section according to</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output:</td>
<td>Discretization with:</td>
</tr>
<tr>
<td></td>
<td>total number of segment “j”</td>
</tr>
<tr>
<td></td>
<td>length of time steps “i”</td>
</tr>
<tr>
<td></td>
<td>length of segment $\Delta l_{\text{segment}}$</td>
</tr>
<tr>
<td></td>
<td>size of segment $\Delta A_{\text{segment}}$</td>
</tr>
<tr>
<td>Boundary conditions (i.e. $Q_n = 0 \ m^3/s$ along symmetry axis)</td>
<td></td>
</tr>
<tr>
<td>Initial data:</td>
<td>Temperature $T_0 = 20{\degree}\text{C}$</td>
</tr>
<tr>
<td></td>
<td>Density $\rho_0$ in kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>Tensile strength $f_t$ in N/mm$^2$</td>
</tr>
<tr>
<td></td>
<td>Initial humidity $\phi_0$ in % by mass</td>
</tr>
<tr>
<td></td>
<td>Ambient air pressure $p_0 = 101325$ Pa</td>
</tr>
</tbody>
</table>

Figure G1: Discretization of the modeled section

2. Concrete temperature:

<table>
<thead>
<tr>
<th>Required data:</th>
<th>Geometry of the modeled section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Temperature-dependent material properties (i.e. EN 1992-1-2 [6])</td>
</tr>
<tr>
<td></td>
<td>- Density $\rho(T)$ in kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>- Thermal conductivity $\lambda(T)$ in W/(m·K)</td>
</tr>
<tr>
<td></td>
<td>- Moisture content and specific heat $c(T)$ in J/(kg·K)</td>
</tr>
<tr>
<td>Heating rate (i.e. ISO-fire [5])</td>
<td></td>
</tr>
<tr>
<td>Radiation and emissivity of the flame</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Temperature analysis with numerical analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output:</td>
</tr>
<tr>
<td>Concrete temperature $T_{(t)}$</td>
</tr>
<tr>
<td>- in each segment “j” of the modeled cross section</td>
</tr>
<tr>
<td>- at each time step “i”</td>
</tr>
<tr>
<td>- starting with $T_{(t=0, j)} = 20{\degree}\text{C}$ in all segments $j = 1 - j = n$</td>
</tr>
</tbody>
</table>
3. Moisture due to decomposition and porosity:

**Required data:**
- Tabulated data for
  - $v'$: specific volume of the water in m$^3$/kg (tabulated data)
  - $v''$: specific volume of vapor in m$^3$/kg (tabulated data)

**Concrete porosity $\varepsilon$:**
- at different temperatures $\varepsilon(20^\circ C)$, $\varepsilon(500^\circ C)$

**Output:**
- Specific pore volume $v(T)$
- Porosity as function of temperature $\varepsilon(T)$
- Vapor content in pores “$x$”

### Determining the specific pore volume $v(T)$:

$$v(T) = \frac{\varepsilon(T) \cdot \Delta l_{slice} \cdot \Delta A_{slice}}{m_w(T)} = \frac{\varepsilon(T) \cdot (\Delta l_{slice} \cdot \Delta A_{slice}) [m^3/kg]}{m_w(T)} \quad (G - 1)$$

**With**
- $\varepsilon(T)$: Porosity of concrete as a function of temperature
- $m_w(T)$: temperature-dependent total mass of the moisture (liquid and vapor) inside concrete in kg

### Determining the temperature-dependent porosity $\varepsilon(T)$:

$$\varepsilon(T) = \begin{cases} 
\varepsilon(20^\circ C) \quad [-] & \text{for } T \leq 100^\circ C \\
\frac{\varepsilon(500^\circ C) - \varepsilon(20^\circ C)}{400} \cdot (T - 100) + \varepsilon(20^\circ C) \quad [-] & \text{for } T > 100^\circ C 
\end{cases} \quad (G - 2)$$

**With**
- $\varepsilon(20^\circ C)$: Porosity of concrete at $T = 20^\circ C$
- $\varepsilon(500^\circ C)$: Porosity of concrete at $T = 500^\circ C$

### Determining the vapor content “$x$”:

$$x = \frac{v - v'}{v'' - v'} = \frac{m''}{m' + m''} = \frac{m''}{m_w} [-] \quad (G - 3)$$

$$m'' = x \cdot m_w \quad (G - 4)$$

**With:**
- $x$: vapor content
- $v$: specific pore volume in m$^3$/kg
- $v'$: specific volume of the water in m$^3$/kg (tabulated data)
- $v''$: specific volume of vapor in m$^3$/kg (tabulated data)
- $m'$: mass of water (liquid) in kg
- $m''$: mass of vapor in kg
- $m_w$: mass of total moisture
4. Pore pressure development due to high temperatures:

Pressure according to the ideal gas law:

\[
P(T) = \frac{R_s T}{v} \left[ \frac{N}{m^2} \right] \tag{G - 5}
\]

With:
- \(p\) pore pressure in N/m\(^2\)
- \(R_s\) specific gas constant for vapor with \(R_s = 462 \text{ J/(kg·K)}\)
- \(T\) absolute temperature in K
- \(v\) specific pore volume in m\(^3\)/kg

Summarized pressure development for heated concrete according to vapor content:

\[
P(T) = \begin{cases} 
101325 \text{ [N/m}^2\text{]} & \text{for } T < 100^\circ\text{C or } m_w = 0 \\
\text{saturation vapor pressure} & \text{for } T > 100^\circ\text{C and } 0 \leq x \leq 1 \\
\frac{R_s T}{v} \left[ \frac{N}{m^2} \right] & \text{for } T > 100^\circ\text{C and } x > 1 
\end{cases} \tag{G - 6}
\]

5. Moisture transport in porous media:

Initial moisture content \((m_{w(t=0)})\) and initial specific volume \((v(t=0))\):

\[
m_{w(t=0)} = \phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice} \quad \text{[kg]} \tag{G - 7}
\]

\[
v(t=0) = \frac{\varepsilon_0}{\phi_0 \cdot \rho_0} \left[ \frac{m^3}{kg} \right] \tag{G - 8}
\]

With:
- \(\phi_0\) initial moisture content in %
- \(\rho_0\) density of water as liquid at \(T = 20^\circ\text{C}\)

Moisture content as a function of temperature \((m_{w(T)})\)

\[
m_{w(T)} = \begin{cases} 
\phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice} & \text{if } T \leq 105^\circ\text{C} \\
\left(\phi_0 \cdot \rho_0 \cdot \Delta l_{slice} \cdot \Delta A_{slice}\right) + a \cdot (T - 105)^b \cdot \varepsilon(T) & \text{if } T > 105^\circ\text{C} 
\end{cases} \tag{G - 9}
\]

With:
- \(a; b\) Parameter determining the losses in mass at high temperature (Table F2)

Moisture content between time steps \((\Delta t)\)

\[
\Delta m_{w(T)t_i} = m_{w(T)t_i} - m_{w(T)t_{i-1}} \tag{G - 10}
\]
Moisture transport in porous media:

\[
Q(T) = \frac{k(T) \cdot \Delta p(T) \cdot \Delta A_{\text{slice}}}{\eta_v(T) \cdot \Delta t_{\text{slice}}} \quad \left[ \frac{m^3}{s} \right]
\]  

(G - 11)

with:

- \( Q \) volumetric flow rate of vapor in \( m^3/s \)
- \( k \) permeability of concrete
- \( \Delta p \) pressure gradient in MPa
- \( \eta_v \) dynamic viscosity of vapor in \( (N \cdot s)/m^2 \) (tabulated data)

Pressure gradient between two segments:

\[
\Delta p_{j \,(\text{out})} = \begin{cases} 
(p_j - p_{j-1}) & \text{if } p_j > p_{j-1} \\
0 & \text{if } p_j < p_{j-1}
\end{cases}
\]

(G - 12)

\[
\Delta p_{j \,(\text{in})} = \begin{cases} 
(p_j - p_{j+1}) & \text{if } p_j > p_{j+1} \\
0 & \text{if } p_j < p_{j+1}
\end{cases}
\]

(G - 13)

Moisture transport according to pressure gradient:

\[
Q_{j \,(\text{out}) \,(T)} = \frac{k(T) \cdot \Delta p_{j \,(\text{out}) \,(T)} \cdot \Delta A_{\text{slice}}}{\eta_v(T) \cdot \Delta t_{\text{slice}}} \quad \left[ \frac{m^3}{s} \right]
\]

(G - 14)

\[
Q_{j \,(\text{in}) \,(T)} = \frac{k(T) \cdot \Delta p_{j \,(\text{in}) \,(T)} \cdot \Delta A_{\text{slice}}}{\eta_v(T) \cdot \Delta t_{\text{slice}}} \quad \left[ \frac{m^3}{s} \right]
\]

(G - 15)

Mass of migrated vapor between segments:

\[
\Delta m_v^{\,(\text{out}) \,(T)} = Q_{j \,(\text{out}) \,(T)} \cdot \Delta t_i \cdot \rho_v(T) \quad [kg]
\]

(G - 16)

\[
\Delta m_v^{\,(\text{in}) \,(T)} = Q_{j \,(\text{in}) \,(T)} \cdot \Delta t_i \cdot \rho_v(T) \quad [kg]
\]

(G - 17)

with:

- \( \Delta m_v^{\,(\text{out})} \) mass of outflowing vapor from segment \( j \) to segment \( j-1 \) in kg
- \( \Delta m_v^{\,(\text{in})} \) mass of inflowing vapor from segment \( j \) to segment \( j+1 \) in kg
- \( \Delta t_i \) time step = \( t_i - t_{i-1} \)
- \( \rho_v \) vapor density in kg/m³
New actual moisture content in segment for time step t+1:

\[ m_{w,j}^{t+1} = m_{w,j}^{t} + \Delta m_{w,j}^{t} - \Delta m_{v,\text{out}}^{t,j} - \Delta m_{v,\text{in}}^{t,j} + \Delta m_{v,\text{out}}^{t+1,j} + \Delta m_{v,\text{in}}^{t-1,j} \quad [kg] \]  

with:

- \( m_{w,j}^{t+1} \): mass of actual moisture content in segment j at time \( t+1 \) in kg
- \( m_{w,j}^{t} \): mass of moisture content in segment j at time \( t \) in kg
- \( \Delta m_{w,j}^{t} \): mass of additional moisture due to drying in segment j due to time step \( \Delta t \) in kg
- \( \Delta m_{v,\text{out}}^{t,j} \): mass of outflowing vapor from segment j to segment j-1 due to time step \( \Delta t \) in kg
- \( \Delta m_{v,\text{in}}^{t,j} \): mass of inflowing vapor from segment j to segment j+1 due to time step \( \Delta t \) in kg
- \( \Delta m_{v,\text{out}}^{t+1,j} \): mass of outflowing vapor to segment j from segment j+1 due to time step \( \Delta t \) in kg
- \( \Delta m_{v,\text{in}}^{t-1,j} \): mass of inflowing vapor to segment j from segment j-1 due to time step \( \Delta t \) in kg

Excess moisture content \( m_{\text{excess}} \):

A “sending” segment is migrating vapor to a “receiving” segment. This “receiving” segment would then have a higher moisture content than physically possible. Moisture is migrated back to the “sending” segment in the form of “excess water” \( m_{\text{excess}} \) in the same time step.

\[
m_{\text{excess}} = \begin{cases} 
\frac{1}{v(T)} - \frac{1}{v'(T)} & \text{for } x < 0 \text{ according to (4-4)} \\
0 & \text{for } x \geq 0 
\end{cases} \quad [kg]
\]

with:

- \( v(T) \): temperature-dependent specific pore volume in \( m^3/kg \)
- \( v'(T) \): temperature-dependent specific pore volume of water (liquid) in \( m^3/kg \)

6. Failure criteria:

<table>
<thead>
<tr>
<th>Required data:</th>
<th>Pressure ( p_{IJ} ) in MPa in each segment “j” at each time step “i”</th>
</tr>
</thead>
<tbody>
<tr>
<td>Occurrence of excess water ( m_{\text{excess}} ) in segments</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output:</th>
<th>Failure if pore pressure exceeds tensile strength ( p &gt; f_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abortion due to the presence of excess water ( m_{\text{excess}} ) in segments</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX H - VALIDATION OF THE ANALYTICAL MODEL FOR EXPLOSIVE SPALLING

APPENDIX H.1 - Comparison with own tests on concrete cylinders

Introduction

The results from the modeled tests as presented in Table 25 and Table 26 are analyzed in the following, including a summary of critical parameters. This analysis includes:

- Maximum pressure inside the modeled section and location of this section
- Temperature during peak pressure with corresponding tensile strength
- Possible occurrence of moisture clog in the form of excess water and analysis in terms of failure criteria

The maximum pore pressure during heating is measured at the end of the plateau phase, just before all moisture evaporated, the temperature increases and the pressure inside the pores decreases. This was discussed in chapter 3.4.4. In terms of the analytical model and the analysis of these modeled tests, the temperature at peak pressure is compared with the plateau temperature from tests, in order to determine similarity in temperature.

Model 1

Table H1 summarizes the main results from model 1 on the M1 - V2 concrete.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heating rate</td>
<td>$T = 0.5$ K/min</td>
</tr>
<tr>
<td>Initial tensile strength</td>
<td>$f_t = 5.2$ MPa</td>
</tr>
<tr>
<td>Peak pressure</td>
<td>$p_{\text{max}} = 2.41$ MPa in 58 mm from heated surface</td>
</tr>
<tr>
<td>Temperature at peak pressure</td>
<td>$T = 222^\circ$C</td>
</tr>
<tr>
<td>Reduced tensile strength</td>
<td>$f_t(222^\circ)$ = 4.68 MPa (no spalling)</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>8.0 s</td>
</tr>
</tbody>
</table>

According to the analytical model, the specimen does not tend to develop explosive spalling, which agrees with the results of the tests. However, during testing, the plateau temperature was observed at a higher temperature of $T = 260 - 270^\circ$C. The model indicated a peak pressure already at $T = 222^\circ$C. At this temperature, a pressure of $p = 2.41$ MPa was modeled, which is far below the reduced tensile strength of the concrete of $f_t(222^\circ)$ = 4.68 MPa at this temperature.

The pore pressure development inside the concrete exhibits, as expected, a rather low pressure at the heated surface and a higher pressure inside the specimen due to moisture migration. It was noticed that the segment $j = 1$ at the heated surface dried already at a temperature of $T = 137^\circ$C and a corresponding pressure of $p_{\text{max}} = 0.3$ MPa, hence no further increase in pressure is observed in this segment. Figure H1 (left) shows the pressure development inside all modeled segments during the exposure time. Each data point represents the peak pressure in one segment, starting with the pressure in the first segment. The first, last and the segment with peak pressure are highlighted in this figure.
Appendix H - Validation of the analytical model for explosive spalling

The segment at 58 mm distance to the heated surface is the last segment that dries and hence the highest pore pressure is observed in this segment. Figure H1 (middle) shows the total moisture content in kg / segment with increasing temperature. Up to a temperature of $T = 100^\circ C$ the initial moisture remains constant. Higher temperatures cause an increase in moisture content due to the decomposition of the concrete and release of physical water. As explained, this additional moisture will migrate via the pores and cause the increase in pressure at higher temperatures. At a temperature of about $T = 180^\circ C$, further migration processes in this segment are noticed up to a temperature of $T = 213^\circ C$. Due to the presence of sufficient moisture inside the segment, the pressure continues to rise as shown in Figure H1 (right).

This increase due to migration is not noticed in the segment close to the heated surface. Apart from the initial moisture content and minor additional moisture up to a temperature of $T = 137^\circ C$, no further increases in moisture are observed. The amounts of moisture migrating from deeper sections of the concrete via the first segment to the outside at high temperature do not lead to a noticeable increase in moisture content or pressure. In addition, as mentioned in chapter 4.2, one boundary condition of the analytical model assumes that dry pores remain dry even though migration processes are still possible due to pressure gradients.

The pressure follows the saturation vapor pressure curve in all segments until its peak pressure and decreases within a few time steps to the ambient air pressure, which is in stark contrast to the tests on pore pressure as analyzed in chapter 3.4.4. According to the analysis using the model, the segment where the peak pressure was noticed is the last segment to dry out. This was at a temperature in this segment of $T = 222^\circ C$. From this state onwards, the pressure inside all segments is close to ambient air pressure of $p \approx 0.1$ MPa.

This difference in post peak behavior compared to tests is mainly caused by the stepwise analysis of the pore pressure development and migration processes, leading to the rapid decrease in the pressure.

The occurrence of excess water $m_{\text{excess}}$ was not noticed during this analytical analysis.

Figure H1: Model 1: M1 - V1 - $T = 0.5$ K/min - Pressure development in segment with highest pressure
left: time / peak pressure development in all segments
middle: moisture content in peak segment
right: pressure development in peak segment
After this preliminary comparison of the results from the model to tests it is already noticed that the increasing branch of the pressure curve follows the saturation vapor pressure curve, even with this very low heating rate. It is mentioned in the literature that low heating rates cause an increase in pressure far below the saturation vapor pressure curve, since the vaporization rate of moisture is less than the migration due to the permeability. However, the permeability for the M1 concrete as modeled here is far lower compared to references in literature.

Model 2

For the next model, the M1 - V2 concrete heated with $\dot{T} = 0.75$ K/min was modeled and the main results are summarized in Table H2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heating rate</td>
<td>T = 0.75 K/min</td>
</tr>
<tr>
<td>Initial tensile strength</td>
<td>$f_t = 5.2$ MPa</td>
</tr>
<tr>
<td>Peak pressure in MPa</td>
<td>$p_{\text{max}} = 2.06$ MPa in 74 mm from heated surface</td>
</tr>
<tr>
<td>Temperature at peak pressure</td>
<td>$T = 214^\circ$C</td>
</tr>
<tr>
<td>Reduced tensile strength</td>
<td>$f_{t(214^\circ)} = 4.68$ MPa (no spalling)</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>4.0 s</td>
</tr>
</tbody>
</table>

A lower peak pressure of only $p_{\text{max}} = 2.06$ MPa was observed. This pressure is far too low to cause explosive spalling, which is contrast to the experimental results. In terms of the time/pressure development as shown in Figure H2 (left), it is interesting to note that the increase in pressure is more pronounced in the inner segments. After a heating time of $t = 300$ min, a rapid increase in pressure is observed. This is in contrast to model 1 as shown in Figure H1 (left), where an almost linear increase in pressure during the entire exposure time is observed.

The highest pore pressure is observed in the innermost segment, with a distance of 74 mm from the heated surface at a temperature of about $T = 214^\circ$C. The plateau temperature in the test was observed at slightly higher temperatures in the range of $T = 260 - 270^\circ$C.

During pressure buildup, a few negative values for vapor content $x$ were noticed in this segment, with the corresponding occurrence of excess moisture $m_{\text{excess}}$. These occurrences of $m_{\text{excess}}$ are noticed by the sudden decrease in pore pressure in one time step and the increase to saturation pressure in the next.

This excess water was noticed in several time steps randomly in any segment. Two factors influence this occurrence, namely the initial moisture content and the rather large steps in temperature of $\Delta T = 4.0^\circ$C of tabulated data on vapor properties at high temperatures. This leads to several local peaks in the water content. It was tried to decrease the length of the time steps to $\Delta t = 0.6$ s. However, no difference in pressure development and local moisture peaks was observed.
Appendix H - Validation of the analytical model for explosive spalling

Figure H2: Model 2: M1 - V2 - Pressure development in segment with highest pressure
left: time / peak pressure development in all segments
middle: moisture content in peak segment
right: pressure development in peak segment

Model 3

Table H3 summarizes the results from model 3 on the M1 concrete heated with $\dot{T} = 1.00$ K/min.

<table>
<thead>
<tr>
<th>Table H3: Model 3: M1 - V2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heating rate</td>
</tr>
<tr>
<td>Initial tensile strength</td>
</tr>
<tr>
<td>Peak pressure in MPa</td>
</tr>
<tr>
<td>Temperature at peak pressure</td>
</tr>
<tr>
<td>Reduced tensile strength</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
</tr>
</tbody>
</table>

Model 3 is almost identical to model 2. The analytical model indicated no spalling for this heating rate, which is in contrast to the experimental results. A slightly higher peak pressure and temperature was analyzed. The plateau temperature could not be measured in these tests, since the specimen spalled before the plateau phase developed.

As with the previous model 2, the peak pressure was observed in the innermost segment. While the pressure development in the segments close to the heated surface did not change significantly compared to the analysis with lower heating rates, an even more pronounced increase in peak pressure in the inner segments was observed in the model. As mentioned previously, if spalling occurs, it is initiated in the inner sections of the specimen.

The presence of a moisture clog due to the occurrence of excess moisture was not observed in this analysis. It should be noted that thermal stresses become noticeable with the heating rates in the tests, leading to different spalling mechanisms, which are not covered by this analytical model.

The corresponding graphs on the pressure development are shown in Figure H3.
Figure H3: Model 3: M1 - V2 - Pressure development in segment with highest pressure
left: time / peak pressure development in all segments
middle: moisture content in peak segment
right: pressure development in peak segment

The M1 concrete mix without steel fibers (M1 - V1) is characterized by a noticeable decrease in permeability in the temperature range $150^\circ\text{C} < T < 275^\circ\text{C}$. This lower permeability minimizes any migration processes. This influence on the analytical model was studied. Table H4 gives an overview on the validation models.

Model 4 - 5:

Table H4: Model 4 - 5 - M1 - V1 concrete mix

<table>
<thead>
<tr>
<th>Model</th>
<th>Model 4</th>
<th>Model 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Heating rate $\dot{T}$</td>
<td>0.25 K/min</td>
<td>0.5 K/min</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$</td>
<td>7.1 MPa</td>
<td></td>
</tr>
<tr>
<td>Peak pressure $p_{\text{max}}$</td>
<td>2.79 MPa</td>
<td>3.26 MPa</td>
</tr>
<tr>
<td>Temperature T in °C at peak pressure</td>
<td>226°C</td>
<td>234°C</td>
</tr>
<tr>
<td>Reduced tensile strength $f_{\text{red}}$ in MPa</td>
<td>5.1 MPa</td>
<td>5.1 MPa</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>74 mm</td>
<td>56 mm</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>15.0 s</td>
<td>8.0 s</td>
</tr>
</tbody>
</table>

The lower permeability for the M1 - V1 concrete is evident in the modeled results, since a higher peak pressure is observed. Even though the resistance to explosive spalling in the form of tensile strength is higher compared to the maximum pressure, the expected peak pressure is in a critical range where spalling must be considered.

It is interesting to note that an increase in heating rate of $\Delta \dot{T} = 0.25$ K/min to $\dot{T} = 0.50$ K/min leads to a significant higher pore pressure. In addition, the higher heating rate shifts the segment with peak pressure closer to the heated surface.

The tests indicated a plateau temperature in the range $T = 255 - 265^\circ\text{C}$. Even though the model underestimates the temperature during peak pressure, there is close agreement with the test results.
The formation of a moisture clog was not observed in the analysis. All corresponding graphs on the pressure development are shown in Figure H4.

Figure H4: Model 4: M1 - V1 - Pressure development in segment with highest pressure
  left: time / peak pressure development in all segments
  middle: moisture content in peak segment
  right: pressure development in peak segment

Figure H5: Model 5: M1 - V1 - Pressure development in segment with highest pressure
  left: time / peak pressure development in all segments
  middle: moisture content in peak segment
  right: pressure development in peak segment

Figure H6 shows the actual moisture content and corresponding pressure in the innermost segment of the model 4 analysis. The peak pressure was observed in this segment. It is interesting to note that the pressure increases as long as moisture is available inside the concrete pores, starting with the initial moisture content of $m_w \approx 0.06$ kg/segment. The peak moisture content $m_w$ in this segment is about $m_w \approx 0.2$ kg/segment, with a corresponding pressure of $p \approx 2.0$ MPa. This pressure continues to rise to $p = 2.79$ MPa during the drying phase.

This observation agrees with the review of pore pressure measurement. Here it is just mentioned that peak pressure is usually observed before the pore dries out at the end of the moisture-induced plateau. In addition,
Appendix H - Validation of the analytical model for explosive spalling

this confirms that the temperature at peak pressure should be close to the plateau temperature as observed in tests.

Figure H6: Model 4 - moisture content / corresponding pressure diagram for inner segment

Models 6 - 11:

Table H5 gives an overview of the model 6 - 11 on the M2 concrete mix.

<table>
<thead>
<tr>
<th>Model</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete mix</td>
<td>M2 - V2</td>
<td>M2 - V2</td>
<td>M2 - V1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Heating rate $\dot{T}$ in K/min</td>
<td>1.0</td>
<td>2.0</td>
<td>3.0</td>
<td>1.0</td>
<td>2.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$ in MPa</td>
<td>5.3</td>
<td>4.2</td>
<td>2.0</td>
<td>4.2</td>
<td>2.0</td>
<td>4.2</td>
</tr>
<tr>
<td>Peak pressure $p_{\text{max}}$ in MPa $^1$</td>
<td>0.74</td>
<td>1.31</td>
<td>1.52</td>
<td>0.3</td>
<td>0.32</td>
<td>0.36</td>
</tr>
<tr>
<td>Temperature $T$ in °C at peak pressure</td>
<td>167</td>
<td>192</td>
<td>199</td>
<td>135</td>
<td>136</td>
<td>140</td>
</tr>
<tr>
<td>Plateau temperature in tests in °C $^2$</td>
<td>245-255</td>
<td>250-260</td>
<td>-</td>
<td>220-225</td>
<td>195-205</td>
<td>-</td>
</tr>
<tr>
<td>Reduced tensile strength $f_{t(T)}$ in MPa</td>
<td>4.77</td>
<td>4.77</td>
<td>4.77</td>
<td>3.45</td>
<td>3.44</td>
<td>3.42</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>6</td>
<td>4</td>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>4.0s</td>
<td>2.0s</td>
<td>1.0s</td>
<td>4.0s</td>
<td>2.0s</td>
<td>1.0s</td>
</tr>
</tbody>
</table>

1) Pressure in mm from the heated surface
2) As presented in Table 13

Similar to the observations made using the model of the M1 concrete tests, it can be seen that an increase in heating rate leads to a higher peak pressure.

For models 6 - 8 for the M2 - V2 concrete mix, the models indicated no spalling. This is in contrast to the experimental results, since spalling started at a heating rate of $\dot{T} = 2.0$ K/min.

The highest heating rate of $\dot{T} = 3.0$ K/min leads to a peak pressure of $p = 1.52$ MPa, where spalling cannot be excluded in general.

Almost no increase in pore pressure was noticed with the model using input parameters based on the M2 - V1 concrete mix when compared to the analyzed peak pressure for the M2 - V2 concrete. The analysis indicated
excess moisture at rather low temperatures of only $T = 135^\circ C$, starting at the outer segment and continuing to the inside. The results of this analysis do not match the observations from tests.

The M2 concrete generally has a higher initial moisture content in the range of 2.5 - 3.0% (Table 21), which is rather high for a HPC mix. However, the permeability of the M2 concrete mix is low. This shifts the peak pore pressure to segments close to the heated surface, since these areas are directly affected by high temperatures and the vapor does not migrate outwards.

The high initial moisture content (Table 21) and additional moisture due to decomposition and drying of the concrete (Figure 24) leads to a continuous increase in pore saturation in all concrete layers. It must be considered that the temperature gradient is rather low, leading to an almost constant degree of moisture content in all segments. According to the thermodynamic equilibrium an increase in pore saturation leads to a decrease in the mass of vapor while the mass of water increases.

The permeability of the M2 concrete mix is very low and migration processes are limited, even to the outside. The gain in additional moisture due to decomposition exceeds the release of vapor due to low pressure gradients, low permeability and low vapor content at a specific temperature. At a certain temperature, the pores inside the concrete close to the heated surface are totally saturated with water, so that no vapor can migrate that would lead to a decrease in moisture. The presence of excess water is noticed at this time step in the model; however, the excess moisture comes from drying processes of the pore itself and cannot be “sent” to any other segment. It is interesting to note that all segments are completely saturated within very few time steps and the pressure development plunges to the ambient pressure. This is observed in the time / peak pressure development figure. Peak pressure is reached for all segments within a few time steps, noticed as a vertical line as shown in Figure H8.

This combination of high moisture content and low permeability would not necessarily lead to explosive spalling with low heating rates in tests, but to significant microcracking, which would release the water and increases the permeability. The pressure inside the pores will not follow the saturation vapor pressure curve. This effect was noticed with the M2 concrete mix as shown in Figure 32; the data points are located at rather high temperatures and well below the saturation vapor pressure curve. The pressure gradient follows a “case 3” pressure development as explained in chapter 3.4.3. This microcracking that occurs in tests is not taken into account in the analytical model.

Decreasing the initial moisture content, as input parameter, to 1.5% would lead to pore pressure development in all segments without the occurrence of excess moisture. However, a decrease in initial moisture leads to lower peak pressures in deeper sections.

This phenomenon is similar to the occurrence of a moisture clog. Even though this clog in the form of a saturated layer develops in deeper sections of the concrete and mainly due to vapor migration, the effect of minimizing further vapor migration is the same.

It should be noted that the combination of high initial moisture content, low permeability, low heating rates and neglecting microcracking in addition to the use of tabulated data marks the worst possible case in terms of modeling. Higher heating rates lead again to reasonable results, since they cause a noticeable temperature increase between two time steps with pressure gradients and a sensitive use of the tabulated data.
The figures for the pore pressure development for models 7 and 10 are shown in Figure H7 and Figure H8. These figures represent all verification tests for the M2 concrete parameters. The vertical line of the time / peak pressure curve indicates the abortion of the analytical analysis due to the moisture overflow in all segments.

Figure H7: Model 7: M2 - V2 - Pressure development in segment with highest pressure
- left: time / peak pressure development in all segments
- middle: moisture content in peak segment
- right: pressure development in peak segment

Figure H8: Model 10: M2 - V1 - Pressure development in segment with highest pressure
- left: time / peak pressure development in all segments
- middle: moisture content in peak segment
- right: pressure development in peak segment
Models 12 + 13:

Table H6 summarizes the results from modeling M3 concrete specimens.

<table>
<thead>
<tr>
<th>Model</th>
<th>Overview - M3 concrete mix</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete mix</td>
<td>Model 12</td>
</tr>
<tr>
<td>Heating rate $T$</td>
<td>8.0 K/min</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$</td>
<td>5.8 MPa</td>
</tr>
<tr>
<td>Peak pressure $p_{max}$</td>
<td>1.40 MPa</td>
</tr>
<tr>
<td>Temperature $T$ in °C at peak pressure</td>
<td>195°C</td>
</tr>
<tr>
<td>Plateau temperature in tests</td>
<td>205-210°C</td>
</tr>
<tr>
<td>Reduced tensile strength $f_{t(T)}$</td>
<td>5.2 MPa</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>6</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>0.5s</td>
</tr>
<tr>
<td></td>
<td>Model 13</td>
</tr>
<tr>
<td>Concrete mix</td>
<td>M3 - V1</td>
</tr>
<tr>
<td>Heating rate $T$</td>
<td>8.0 K/min</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$</td>
<td>5.6 MPa</td>
</tr>
<tr>
<td>Peak pressure $p_{max}$</td>
<td>0.35 MPa</td>
</tr>
<tr>
<td>Temperature $T$ in °C at peak pressure</td>
<td>139°C</td>
</tr>
<tr>
<td>Plateau temperature in tests</td>
<td>135-145°C</td>
</tr>
<tr>
<td>Reduced tensile strength $f_{t(T)}$</td>
<td>4.6 MPa</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>0</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>0.5s</td>
</tr>
</tbody>
</table>

The analytical modeling of M3 concrete specimens shows overall good agreement between the modeled data and the results from tests in terms of temperature at peak pressure and plateau temperature. The temperatures almost coincide.

Spalling was not observed with both concrete specimens. The analytical model indicated a pore pressure well below the tensile strength.

The figures with the pore pressure development are shown in Figure H9 and Figure H10. The vertical line of data points indicates the end of the analysis time. This is in contrast to the M2 concrete, in which the analysis was aborted due to moisture overflow.

![Figure H9](image-url)

**Figure H9:** Model 12: M3 - V2 - Pressure development in segment with highest pressure
- left: time / peak pressure development in all segments
- middle: moisture content in peak segment
- right: pressure development in peak segment
APPENDIX H.2 - Additional models for the pore pressure in heated concrete cylinders and slabs

In order to analyze the general adaptivity of the model for higher heating rates and the general possibility to use the model for assessing the possible risk of spalling in real fire scenarios, some additional case studies were performed.

In a first step, the M1 concrete, including all four sub-mixes V1 - V4, was analyzed by rapid heating with $\dot{T} = 8.0 \text{ K/min}$ to analyze the influence in pore pressure due to the presence of PP fibers. Table H7 summarizes the main results of this analysis.

As mentioned above, the analyses as presented in Table H7 and Table H8 are based on a concrete slab with a thickness of $h = 100 \text{ mm}$ exposed on one side.

Table H7: Modeled pore pressure in M1 concrete mix with rapid heating of $\dot{T} = 8.0 \text{ K/min}$

<table>
<thead>
<tr>
<th>Model</th>
<th>Model 14</th>
<th>Model 15</th>
<th>Model 16</th>
<th>Model 17</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete mix</td>
<td>M1 - V1</td>
<td>M1 - V2</td>
<td>M1 - V3</td>
<td>M1 - V4</td>
</tr>
<tr>
<td>Heating rate $\dot{T}$</td>
<td></td>
<td></td>
<td></td>
<td>$8.0 \text{ K/min}$</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$</td>
<td>7.1 MPa</td>
<td>5.2 MPa</td>
<td>5.0 MPa&lt;sup&gt;1)&lt;/sup&gt;</td>
<td>5.0 MPa&lt;sup&gt;1)&lt;/sup&gt;</td>
</tr>
<tr>
<td>Peak pressure $p_{\text{max}}$</td>
<td>4.66 MPa</td>
<td>4.89 MPa</td>
<td>1.40 MPa</td>
<td>1.09 MPa</td>
</tr>
<tr>
<td>Temperature $T$ in °C at peak pressure</td>
<td>252°C</td>
<td>253°C</td>
<td>193°C</td>
<td>181°C</td>
</tr>
<tr>
<td>Reduced tensile strength $f_{t(T)}$</td>
<td>4.9 MPa</td>
<td>4.7 MPa</td>
<td>3.9 MPa</td>
<td>3.9 MPa</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>10 mm</td>
<td>14 mm</td>
<td>10 mm</td>
<td>22 mm</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>0.5s</td>
<td>0.5s</td>
<td>0.5s</td>
<td>0.5s</td>
</tr>
</tbody>
</table>

<sup>1)</sup> Assumption based on M1 - V2 concrete

The analysis on the modeled M1 samples heated with $\dot{T} = 8.0 \text{ K/min}$ clearly indicated spalling with both PP-fiber-free mixes. The expected pressure inside the concrete clearly exceeds the reduced tensile strength. These results are in the same range predicted in the model by Dwaikat and Kodur [33]. In contrast, both
concrete mixes with an increased permeability due to the added PP fibers showed a significantly lower peak pore pressure.

The two mixes without PP fibers (V1 + V2) most probably tend to explosive spalling, even though the reduced tensile strength seems to be a bit higher compared to the maximum pore pressure with the -V1 mix. Adding PP fibers to the concrete reduced the maximum pressure below a critical level. It is interesting to note that type C PP fibers (see Table 14) as used in the -V4 mix seems to provide a slightly better performance in terms of pressure relief.

In all analyzed models, the peak pressure is usually expected within the first third of the specimen. In addition, the temperature on reaching peak pressure is close to plateau temperatures as observed in the tests on spalling (see Table 13). The presence of PP fibers lowers this temperature due to the increased permeability. The time and pressure developments for these modeled case studies are shown in Figure H11 and Figure H12.

Figure H11: Models 14 + 15: M1 concrete cylinder heated according to the ISO fire curve time / peak pressure development in all segments

Figure H12: Models 16 + 17: M1 concrete cylinder heated according to the ISO fire curve time / peak pressure development in all segments
Models 18 - 22:

Table H8: Modeled pore pressure in M1 - M3 concrete mixes with ISO fire exposure

<table>
<thead>
<tr>
<th>Model</th>
<th>Model 18</th>
<th>Model 19</th>
<th>Model 20</th>
<th>Model 21</th>
<th>Model 22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete mix</td>
<td>M1 - V1</td>
<td>M1 - V2</td>
<td>M1 - V4</td>
<td>M2 - V4</td>
<td>M3 - V2</td>
</tr>
<tr>
<td>Heating rate $\bar{T}$</td>
<td>ISO</td>
<td>ISO</td>
<td>ISO</td>
<td>ISO</td>
<td>ISO</td>
</tr>
<tr>
<td>Initial tensile strength $f_t$</td>
<td>7.1 MPa</td>
<td>5.2 MPa</td>
<td>5.0 MPa</td>
<td>5.0 MPa</td>
<td>5.0 MPa</td>
</tr>
<tr>
<td>Peak pressure $p_{\text{max}}$</td>
<td>5.17 MPa</td>
<td>4.61 MPa</td>
<td>2.90 MPa</td>
<td>5.33 MPa</td>
<td>8.09 MPa</td>
</tr>
<tr>
<td>Temperature $T$ in °C at $p_{\text{max}}$</td>
<td>256°C</td>
<td>259°C</td>
<td>232°C</td>
<td>268°C</td>
<td>286.1°C</td>
</tr>
<tr>
<td>Reduced tensile strength $f_t(T)$</td>
<td>4.9 MPa</td>
<td>4.7 MPa</td>
<td>3.9 MPa</td>
<td>3.9 MPa</td>
<td>3.9 MPa</td>
</tr>
<tr>
<td>Peak in mm from surface</td>
<td>10 mm</td>
<td>96 mm</td>
<td>4 mm</td>
<td>0 mm</td>
<td>4 mm</td>
</tr>
<tr>
<td>Length of time step $\Delta t$</td>
<td>2.0 s</td>
<td>2.0 s</td>
<td>2.0 s</td>
<td>2.0 s</td>
<td>2.0 s</td>
</tr>
</tbody>
</table>

1) Assumption based on M1 - V2 concrete
2) Termination due to moisture overflow

The results of the analysis of modeled concrete slabs with a thickness of $h = 100$ mm are presented in Table H8. In terms of the modeled M1 concrete slabs, there are similarities to the analysis on concrete cylinders presented in Table H7.

In the case of the M1 - V2 concrete, the peak pressure was observed in the modeled segment close to the cold surface, which is in contrast to the usual findings and theory. However, as shown in Figure H13 in the time / pressure curve, all segments show a nearly constant pressure level over the entire modeled cross section with a slight decrease in the inner sections because of moisture migration. The peak pressure close to the heated surface is approximately the same as the pressure analyzed at 96 mm distance.

It is interesting to note that the peak pressure for the model based on the M1 - V4 mix (model 20) is high. Spalling cannot be excluded in general at this high pressure. As shown in the time / peak pressure curve in Figure H13, the peak pressure is expected to be close to the heated surface and decreases to $p \approx 1.0$ MPa in the inner section, before it increases again towards the cold surface due to moisture migration into cooler sections of the concrete. All pores can be assumed dry and the pressure decreases after an exposure time of about $t = 88$ min.

It should be noted that the analysis of permeability is based on a concrete mix including 2.0 kg/m$^3$ of PP fibers. This level marks the usual minimum amount of PP fibers, which should be added to concrete in order to reduce the risk of spalling. It can be assumed that higher amounts of PP fibers increase the permeability and lead to lower pore pressures. However, further tests are required.

The M2 concrete mix has a high initial moisture content compared to the other concrete mixes, which already causes misleading results with the model shown Table H5. The termination of the modeling was caused by an oversaturated state of the pores and was observed again with model 21 for the M2 - V4 concrete. After a fire exposure time of only $t = 6$ min, the modeling was stopped. This time a rather high peak pressure of $p = 5.33$ MPa was observed.
The model of the M2 - V4 concrete mix is based on material properties of a concrete including 2.0 kg/m³ of PP fibers. It seems that even with the increased permeability of the concrete, the release of these high amounts of moisture is not possible.

The same phenomenon of excess moisture leading to an oversaturated state of the concrete pores is also observed with model 22 for the M3 - V2 concrete. It causes termination of the model after $t = 17$ min, even though the peak pressure was already observed after $t = 8$ min. However, this high peak pressure of $p = 8.09$ MPa is far too high to be realistic.

The time / peak pressure curve shown in Figure H14 indicates the termination of the model noticed by the vertical lines of data points.

Figure H13: Model 18 - 20: M1 concrete slab heated according to the ISO fire curve time / peak pressure development in all segments

Figure H14: Model 21 - 22: M2 and M3 concrete slab heated according to the ISO fire curve time / peak pressure development in all segments
Models 23 - 26:

In order to compare the beneficial effects of PP fibers and lining as protective measures against spalling, the influence of different layers of lining is modeled in case studies 23 and 24. The material data according to the M1 - V1 mix were chosen, since this mix shows the lowest permeability. Models 25 and 26 are similar to the model for tests on concrete slabs mentioned in chapter 3.3.2 and presented in the test report [75]. Table H9 summarizes all results.

Table H9: Modeled pore pressure with M1 and M2 concrete exposed to ISO fire

<table>
<thead>
<tr>
<th>Model</th>
<th>Model 23</th>
<th>Model 24</th>
<th>Model 25</th>
<th>Model 26</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete mix</td>
<td>M1 - V1</td>
<td>M1 - V1</td>
<td>M2 - V1</td>
<td>M2 - V1</td>
</tr>
<tr>
<td>Heating rate Ṫ</td>
<td>ISO</td>
<td>ISO</td>
<td>ISO</td>
<td>ISO</td>
</tr>
<tr>
<td>Thickness of protective lining</td>
<td>10 mm</td>
<td>20 mm</td>
<td>10 mm</td>
<td>10 mm</td>
</tr>
<tr>
<td>Initial tensile strength f&lt;sub&gt;t&lt;/sub&gt;</td>
<td>7.1 MPa</td>
<td>7.1 MPa</td>
<td>4.2 MPa</td>
<td>5.3 MPa</td>
</tr>
<tr>
<td>Peak pressure p&lt;sub&gt;max&lt;/sub&gt; within 120 min exposure</td>
<td>4.12 MPa</td>
<td>2.10 MPa</td>
<td>6.41 MPa</td>
<td>1.03 MPa</td>
</tr>
<tr>
<td>Temperature T in °C at peak pressure</td>
<td>245°C</td>
<td>212°C</td>
<td>281°C</td>
<td>181°C</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>32.5 mm</td>
<td>10 mm</td>
<td>0 mm</td>
<td>0 mm</td>
</tr>
<tr>
<td>Peak pressure p&lt;sub&gt;max&lt;/sub&gt; within 240 min exposure</td>
<td>4.12 MPa</td>
<td>2.10 MPa</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Temperature T in °C at peak pressure</td>
<td>245°C</td>
<td>212°C</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Peak in mm from the heated surface</td>
<td>32.5 mm</td>
<td>10 mm</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Reduced tensile strength f&lt;sub&gt;t(T)&lt;/sub&gt;</td>
<td>4.9 MPa</td>
<td>5.4 MPa</td>
<td>2.7 MPa</td>
<td>4.7 MPa</td>
</tr>
<tr>
<td>Length of time step Δt</td>
<td>2.0 s</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) Cf. test 1 on concrete slabs presented in the technical report [75]
2) ISO fire + 10 mm protective lining similar to test 2 on concrete slabs in the technical report [75]
3) Termination due to moisture overflow

A lining thickness of 10 mm is usually considered as insufficient for an adequate concrete protection. The model indicated a pressure of up to p = 4.12 MPa within the first third of the concrete slab. Even though the probable resistance seems to be adequate, spalling must be considered at these high pressures. If spalling occurs, it is initiated in deeper sections of the concrete, leading to a large release of concrete cover including the lining.

It was further noticed that some modeled segments still contain moisture after a modeled exposure time of t = 120 min. However, this remaining moisture in deeper segments of the modeled cross section does not lead to significant increase in pore pressure in the long run. All segments are dry after t = 170 min without any increase in peak pressure. The corresponding time / peak pressure curve is shown in Figure H15.

Increasing the thickness of the lining to 20 mm reduces the temperature rise at the surface and minimizes thermal gradients. As a result, the peak pressure was reduced to p = 2.1 MPa after an exposure time of t = 114 min. A total drying of all pores was observed after t = 180 min.

The time / peak pressure curve shown in Figure H15 indicates a peak pressure of the first segment close to the heated surface after t = 56 min with an uncritical pressure of p = 0.6 MPa. The next segment in which peak pressure was noticed is the segment at 10 mm depth from the heated surface with a pressure of p = 2.1 MPa.
With these models for protected concrete it can be concluded that a lining leads to a general decrease in maximum pore pressure. However, it might also lead to delayed spalling, since the pressure still increases to critical levels.

![Graph showing time vs. peak pressure for Case studies 23 and 24](image)

Figure H15: Model 23 - 24: M1 concrete slab with protective lining heated according to the ISO fire curve - time / peak pressure development in all segments

**Models 25 + 26:**

Models 25 and 26 refer to similar test models for M2 concrete slabs. These tests are presented in detail in the test report [75] and the results are summarized in chapter 3.3.3.

Similar to other models with an input parameter based on the M2 concrete, the high amount of initial moisture leads to termination of the model. In terms of the unprotected concrete slab (model 25), an unrealistic high pressure of $p = 6.4$ MPa was observed in the first segment after $t = 6$ min. Even though the iteration of the model stopped after this time step, it indicates probable failure of this segment due to the formation of a moisture clog, combined with high pore pressure close to the heated surface. In terms of tests, layer-by-layer spalling was noticed after $t = 15$ min fire exposure.

The iterative process for the model on the protected slab (model 26) stopped after $t = 27$ min with a peak pressure of $p = 1.03$ MPa in the first segment. In tests failure was noticed after $t = 119$ min.

The main reason for the huge difference in predicted failure time of the model compared to test is the lack of microcracking, which is not included in the model. This would promote releasing the excessive moisture.
Figure H16: Model 25 - 26: M2 concrete slab with protective lining heated according to the ISO fire curve - time / peak pressure development in all segments
CURRICULUM VITAE

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