FIRE RESISTANCE OF EXISTING STRUCTURES

Assessing the fire resistance of existing concrete buildings

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PREFACE

This thesis is written as part of the Master "Structural Engineering" at the faculty of Civil Engineering and Geosciences of TU Delft. It presents my ten month of graduation research, developed under guidance of IOB section Structural Engineering. First of all, I want to thank André Lankhof for the opportunity to perform my research at IOB and his role as thesis supervisor. Secondly, I would like to thank Patrick Meerkerk, Jurjen Meuldijk, and Reint Sagel for the interesting conversations and their motivating assistance. Furthermore, I want to thank Paul Lagendijk, Rob Nijsse, and Sander Pasterkamp for their counselling as members of my graduation committee.

Last but not least, I am grateful to my friends and family for their support, in particular my sister with her writing skills and my parents, without whose confidence in my capabilities, I could never have finished this thesis.

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SUMMARY

Many vacant buildings that are reused are constructed in concrete. With concrete, one thought to have found a solution for the fire resistance of buildings. Today it appears that the fire resistance of concrete is not always a matter of course. Therefore, it is important to properly asses the fire resistance of existing concrete buildings.

With the intention to help the structural engineers in the future and to smoothen the process of the fire resistance assessment, this thesis summarizes and discusses the main points of attention concerning the determination of the fire resistance of existing concrete buildings. This is done by means of relevant literature, interviews, and a case study.

The results of this research show that a structural engineer encounters several difficulties while assessing the fire resistance, beginning with the fire resistance requirements. Although the fire resistance requirements are slightly adapted and supplemented over the years, these requirements are still unclear and the backgrounds of these performance requirements are not explained in the Building Decree. In combination with the limited knowledge of the municipalities and the fire brigade, this could lead to unmotivated high demands and a difficult use of the principle of equivalence.

Besides the knowledge of the municipalities and the fire brigade, the knowledge of the structural engineer himself is also limited. Although there are many temperature effects known, it appears to be difficult to get an idea of the fire resistance of the concrete material, as well as the structural behaviour under fire conditions. For these reasons, it will take some time before the calculation methods in the Eurocode could be supplemented with more detailed methods. Even though the current Eurocode recommends to assess the structure as a whole and notes that the imposed deformations should be taken into account, the Eurocode mainly gives simple calculation methods concerning individual elements or small parts of the structure.

However, despite all these difficulties, this study also makes clear that it is still possible to deal with a fire resistance assessment of an existing concrete building in a more efficient way, partly due to the fact that the consequences of thermal expansions do not always have to be disadvantageous.

Based on all the collected information, this thesis is concluded with a recommended approach for the structural engineer to assess the fire resistance of an existing concrete building.

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1 INTRODUCTION

1.1 Research motivation

The Netherlands has seen an increasing trend in vacant buildings in recent years. This vacancy mainly occurs in the office sector and the retail sector. The retail sector shows a percentage of 9.2 % of vacant floor space, which corresponds to more than 3 million square meters [1]. For the office sector, this percentage counts 17.2 %, corresponding to a floor space of 8 million square meters, which directly is the highest percentage in Europe [2, 3]. A part of these empty buildings might be well exploited in order to fulfil the need for temporary or permanent housing [4].

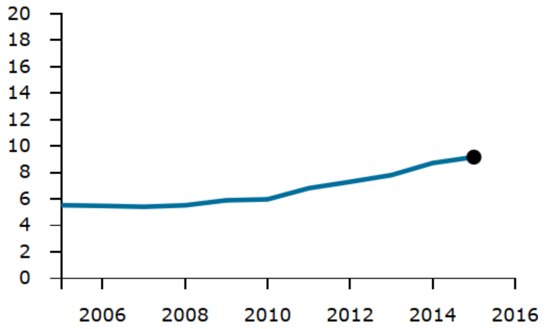


Figure 1 | Vacancy rate of retail buildings, shown as percentage of total rentable retail area [1]

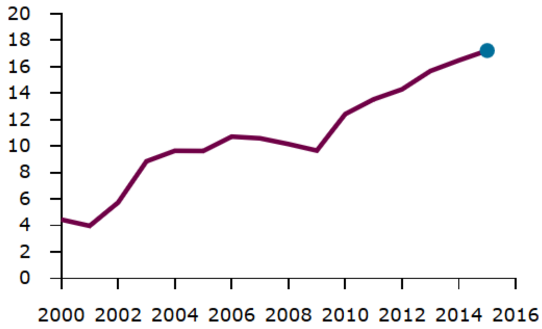


Figure 2 | Vacancy rate of office buildings, shown as percentage of total rentable office area [3]

Nowadays, more and more existing buildings are therefore being reused. The redevelopment is particularly focused on creating luxury apartments in historic complexes and the transformation of old factories into new workspaces [5]. This development often leads to cheaper solutions than building completely new buildings [6].

The reuse of existing buildings (both for a function change or a structural alteration) often requires renovations due to their age. These renovations must meet the minimum requirements which are included in the Building Decree [7]. This also holds in terms of fire safety. However, this topic did not always get the same amount of attention in the twentieth century as it does now, because the knowledge concerning the consequences of a fire in a concrete building was limited.

This observation forms the motivation of this research. With this thesis, it is intended to look into how this former lack of attention influences the fire resistance of existing buildings. Paragraph 1.2 will explain the scope of this research, which subsequently leads to the research objectives in paragraph 1.3. The research methodology of this thesis is addressed in paragraph 1.4.

1.2 Scope of the research

In the seventeenth century, there already existed several fire safety provisions in the Netherlands [8]. These provisions were based on preventing and limiting a fire and were only used in cities. One should think of measures like a prohibition of wooden and thatched roofs, a prohibition of tarring houses, and alignment rules. It was not until the end of the nineteenth century, however, that one started to think about preventing and limiting a fire on a larger scale [9, 10]. Because of several dramatic fires in the nineteenth century, fire prevention became a subject of broad interest in the twentieth century. The development of fire safety regulations had therewith begun [8-10].

In addition to this development, the twentieth century experienced another important development, namely the emergence of reinforced concrete. The high strength of the material (in relation to building materials such as timber), the long service life and the fact that it could be formed in all kinds of shapes, were the major advantages of the reinforced concrete at the beginning of the twentieth century [11].

It became apparent later on that the behaviour of concrete during a fire is another essential benefit of this material. From many previous fires, it appeared that the building materials wood and steel had different advantages and disadvantages concerning a fire. Wood is a material that burns, which means that it has a bad reaction to fire, but the charred wood provides a good fire resistance for the remaining part of a structural element [12]. With steel, this is precisely the opposite: this material does not burn, which means that the reaction to fire is good, but the resistance to fire is very poor [12]. Concrete combines the two qualities: it does not burn and it has a good resistance to fire. Municipalities therefore thought they had found a solution for the fire safety of buildings. This, together with the other assets of concrete of which some were mentioned earlier, leads to the fact that concrete became the main construction material of the twentieth century [10-12].

However, today it is known that the fire resistance of concrete is not a matter of course. For example, the reinforcement steel will lose its strength, the concrete surface can be damaged by “spalling”, and thermal stresses can lead to deformations or even failure of a structure [10, 13, 14].

Because of the reasons stated above, this project will focus on the fire resistance of existing concrete buildings, which explains the title of this thesis: “Fire resistance of existing structures – Assessing the fire resistance of existing concrete buildings”.

1.3 Research objectives

Based on the motivation and scope of this research, the following main objective is central to this report:

- *Summarize the main points of attention of the fire resistance assessment procedure of an existing concrete building for the structural engineer.*

This main objective is divided in five secondary objectives, which are related to specific subtopics. These subtopics and secondary objectives are as follows:

- The fire resistance requirements

  Summarize the current fire resistance requirements and their method of application, in comparison with the historical fire resistance requirements.

- The temperature effects on reinforced concrete

  Briefly explain the different temperature effects on reinforced concrete and summarize the several factors which influence them.

- The structural behaviour of a concrete building under fire conditions

  Describe how the thermal expansions can influence the structural behaviour and fire resistance of a concrete building.

- The building standards

  Discuss the emergence of the fire resistance in the building standards and consider which aspects are still missing.

- Estimation of the fire resistance

  Examine whether it is possible to give an estimation of the fire resistance of an existing concrete building, despite the missing knowledge concerning different aspects of the fire resistance.

1.4 Research methodology

These subtopics are covered in different chapters of this thesis. This is done by means of relevant literature, interviews, and a case study. To fully understand this thesis, it is important to know the basics of the fire physics. For this reason, chapter 2 will briefly explain these basics. In chapter 3, the fire resistance requirements are discussed. Chapter 4 and 5 focus on the temperature effects on reinforced concrete and the structural behaviour of a concrete building under fire conditions. The development and application of the building standards are treated in chapter 6. Finally, the acquired knowledge of chapter 4, 5, and 6 is applied on the case study, which will be dealt with in chapter 7. With this case study, it will be examined whether it is possible to give an estimation of the fire resistance of an existing concrete building, despite the missing knowledge concerning different aspects of the fire resistance, mentioned in chapter 4, 5, and 6.

2 FIRE PHYSICS

2.1 The fire triangle

Fire is a chemical reaction between a fuel and oxygen in which energy (heat) is released [9, 15]. This reaction takes place when a combustible material, oxygen, and heat come together. These three aspects together form the well-known ***fire triangle***, as is shown in Figure 3.

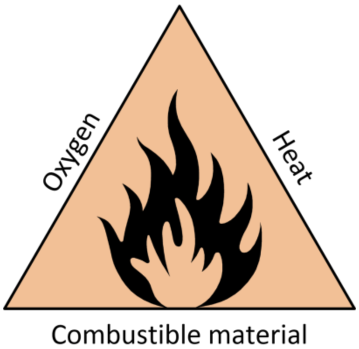


Figure 3 | The fire triangle

The combustible material is often a solid or liquid [9, 15]. A material in a solid or liquid state cannot spontaneously combust. However, when these materials reach a certain temperature, flammable gasses are produced. The temperature at which a fire starts, is called the ***ignition temperature***.

The distribution and the type of the materials across the room are essential for the way in which the fire develops [9, 15]. Porous and wooden materials in furnishings contribute to a rapid growth of a fire, while plastics starts to drip, which can cause a fire on the floor. The position of the materials relative to the local fire also has a huge influence, because flames will spread much faster across a vertical than a horizontal surface.

When a fire has begun, it will proceed as long as there is enough fuel and oxygen and the temperature remains high enough [9, 15]. During the fire, the combustible element carbon, which is the most important element in combustible materials, initially changes into carbon dioxide (CO_2). But in case of a lack of oxygen, the well-known gas carbon monoxide (CO) is produced, which is very dangerous for man.

2.2 Fire development

In a closed room, the process of the fire development goes through the following three phases [9, 16]:

- the growth phase;
- the fire phase;
- the decay phase.

These different phases will be explained in this paragraph. Each phase is associated with a certain temperature range. The relation between the temperature and the time in a fire in a closed room is shown in a fire curve in Figure 4. This curve is a model, which means that it is a simplification of the reality. More information about fire curves can be found in paragraph 2.4.

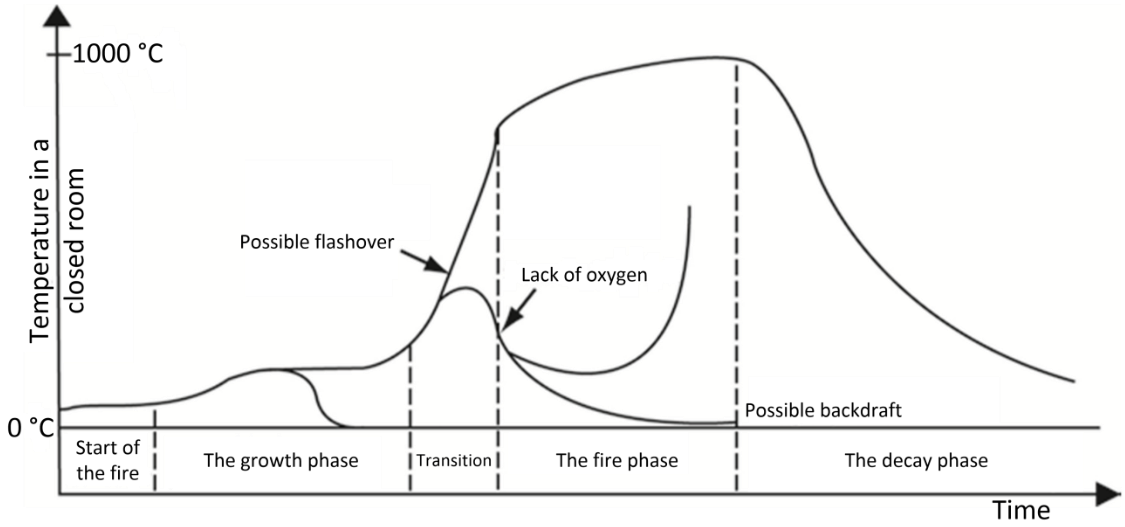


Figure 4 | Fire curve with the three different phases of the process of the fire development [9]

2.2.1 The growth phase

As was stated earlier, a fire originates when all of the three elements of the fire triangle are present. The temperature in the room increases and a small amount of materials starts to burn. The first gasses and smoke appear. The gasses rise and form a hot cloud at the ceiling, because they are lighter than the cold air. This layer radiates heat to other objects in the room, which will ignite as well. This leads to even more gasses that ascend, causing the upper cloud getting hotter, thicker, more flammable and more concentrated. At a temperature between 200 °C and 300 °C, there is so much accumulation of combustible gasses that a combustible gas-air mixture can occur. Whether or not the fire continues from this stage, depends on several factors. One important factor is the presence of enough oxygen. During the beginning of the fire, all of the oxygen is extracted from the air in the room. But later on, air could flow in via openings as broken windows and doors as well. Through the same openings, hot gasses are able to escape when the interface between the hot and cold layer has reached the top of these openings, due to the increase of the thickness of the hot layer [9, 16]. This situation is schematized in Figure 7A.

2.2.2 The fire phase

The fire phase takes its name from the fact that the fire is fully developed in this period [9, 16]. The growth stage has reached its maximum and all of the combustible materials have been ignited. The transition of a local fire to a fully developed fire takes place between the growth phase and the fire phase, and is known as the ***flashover***. During a flashover, the temperature in the smoke layer under the ceiling of the room is so high that the heat radiated by that smoke layer ignites all the inflammable objects in the whole space, including those under the smoke layer, within a very short period of time, as a result of thermal decomposition. Because this definition refers to the transition of a local fire into a fully developed fire in a room, it means that a flashover could only take place in an enclosed compartment. Without a flashover, the temperature would rise slower. But even then, the temperature could increase to 900 °C in just 15 minutes.

2.2.3 The decay phase

The temperature and the intensity of the fire subside when approximately 80% of the fuel has been expended. At this moment, the decay phase starts [9, 16]. This phase could also start due to a lack of oxygen. In this situation, the remaining combustible materials can be red-hot, but the flames are extinguished by the lack of oxygen. Although the temperature drops, it is important to note that it remains high enough to cause a ***backdraft*** in combination with the remaining flammable gasses. This is a mechanism in which a sudden exposure to oxygen in a space without any oxygen, but with a high temperature and a significant amount of flammable gasses, leads to an almost explosive combustion.

Figure 5 | Backdraft [17]

2.3 Reaction to fire and resistance to fire

Reaction to fire and resistance to fire are two definitions that are often used in case of fire safety. For this reason, it is important to know what they mean and how they differ.

Reaction to fire refers to the characterisation of a material on a relatively small scale [16]. It is a measure of all the properties of a material which are relevant for the ignition and the development of a fire, such as the inflammability, the means of propagation of flames along the surface of the material, the energy release due to the combustion, and the smoke production.

Resistance to fire refers to the characterisation of the ability of a structure to maintain its function during a fire [16]. This contains information about the ability of a fire separating structure to stop the spread of fire, or the ability of a load bearing structure to resist the fire without collapsing.

Because the focus of the fire safety requirements and calculation methods which are treated in this thesis is on the structural behaviour of buildings in case of a fire, the fire resistance is mainly concerned. However, the properties of a material will affect the performance of structural elements. Therefore, the behaviour of structural materials exposed to fire will be discussed in chapter 4.

2.4 Fire curves

In practice, fires would never have exactly the same temperature development as each other. The fire temperature depends on specific circumstances, such as the ventilation conditions, the distribution and quantity of combustible materials, and the insulation of the compartments. In order to be able to design structures in a generic way, and to evaluate the performance of fire resistant structures and compare them with the fire safety requirements, it is necessary to simplify the temperature development. For this reason, several fire curves are generated [16].

2.4.1 Nominal temperature-time curves

In general, the design of a structure is based on the thermal actions given by a nominal fire exposure. The nominal temperature-time curves show the temperature development of a fire as a function of time. They do not contain a single relationship to the characteristics of the building which is considered. Therefore, it is clear that these curves are very simplified models for the representation of a fire [18, 19]. Different types of these curves are shown in Figure 6 and are discussed below.

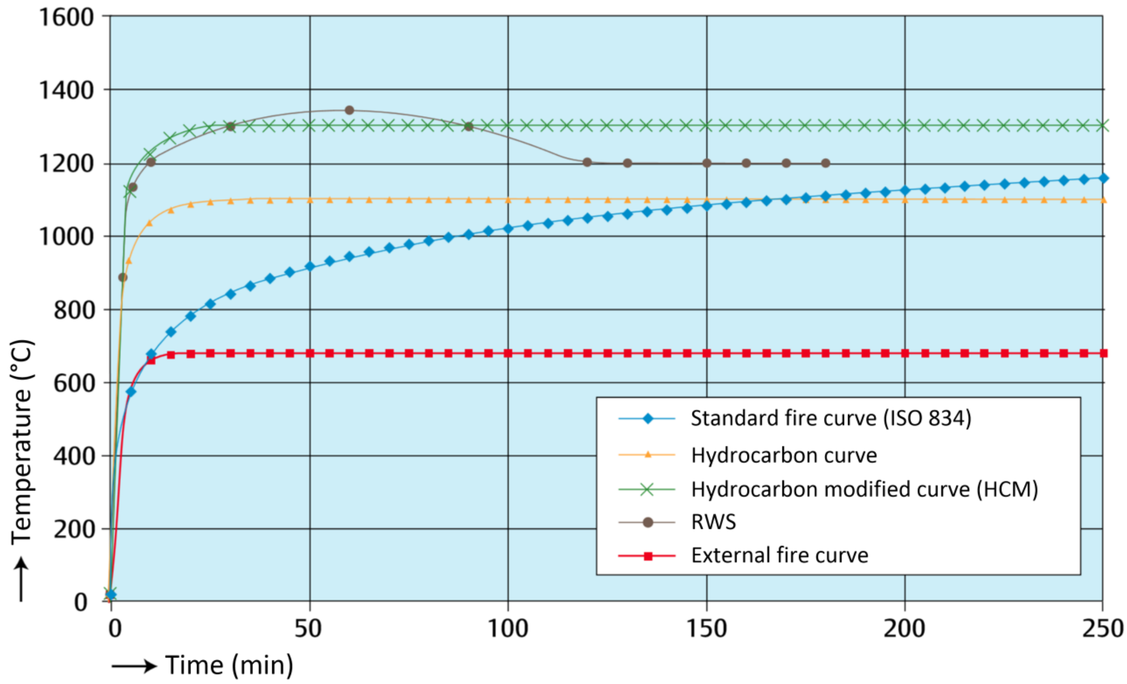


Figure 6 | Several nominal temperature-time curves [18]

The ***standard fire curve*** or ***ISO 834*** is the fire curve which is normally used for testing structural elements in case of a fire, as well as for the calculations of structures during a fire. This most common nominal curve is the result of the standard fire test which was developed in 1930. The test was based on the combustion of cellulosic materials, such as wood and paper, which often occur in buildings. In that time, it was already noted that there was no direct demonstrable relationship between the theoretical curve and a fire in practice. However, this curve is still used, although the application of the curve usually leads to conservative results compared to real fires. Therefore, the discussion about the use of this curve as the base of fire tests, fire safety requirements, and calculation methods, still continues and is reflected in attempts to position the natural fire curve as the basic model in the last couple of years [9, 18].

The ***external fire curve*** is a temperature-time curve which is used for – as the name already suggests – fires which occur in the outside, such as a fire under a bridge [18].

For fires which are caused by the ignition of hydrocarbons (such as tankers and oil storages), tougher fire curves than the ISO 834 should be assumed, since the fuel has a very high calorific value in these cases. For this purpose, ***hydrocarbon curves*** are used, which have a maximum temperature of 1100 °C. In the French regulations, an even more severe curve is used, which has a maximum temperature of 1300 °C. This curve is known as the ***HCM***: the ***hydrocarbon modified curve*** [18].

Finally, the grey line represents the ***RWS-curve***. This curve is developed by Rijkswaterstaat for specific use for fires in tunnels, based on a research of TNO [18].

2.4.2 Simplified natural fire models

When other aspects of a fire need to be taken into account and the relation between temperature and time is not enough, natural fire models are used [9, 15, 18]. This is mainly the case in special situations, where the application of the standard fire curve is too unrealistic. These models provide a more realistic approach of a fire. Unlike the nominal fire curves, which only take into account a fully developed fire, natural fire models have a growth- and decay phase.

Simplified natural fire models are based on a small number of specific physical parameters with a limited field of application [9, 15, 18]. Two familiar simplified models are the compartment fire and the local fire. The ***compartment fire*** is based on a uniform temperature distribution as a function of time, while the ***local fire*** is based on a non-uniform distribution. A good example of a compartment fire is shown earlier in Figure 4.

2.4.3 Advanced natural fire models

For more sophisticated calculations, use can be made of advanced natural fire models. There are various types of advanced models. In ascending order of complexity, these are the one-zone models, the two-zone models and the field models [9].

The basics of zone models consists of several sub-sections that may vary in time, such as a fire of a known area and output, a plume of smoke, and a hot layer of smoke [9]. In a ***one-zone model***, a homogeneous temperature distribution in a room is assumed. A ***two-zone model*** is based on stratification by a horizontal separation between a hot zone, the smoke layer, and a cold zone, the layer without smoke. When a fire develops, the smoke layer thickens and the temperature of this layer increases, while the smoke-free layer gets smaller. This process is described earlier in paragraph 2.2. When the conditions are such that flashover occurs, the two-zone model changes into a one-zone model. Both of these models are illustrated below.

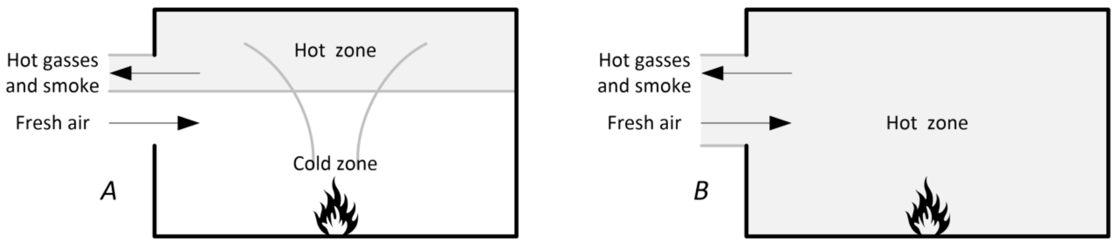


Figure 7 | Schematic representation of a two-zone model (A) and a one-zone model (B)

Field models, also known as ***computation fluid dynamics (CFD)*** models, are models in which three-dimensional areas are divided into cells [9]. The conditions of the heat and the smoke can be calculated for each cell. In addition to the temperature distribution, other properties can be considered as well, such as the heat flow. Structural elements and objects within these elements which could influence this heat flow, could be incorporated in the calculations. The required accuracy determines to what level of detail the geometry should be set. It is important to keep the input of the geometry as part of the field model as simple as possible, without any loss of relevant information, because extensive details of the geometry consume a lot of processing power of the computer.

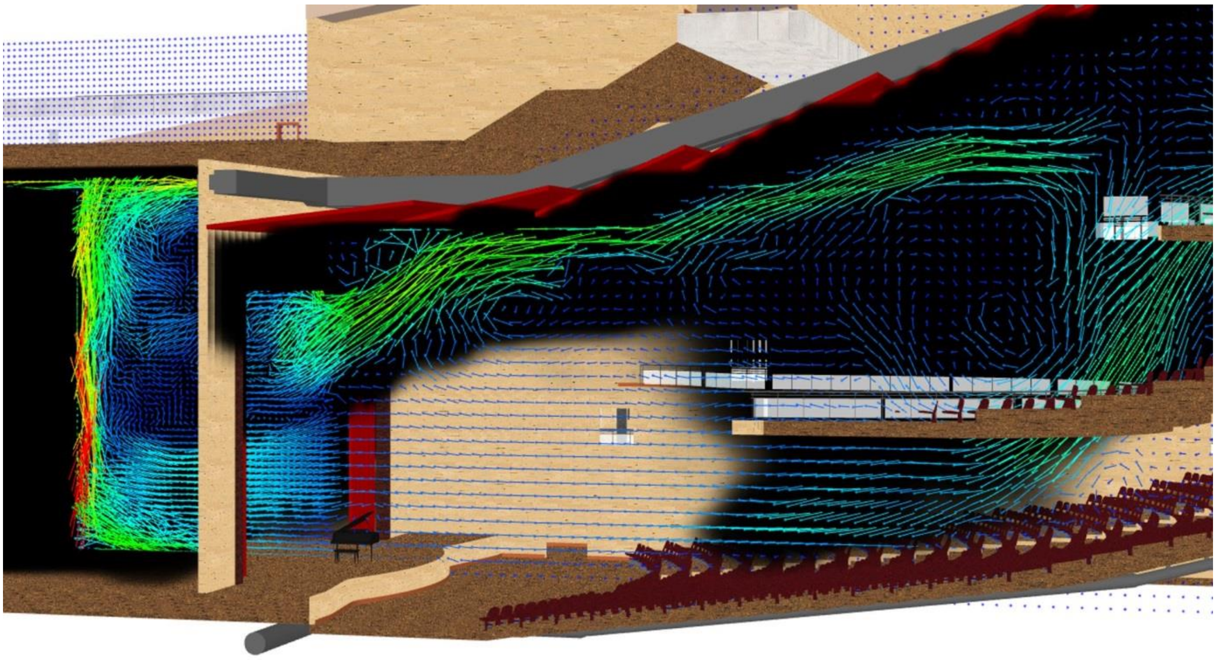


Figure 8 | Example of a field model [20]

3 FIRE REGULATIONS

3.1 The Dutch building regulations

Fire prevention in buildings is regulated by the government through a system of laws and regulations, which hold for the construction, the furnishing, and the use of buildings, together with several regulations for the internal and external emergency response. This system consists of three different parts [9, 21]:

- The building regulations
- The regulations on the working conditions
- The fire service assistance regulations

It goes without saying that for the structural design and safety of a building, only the first one is of importance. These buildings regulations are part of a system of regulations, by virtue of the Dutch Environmental Permitting Act (in Dutch: “Wet algemene bepalingen omgevingsrecht” or “Wabo”). This act was introduced in 2010 and mainly focusses on issues as several permits (as construction permits, environmental permits and listed building consents), which are merged into one integrated permit, the so-called “environmental permit for physically location-specific projects” (in Dutch: “de omgevingsvergunning voor fysiek locatiegebonden projecten”) [9, 22]. In The Living Environment Law Decree (in Dutch: “Besluit omgevingsrecht” or “Bor”) and the ministerial regulation on environment law (in Dutch: “ministeriële regeling omgevingsrecht” or “Mor”), this act is elaborated further. The Living Environment Law Decree contains, among other things, provisions on the obligation to obtain a permit, while the ministerial regulation on environment law regulates the submission requirements for an environmental permit [23, 24].

Technical regulations are given in the Dutch Housing Act (in Dutch: “Woningwet”) and the Dutch Building Decree (in Dutch: “Bouwbesluit”). The Housing Act contains specific technical instructions for the construction and the use of buildings, while the official building rules – including those on fire safety - are mentioned in the Building Decree [9, 21]. For this reason, this decree will be discussed in detail in paragraph 3.2.

Finally, there are municipal building regulations (in Dutch: “bouwverordeningen”). Based on the Housing Act, municipalities are obliged to establish these, for which they could make use of the sample municipal building regulations, formed by the Association of Netherlands Municipalities (in Dutch: “Vereniging van Nederlandse gemeenten” or “VNG”). However, these regulations are of limited significance for the fire safety in the field of the structure of a building nowadays – the fire safety requirements that are mentioned here are mainly focused on installations and the use of buildings [9, 25][*].

[*] This also came up in a conversation at 11-12-2015 with ir. Jurjen Meuldijk, fire safety consultant at IOB

3.2 The Dutch Building Decree

3.2.1 Brief history of the formation of the Dutch Building Decree

In paragraph 1.2, it was already mentioned that as a result of several utility buildings which burned down at the end of the nineteenth century, the interest in fire safety increased a lot around this time [10]. City governments did not have a Building Decree, but they did already have some technical building codes on a local scale which were focusing on public order, safety and health. These codes did not have specific fire safety requirements, but they contained a number of measures meant to reduce the risk of fire. For example, some governments forbade the building of wooden houses and the tarring of buildings.

Because of the industrialization in this century, the attention to the homecraft slowly faded out. The housing conditions became dramatic. These bad conditions led to the introduction of the first Housing Act in 1901, a law which required that every municipality had to establish rules which had to be met in the construction, renovation and expansion of a house [8]. Since then, there could not be built without a license anymore (with some exceptions, such as several maintenance activities).

When the usage of steel and reinforced concrete increased, the differences between the municipal building regulations appeared to be so big, that it strongly hindered the building industry. As a result, the Decree of Uniform Building Regulations (in Dutch: “Besluit Uniforme Bouwvoorschriften”) came into force in 1956 [8]. This ensured that when a building plan fulfilled the provisions of the decree but not the provisions of the municipal building regulations, it had to be approved.

But this did not seem to solve the problem. The model-building regulations (in Dutch “Model-bouwverordeningen”) did not appear to be the right solution as well. The model-building regulation of 1965 was the first Dutch document with uniform fire resistance demands [8]. It was recommended to all municipalities to at least adopt the technical requirements of the model-building regulations into their own municipal building regulations without any changes. Unfortunately, working according the building regulations remained a difficult case, because the differences between the municipalities were still big enough to obstruct things like innovation and optimization.

This is why the Building Decree (in Dutch: “Bouwbesluit”) was introduced in 1992 and is still used nowadays. In this document, the technical requirements are nationally standardized. Since the introduction of this decree, municipalities are not allowed to adopt technical requirements in their own building regulations anymore [26].

The Building Decree of 1992 was the first document with national standardized requirements for the fire safety of buildings in the Netherlands [8]. These requirements, both those of the first decree as those of the current one, will be discussed in the remainder of this chapter.

3.2.2 Main objective of the Building Decree concerning fire safety

In the general notes of the Building Decree, the main objectives of the fire safety rules are stated as follows [26]:

- Preventing casualties (people being killed or injured)
- Preventing fire spreading to another plot

It is often assumed that preventing damage is a main objective as well, but this is a mistake. Damage can lead to economic and social consequences, but it does not threaten the physical safety of people. For this reason, damage prevention is not a task of the government, but is the responsibility of the owner [9, 26].

3.2.3 General fire safety requirements

To meet these objectives, the Building Decree contains several fire safety regulations, which can be found in three different chapters [26]:

- Chapter 2: Technical building regulations in terms of safety
- Chapter 6: Requirements for installations
- Chapter 7: Regulations for the use of buildings

Each chapter contains several functional requirements. With these requirements, the legislator indicates what is contemplated by the relevant regulations. The following table shows the functional requirements of the paragraphs which are related to the fire safety [15]. Because this master research focuses on the structural point of view of the fire safety, chapter 6 and 7 are excluded.

Table 1 | Technical building regulations in terms of fire safety [15]

Section		Functional requirement
2.2	Strength in case of a fire	A building which will be built, or an existing building, can be left and searched during a reasonable period in case of a fire, without any danger of collapse
2.8	Limiting the occurrence of fire-hazardous situations	A building which will be built, or an existing building, is such that the occurrence of a fire-hazardous situation shall be sufficiently limited
2.9	Limiting the development of fire and smoke	A building which will be built, or an existing building, is such that fire and smoke cannot develop quickly
2.10	Limiting the spread of fire	A building which will be built, or an existing building, is such that the chance of a rapid spread of fire is sufficiently limited
2.11	Further limiting the spread of fire and limit the spread of smoke	A building which will be built, or an existing building, is such that the spread of fire is restricted in farther extent than is envisaged in section 2.10, and that it is possible to flee safely
2.12	Escape routes	A building which will be built, or an existing building, has such escape routes that a safe place can be reached in case of a fire
2.13	Emergency assistance in the event of a fire	A building which will be built, and an existing road tunnel with a length of more than 250 m, are such that emergency services can save people and fight fire in a reasonable time

Section		Functional requirement
2.14	High and underground buildings	A building which will be built, in which a floor of an area of use is 70 m above or 8 m below the measuring level, is arranged in such a way that the structure is fireproof
2.16	Safety zone and attention area of a flammable liquid fire	A building which will be built in a safety zone or in a flammable liquid fire attention area, or above the full width of a basic transport route if the safety zone is only a part of the width of that basic transport route, is such that the risk which arises from the transport of hazardous substances, is limited for the people inside the building

In each section, these functional requirements are elaborated in performance requirements. By fulfilling these performance requirements, the functional requirements are also met [15].

These performance criteria provide a distinction between safe and unsafe [27, 28]. An intermediate zone (more or less safe) does not exist and the relationship between the individual variables is not considered. The advantage of this approach is the relatively simple design of the fire regulations. However, it is the question if in all of the conceivable cases that comply with the Building Decree, the same level of safety is achieved.

The fulfilment of all the performance requirements would not lead to situations in which the fire would not take a single casualty. The safety level of the Building Decree is based on a socially acceptable risk. This risk can be defined by an allowable failure rate for each functional requirement or sub-goal. The allowable failure rate can be determined by the level of facilities of a building which directly meets the performance requirements, which can be seen as the reference level. By varying the failure rates, an acceptable risk could be achieved in multiple ways, even if not all of the performance criteria are met. If not all the performance requirements are met, but the authorities can show that there is compliance with the functional requirement, the solution complies to the equivalence article of the Building Decree [27, 28]. This article will be mentioned in paragraph 3.2.7.

3.2.4 Relevant requirements for loadbearing structures of concrete

In the building standards, there are generally three different performance criteria distinguished for the loadbearing structures in relation to the fire safety: the loadbearing capacity (criterion R), the integrity (criterion E), and the thermal insulation (criterion I). These criteria, the fire resistance criteria, are defined in the NEN-EN 13501-2 as follows [29]:

- ***Loadbearing capacity (R)*** is the ability of the element of construction to withstand fire exposure under specified mechanical actions, on one or more faces, for a period of time, without any loss of structural stability.
- ***Integrity (E)*** is the ability of the element of construction that has a separating function, to withstand fire exposure on one side only, without the transmission of fire to the unexposed side as a result of the passage of flames or hot gasses. They may cause ignition either of the unexposed surface or of any material adjacent to that surface.

- ***Thermal insulation (I)*** is the ability of the element of construction to withstand fire exposure on one side only, without the transmission of fire as a result of significant transfer of heat from the exposed side to the unexposed side. Transmission shall be limited so that neither the unexposed surface nor any material in close proximity to that surface is ignited. The element shall also provide a barrier to heat, sufficient to protect people near to it.

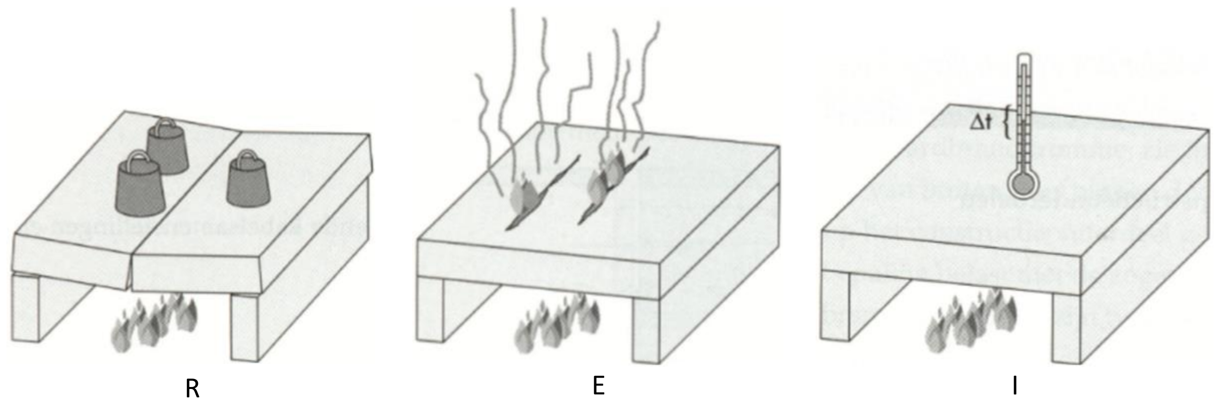


Figure 9 | Visualization of the three performance criteria [15]

Criterion R is required when during a fire, a mechanical strength for a structure is necessary. This structure must be designed and constructed in such a way, that it maintains its loadbearing capacity to fire during the exposure. This criterion therefore applies to all loadbearing elements.

Criteria E and I are of importance for compartmentation. Compartmentation is an important aspect in building designs. Buildings need to be divided into fire compartments, which are meant to be the maximum expansion area of a fire. Compartmentation limits the spread of the fire, leading to a restriction of the fire damage and safe escape routes. When compartmentation is required, the partitioning elements (joints included) must be designed and constructed in such a way that they maintain their separating function during the exposure to a fire. Therefore, it is required that no cracks, holes or other openings arise, through which the fire can be let through in shape of hot gasses or flames. This refers to the integrity (E). A graduate research of A.Y. Botma has shown that this criterion is often underestimated [30]. Besides this criterion, the isolating function must not fail, which could lead to a temperature rise above the ignition temperature at the non-exposed side of partitioning element. This refers to the thermal insulation (I). Both criteria apply to the loadbearing elements with a partitioning function, like walls, floors, facades, and roofs.

It is important to keep in mind that these two criteria does not only relate to the fire spread through walls or floors inside a building, but to the fire spread outside a building as well. When flames pass the façade, the fire could spread to the storey above, or even to the building beside the one that is on fire. This phenomenon has caused major city fires in the past. A very well-known example is the one which occurred in London in 1666, which destroyed more than 80 percent of the historical core of the British capital [31]. To avoid these situations, one started to take measures like applying certain distances between buildings and prohibiting tarring houses. Nowadays, these situations are mainly avoided by applying the criteria E and I, by means of fire-resistant windows, doors, walls, or roofs.

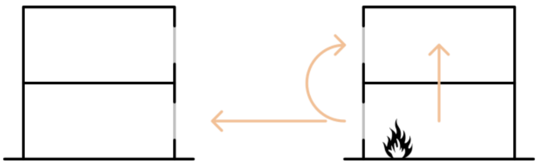


Figure 10 | Different ways of the spread of a fire

The terms R30, R60, E30, E60, and I30, I60, indicate that an element needs to meet the criteria R, E, or I for 30, 60, or more minutes. The indication REI30 means that an element needs to fulfil all three criteria for at least 30 minutes. In this case, the most critical criterion is decisive [18].

However, the NEN 6068 mentions that the fire resistance with respect to the separating function of a structural element is at most equal to the fire resistance with respect to failure [32]. Ir. Jurjen Meuldijk, fire safety consultant at IOB, also mentioned that for a loadbearing concrete element, criterion R is the most influential one. For this reason, this master research will be limited to the fire resistance requirements with respect to failure. So when in the remaining chapters fire resistance is mentioned, this will only indicate the fire resistance with respect to collapse.

3.2.5 Fire resistance requirements

The requirements of the Building Decree which correspond to the criterion R, can be found in Section 2.2 – Strength in case of a fire [26]. In this section, the ***fire resistance*** is defined as a time period in which a building is on fire and should not collapse, to give people the ability to flee and search the building, without the risk of structural collapse. Noteworthy is that the background of these requirements are nowhere explained in the Building Decree; only the values of the fire resistance are given in minutes and form the performance requirements. These performance requirements are divided in three performance levels: new buildings, alteration or renovation, and existing buildings [26, 33]. In the next paragraphs, the requirements will be mentioned per performance level, in combination with their backgrounds (as far as they are known) which are missing in the Building Decree. The application of the performance levels will be discussed later in paragraph 3.2.7.

3.2.5.1 Parts of a structure which require fire resistance demands

Before mentioning the performance requirements, it is important to know that the starting point of the fire resistance requirements is that the ***sub-fire*** or ***fire compartment*** in which a fire occurs may collapse, as long as this, within a certain time frame, does not lead to the collapse of structures outside this compartment. In other words, progressive collapse needs to be prevented [26]. A fire compartment is defined as a part of one or more structures meant as the maximum expansion area of a fire. A sub-fire compartment is a part of a building that is within the boundaries of a fire compartment, meant to limit the spread of smoke and the maximum expansion area of a fire [15]. To clarify this starting point, imagine that there is a fire at the top floor of a building which has three storeys (situation A of Figure 11). The structures that are adjacent to this fire compartment are the walls, the floor and the roof of the third storey, all marked with a red line. Failure of these structures will not lead to the collapse of other structures (the black lines). This means that the third storey does not have to fulfil any fire resistance requirements. But when there is a fire on the second floor (situation B) and the adjacent structures of the second floor collapse, the non-adjacent structures of the third floor will come down as well. The same holds for situation C, where failure of the adjacent structures of the first storey will lead to the collapse of the non-adjacent structural elements of the second and third floor. This means that the building structures adjacent to the first and second floor (marked with a blue line in D), must meet certain fire resistance requirements [34].

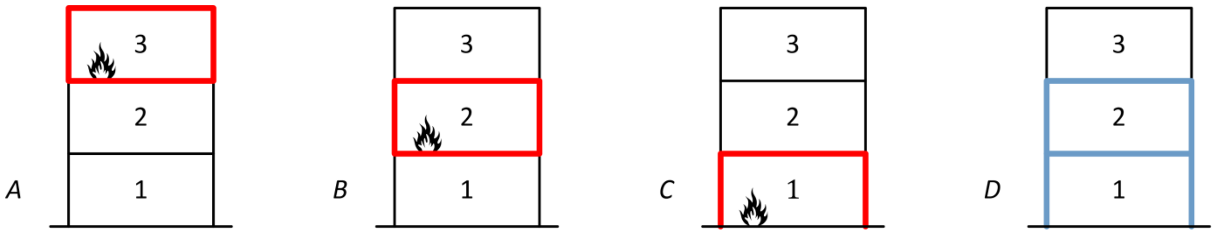


Figure 11 | Schematization that shows which structural parts of a building consisting of 3 large fire compartments need to fulfil the fire resistance requirements [34]

This means that in a case of a building which only has one storey, the structure generally does not need to fulfil any fire resistance requirements (except maybe the requirements related to the fire spreading to adjoining buildings). This followed from a thought that originated in the mid-twentieth century [9]. The thought was that it was not necessary to impose special requirements on the fire resistance of the loadbearing structure, since if the loadbearing structures were exposed to a temperature that was so high that they might fail, this temperature would also be too high for the firefighters to take action. And at the places were firefighters could take action, the temperature would be so low that there was no risk of structural collapse. Although current fire fighters have protective clothing and breathing equipment in contrast to the firefighters of the mid-twentieth century, which means that burning compartments can be entered further nowadays, this principle is still applied at present.

With this principle, it is assumed that a building which only has one storey can be left almost immediately after the occurrence of a fire, so the people inside the building should already be gone when the firefighters arrive. For example, The Basis for Fire Safety (a knowledge document providing substantiation, argumentation, and background information about fire prevention, published by the Dutch institute of physical safety) mentions a period of just 3 minutes after the occurrence of a fire in which people are alarmed and were able to leave their house. In case of a building with non-self-sufficient people, this time is estimated as 5 minutes [9]. However, one should note that if a building without any fire requirements cannot be left directly in case of a fire, a requirement for an escape route still needs to be fulfilled. An escape route which is located within a sub-fire compartment where a fire occurs, is allowed to become unusable as a result of failure, because this route is already inefficient due to the fire and smoke [26].

3.2.5.2 Fire resistance values for new buildings

The values of the performance requirements (so the length of the fire resistance) are depending on the function type of the building, and the building height [15, 26]. The function types can be divided in three main categories: housing, utility buildings with sleeping accommodations (for example, a hotel, a prison, or a hospital), and utility buildings without sleeping accommodations (for instance, a school building, an office building or a shopping centre). Instead of the total building height, the requirements are based on the height of the highest floor of an accommodation area with respect to the adjacent terrain at the location of the building entrance. The level of the adjacent terrain is known as the ***measurement level***. If a building can only be entered by a stairway or a ramp, the measurement level equals the height of the adjacent terrain at the bottom of this stairway or ramp. The tables below give the fire resistance requirements for new buildings, based on their function types and the height of the highest floor of an accommodation area with respect to this measurement level [15, 26]:

Table 2 | Fire resistance requirements for new housing, according to the Building Decree of 2012 [15, 26]

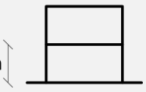
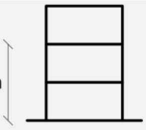
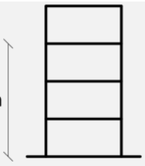
Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes	Reduced fire resistance at a permanent fire load density of ≤ 500 MJ/m^2, expressed in minutes
	60	30
≤ 7 m 	60	30
7-13 m 	90	90 (no reduction)
> 13 m 	120	120 (no reduction)

Table 3 | Fire resistance requirements for new utility buildings with sleeping accommodations, according to the Building Decree of 2012 [15, 26]

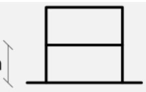
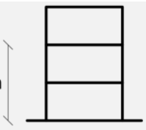
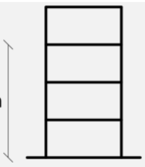
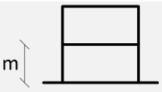
Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes	Reduced fire resistance at a permanent fire load density of ≤ 500 MJ/m^2, expressed in minutes
	60	30
≤ 5 m 	60	30
5-13 m 	90	60
> 13 m 	120	90

Table 4 | Fire resistance requirements for new utility buildings without sleeping accommodations, according to the Building Decree of 2012 [15, 26]

Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes	Reduced fire resistance at a permanent fire load density of ≤ 500 MJ/m^2, expressed in minutes
	-	-
> 5 m 	90	60

As can be seen from the tables, the fire resistance values for all new utility buildings and the new houses and apartment blocks where the highest floors of the accommodation areas are up to 7 m above the measurement levels, can be reduced by 30 minutes, if can be shown that the permanent fire load density is below or equal to a value of 500 MJ/m^2. This is based on the assumption that a structure of concrete, steel, or masonry, which most likely results in a permanent fire load density value below 500 MJ/m^2, does not contribute to the fire [8, 35]. A description of the permanent fire load density will be given in 3.2.5.5. Other ways to reduce the fire resistance requirements (such as the application of a sprinkler system) may be employed by making use of the principle of equivalence, an article of the Building Decree which will be discussed in paragraph 3.2.8.

In addition to the specific demands, it is required that a floor, stairway or ramp on or under an escape route does not fail within 30 minutes due to a fire in a sub-fire compartment in which the escape route is not situated [26]. The value of 30 minutes is based on the following principles in the Building Decree:

- The fire must be discovered and the people endangered by it, as well as the fire service, must be alerted within 15 minutes of the start of the fire.
- The people endangered by a fire must be able to flee without the fire service's assistance within 15 minutes of having been alerted.
- The fire service is present and operational within 15 minutes since the fire is reported.

Just as the principle which is used to determine for which part of the building the requirements hold, these principles are mostly based on assumptions during the twentieth century [9]. In the first half of this century, it was assumed that in places which had a properly equipped fire service, the offensive would not start later than 15 minutes after the fire had been supported. For that reason, the door of an escape room had to withstand a fire for at least 15 minutes. Later, after the Second World War, the time needed for discovering the fire was added. For the time of this event, the moment of flashover was assumed, counted with a temperature of 750 °C. A flashover is a sudden change in the steady growth of a fire. In a flashover, the temperature in the smoke layer under the ceiling of the space is so high that the heat radiated by that smoke layer ignites the inflammable objects in the whole space, including those under the smoke layer, within a very short period of time, as a result of thermal decomposition [16]. The standard fire curve (paragraph 2.4.1) showed that this temperature was reached after 15 minutes. This resulted in the three mentioned principles and a fire resistance value for escape routes of 15 + 15 = 30 minutes, which is still used nowadays.

Another principle in the Building Decree also originated in the mid-twentieth century, based on a slightly different line of reasoning. This principle is as follows [9]:

- The fire service must have the fire under control within 60 minutes of the fire starting, which implies that the fire must be prevented from spreading further. At such time, the last people endangered by the fire must have been rescued with the fire service's assistance.

The motivation behind it was that experience had shown, that fighting a fire inside a building did not always enable a positive result to be achieved, and the fire service had to completely or partially withdraw from the building to enable the fire to be completely or partly fought from the outside. It was assumed that it could be decided if this would be necessary, 30 minutes after the start of the operation. Together with the 15 minutes of discovering and alerting, and the 15 minutes of arrival of the fire fighters, this led to the 60 minutes mentioned in the principle [9]. It explains the fire resistance value of 60 minutes for utility buildings with sleeping accommodations and houses or apartment blocks, where the highest floors of the accommodation areas are up to 5 and 7 m above the measurement levels. However, the idea behind the additional 30 and 60 minutes for higher buildings and the floor heights above the measurement level is not clear. It is known that some of these values are based on several motivations, which are mentioned in the framework below, while others were determined by history. Unknown is how and in which extent these aspects have contributed to these values. There seems to be no logic in the requirements. For example, utility buildings without a sleeping accommodation have no requirement below 5 meter. But above this height, suddenly a value of 90 minutes holds. There is no scientific explanation for this leap. It would be more logical if the fire resistance values would increase as a function of the building heights, without any gaps of 30 minutes or more. Also clear explanations for the differences between utility buildings with and without sleeping accommodations are missing [8].

Several motivations which were relevant for the determination of the values of the fire resistance requirements and the related heights above the measurement levels [8]:

- The possibility to leave a building safely
- The necessary safety of emergency services (fire fighters, police, health authorities)
- The necessary safety of accidental passers
- Unacceptable social consequences of a fire (the collapse of a tall building will have a huge social impact)
- Simplicity of the regulations (nuances of fires which are not based on the standard fire curve would make the regulations complex)

3.2.5.3 Fire resistance values for alteration or renovation

For the performance level of alteration or renovation, no fire resistance values are given in tables. There has to be made use of the legally obtained level (in Dutch: “het rechtens verkregen niveau”) [26]. This level meets the requirements that were applied when the building was constructed. The level should not be lower than the requirements applicable to existing buildings and it should not exceed the level of requirements applicable to new buildings. In paragraph 3.2.7, this term will be further illustrated.

3.2.5.4 Fire resistance values for existing buildings

Lastly, the requirements for the existing buildings are mentioned. This performance level does have fire resistance values given in tables, just as the level for new buildings. These values are shown in Table 5, Table 6, and Table 7.

Table 5 | Fire resistance requirements for existing housing, according to the Building Decree of 2012 [15, 26]

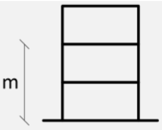
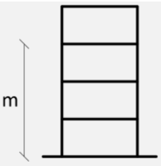
Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes
	-
7-13 m 	30
> 13 m 	60

Table 6 | Fire resistance requirements for existing utility buildings with sleeping accommodations, according to the Building Decree of 2012 [15, 26]

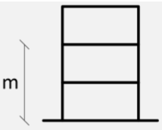
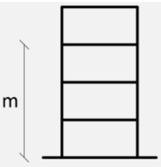
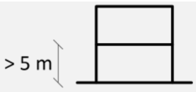
Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes
	-
5-13 m 	30
> 13 m 	60

Table 7 | Fire resistance requirements for existing utility buildings without sleeping accommodations, according to the Building Decree of 2012 [15, 26]

Highest floor of accommodation area above measurement level	Fire resistance, expressed in minutes
	-
	30

Compared to the tables of the new buildings, there are three important differences to notice. At first, all of the fire resistance values are reduced with one hour. Secondly, there are no requirements for houses or apartment blocks where the highest floors of the accommodation areas are up to 7 m above the measurement levels and utility buildings with sleeping accommodations where the highest floors of the accommodation areas are up to 5 m above the measurement levels. And thirdly, the reduction of 30 minutes for buildings with a permanent fire load density below or equal to 500 MJ/m^2, does not hold for existing buildings.

The fire resistance demand concerning a floor, stairway, or ramp on or under an escape route, amounts 20 minutes for existing buildings, in contrast to the 30 minutes which are required for new buildings [26].

These demands could also raise questions. The starting point for existing buildings according to the building Decree, is that the building structure has to be “only just save enough” [26]. In this way, measures with large financial consequences are being avoided. But the differences with the demands for new buildings are significant. Based on the principle of the required time to escape, the values of 30 minutes could be accepted. But a value of 20 minutes for the escape routes of existing buildings would suggest that these are not safe enough, based on the same principle.

3.2.5.5 Fire load density

The ***fire load density*** which is mentioned in the context of the reduction of the fire resistance requirements, is defined as “the amount of heat released per unit of floor area during the combustion of all the present combustible materials in a (considered part of the) structure”, according to the NEN 6090 [15]. Distinction is made into two types of fire load density: the permanent and the variable fire load density. The differences between these two are explained by Table 8.

Table 8 | Types of fire load density [15]

Type of fire load density	Description	Examples
Permanent	The contribution of all components that are required by the environmental permit	• Floor insulation • Wood frame • Roofing
Variable	The contribution of all other structural parts and interior of the building	• Lightweight partition walls • Inventory of a storage building • Workout machines in a sport centre

Some structural elements may be counted to both the permanent and variable fire load. In that case, the function of the structural member determines which type it should be granted to. See the following table for two examples.

Table 9 | Examples of the classification of structural elements to a type of fire load density [15]

Structural element	Permanent	Variable
Floor covering	Floor covering needed to meet a demand of the Building Decree (like reverberation time)	Floor covering which does not need to fulfil a requirement of the Building Decree
Partition walls	Partition wall between two accommodation areas (located in two different fire compartments for example)	Partition wall between two accommodation rooms in the same accommodation area

For the reduction of the fire resistance demands, only the permanent fire load density is relevant [15].

3.2.6 Differences between the fire resistance requirements of the actual and former Building Decrees

As was mentioned in paragraph 3.2.1, the model-building regulation of 1965 was the first Dutch document with uniform fire resistance demands [8]. The intention was that the municipalities at least met these minimum requirements in their own municipal building regulations. Values of 20, 30, 60, and 120 minutes were assigned based on the building type (public buildings or housing), the function of the compartments and the location of the walls and floors. There was hardly made any distinction between different heights; only the public buildings where divided in groups of buildings with the highest floor (except attics) above or under a height of 12.5 m with respect to the ground level. According to Reint Sagel, retired structural engineer, this height was based on the maximum reach of a ladder truck; above this height (which corresponds to four floors), elevators should be used. Later on, in the first Building Decree, this value was replaced by 13 m. Due to the lack of decimals, this value was not applied very strictly. After all, a value of 13.49 m is still rounded up to 13 m.

The fire resistance requirements of the first model-building regulation were only updated in the 14th supplement, which was introduced in 1977 [36]. These adaptions were related to the requirements of the houses and the apartment blocks. First of all, the apartment blocks were divided in low and high ones. Besides, distinction was made between "fire resistance" and "fire resistance with respect to collapse", which can be seen as a distinction between criterion R and the criteria E and I. Finally, the fire resistance requirements could be reduced if the permanent fire load density was below or equal to a value of 100 MJ/m^2, which had to be calculated according to the NEN 3891 ("Fire Safety of Buildings"), which came out in 1971. All these requirements remained unchanged in all the subsequent model-building regulations until 1992.

The fire resistance requirements of the Building Decree of 1992 were based on the model-building regulation of 1992 [36]. These requirements were more concise than the current requirements. The content of the decree was divided in several chapters, each about another function type. Only in two of these chapters, chapter II (Technical requirements regarding the construction of houses and apartment blocks) and chapter VI (General technical requirements concerning the construction of utility buildings), some specific fire resistance values were given. In the version of the Building Decree of 1992, which was valid from 01-01-2000 [37], the values in chapter II and VI were as follows:

Table 10 | Fire resistance requirements for housing, according to chapter II of the Building Decree of 1992 [37]

Main loadbearing structure	Fire resistance, expressed in minutes
Building structure of which the collapse leads to the unusability of an escape opportunity	30
Main loadbearing structure of a house or apartment block which is not situated in the house or in the apartment block	60
Main loadbearing structure of a house or apartment block where the highest floor of an accommodation area is up to 13 m above the measurement level	90
Main loadbearing structure of a house or apartment block where the highest floor of an accommodation area is more than 13 m above the measurement level	120

For public buildings, these values were mentioned in chapter VI:

Table 11 | Fire resistance requirements for utility buildings, according to chapter VI of the Building Decree of 1992 [37]

Main loadbearing structure	Fire resistance, expressed in minutes
Building structure of which the collapse leads to the unusability of an escape opportunity	30
Main loadbearing structure of a building which also has sleeping accommodations, where the highest floor of an accommodation area is up to 5 m above the measurement level	60
Main loadbearing structure of a building where the highest floor of an accommodation area is between 5 and 13 m above the measurement level	90
Main loadbearing structure of a building where the highest floor of an accommodation area is more than 13 m above the measurement level	120

There were no clear demands for utility buildings with sleeping accommodations, where the highest floors of the accommodation areas were more than 5 m above the measurement levels, just as there were no specific demands for existing buildings or buildings that were to be altered or renovated. All of the values in the table were allowed to be reduced by 30 minutes, just as the current values for all new utility buildings and the new houses or apartment blocks with a height up to 7 m can be reduced now. But before the adaption of 01-07-1997, the permanent fire load density had to be below or equal to a value of 100 MJ/m^2, instead of the value of 500 MJ/m^2 nowadays [38]. The reason why this was changed is because of the contribution to the permanent fire load density by wooden windows, doors, roofs, window frames, and interior walls as boundaries of rooms. In the regulations before the introduction of the Building Decree (the standard NEN 3891), these components were not considered for the determination of the permanent fire load density. But according to NEN 6090, these structural elements were not allowed to be ignored anymore. As a result, the reduction of the fire resistance assuming a value of 100 MJ/m^2, became practically infeasible. For this reason, a new value had to be proposed. By comparing the NEN 3891 with the new NEN 6090, it turned out that a building which had no permanent fire load according to the NEN 3891, had a permanent fire load density of 500 MJ/m^2 according to the NEN 6090. That is why a new value of 500 MJ/m^2 was introduced [39, 40].

In the Building Decree of 2003, a lot of things had changed with respect to the version of 1992 [41, 42]. First of all, the content was now divided in certain topics instead of function types. Secondly, a new maximum floor height of the accommodation area of 7 m above the measurement level was introduced for new housing. For new utility buildings, the values of 60 and 120 minutes became valid for utility buildings with sleeping accommodations, instead of utility buildings without these functions. All new utility buildings without sleeping accommodations where the highest floors of the accommodation areas were more than 5 m above the measurement levels, had to withstand a fire for at least 90 minutes. Finally, the fire resistance values of houses and apartment blocks where the highest floors of the accommodation areas were more than 7 m above the measurement levels were not allowed to be reduced anymore.

The new maximum floor height of 7 meter for housing was introduced to prevent less safe situations for fire fighters [42]. The reasons of the other additions and changes concerning the requirements for the new buildings, however, are not clear. As was mentioned in paragraph 3.2.5.2, explanations for the differences between utility buildings with and without sleeping accommodations are missing. Additionally, it is unknown why the reduction principle concerning the permanent fire load density did not hold for all the housing anymore. Since this principle is no longer valid for houses and apartment blocks where the highest floors of the accommodation areas are more than 7 m above the measurement levels, more expensive solutions need to be found to fulfil the fire resistance requirements. Due to the fact that this adaption has significant consequences, it is strange that this adaption is never clearly explained [43].

Beside these adaptions of demands for new buildings, specific fire resistance values were introduced for existing buildings. All of the values (both for new and for existing buildings) correspond to the current requirements. The only differences between the actual Building Decree and the Building Decree of 2003, is the mention of buildings that are to be altered or renovated, the references to other building standards (which will be discussed in chapter 6), and some changes in the definitions, including the disappearance of the term "main loadbearing structure", which is mentioned in Table 10 and Table 11. The term mainly referred to the part of the structure which is described earlier in paragraph 3.2.5.1, but in a slightly different way. This will be discussed below.

3.2.6.1 Former definition of the main loadbearing structure

The Building Decree of 1992 and the Building Decree of 2003 both referred to the standard NEN 6702 for the definition of the term ***main loadbearing structure***. This term was defined as "*a part of the building structure of which the collapse leads to the failure of structural parts which are not located in the direct vicinity of the collapsed part*" [44]. Strict application of this definition could lead to misinterpretations in practice. For example, imagine a composite slab with primary and secondary girders (situation A in Figure 12). According to the strict application, the secondary girder lies in the direct vicinity of the primary girder, while the slab lies in the indirect vicinity of the primary girder. In this way, structural engineers sometimes counted the primary girders to the main structure, because the collapse of these girders would not only lead to the collapse of the secondary girders, but also to the collapse of the slab in the indirect area. With this theory, a slab which is only supported by primary girders (situation B in Figure 12) would not have a main load bearing structure, although the fire resistance of this situation must be equal to that of situation A.

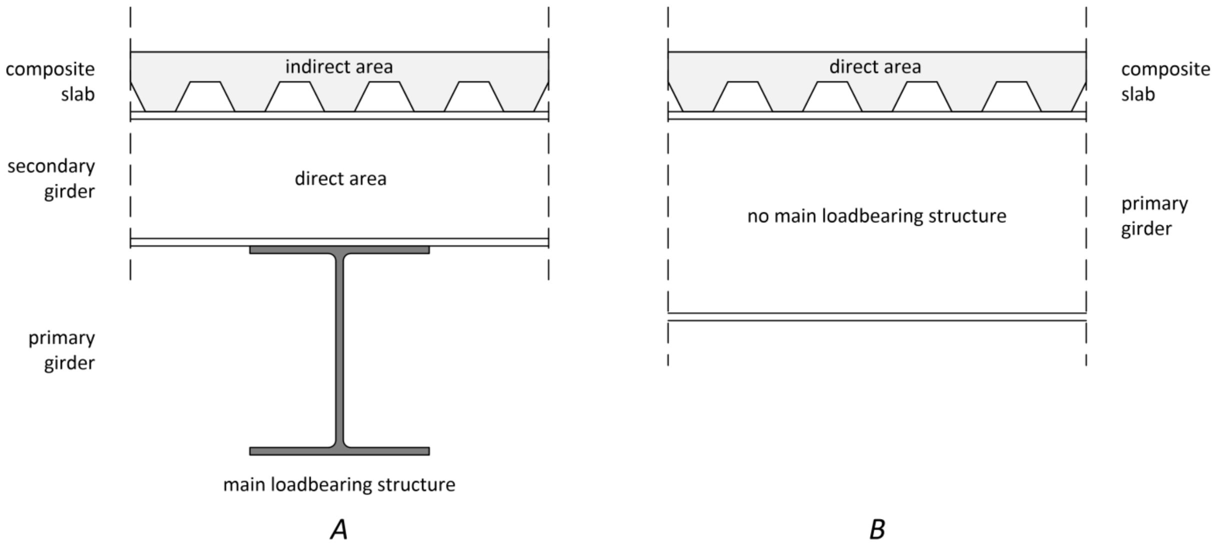


Figure 12 | Misinterpretation of the definition of the main loadbearing structure

The term was clarified in 2005 by splitting it into "main loadbearing structure under fire conditions" and "main loadbearing structure under other conditions" (like a collision for example). The ***main loadbearing structure under fire conditions*** was defined as [34]:

"*a part of the building structure situated in or adjacent to a fire area with fire compartments and sub-fire compartments, of which the collapse results in the collapse of a building structure which*

- *a. is not located in the same fire compartment as the considered fire area (this does not hold for residential functions);*
- *b. takes care of the maintenance of non-directly adjacent sub-fire compartments and other non-directly adjacent areas, in case the fire area is a sub-fire compartment or a part of a sub-fire compartment;*
- *c. takes care of the maintenance of premises that are non-directly adjacent to the fire area, but are located in the (sub-)fire compartment, in case the fire area is part of a (sub-)fire compartment that contains more than three floors; the most unfavourable location of the fire area in combination with three floors may be assumed in this situation.*"

The Building Decree of 2012 does no longer refer to the standard NEN 6072, but a strongly simplified version of the standard text is included in the text of the Building Decree [26]. According to Article 2.3 (1) of the current Building Decree, points a and b of the former definition of the main loadbearing structure under fire conditions are worked out in the same way, although before the introduction of the Building Decree of 2012, the regulations could lead to unintended effects (which usually had to do with an incorrect amount of coated beams). Point c no longer applies. Due to the strong simplification, the method may be more user-friendly nowadays, but it could accidentally lead to stronger requirements. The term ***building structure***, which replaces the term main loadbearing structure under fire conditions, includes any part of a structure which is intended to bear loads. However, for example, this means that a balcony is a building structure as well, which leads to the fact that a detached house may collapse as long as the part with the balcony maintains for a prescribed period, because the balcony is not located in a fire compartment. Therefore, it is recommended to consider to apply no requirements for some building structures [45].

3.2.7 Performance levels

In paragraph 3.2.5, it was mentioned that the performance requirements are divided in three performance levels: new buildings, alteration or renovation, and existing buildings. To apply these demands correctly, it is important to understand what these levels mean and how they are defined. This will be explained in this paragraph.

An existing building at least has to satisfy the requirements of the Building Decree for the level of existing buildings. When a building will be adapted and there will be “built”, the structure in principle needs to fulfil the demands of new buildings, according to Article 1b (1) of the Housing Act [46]. Or in fact, not the whole structure, but according to Article 4 of the Housing Act, only the parts that are being (re-)constructed. However, Article 1.12 of the Building Decree states that a rebuilt structure needs to meet the requirements for new buildings, *unless* the Building Decree mentions otherwise in particular sections. Indeed in different sections, an article is included in which specific demands for alterations and renovations are mentioned. These articles were introduced for the first time in the Building Decree of 2012 [47]. Before that time, there was a possibility for municipalities to grant exemptions to severe demands, but this authority disappeared because of the fact that the government was not satisfied about the way the authority was used (more information can be found in paragraph 6.3.1).

So since 2012, when the mentioned authority disappeared, a specific requirement level for buildings which would be altered or renovated was implemented, formed by these articles [47]. The NEN 8700 states that for buildings younger than 15 years, this level is only used for assessments; alterations or renovations take place in accordance with the requirements which hold for new buildings (see also paragraph 6.3.2.1). For older buildings, this level is used for alterations and renovations, while the level of existing buildings is used for assessments here. Deviation to this requirement level is only allowed for structures which are older than 15 years, and a good motivation which explains why the demands could not be met, is needed.

An interesting question is what the verb “build” really means in this context. In the Housing Act, it is defined as “*placing, entirely or partly establishing, renewing, adapting or enlarging*” a structure [46]. These operations require a license, as long as they need to meet certain requirements. A simple example, given by ir. Meuldijk (fire safety consultant at IOB), is a partition wall. If this wall does not need to meet any requirements, it does not need a license as well. These license-free situations are mentioned in the Decree of Uniform Building Regulations. But when this wall is used to divide an area in different fire compartments, for example, it has to fulfil certain fire safety demands, which means it also requires a license.

What does this mean for changes in the function of buildings? It means that the function change itself is not covered by “building a structure”, and if this change does not involve structural adjustments (or only license-free adjustments), the change of function does not have to satisfy the requirements for new buildings or altered or renovated buildings, but only the minimum requirements for existing buildings [47]. This level of safety is also known as “the level of disapproval”.

A structure should be disapproved when the requirements of this level are not met, taking into account:

- a remaining lifetime of 1 year;
- loads associated with a reference period of 15 years*;
- the strength corresponding to the strength of the actual structure.

A building only needs to be disapproved when the safety is really at stake. This particularly applies to acute situations which require short-term measures. Therefore, it is obvious to align a period which is used to evaluate the minimum security to a short-term situation. For short-term situations, a design life of 1 year is adjusted according to the NEN-EN 1990 [48]. This value is also used for existing buildings, where it is defined as “remaining lifetime” instead of design life, which explains the value of 1 year in the enumeration above [49]. More information can be found in paragraph 6.3.3.4.

So the level of disapproval is a minimum safety level, meant to prevent drastic measures after the occurrence of unforeseen difficult events, for incidents of which the fulfilment of a higher safety level leads to extremely high costs. Although a function change of a building in general only needs to fulfil the requirements of this level, it is strongly advised to stay above this limit, with respect to the principle of proportionality. This principle means that the demands of a structure must correspond to the intended function of a building. In the NEN 8700, a building standard which is relevant for the level of alteration or renovation, and which will be discussed in paragraph 6.3, it is clearly meant to keep this principle in mind [50]. For this reason, it is inappropriate to use requirements, which were originally meant for incidents, for the transformation of a building.

Anyway, it is relatively rare that nothing needs to be rebuilt or adapted when the function of a building is changed. And if nothing has to be changed in the beginning, it would not directly mean that it stays that way. When, for example, a school is used as a residential area, without any (structural) changes being done, it is allowed to live there, without taking into account further requirements of the Building Decree. However, the structure still needs to satisfy the minimum demands of existing structures for the new function (in this example, the residential area). When these requirements are not met, the structure needs to be adapted, which directly means that it needs to fulfil the requirements for alteration or renovation, because structural changes need to be made [51]. This leads to the fact that function changes in most cases need to fulfil the requirements

* An exception applies to consequence class 1A (CC1A), where 1 year is allowed (see paragraph 6.3.3.5)

of buildings that are to be altered or renovated. In case of the fire resistance demands in Section 2.2 of the Building Decree, this performance level constantly refers to the legally obtained level, which was mentioned earlier in paragraph 3.2.5. This term will be further illustrated below.

3.2.7.1 Legally obtained level

In the Building Decree, the ***legally obtained level*** is defined as "*the level which is the result of the application of the relevant, on that moment applicable technical requirements, and which is not below the level of requirements for existing buildings, and not above the level of requirements for new buildings*" [26].

The first part of this definition describes the actual quality level, prior to the alteration or renovation. This quality level is obtained by construction and reconstruction by legal means, which implies the use of the requirements which were valid in that time. The second part states that the result of the alteration or renovation activities must not come below the level of existing buildings and the upper limit lies at the level of new buildings. So practically, a building satisfies the legally obtained level if:

- the quality level of a structural part would not be reduced by the alteration or renovation, and
- the result does not come below the minimum level of existing buildings (or a specific demand in the Building Decree which holds as a minimum*) [51].

When it comes to a new structural part which will be added to an existing building, there is no actual quality level, so the requirements for existing buildings hold. For an existing part, the actual quality level can be obtained by investigating the structure, or by looking into the requirements which were valid at that time. And when there were no requirements yet, it is possible to look into the license of the building, which often contains the demands which were apprehended [51, 52]. It should however be aware that the current quality level of an existing building is usually lower than the quality level of the original design, due to the deterioration of the concrete over the years (see paragraph 4.6) in combination with deviations following from an inaccurate execution.

According to fire safety consultant Meuldijk, before 1992, building licenses were not drawn up as properly as they are now. In combination with the lack of requirements in that period, it could occur that there is a deficiency of information to determine a legally obtained level. In this case, there is often made use of the principle that when the new function has less severe demands than the former one (for example, in case of transforming an office building with sleeping accommodations into a hotel), which can be fulfilled at the level of new buildings, it goes without saying that the level of alteration or renovation will be satisfied as well. But it could also occur that the new function requires tougher demands, for example in case of transforming a hotel in a cell block. For these situations, this principle cannot be used. When there is not enough information for a legally obtained level and this principle cannot be used, the performance level of existing buildings has to be used. It is up to the building contractor to ask for tougher demands.

* In Section 2.2 of the Building Decree, no such specific demands are given, which means that the level of existing buildings holds as the minimum level in all cases of fire resistance.

Finally, the authorities need to verify the fire resistance of a building design or assessment. If the design or assessment is based on the performance level of existing buildings, the authorities may summon the owner of an existing building to take extra measures, in addition to the requirements for existing buildings in the Building Decree, based on section 13 of the Housing Act [9, 46, 53]. In these situations, they are advised by the fire brigade. The level of these measures should not exceed the legally obtained level or – in case there is not enough information to form a legally obtained level – the performance level of new buildings. When the authorities ask for additional measures, a specific motivation is required.

3.2.8 Principle of equivalence

If a building cannot meet the performance requirements or if there are less expensive solutions to ensure the fire safety which do not have any requirements mentioned in the Building Decree, an appeal can be made on the equivalence principle, which is included in the Building Decree [54, 55]. The applicant must demonstrate that there is at least equivalent safety as envisaged by the official regulations. An example of such a solution is the use of sprinkler systems. Sprinkler systems are often applied. They control the fire at an early stage and extinguish it in many cases. The fire spread and fire damage stays limited. The chance of a fully-developed fire is very small due to these systems. This can lead to a reduction of the fire resistance requirements (in this case not only the resistance with respect to collapse, but mainly the resistance concerning the spread of the fire), if can be shown that the safety at least equals the safety that would be reached by the official fire resistance requirements.

At the time of the introduction of the Building Decree of 1992, equivalence was seen as the equivalence to one single performance requirement, and the equivalent solution had to be sought in a structural measure, a technical measure, or both [9]. Later on, the equivalent solution could also meet the functional requirement. The Building Decree of 2012 offers even more opportunities to find an equivalent solution [26]. Functional requirements can give an indication of a certain section of the Building Decree, but not in terms of an integrated approach. Besides, functional requirements can be interdependent. For example, a measure to reduce the development of fire and smoke also influences the escape safety. Because of these reasons, an equivalent solution does not need to be based on functional requirements, according to the current Building Decree. For solutions, the interaction and exchange of structural, installation-technical, and organizational safety measures is possible. It should be noted that these solutions are adapted to specific buildings, which means that not every solution is possible for every building.

Although the above may suggest that applying the principle of equivalence has become easier, this is not the case, according to fire safety consultant Meuldijk. He states that because of the huge amount of possibilities, the motivation for certain measures and the deviations from the current requirements, has to be much clearer, while in the past, it only was required to show that a certain performance requirement was met. Returning to the example of the sprinkler systems, the consequences and probabilities of failure of the systems have to be fully investigated based on a comprehensive risk analysis. The question whether the proposed solution is sufficient for safe escape routes and safe situations for the fire brigade, needs to be answered. Because there are no set rules, this will be done by the municipality, based on the opinion of the fire brigade. However, research by the Dutch Organisation of Fire Safety (in Dutch: Nederlandse Organisatie voor Brandveiligheid, abbreviated as NOVB) has shown that the equivalence is often interpreted differently. Therefore, early consultation with the municipality and the fire brigade is recommended [54, 55].

Interesting to note is that, as will be discussed in paragraph 6.3.1, situations where a calculative approach is used to determine the fire resistance concerning the performance level of existing buildings, also fall under the principle of equivalence. This means that the authorities are not only allowed to impose higher demands for existing buildings according to section 13 of the housing Act, but also according to the principle of equivalence.

3.3 Degree of attention

The lack of background information concerning the fire resistance requirements which are mentioned multiple times in this chapter, seems to restrict the attention to fire resistance in practice. Designers and assessors mainly limit themselves to the establishment of the performance requirements. Research has shown that engineers do not have a clear and uniform insight to deal with aspects concerning the strength of a structure in case of a fire [56]. The lack of consideration of these aspects in the education of engineers also plays a role. Very few engineers are able to draft and asses normative fire scenarios, which is why the determination of the fire resistance will mainly continue to be based on the standard fire curve.

Secondly, structural engineers are often not placed in a position in which they are able to verify all the aspects and respond to them, even though it is often to the structural engineers to convince other parties of different possibilities. Besides, the building contractors can ask for tougher demands as mentioned in paragraph 3.2.7.1, but this is rarely the case [56]. They generally consider the price of a bigger importance, due to the (in their opinion) small likelihood of the occurrence of a fire. And when they do ask for extra measures to increase the fire resistance, these measures are frequently applied to the whole structure, without performing further design or calculation work.

The view of the authorities concerning the fire resistance is often limited as well [56]. This means that the authorities are not always able to fully verify the calculations of the engineers. Moreover, the Housing Act offers several ways to perform an acceptability review of the design or assessment, without the need to check all the calculations of the engineers. The decisions of the authorities are influenced by the advices of the fire brigade.

However, the fire brigade is highly risk-averse. This can be an explanation for the limited level of knowledge at the fire brigade concerning the behaviour of structures. Firefighters are not trained to pay attention to the reactions of a building structure during a fire. The only information that is provided to the firefighters in terms of the fire resistance, concerns specific signals that could lead to a collapse [56].

So, it appears that the attention to the fire resistance in practice is very limited. Besides, the fire resistance is considered from very different perspectives by structural engineers, building contractors, authorities, and the fire brigade. This complicates the collaboration and consultations between these different parties. However, this complication would not necessarily have to mean that the fire resistance of a building is insufficient. It could also lead to a fire resistance of a building structure which is much higher than required. Because existing buildings has much less structural possibilities than new buildings, the adjustments that are needed to reach this level of fire resistance could be very expensive. In order to limit the costs, it is effective to apply a safety level as low as possible. For this reason, a correct and realistic determination of the fire resistance could be of greater importance for existing buildings than for new buildings.

4 TEMPERATURE EFFECTS ON REINFORCED CONCRETE

Concrete does not burn and has a good resistance to fire, as was mentioned earlier in paragraph 1.2. These two advantages are generally known. But there are more properties of concrete which are advantageous in case of a fire. All these natural properties together ensure that a fire in a concrete structure is easier to extinguish, because the material will resist the fire for a longer period of time than timber or steel (without protection). It also reduces the fire damage and the risk of environmental pollution, by applying a separation into compartments, which will stop the spread of the fire. These natural properties are as follows [57]:

- concrete does not burn and does not increase the fire load;
- concrete has a high fire resistance;
- concrete does not lead to molten material drops, which can spread the fire;
- concrete does not produce smoke or toxic gasses;
- concrete is a (heat)insulation material;
- concrete protects other materials against fire.

Nevertheless, it is inevitable that a concrete structure will be damaged by a fire. The degree of the damage depends on the severity of the fire and the height of the temperature. Common types of damage are [58]:

- reduction of the compressive and tensile strength of concrete;
- reduction in the modulus of elasticity;
- micro-cracking within the concrete microstructure;
- spalling;
- loss of bond between concrete and steel;
- loss of residual strength of steel reinforcement.

These types of damage will be discussed in this chapter.

4.1 Degradation of the concrete due to high temperatures

Concrete mainly exists of cement stone and aggregates. The reduction of the compressive strength, the tensile strength, and the modulus of elasticity, mainly occurs due to the formation of internal cracks in the cement stone. At very high temperatures, however, cracks could also occur at the interface between the aggregates and the cement stone. These crack formations are the consequences of several chemical and physical reactions of the cement stone and the aggregates, caused by temperature increases [59]. The reactions will be briefly explained below (and are summarized in Figure 13).

4.1.1 Chemical transformations of cement stone and aggregates

Below a temperature of 100 °C, the cement stone will slightly expand, while the free water in the capillary pores evaporates [57, 59]. Free water is the water in the concrete, which is not chemically bounded. The exposure of concrete to this temperature is generally harmless. Above this temperature, the cement stone will shrink noticeably, because besides the free water, the chemically bounded water evaporates as well.

When the temperature rises above 300 °C, the calcium-silicate-hydrate (C-S-H) gel dissolves [57, 59]. This gel is mainly responsible for the strength of the concrete [60]. In addition to this dissolution, the iron-containing compounds in the cement stone oxidizes. The cement stone shrinks, while the aggregates expand.

At a temperature of 400 °C, calcium hydroxide ($Ca(OH)_2$) starts to dissolve into lime (CaO) and water (H_2O). This chemical reaction is called “dehydration” [57, 59].

If the concrete reaches a temperature of 575 °C, the siliceous aggregates undergoes a crystalline conversion of quartz α into quartz β [57, 59]. This is coupled with a sudden increase of their volume with about 5,7%. This increase may cause damage to the concrete. Examples of siliceous aggregates are river gravel, sandstone and quartzite rocks. Limy aggregates, however, are stable up to 700 °C. Dolomite and limestone are two examples of such aggregates.

Above 700 °C, the “decarbonisation” of limestone takes place [57, 59]. This is a chemical reaction, in which the limestone ($CaCO_3$) is decomposed into calcium oxide (CaO, also known as “quick lime”) and carbon dioxide (CO_2).

If the concrete is exposed to these temperatures and cools down afterwards, the quick lime which is formed by the dehydration and the decarbonisation, combines with the ambient humidity, and forms calcium hydroxide [57, 59]. This reaction is associated with a significant volume increase of 44%, which causes the concrete to disintegrate.

So in short, the really harmful chemical reactions start at a temperature of 300 °C. This is why after a fire, the concrete zones which are exposed to temperatures of 300 °C or higher, need to be eliminated and replaced. In addition to all the reactions mentioned above, the cement stone could also melt. This only happens, however, when an extremely high temperature of at least 1100 °C is reached [57, 59].

4.1.2 Physical interactions

The cement stone shrinks and the aggregates expand at a rising temperature, as was already mentioned in the previous paragraph. The degree to which this occurs, depends on the composition of the cement stone and the type of aggregates. Because of the adhesion between the cement stone and the aggregates, the shrinkage of the cement stone and the expansion of the aggregates induces tensile stresses in the concrete. These tensile stresses lead to the formation of cracks [60].

However, the cement stone can adapt itself to the large differences in the thermal deformations. This phenomenon is known as “***load induced thermal strain*** (***LITS***)” or “***transient thermal strain*** (***TTS***)”, and involves a mainly irreversible, largely time-independent strain-component, that develops if concrete is heated while being loaded [59, 61]. It can be seen as a form of relaxation of the concrete under imposed deformations, which only occurs at the first temperature rise. Because the strain is irreversible, the strain is not recovered during the cooling down. This means that during the cooling down (and at later temperature rises), due to the absence of the LITS or TTS, internal stresses are developed, which lead to cracks between the cement stone and the aggregates. These cracks affect the mechanical properties of the concrete.

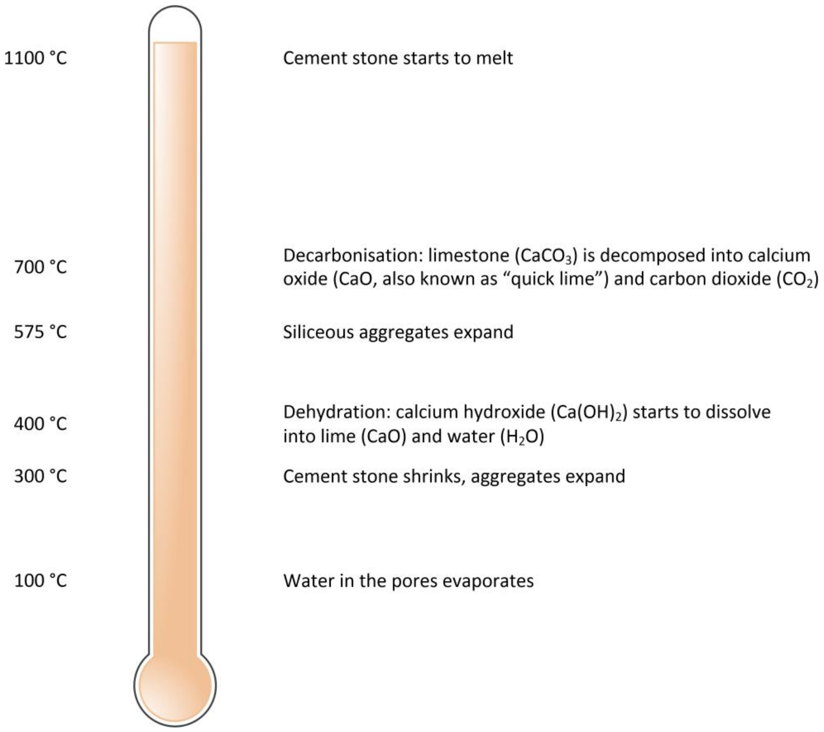


Figure 13 | Overview of the most important (chemical) transformations of cement stone and aggregates [57]

4.2 Temperature distribution

In unprotected steel structures, the temperature over the entire cross-section of the profiles rises very quickly. At a severe fire, a critical limit at which the material strength decreases by half, will be reached within 15 minutes. In concrete structures, this is not the case. The heating of the concrete takes place much slower, and in a heterogeneous manner along the cross-section. Dehydration and decarbonisation (as well as the crystalline conversion of siliceous aggregates), mentioned in paragraph 4.1.1, are endothermic reactions. This means that these reactions absorb heat energy. Due to these absorptions, the outer layer of the concrete works as an insulating layer and a heat shield, causing a heterogeneous temperature distribution (Figure 14). Because of this distribution, the adverse effects of the heat, of which several were mentioned in the former two paragraphs, generally only occurs in the outer layer of the concrete, which has a thickness of 30 to 50 mm [57, 59, 62].

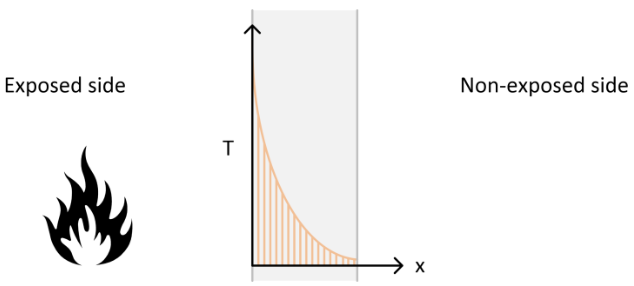


Figure 14 | Rough schematization of the temperature distribution

4.3 Spalling

In approximately 80% of the reported fire incidents in concrete buildings, different parts of the concrete structure broke off from the cross-section, when the concrete surface was exposed to a fire [63]. This type of damage can occur rapidly and is known as "***spalling***". Spalling can occur due to two different causes, which are [64, 65]:

- thermal stresses in the concrete;
- high pore pressures in the concrete.

As is partly explained earlier, concrete will expand when it is exposed to temperature rises. If the expansions are restricted, they will lead to a high level of compression near the surface which is exposed to the fire (see also paragraph 4.5). These thermal stresses can be so high, that they cause compressive failure in the most heated part of the concrete.

The water in the concrete expands much more than the concrete itself. Due to the expansion of water (evaporation), a high pressure in the pores of the most heated part of the concrete cross-section will build up. Because of this high pressure, the water in the pores will flow through the concrete in the direction of the lower pressure. The lower pressure can be found at the fire exposed surface, but also deeper in the concrete where the temperature is still low. In this way, a part of the water is pushed out of the concrete, while the other part is pushed deeper into the concrete, which leads to a higher saturation of the pores in the cold concrete. When the high temperature penetrates deeper into the concrete during the fire, these pores will be heated as well. Due to the high saturation and temperature, the pressure in these pores increases even more. If the concrete has sufficient permeability, this does not lead to any problems. But if the water cannot be transported easily enough in both directions, which means it cannot keep up with the speed of the temperature rise, the pressure will keep increasing and finally causes an explosion of the concrete, which blows away the most heated part from the concrete area with the highest pore pressure [64, 65].

These two causes jointly contribute to the spalling of concrete in practice. Depending on the moisture content in the concrete, the porosity, the permeability, the stress conditions, the type of aggregate, the dimensions of the element, the rate of the temperature rise, and the strength of the concrete, the level of damage and energy release of spalling may vary (these factors will be discussed later in this section). Therefore, different forms are recognised, which are set out into four categories [64, 65].

4.3.1 Types of spalling

4.3.1.1 Aggregate spalling

Aggregate spalling (Figure 15a) is also known as aggregate splitting or flaking, and involves the bursting or splitting of aggregates at the heated concrete surface [65]. This form of spalling generally leaves coin-sized craters on the surface of the concrete, with a maximum depth of 5 to 10 mm. It neither removes huge amounts of the concrete cross-section, nor exposes any reinforcement. Consequently, aggregate spalling has practically no effect on the fire resistance of concrete elements.

4.3.1.2 Corner spalling

If a structural element like a beam or column is exposed to a fire on four sides, the temperature of the concrete surface rises quickly, and the concrete wants to expand. This expansion is restricted by the core of the element, which remains cold. This leads to tension at the core and compression at the outside of the column or beam. These thermal stresses are added to the stresses which follow from the applied loads. As a result, the outer layer of the concrete, of which the resistance decreases during the temperature increase, is subjected to very high stresses. These stresses, in combination with the loss of bond between concrete and steel and the expansion of the reinforcement, cause the concrete which covers the reinforcement at the corners of the element, to tear off (Figure 15b). This type of spalling is very serious, because it induces a reduction of the cross-section and exposure of the reinforcement bars [59, 65].

4.3.1.3 Surface spalling

Surface spalling (Figure 15c) is a violent form of spalling where huge parts of concrete layers are dislodged, due to the pore pressures and thermal stresses [64, 65]. The impacts of surface spalling are made even more severe by the fact that it occurs progressively (this is why it is also known as “progressive spalling”). If one layer of concrete spalls off, a new concrete surface is exposed to the fire and the process starts all over again. In this way, the damaged area may extend several square meters and reach large depths, exposing huge amounts of reinforcement and reducing the cross-section intensively.

4.3.1.4 Explosive spalling

As the name suggests, explosive spalling (Figure 15d) is the most violent form of spalling [64, 65]. It occurs in concrete elements which are exposed to a fire from multiple sides. This means that the high temperature will penetrate the cross-section from multiple sides as well. Hereby, the water in the pores is pushed deeper in the concrete from multiple directions. The zone with the high water saturation will reach the centre of the cross-section at a certain time. At that moment, the water cannot be pushed any further away from the exposed surfaces, which results in a very high water pressure in the middle of the cross-section. This pressure can lead to an explosion that destroys the total cross-section at once, without any preceding warning signs.

a *b* *c* *d*

Figure 15 | Four different categories of concrete spalling, from left to right: aggregate spalling, corner spalling, surface spalling, and explosive spalling [65]

4.3.2 Factors that influence spalling

At the beginning of this section, it was already explained that different types of spalling mainly occur due to thermal stresses and high pore pressures in the concrete, depending on the moisture content in the concrete, the porosity, the permeability, the stress conditions, the type of aggregate, the dimensions of the element, the rate of the temperature rise, and the strength of the concrete. In this paragraph, these factors will be globally discussed one by one.

4.3.2.1 Moisture content

If a concrete has a high moisture content, it is more likely to spall since high pore pressures caused by a moisture clog form one of the main causes of spalling [66]. The EN 1992-1-2 indicates that below a moisture content of 3% by weight, spalling will not occur [67]. However, it is important to note that very dense high-strength concrete (HSC) has a higher rate of strength loss than normal concrete at temperatures up to 400 °C, which is often caused by explosive spalling with much lower moisture contents. High-strength concrete contains additives such as silica fume and water reducing admixtures, which result in an increased compressive strength in the range of 60 to 120 N/mm^2. However, this also leads to a smaller free-pore volume, so the pores become filled with high-pressure water vapour faster than in normal weight concrete, which makes high-strength concrete more susceptible to spalling [66]. This means that the limit of 3% is not valid for all situations.

4.3.2.2 Porosity and permeability

A high ***permeability*** caused by a more ***porous*** concrete, affects the rate of the vapour release to a large extent [68, 69]. This reduces the build-up of the vapour pressure within a concrete section. Concrete with a high permeability is therefore very unlikely to exhibit any symptoms of spalling. A disadvantage though, is that a porous concrete will give a poor performance with respect to durability.

4.3.2.3 Stress conditions

High (thermal) ***stress conditions*** have already been mentioned as a main cause of spalling. From several fire tests and fire observations, it has been noted that spalling is more severe in parts of the concrete which are under compression [65]. This could be partly explained by the fact that in these concrete parts, cracks could not open up to release any internal pressures. However, one most not directly think that spalling could not occur in sagged concrete parts with cracks, because these cracks do not have to be continuous. This means that pressure could still be built in cracked areas.

4.3.2.4 Type of aggregate

Concerning aggregate types, the experimental data could sometimes be inconsistent. However, it could be generally noted that the aggregate most likely to cause spalling is siliceous aggregate, while limestone produces less spalling and lightweight concrete the least [68, 69]. This can probably be related to the porosity of the aggregates; siliceous aggregate is rather well impermeable in contrast to the others. Another reason could be the amount of thermal expansion of these aggregates. However, limestone and lightweight aggregates may give problems, because the pore structure of the aggregates provides enough space for the storage of free water, especially in young concretes.

4.3.2.5 Dimensions of the concrete element

Experience has shown that sharp profiles will produce more spalling than rounded or chamfered edges and that the more faces of a member are exposed to fire, the more likely spalling is to occur [68, 69]. Furthermore, spalling is severe in thin concrete sections, because the depth of the spalling contains a greater proportion of the section dimensions and due to the fact that there is a smaller amount of cool area for the moisture to migrate to [65]. However, thick sections may also encounter problems, because high concrete covers are likely to produce greater amounts of spalling. For this reason, the Eurocode places restrictions in case high concrete covers are needed at high fire resistance periods in order to keep the temperature in the reinforcement steel low [67]. These restrictions concern the placement of a light mesh at the surface of the concrete cover in case the axis distance exceeds 70 mm, in order to retain the cover. The mesh size should not be bigger than 100 mm and the bar diameter not smaller than 4 mm. This supplementary reinforcement also makes the concrete easier to repair after a fire. Unfortunately, this type of reinforcement is difficult to place, especially in thin sections such as ribbed floors.

4.3.2.6 Rate of the temperature rise

High heating rates give the pore pressures less chance to dissipate to the relatively cool internal regions of a concrete element. This is why the heating rate significantly influences the occurrence of spalling. However, the moment of spalling occurs in a certain temperature interval, which is independent on the heating rate [68, 69].

4.3.2.7 Strength of the concrete

As said before, high strength concretes are more likely to spall, probably due to the low permeability. In combination with the spalling consequences of a high concrete cover, one should not simply think that a concrete element with a high concrete strength and a high concrete cover automatically has a good fire resistance. This element has a bigger risk of spalling [65, 68, 69]. So in terms of spalling, concrete of a poor quality, in fact, has a relatively good quality, because it is much less susceptible to spalling than high strength concrete.

4.4 Temperature effects on steel reinforcement

Steel reinforcement forms the weak link in concrete structures in case of a fire. Steel starts losing its strength when the temperature is above 300 °C. At 750 °C, the tensile strength is almost reduced to zero [58]. However, steel can fully recovers its yield strength when it cools down from temperatures of up to 450 °C in case of cold worked steel and up to 600 °C for hot rolled steel. But when the temperatures are higher, the loss in the yield strength is permanent. The modulus of elasticity is also significantly reduced at these elevated temperatures. The temperature of the reinforcement steel in a concrete structure depends on the duration of the fire, the fire load, the cross-sectional shapes, and the concrete cover. The larger the concrete cover, the more slowly the steel temperature will rise. This makes sense, if one keeps in mind the temperature distribution as described in paragraph 4.2 [10, 58].

Besides the strength of the steel itself, the bond between steel and concrete can be affected at temperatures higher than 300 °C [58]. Below this temperature, the thermal expansion coefficient of steel is nearly equal to the coefficient of concrete. But when the temperature rises above the value of 300 °C, the thermal expansion coefficient of steel varies with the temperature in a different way than the expansion coefficient of concrete. Changes in the thermal expansion coefficient of steel at high temperatures are related to changes in the phase of the steel (it starts to yield) and the crystal composition of the steel.

The thermal conductivity of steel is much bigger than the conductivity of concrete. If a part of the reinforcement is exposed to the fire (by loss of the concrete cover due to spalling, for example), the heat will spread through the whole bar. The steel temperature rapidly becomes high enough to dehydrate the cement paste, even at the place where the reinforcement is still covered. This effect, as well as the difference in the thermal expansion coefficients, may adversely affect the local bond between concrete and steel at high temperatures [70]. This problem is rather worse in prestressed concrete structures, where enough bond strength is needed in the anchorage length to transfer the prestressing force into the concrete. However, there are barely, if any, known cases of failures which had directly occurred due to the loss of bond [69].

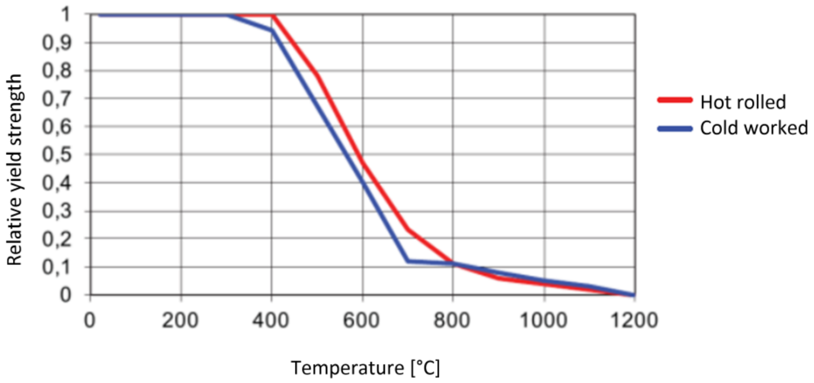


Figure 16 | Relative yield strength of the reinforcing steel as a function of the temperature [16]

4.5 Imposed thermal deformations

Besides all of the mentioned types of damage, temperature rises lead to another important phenomenon concerning concrete structures: the imposed thermal deformations [61]. These deformations occur not only in concrete structures, but in all types of structures. Due to the expansion of the construction material at high temperatures, the structural elements will deform. In a building, these thermal deformations will often be prevented by the surrounding structure. This can lead to large compressive and tensile stresses in transverse and longitudinal direction, as well as additional moments [62]. To understand this phenomenon, it is important to take a closer look at the temperature distribution.

Figure 14 already showed a rough schematization of the non-linear temperature distribution inside the cross-section of a concrete element. This temperature distribution is divided into three components, to gain more insight into the response of a structural element on the temperature load. These components are [61]:

- The mean temperature (ΔT_m);
- The temperature difference (ΔT_b);
- The eigen temperatures (ΔT_e).

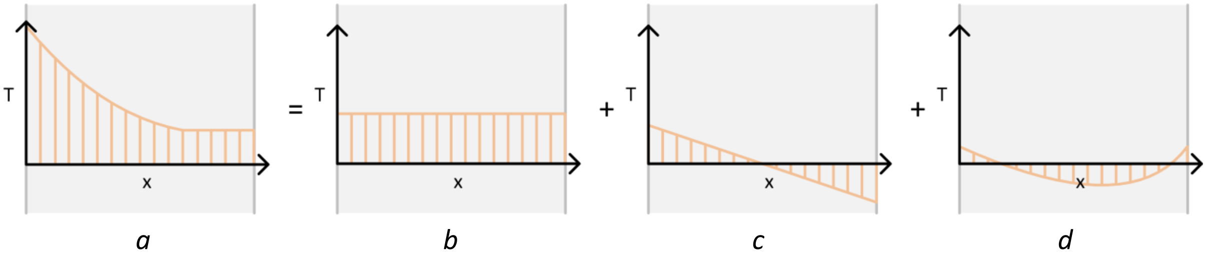


Figure 17 | The temperature distribution of a cross-section (a), divided in three components: the mean temperature (b), the temperature difference (c), and the eigen temperatures (d) [61]

The mean temperature (Figure 17b) is a constant temperature component, which causes an elongation of the element in case of a temperature rise due to a fire. If these deformation is restrained, an axial compressive force develops (Figure 18) [61, 62].

The linear temperature distribution component shown in Figure 17c, is the temperature difference. If a structural element is free to deform, this component causes a curvature (Figure 18). A curvature leads to rotations at the beginning and the end of the element. If these rotations cannot occur because they are restrained by the supports, moments are induced at these locations (Figure 18) [61, 62].

Eigen temperatures (Figure 17d) form the remaining part of the temperature distribution (in the form of a formula: $\Delta T_e = \Delta T - (\Delta T_m + \Delta T_b)$). The summation of these temperatures over a cross-section of a structural element is always zero, both concerning the axial direction and the bending [61, 62]. This means that the stresses caused by the eigen temperatures, called "***the eigen stresses***", do not induce a elongation or curvature, which contribute to the bending moment and normal (axial) force in case they are restrained. Nevertheless, this temperature component is certainly not unimportant. The eigen stresses caused by heating, exist of compressive stresses in the outer layer of the concrete and tensile stresses at the inside. These compressive stresses can be dangerous, because a large compression in a surface which is exposed to a fire, may cause spalling (as was already told in paragraph 4.3). They also postpone the moment at which cracks due to an externally applied moment will occur, while the concrete in the middle of the cross-section could already have been cracked, due to the tensile stresses caused by the eigen temperatures. These situations can lead to unexpected cracking of the structure [61, 62].

The extent of all these effects mainly relate to the location and the duration of a fire. If a fire occurs in the middle of a large building, the heated structural element is completely surrounded by other elements, which will block the thermal deformations. In case of a fire at the edge of a building, these deformations will be restrained less. And a short fire has smaller consequences than a long one, even if this fire is more severe. This is because of the fact that concrete needs time to heat up [61, 62, 64].

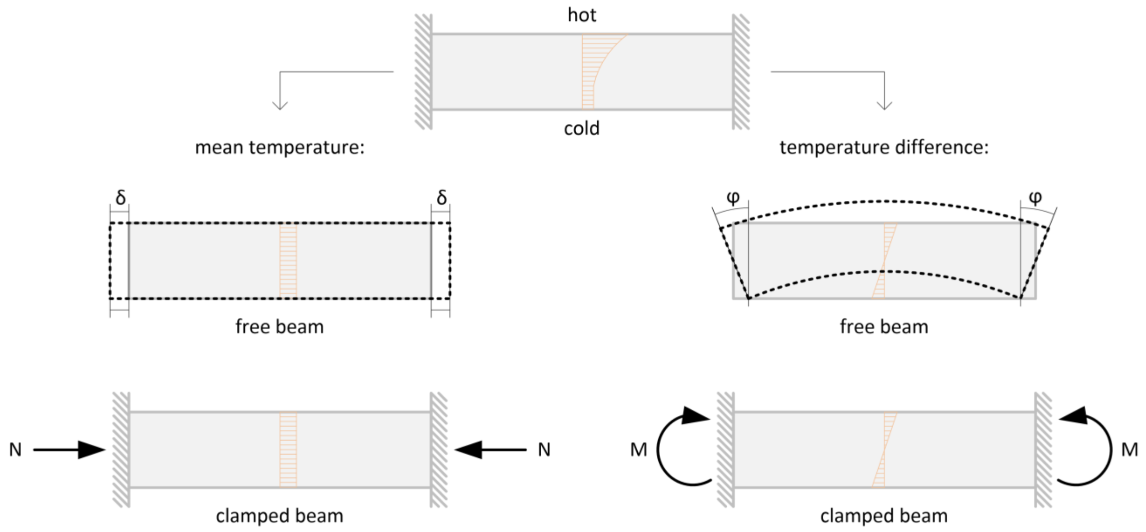


Figure 18 | Forces and moments caused by imposed thermal deformations [61]

4.6 Factors influencing the fire resistance of historic concrete

To determine the fire resistance of existing concrete buildings, it is important to take into account the differences between historic and current reinforced concrete, which can influence the mentioned temperature effects. In addition, although it is known that the compressive strength of the concrete still increases after the characteristic strength at 28 days is reached (see Figure 19), the concrete may be significantly weakened after many years by several forms of deterioration. This could influence the temperature effects as well. For these reasons, the main differences between historic and current reinforced concrete will be mentioned in paragraph 4.6.1, while paragraph 4.6.2 deals with the several forms of deterioration. Finally, paragraph 4.6.3 discusses the complexity of the spalling of aged concrete, which is associated with these differences and forms of deterioration.

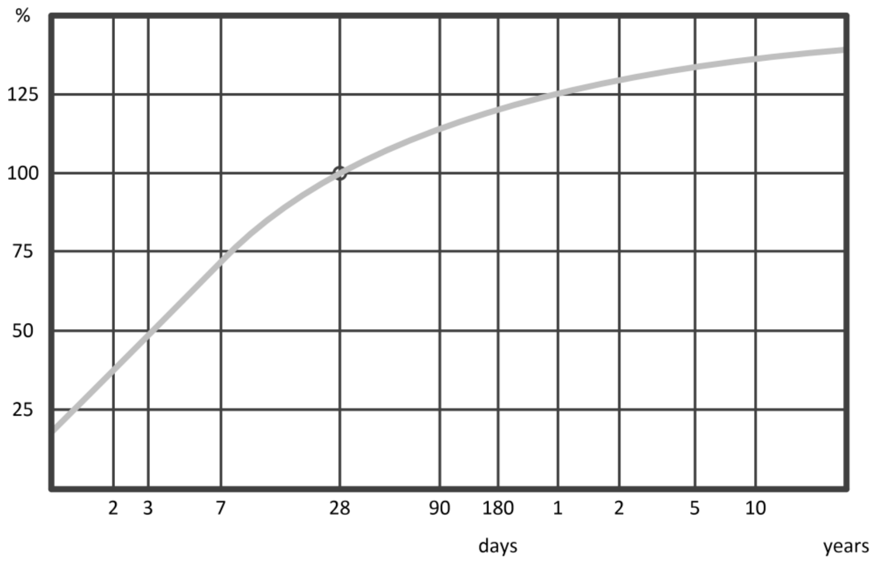


Figure 19 | Increase of the concrete strength in the long term [71]

4.6.1 Main differences between historic and current reinforced concrete

4.6.1.1 Compressive strength

The values of the compressive strength of the concrete, which can be reduced by fire exposure as was mentioned before, were considerably lower than they are now. Until 1950, concrete had a minimum cubic compressive strength of 15 N/mm^2 in the Netherlands [72]. This cubic compressive strength represented an average value which followed from pressing concrete cubes with ribs of 200 mm in a series of three pieces after curing of 28 days. In the GBV ("Gewapend Betonvoorschriften", see paragraph 5.1.1) of 1962, new strength classes were defined. These classes were K160, K225, and K300, with corresponding cubic compressive strength values of 16 N/mm^2, 22.5 N/mm^2, and 30 N/mm^2, which were also determined by means of concrete cubes with ribs of 200 mm [73]. Upon further developments in the concrete mixture, eight new strength classes were defined in the VBC ("Voorschriften Betonconstructies") of 1974: B12.5 up to B60 [73]. These values represented the characteristic cubic compressive strength (f_{ck}), which was calculated by the average cubic compressive strength (f_{cm}) minus 1.53 times a standard deviation of 5 N/mm^2. For the subsequent classes B15 to B65 which were introduced in the VBC of 1995, use was made of the assumption $f_{cm} = f_{ck} - 8\ N/m^2$ [72]. Both the values of 1974 and 1995 were determined using cubes with ribs of 150 mm, which caused slightly different results than cubes with ribs of the former used 200 mm. To still be able to compare the mutual average cube compressive strengths, the strengths up to 1962 are adjusted by multiplying them with a factor of 1.05, which was introduced in the VBT ("Voorschriften Betontechnologie") of 1995 [73].

Nowadays, one uses the Eurocode which mainly focuses on the cylindrical compressive strength values. These values, divided into C12 to C90, are 15 to 20% lower than cubic compressive strength values [74]. In order to get a good picture of the development of the compressive strength of the concrete from 1912 up to the Eurocode of 2012, Table 12 shows the mean cubic compressive strength values of the highest strength class from the mentioned standards. Use is made of the factor 1.05 for the values up to 1962 and the cubic compressive strength values which correspond to the cylindrical compressive strength values of the Eurocode for the values of 2012.

Table 12 | Mean cubic compressive strength values of the highest strength classes of the mentioned standards [73, 74]

Standard	Strength class	Mean cubic compressive strength in N/mm^2
GBV 1912	-	26.3
GBV 1962	K300	31.5
VBC 1974	B60	67.5
VBC 1995	B65	73.0
Eurocode 2012	C90/105	113.0

4.6.1.2 Reinforcement steel

The tensile strength of a reinforced concrete element mainly depends on the reinforcement. The reinforcement bars and wires which were used in the first concrete elements were made of steel and iron. Steel was preferred, because this material was stronger and adhered well to the concrete, although it turned out that steel is more sensitive to corrosion than iron [72]. Since 1930, the quality of the reinforcement steel gradually increased. This mainly concerned the reduction of the brittleness of the material, due to the better production methods. The tensile strength of the applied reinforcement steel increased as well. Nowadays, the most commonly used reinforcement steel has a tensile strength of 580 N/mm^2 [73]. Halfway the twentieth century, the most commonly used reinforcement steel had a tensile strength of 360 N/mm^2, although steel with a tensile strength of 500 N/mm^2 was already used in 1912 [72, 73, 75].

In addition to the increased quality of the material, the smooth profiles were increasingly replaced by ribbed steel, which led to a twice as high bond strength. A higher bond strength increases the strength of the reinforced concrete. However, this increased bond strength is not always an advantage. It appeared to be a major contribution to the failure of concrete elements at high temperatures [76]. Imagine a concrete slab with ribbed bars and a concrete slab with smooth bars. As was discussed earlier in this chapter, a fire could lead to cracks in the concrete, after which the reinforcement is also exposed to high temperatures. Because of the better bond strength of the ribbed bars, the straining of the ribbed reinforcement occurs over a smaller free length and fracture strains could already happen at small crack widths. The smooth reinforcement would not develop local strains of the same order, because the free lengths of the bars can become much higher. This means that the slab with ribbed bars could retain its cracks because the reinforcement is fractured, while the cracks of the slab with smooth reinforcement could reclose after the fire. Figure 20 shows this difference between these types of reinforcement as a result of a heating test at the University of Sheffield [76].

Ribbed reinforcement Smooth reinforcement

Figure 20 | Concrete slabs with different types of reinforcement, exposed to high temperatures [76]

4.6.1.3 Concrete cover

Because the concrete cover prevents the reinforcement steel to heat up and lose its strength, the cover plays a significant role in the field of the fire resistance. This was known early on, since the GBV of 1912 already contained a minimum value of 10 mm for the concrete cover of slabs and a minimum value of 15 mm for the concrete cover of beams and columns, to protect the steel bars against harmful conditions, of which fire was seen as the main threat [75]. These values were increased by 10 mm for cases covered by the exposure class “high temperatures”, the first given exposure class introduced by the GBV of 1918. Besides the matter of fire protection, one was aware that the concrete cover was important to minimize the chances of corrosion, although the underlying theory was not fully understood. Engineers were assumed that the protection against corrosion was provided by the density and quality of the concrete cover, instead of the thickness of the concrete cover which – as one knows now – contains an alkaline environment creating a protective layer for the reinforcement steel. In 1930, when exposure to sea water, humidity, and flue gasses were found to be harmful as well, the exposure classes were extended by “aggressive environments” and “no control after casting”, followed by “wind and weather” in 1950. Nowadays, engineers use an even more extensive collection of exposure classes, which are shown in Table 13, followed by an indication of the corresponding concrete cover values in Table 14 [74]. In Table 15, the historic concrete cover values in the different standards are given. At this table, it is important to note a difference in the definition of the concrete cover. Nowadays, the concrete cover is defined by the Eurocode 2 as "the distance between the surface of the reinforcement - including links, stirrups, and surface reinforcement - closest to the nearest concrete surface" [74]. However, before 1950, the main reinforcement instead of the outer reinforcement was taken as a reference point. Before 1918, it was not even defined. This means that concrete covers were not completely free of steel. Besides this difference, the concrete covers applied before 1930 could be even smaller than the given values, because it was not yet explicitly stated that the plaster coating must not be accounted to this layer [72].

Table 13 | Current exposure classes according to the Eurocode [74]

Exposure class	Description
1. No risk of corrosion or attack	
X0	*For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack; For concrete with reinforcement or embedded metal: very dry*
2. Corrosion induced by carbonation	
XC1	*Dry or permanently wet*
XC2	*Wet, rarely dry*
XC3	*Moderate humidity*
XC4	*Cyclic wet and dry*
3. Corrosion induced by chlorides	
XD1	*Moderate humidity*
XD2	*Wet, rarely dry*
XD3	*Cyclic wet and dry*
4. Corrosion induced by chlorides from sea water	
XS1	*Exposed to airborne salt but not in direct contact with sea water*
XS2	*Permanently submerged*
XS3	*Tidal, splash and spray zones*
5. Freeze / Thaw attack	
XF1	*Moderate water saturation, without de-icing agent*
XF2	*Moderate water saturation, with de-icing agent*
XF3	*High water saturation, without de-icing agents*
XF4	*High water saturation with de-icing agents or sea water*
6. Chemical attack	
XA1	*Slightly aggressive chemical environment according to EN 206-1, Table 2*
XA2	*Moderately aggressive chemical environment according to EN 206-1, Table 2*
XA3	*Highly aggressive chemical environment according to EN 206-1, Table 2*

Table 14 | Current minimum concrete cover values according to the Eurocode [74]

X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
10 mm	15 mm	25 mm	30 mm	35 mm	40 mm	45 mm

Table 15 | Minimum concrete cover values according to several standards of the twentieth century [75, 77-79]

	Reference point	Slabs				Beams				Columns				Additional cover
		No special exposure class	*Exposed to wind and weather*	*No control of surface after casting possible*	*Aggressive environments*[a]	*No special exposure class*	*Exposed to wind and weather*	*No control of surface after casting possible*	*Aggressive environments*[a]	*No special exposure class*	*Exposed to wind and weather*	*No control of surface after casting possible*	*Aggressive environments*[a]	*Surface treatment which damages the cement film*
GBV 1912	◦	10[b]				25				15				
GBV 1918	●	10[b]			20[c]	25			35[c]	35			50[c]	
GBV 1930	●	10		20	20	25		35	35	35			50	
GBV 1940	●	10		20	+10	25		35	+10	35			+10	+10
GBV 1950	○	d≤12 cm: 10[d] d>12 cm: 15	15[d]	20	+10	20	25	40	+10	30	35	40	+10	+10
GBV 1962	○	10	15	20[e]	+10	20	25[e]	30	+10	25	30	35	+10	+10
VB 1974/1984	○	15[d]	25[d]	+5[d]	30[d,f]	25	30[f]	+5	35[f]	30	35	+5	40[f]	+5
VBC 1995	○	15[g,h]	25[g,h]	+5[h]	30[g,h]	25[g,h]	30[g,h]	+5[h]	35[g,h]	30[g,h]	35[g,h]	+5[h]	40[g,h]	+5[h]

◦ None defined ● Main Reinforcement ○ Outer reinforcement **a)** High temperatures due to fire, sea water, aggressive water or gases. **b)** Valid for all other elements which are neither beams nor columns. **c)** Only for possible exposure to fire. **d)** Valid for walls and slabs. **e)** Includes now also contact to soil and ground water. **f)** +5 mm if the characteristic cubic compressive strength is below 17.5 N/mm^2 **g)** Due to the exposure classes of VBC 1995, "No special exposure class" must be read as "No risk of corrosion", "Exposed to wind and weather" as "Corrosion induced by carbonation", and "Aggressive environments" as "Wet in combination with de-icing agent, Seawater, and Aggressive environments". **h)** +5 mm if the characteristic cubic compressive strength is below 25 N/mm^2

From the tables, it appears that historic structures have been designed with smaller concrete cover values. The required thicknesses are generally too small compared to the modern environmental classes, especially in case of concrete floor slabs. Besides, the concrete of the older structures is generally more porous due to the higher water/cement ratio of that time, which negatively affects the resistance to the outside influences as well. Finally, one should take into account that the concrete cover was (and is) not always executed properly, which could lead to large deviations of the concrete cover [72].

A small but not insignificant remark is that there are situations where the concrete cover is of less importance. As will be explained in section 5.2, thermal expansions of a restraint concrete element could lead to lower forces in the reinforcement steel. In such situations, the thickness of the whole concrete element - which influences the heat transmission - is more essential than the concrete cover protecting the reinforcement steel.

4.6.1.4 Type of aggregate

Besides the mentioned thickness of the concrete cover and the water/cement ratio, the heat transmission also depends on the type of aggregate [80]. Commonly used aggregates were gravel and crushed stone, of which crushed stone was initially preferred despite its higher price because its rough surface offered a bigger area to bond with the cement, and the freshly broken surface was less contaminated. It was not yet known that the use of gravel requires less water and thereby reduces the risk of segregation and bleeding. However, gravel was used more later on, because of its low costs and the fact that it requires less cement due to its round shape and different particle sizes, so that the voids were filled with the smaller particles of the gravel instead of the cement [75]. Gravel mainly consisted of siliceous aggregates, which have a slightly lower fire resistance than carbonate aggregates, as can be seen in Figure 21. This can be explained by the difference in the heat transmission, caused by the decarbonisation mentioned earlier in paragraph 4.1.1: the liberated carbon dioxide (CO_2) absorbs heat in much the same manner that water absorbs heat when it is converted to steam and thus retards the transmission of heat [80].

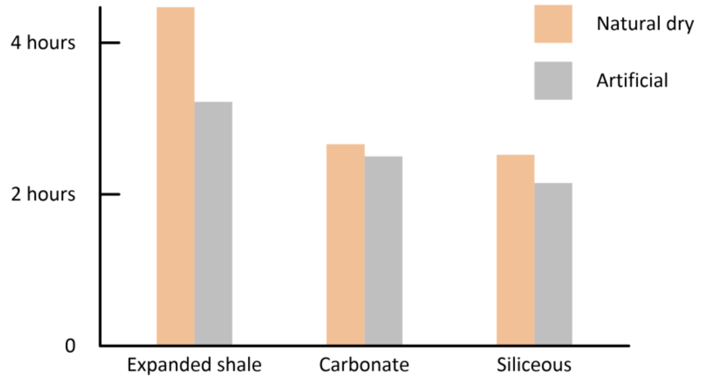


Figure 21 | The effects of the type of aggregate and drying method on the fire endurance of 6 inch thick concrete slabs as determined by a 250° C temperature rise on the unexposed surface of the slab [80]

In addition to these normal weight aggregates (carbonate and siliceous aggregates), Figure 21 shows a lightweight aggregate (expanded shale). In general, lightweight aggregates are more fire resistant than normal weight aggregates, due to their insulating properties and stability at high temperatures. Although several lightweight aggregates were already known in the beginning of the twentieth century, they were rarely applied in reinforced structures due to the low protection against corrosion and the low strength of the material [72, 81].

4.6.1.5 Moisture content

Serious attention to the amount of water in the concrete was only paid after 1950, when one found out that high water/cement ratio values led to very porous concrete. Before that time, concrete mixtures contained a lot of water to improve the workability of the material [80]. Looking to Figure 21, it appears that the heat transmission is retarded by concrete with a high moisture content, since the natural dried concrete contains more water than the artificial dried concrete. However, this does not directly mean that it increases the fire resistance of historic concrete, since one must not forget the bigger chance of spalling (besides the extra porosity) mentioned in paragraph 4.3.2.1.

4.6.2 Deterioration of the reinforced concrete

4.6.2.1 Corrosion

The main damage mechanism of reinforced concrete is the ***corrosion*** of the reinforcement (in Dutch often called “betonrot”) [72]. Corrosion is an electrochemical reaction, characterized by the atoms in an anodic and cathodic region which experience a change in oxidation state, in combination with an electric current which connects these regions. This electric current uses the pore water as a medium to be able to move, a so called electrolyte. In general, this water has a pH value above 12, caused by the reaction products of the hydration of the concrete. Under those circumstances, the steel is passivated: it is protected against continuous corrosion by a stable oxide layer. However, this favourable environment can change due to carbonation of the concrete, a high concentration of chlorides in the concrete, or both [82].

4.6.2.1.1 Carbonation

During ***carbonation***, carbon dioxide reacts with the hydroxides which are present in the pores of the cement stone. These hydroxides are converted into carbonates, which decrease the pH value. If this value comes below 10, the protective oxide layer is degraded and the reinforcement steel becomes prone to corrosion [72]. The carbonation itself does not cause any damage – the produced carbonates are not expansive and reduce the porosity of the concrete. However, the corrosion products which result from the initiated corrosion process are much more voluminous than the reinforcement steel, leading to an internal pressure. By this internal pressure, the tensile stresses may become too high, causing cracks in the concrete surface in combination with pieces of concrete which are teared off – known as the phenomenon "spalling" which was described in paragraph 4.3 [82].

The Eurocode recommends a sufficient reinforcement depth and concrete quality to prevent or reduce the corrosion caused by the carbonation [74]. However, paragraph 4.6.1.3 indicated that historic concrete is rather porous and the concrete cover is relatively thin. This makes historic concrete more sensitive to carbonation than current concrete, but this does not always has big consequences. The risk of problems due to the corroding reinforcement is small for concrete which is always fully saturated or concrete which is always dry (such as in a heated indoor climate). Under these circumstances, carbonation cannot take place because air cannot get in or, respectively, no moisture is present. The risk of damage is greatest in an environment where the concrete is alternately wet and dry [82].

4.6.2.1.2 Chloride attack

Chlorides may occur in concrete as a result of intrusion, where the chlorides can be obtained from de-icing salts, sea air, or sea water. They can also occur by contaminants in the aggregates and additions as calcium chloride used to accelerate the hardening process, in particular applied in the second half of the twentieth century (for example for the production of the "Kwaaitaal" and "Manta" prefab floors) [83]. High concentrations of chlorides can, despite the high pH value of the surrounding concrete, deplete the passivation layer on the reinforcement steel very locally. This form of corrosion is also known as ***pitting*** (in Dutch: "putcorrosie") [82]. The local corrosion reaches great depths of the reinforcement steel, reducing its strength. Because this corrosion is very local, the amount of corrosion products is much lower than in case of carbonation, leading to nearly any internal pressure. This means that the subsequent forms of damage caused by the internal pressure - the cracks in the concrete surface and the teared off pieces of concrete – do not take place. Hereby, there is nearly any warning effect in contrast to the carbonation, which is why the urgency of this situation is often not properly assessed, leading to treacherous and structural dangerous conditions. Only in a more advanced stage, a chloride attack can be recognized from the outside of the element by corrosion products which flow out of the concrete through pores or cracks (Figure 22C).

Just like in case of carbonation, historic concrete is more susceptible to this type of corrosion due to the high porosity and thin cover. Besides, the use of the mentioned calcium chloride as an accelerator for the hardening process negatively influences the risk of pitting even more. Nowadays, the addition of these chlorides is, therefore, no longer permitted [82].

4.6.2.1.3 Combination of carbonation and chloride attack

A significant amount of chlorides in the concrete is bounded to the cement stone. However, during carbonation, bounded chlorides come free, leading to a much higher concentration of free chlorides.

So the combination of carbonation and chlorides is dangerous, because chloride concentrations which did not lead to corrosion at first, can lead to corrosion due to carbonation [82].

A B C

Figure 22 | Damage due to carbonation (A), chlorides (B), and a combination of the two (C) [82]

4.6.2.2 Chemical attacks

The concrete itself could also be damaged by chemical reactions between substances that are present in the cement stone and the additives. The chemical attacks generally lead to expansion, crack formation, and leaching reaction products. The cause of the damage is characterized by reaction products which swell up and expand due to the absorption of pore water, leading to an internal pressure. Thereby, humidity is an important condition. For this reason, the main and almost only measure to reduce the damage of chemical attacks is a strong reduction of the moisture ingress [60, 82]. So again, historic concrete is more sensitive due to the relatively high water/cement ratio and the high porosity.

The main forms of the chemical attacks, the ettringite formation and the alkali-silica reaction, will be briefly described below. It goes without saying that these reactions, just like the different forms of corrosion, can occur at the same time.

4.6.2.2.1 Ettringite formation

The formation of ettringite, which takes place in the form of needle-shaped crystals, occurs due to the reaction of sulphate with substances of the cement paste, such as aluminate and calcium. Sulphates can be present in the aggregates or penetrate from the outside. Damage caused by the formation of ettringite is recognizable by micro cracks in the cement stone and the loss of cohesion. The cement stone often turns white and pulverizes, leading to loose pieces of gravel. Larger cracks could occur as well, giving the concrete a layered appearance. From these cracks, calcium deposits at the concrete surface can also develop [60, 82].

4.6.2.2.2 Alkali-silica reaction

At an alkali-silica reaction (also known as “ASR”), a reactive aggregate containing a silica reacts with the alkalis in the pores to a alkali-silica gel, which swells up due to the absorption of water. Besides a high moisture content, an alkali-silica reaction requires a high alkalinity, the presence of Portland cement, and a critical amount of reactive aggregates. Just like in case of the formation of ettringite, a reduction of the cohesion will occur at an alkali-silica reaction. The damage begins at the interface of certain aggregates, leading to visible torn grains. Due to the expansion of the concrete, a characteristic crack pattern occurs, consisting of more or less parallel cracks which are connected by transverse cracks. From these cracks, a secretion of a whitish gel often takes place [60, 82].

However, an alkali-silica reaction does not always lead to significant damage. Sometimes, the formation of the gel is widely distributed and the gel reacts further with the available calcium to calcium silicate hydrates, compacting the cement stone and slowing down the reactions [60, 82].

4.6.2.3 Physical deterioration

Finally, various physical phenomena - in addition to fire - could deteriorate the concrete as well. Well known forms of physical deterioration are (long-term) drying shrinkage and creep [72, 82]. Drying shrinkage occurs in the first weeks or months after the concrete is placed. Due to the loss of water, the cement would generally shrink up to 1%, which does not lead to any damage. However, because the aggregates in the concrete restrain this shrinkage only to 0.05%, tensile stresses inside the cement paste will develop. The magnitude of these stresses can be influenced by creep, the phenomenon where a long-term load, which may be considerably lower than the maximum allowable load, leads to a permanent deformation. The developed tensile stresses can cause cracks when the tensile strength of the material is exceeded.

Teared off pieces concrete and cracks could also indicate frost and de-icing salt damage. Frost can deteriorate the concrete when the material is nearly fully saturated with water during frost periods. When freezing, water expands by about 9%. Damage occurs when the pores do not have sufficient air to provide space to this expansion. Just as with corrosion and chemical attacks, the relatively high water/cement ratio and porosity of concrete which dates from the first half of the twentieth century makes historic concrete extra sensitive for frost damage. Besides, this sensitivity can be increased by the presence of de-icing salts, which provide a different type of damage, namely, flaking and crushing of the concrete surface (see Figure 23C) [82].

Finally, physical damage could of course have mechanical causes as well, such as overloads or uneven settlements.

A B C

Figure 23 | Damage due to alkali-silica reaction (A), shrinkage (B), and frost in combination with de-icing salts (C) [72, 82]

4.6.3 Spalling of aged concrete

Paragraph 4.3.2 already revealed that spalling is a complex topic, because it is influenced by a lot of different parameters. Some of these parameters are mentioned in the main differences between historic and current reinforced concrete, such as the moisture content, the porosity, and the concrete strength. The moisture content of historic concrete (especially before 1950) seemed to be quite high. According to paragraph 4.3.2.1, this could increase the likelihood of spalling. For this reason, one might assume that the change of spalling for historic concrete is bigger than for current concrete. However, the lower concrete strength and the high porosity of the historic concrete suggest the opposite. Besides, the water content reduces with age from drying. Based on this, several experimental researches have indicated that concrete will not spall beyond a critical age [63]. However, other fire tests have shown that concrete over this critical age is still capable of spalling.

For these reasons, it is clear that the chance of spalling of existing structures cannot be easily assessed. To get an idea of the risk of spalling, a fire test is often recommended [63]. This is usually a job for a fire engineer, not a structural engineer. However, this does not mean that a structural engineer could not make a single assumption concerning the spalling risk of an existing building. As was described in paragraph 4.6.2.1.1, corrosion products of carbonation can cause high compressive stresses which can lead to spalling as well. So in contrast to the other factors, it is certain that carbonation practically always leads to a high chance of spalling. This means that when a structural engineer observes this type of corrosion, a high risk of spalling can be assumed.

4.7 Concrete compared to timber and steel

4.7.1 Timber

In contrast to concrete, timber is a combustible material which contributes to the fire load. In other words, timber has a bad reaction to fire [64, 84]. When the outer surface of unprotected timber reaches a temperature of approximately 250 °C, it starts to decompose (pyrolyse) and gasses will be released, which will burn when they are exposed to oxygen. The remaining cross-section gets gradually smaller. The speed at which this happens is called the charring rate. At a certain moment, the unburnt profile which has to carry the load, becomes so small that the structure will collapse. Because this collapse occurs in the form of a brittle fracture, without any significant deformation taking place that could serve as a warning, the material could lead to dangerous situations in case of a fire. Another disadvantage is that the damaged timber members cannot be repaired, in contrast to concrete, where the damaged layer could be eliminated and replaced by a new one (as was told earlier in paragraph 4.1.1).

However, although this material has a bad reaction to fire, it does not mean that the resistance to fire is bad as well. The burnt timber at the outside of a timber profile forms a layer of charcoal. The thermal conductivity of this charcoal is only one sixth of the conductivity of the unburnt solid timber [84]. This means that this layer has an insulating effect, which causes a non-homogenous temperature distribution in the cross-section of a timber element (Figure 24). The core of the element stays relatively cool, which leads to practically no decrease of the strength of the timber. Besides that, the charring rate slightly decreases. Due to the formation of this charcoal layer, timber has a good resistance to fire, compared to building materials such as steel [64, 66, 84].

4.7.2 Steel

Steel is a material with a high thermal conductivity. Combined with the fact that steel members are relatively thin walled, steel structures heat much faster than timber or concrete structures when they are exposed to fire. When they reach a temperature of 1200 °C, the material starts to melt [66]. However, far before reaching this temperature, the modulus of elasticity, the yield stress and the ultimate tensile stress already reduce. The deterioration of these mechanical properties starts at a temperature of 200 °C. At a certain point during the fire, the strength of the material could have been reduced in such an extent, that the resistance of the member reaches a value below the load and the structure fails.

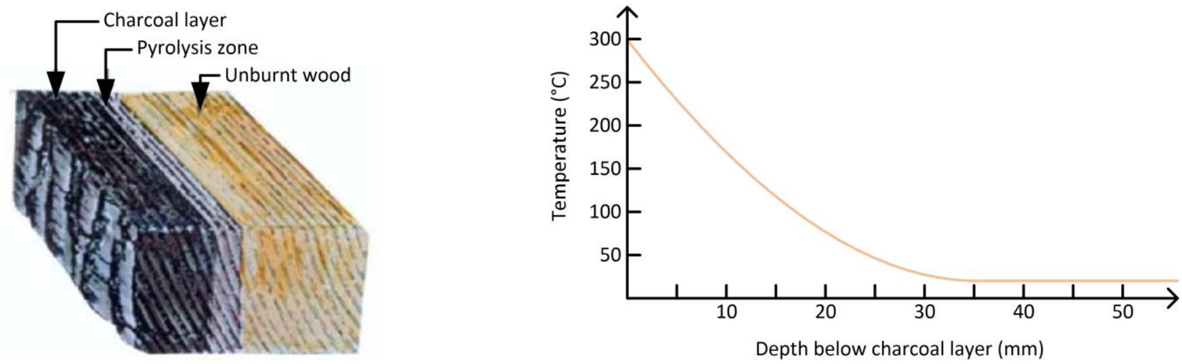


Figure 24 | Representation of the influence of the charcoal layer on the temperature distribution of the timber cross-section [84]

If the structure did not fail during a fire, it could still show large permanent deformations after cooling [64]. These deformations are not caused by thermal expansions of the material, because thermal expansions are reversible. The large deformations occur due to creep. This is a stress, time and temperature dependent mechanism, at which strains develop for structures that are subjected to a constant load in time. Creep can be neglected at room temperature, but during a fire, it could dominate the behaviour of the steel in a very short time. Structural elements which such deformations cannot be re-used after a fire. However, creep does have an important advantage. Because deformations due to creep become excessively large before failure occurs, creep actually forms an excellent warning system.

Steel structures which are exposed to a relatively low temperature or a low stress during a fire, does not show such large deformations. These structures may be re-used. However, it is necessary to take into account the possibility that the material strength may not be completely retrieved after a fire. Attention to bolted connections is also required. Apart from a strength reduction of the bolts, thermal expansion and creep due to a fire may lead to high tensile forces in the bolts after cooling [64, 66].

4.7.3 Summary

All of the relevant aspects of the unprotected construction materials in case of a fire which are discussed earlier in this chapter, are summarized in Table 16. From this table, it becomes clear why preference is being given to concrete structures concerning the fire safety of a building. Concrete is, together with other stony construction materials such as bricks, the only load-bearing material that is able to withstand a fire without additional protection of any kind (Annex A shows an example of a fire damaged building, wherein the stony support structure has remained undamaged). The properties which influence the fire behaviour of the material in a favourable way, do not change in time. Because of this, no additional expenses for maintenance are required. In this way, concrete provides the required fire resistance in an economical manner. It usually suffices to use a certain concrete cover given by tables in different building standards. By applying this simple method, one does not need advanced fire models as discussed in paragraph 2.4 [12]. The assessment of the fire resistance of concrete structures, including the use of these tables, will be extensively discussed in chapter 6.

Table 16 | Summary of unprotected construction materials performance in fire [85]

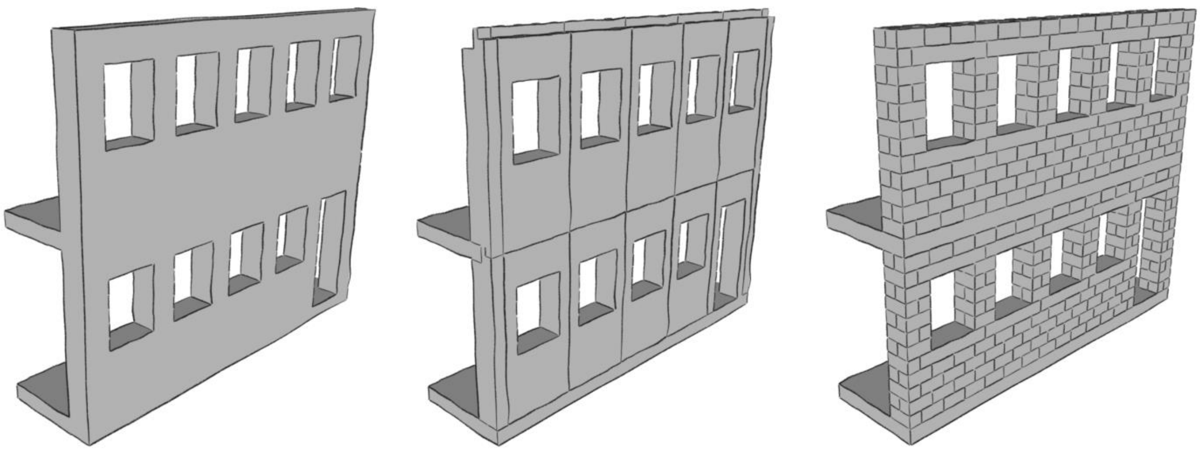
Unprotected construction material	Fire resistance	Combustibility	Contribution to the fire load	Rate of temperature rise across a cross-section	Reparability after a fire
Timber	Low	High	High	Very low	Nil
Steel	Very low	Nil	Nil	Very high	Low
Concrete	High	Nil	Nil	Low	High

5 STRUCTURAL BEHAVIOUR UNDER FIRE CONDITIONS

Different temperature effects on reinforced concrete were discussed in the previous chapter, including thermal expansions. These thermal expansions can influence the structural behaviour of a concrete building. To properly design or asses a concrete building, it is important to know these influences on the structural behaviour as well, besides the effects which only concern an individual element. For this reason, the structural behaviour of concrete buildings influenced by thermal deformations is discussed in this chapter. However, to fully understand these aspects, basic knowledge of concrete building systems (and the difference between them) can be of importance. For this reason, the concrete building systems will be briefly discussed first, after which the aspects concerning the structural behaviour under fire conditions will follow.

5.1 Concrete building systems

It is generally stated that the concrete-using building systems can be divided in three main groups, namely casting construction, prefabrication, and masonry. These groups are visualized in Figure 25.



Casting construction *Prefabrication* *Masonry*

Figure 25 | The three main groups of the building systems

Several differences exist between the casting construction and the prefabrication. These differences will be noted in the appropriate locations in this section. But at first, the historical development of these systems will be explained.

Masonry is a traditional construction method, where the structure is usually formed by brick walls. But instead of these traditional bricks, prefabricated concrete blocks (or other materials) can be used as well. This can be seen as a form of prefabrication which is generally used for housing [86]. For this reason, this building system would not be discussed further in this thesis.

5.1.1 Historical development

The first applications of concrete date from the early nineteenth century. The concrete was used as an unreinforced material which in the Netherlands was applied in hydraulic works, fortifications, bridges, sewers, floors, and tiles. The concrete from this era is known as the so-called '***stamp-concrete***', a dry concrete mixture which was applied in thin layers and was densified with pistils [87].

Around 1860, the cement industry had emerged, in which cement mortar was used for precast elements as tables and ornaments, which were previously made out of stone. Important steps in the development of the use of reinforced concrete were the patents from 1867 of the Frenchman J. Monier on applications of iron networks in concrete elements, together with the patent from 1892 of another Frenchman, J. Hennebique, which was on a method of construction in which reinforced columns, beams and slabs were formed into one monolithic entity [82].

With the introduction of reinforced concrete in the Netherlands around 1880, the material was increasingly used for structures such as bridges, factories, halls, silos, cooling towers, and water towers [86]. The first concrete floor buildings in the Netherlands appeared in the beginning of the twentieth century. To raise the quality of the concrete structures, the first building standards, known as the "Gewapend Betonvoorschriften (GBV)", were introduced in 1912 (see chapter 6).

After 1900, there was a growing amount of available concrete elements in the Netherlands. However, most buildings were still built as cast-in-situ structures until 1940 [87]. Prefabrication was unprofitable, because the labour costs were low compared to the material prices. Furthermore, the possibilities of transport and assembly of large elements were limited. And in addition, the in-situ structures had several appreciated advantages, such as the freedom of shape and the monolithic character, which provided a robust entity.

When the Second World War had passed, the precast concrete industry really evolved [82, 87]. There was a great need for housing and buildings, because the construction of buildings had been insufficient for a couple of years and a huge amount of existing buildings had been destroyed. Besides this need, the shortage in manpower was also an important aspect to be reckoned with. Because of the need of new buildings and the lack of manpower, one started looking for a faster and more effective building method. The use of precast concrete was found to be suitable for this.

Both cast in-situ and precast building systems could be applied in the form of a ***framing structure*** or a ***traditional structure***. In a traditional structure, the separating elements have also a load-bearing function. The framing structure is a coherent set of structural elements such as columns, beams, and floors, in which façade walls and partition walls are placed which only serve to separate the different rooms [86]. Because the elements in a traditional structure perform multiple functions, the traditional structure is very economical. However, the separation of functions in framing structures has some other important advantages. First of all, adapting a building with a framing structure is relatively simple, because non-bearing partition walls can be replaced very easily. Besides, these walls can be carried out lighter and with larger openings. Furthermore, the execution of the construction of the building is easier, because the framing structure can be made without taking into account the outfitting. And finally, the materials in a framing structure can be used in such a way that the material properties can be optimally utilized (for example by using concrete for the framing structure, but another material with a better thermal insulation for the facades or partition walls). Because of all these advantages, most utility buildings were built as framing structures. For housing, however, these advantages were of less importance, which is why traditional structures were commonly used in this sector. This still applies to the current building market [82, 86, 87].

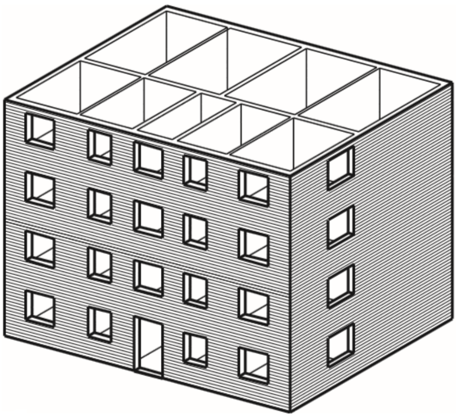


Figure 26 | Traditional structure [86]

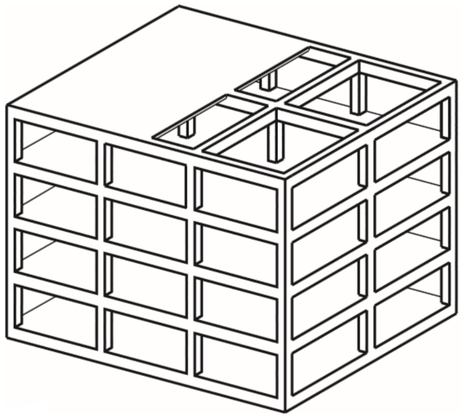


Figure 27 | Framing structure [86]

5.1.1.1 Cast in-situ building systems

In the first concrete buildings, the material was applied on the same way as one was used to build with timber and iron, namely in the form of a ***beam structure***. The big difference with timber and iron is the monolithic character of the concrete structure. This character was already observed by Francois Hennebique in 1892 [73]. He showed that reinforced concrete should not be seen as an assembly of two materials, but as one monolithic structure. After some improvements by the addition of iron stirrups around the reinforcement to resist the shear stresses and the bending of the reinforcement bars at the supports to resist the support moments, he developed a construction system where columns support longitudinal beams, which, together with the cross-beams, support the floor spanned in one direction. This system is known as ***system Hennebique***, the system on which the beam structure was based [73, 88]. Both the primary and the secondary beams were often bevelled towards the supports to save material in the middle. It goes without saying that on this way, the creation of the formworks became very labour-intensive. At that time, however, this was not an issue because the labour costs were much lower than the material costs.

Figure 28 | Beam structure [73]

Figure 29 | Mushroom floor [82]

At a later stage, new material specific building systems were designed, with floors spanned in multiple directions. The most popular one is the ***mushroom floor*** (in Dutch: "paddestoelvloer") [73, 82, 88]. In the Netherlands, this system was first used in 1914 and was originally intended for heavily loaded storehouses. The system exists out of reinforced concrete slabs on point-shaped supports. The columns are rigidly connected to these slabs. This connection is thickened to resist the punching shear forces and forms the so-called "mushroom". Because of the favourable structural properties associated with the multiple span directions, the relatively thin floors could resist high loads. This led to the fact that later on, from aesthetic and practical points of view (such as light penetration and a small construction height), the mushroom floor was widely used in factories and warehouses as well. Nevertheless, most of the buildings up to 1940 were still given a beam structure, in which the beams had a console at the connections in the form of a chamfer [73, 82, 88].

5.1.1.2 Precast building systems

At the beginning of the precast industry, there was a striving to give the structures a monolithic character [87]. Engineers were used to design in cast in-situ and wanted to maintain the advantages of these types of structures for their designs in prefabricated concrete. This often resulted in very complex connections. However, because of the increasing building costs, it was necessary to rationalise the production process. This had a positive influence on the development of precast systems [82, 87]. The focus did no longer lie on creating a monolithic structure, and the restraint connections were replaced by simple pinned connections. Furthermore, the quality of the precast concrete elements increased rapidly after 1945, due to new techniques to compact concrete (vibration and shocking) and the usage of plasticizers, which are substances which make concrete properly workable with less water. Finally, by the application of prestressed concrete, which was developed around 1930, bigger and lighter elements could be made. All of these developments led to the formation of many different precast systems which are characterized by their floor types. In the scope of this thesis, it would be infeasible to discuss all of these systems. However, these systems could be roughly reduced to the following types [88]:

- the ribbed slab floor;
- the combination floor;
- the hollow core slab;
- the solid slab and pipe floor.

These floors are used in both residential buildings and utility buildings. In addition, there are specific types of floors for utility buildings that are suitable for higher loads and larger spans, such as the well-known TT slabs.

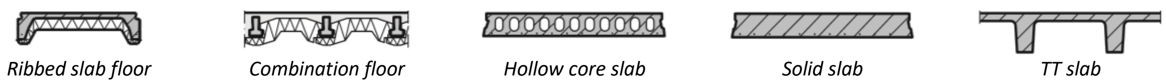


Figure 30 | The four main types of the precast systems and the TT slab [88]

Generally, many floor types have a variant with insulation which has been developed to use as a ground floor [88]. Exceptions are on the one hand the ribbed slab floor and the combination floor, which are specifically intended as a ground floor, and on the other hand the TT slabs, which are designed to use as upper floors.

The top side of most of the precast floor slabs is not finished yet and the elements are not coupled directly after installation. For these reasons, it is often necessary to apply a slight finishing concrete layer which fills the seams between the elements and provides a smooth surface. Besides, some flooring systems require a compressive layer as an in-situ addition which contributes to the compressive strength of the structure and which ensures that the floor is part of a large continuous sheet for an adequate amount of stability [88].

5.1.2 Cast in-situ building systems versus precast building systems

The main difference between cast in-situ structures and precast structures manifests itself in the fact that for cast in-situ structures, carefully dimensioned formworks must be manufactured, set, filled, and unloaded on the building site [87]. All these operations require time and manpower. As was mentioned in paragraph 5.1.1, the costs of manpower were not that high in the beginning of the twentieth century. But since the end of the Second World War, both aspects are leading to significant costs. In case of precast structures, this labour and time-consuming manufacture of formworks on site is replaced by formworks which are made in a factory [87]. These formworks can be used for a large number of times. Furthermore, the conditions in the factory during the fabrication of the mould and the concrete element are much better than the conditions on the building site, which results in a higher quality of the concrete element. Additional benefits of the fabrication in a factory, are the noise and space reduction on site. However, the major disadvantage of the prefabrication of concrete elements is the longer preparation time. Adaptions on site are barely possible, which means that elements and the corresponding moulds must be designed and manufactured very accurately and at an early stage. The number of design rounds with the essential audit work between the architect, contractor, and precast manufacturer are high.

In addition to the abovementioned examples of practical and economical differences between cast in-situ and precast structures, there are also structural differences between these building systems [88]. An important difference which was already mentioned earlier, is the monolithic character of the cast in-situ structures. Because the floors and girders are able to work together, the height of the structure could be reduced, despite the lower quality of the concrete. This difference is related to the different connection types. As was told in paragraph 5.1.1.2, precast structures require simple connections because of the high erection speed. Therefore, they are mainly executed as pinned connections. These connections are unable to transfer moments, in contrast to the fixed connections in a cast in-situ building. Because of this lack of stiffness, it is barely possible or uneconomical to provide the stability of a precast building by frame actions. For low buildings, this can be solved by restraining the columns into the foundation. However, for high-rise buildings it is necessary to place special stabilising structures as shear walls or cores. If these stabilising structures are cast in-situ, the assembly of the precast structure will be interrupted several times, causing the time savings of the construction in precast elements to be lost. For this reason, it may be recommended to construct the stabilising structures as precast concrete elements as well [87, 88].

5.2 Thermal expansions

These structural differences can influence the fire safety of a building. As a simple example, if the openings between the simple connections of a precast structure are too wide, the fire could easily spread to another floor [89]. The EN 1992-1-2 gives values for the maximum width and depth of these gaps. When these limits are exceeded, an additional sealing product could be used to increase the fire resistance [67]. However, this is related to the integrity and insulation of a building, and not to the load bearing capacity. More important is the role of these gaps in case of thermal expansions. If these gaps are small, thermal expansions of a heated concrete element will be restrained, which causes large compressive forces in both the longitudinal and transverse direction, as was shown in paragraph 4.5. Besides the influence of the location, duration, and severity of the fire as was mentioned here as well, the consequences of the thermal expansions depend – in addition to the stiffness of the (surrounding) structural elements – on the way in which the beams and floors are supported. Three conditions of support will be explained below, which are:

- simply supported, thermally unrestrained, flexural members;
- simply supported, thermally restrained, flexural members;
- continuous flexural members.

5.2.1 Simply supported, thermally unrestrained, flexural members

Figure 31 shows the behaviour of a simply supported concrete member which is exposed to a fire from beneath. At both ends the member is free to rotate and to elongate. Because the underside of the member is exposed to fire, the bottom expands more than the top, and the resulting curvature causes the member to deflect downwards. This means that the sagging of the member which – in combination with the reduced strength of the concrete and the reinforcement steel – eventually leads to flexural failure, does not only depend on the loads on the member, but also on the thermal expansion of the member [62, 89]. Because this extra deflection depends on the the temperature difference (ΔT_b) as shown in Figure 18, this deflection will be bigger for flat floor slabs than for ribbed floor slabs with the same thickness.

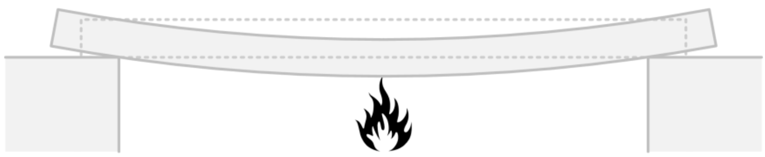


Figure 31 | Behaviour of a simply supported concrete member which is exposed to a fire from beneath

The depth of the expanding area depends on the duration of the fire, as was already stated in paragraph 4.5. To give an impression of the influence of the fire duration related to the depth of the heated area, Figure 32 shows the temperature distribution in a concrete slab of 200 mm thick at a certain depth x, corresponding to a duration of 30, 60, 90, 120, 180, and 240 minutes [67]. From this graph, it can be concluded that the top of a relatively thick floor slab (≥200 mm) would nearly heat up, even after 120 minutes of fire exposure at the bottom of the slab.

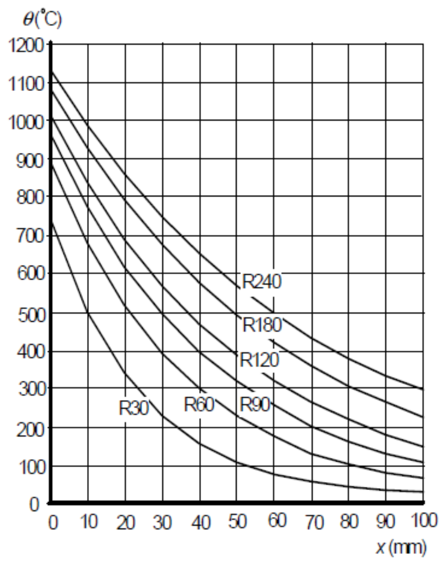


Figure 32 | Temperature distribution in slabs with a thickness of 200 mm for an exposure time of 60 to 240 minutes (x is the distance to the surface exposed to fire) [67]

An important difference between cast-in-situ and precast structures concerning the flexural failure is the required concrete cover [89]. In general, the concrete cover is increased by a certain value to allow deviations which could occur due to a less accurate control of the formwork dimensions, the placement of the reinforcement, the concrete quality, and the curing procedures [74]. This value can be found in the National Annex of the EN 1992-1-1 and is 5 mm in the Netherlands [90]. When it can be assured that a very accurate measurement device is used for monitoring and that non-conforming members are rejected, which holds for precast elements, this additional value may be reduced to 0 mm. In this way, a precast concrete element generally exists of concrete of a higher quality, but it may have a smaller concrete cover. This means that if both a precast concrete element and a cast in-situ concrete element satisfy the minimum cover requirements, the cast in-situ element may have more reserve capacity because the steel temperature will increase less quickly due to the greater amount of concrete cover.

5.2.2 Simply supported, thermally restrained, flexural members

Most reinforced concrete members in actual buildings are constructed and supported in such a way that longitudinal expansions and rotations at the supports are restricted [80]. If these thermal expansions and rotations are restricted, the heated member will exert a compressive force on the adjoining members which is referred to as "***thrust***" T [62, 89]. When the adjoining members can withstand this force and the line of action lies beneath the neutral axis of the member, the thrust force acts similar to a prestressing force, which induces a positive moment. This positive moment is beneficial to simply supported members that need to resist a negative moment. Even if the fire occurs at the edge of a concrete member, the thermal expansion is still partially blocked by the rigidity of the bearing structure and the distribution of horizontal loads by the plane elements. This means that restraints generally improve the performance of concrete members which are subjected to a fire.

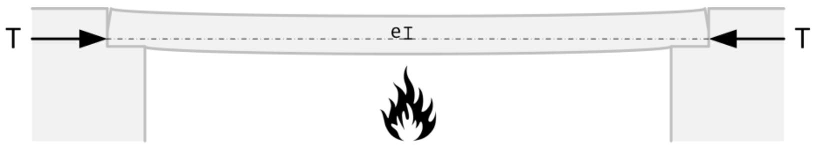


Figure 33 | Simply supported, thermally restrained concrete member with the thrust forces

However, the effect of the restraints can also be negative [62, 89]. If the line of action of the thrust force e lies above the neutral axis, it will generate a negative moment, which reduces the total flexural capacity. Although it seems that the line of action mostly lie beneath the neutral axis, it remains very difficult to determine the exact location, because the position can be influenced by a lot of different parameters, such as the time duration of the fire exposure, the shape of the member, the concrete compressive strength, the amount of the reinforcement, the relative stiffness of the flexural member and the adjoin frame, and the amount of expansion that is permitted. Besides, it should be noted that a large thrust force could cause additional failure modes, such as shear failure of the columns of the structure (Figure 35) or buckling of the compressed member [62, 89].

5.2.3 Continuous flexural members

Continuous flexural members have to deal with changes in reactions and internal forces during a fire, known as "***moment redistribution***" [62]. This phenomenon is shown if Figure 34. The fire beneath the concrete member causes the bottom of the member to expand more than the top, which leads to a resulting curvature which increases the downward reactions at the interior supports. These reactions result in a redistribution of moments: the positive moments at the interior supports increase, while the negative moments decrease. Because the positive moment reinforcement in the top of the concrete member stays cooler than the negative moment reinforcement at the bottom of the member, the positive moment reinforcement is better protected and therefore the capacity to withstand the increasing positive moments is usually sufficient. Besides, the reduction of the negative moments means that the reinforcement in the bottom of the member can be heated to a higher temperature before flexural failure will occur. And even when this steel has inadequate strength left, it does not have to mean that the concrete member will fail. Plastic hinges need to form first at the supports in addition to the one at midspan before a failure mechanism is created. Due to the better protection of the reinforcement in the top of the member, these hinges will mostly occur after the formation of the negative moment hinges. So in short, all of this means that the fire resistance of continuous concrete members (which are common in cast in-situ structures) is usually significantly longer than the fire resistance of simply supported members (which are common in precast structures) with the same cover and the same applied loads, especially when these members are spanned in multiple directions.

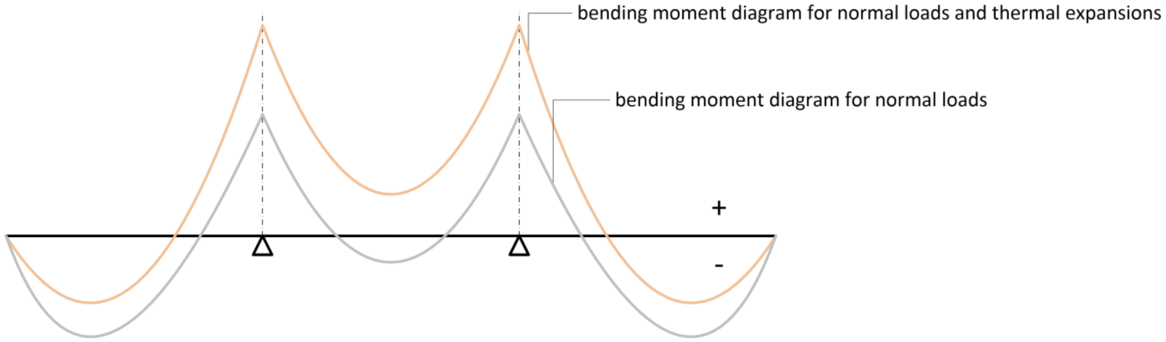


Figure 34 | Moment redistribution of a continuous concrete member

However, in contrast to the simply supported members, considerations need to be given to shear [89]. This is because the explained development of the reaction force at the interior support alters the distribution of internal shear. Furthermore, fires which are raging over several spans can cause major accumulated distortions at the ends of the members. A temperature rise of only 100 °C at a concrete member of 100 meter could already lead to an elongation of 120 mm. Due to these distortions, columns are subjected to very high shear forces, which could lead to failure. This happened to a harbour building in Ghent in 1983 (Figure 36, left) [91]. Although this building was over dimensioned in terms of the minimum concrete cover related to fire, it collapsed too early due to shear failure of the columns. The same happened in 1996 to a library building in Linköping, Sweden (Figure 36, right) [91]. The structure with a floor of 52 meter which was exposed to a fire on both sides was designed to resist a fire for 60 minutes, but it already collapsed after half an hour.

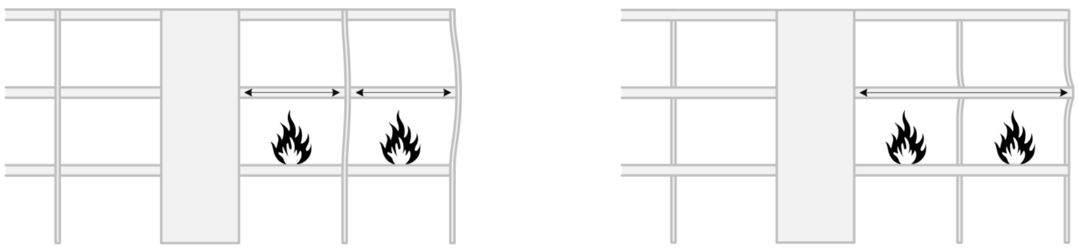


Figure 35 | The formation of shear forces in the columns by the longitudinal expansions of the horizontal members

For this reason, it is important to show attention to the thermal expansions of these continuous members. For example, a continuous member without expansion joints should not be too long. Besides, it is important to note that beams heat up more quickly than flat floors, because these elements are exposed to the fire from multiple sides. This means that although a ribbed floor shows less sagging than a flat floor with the same floor thickness, the longitudinal expansion of this ribbed floor will be significantly larger which increases the risk of shear failure of adjoining columns [62, 89]. However, one should keep in mind that a fire is usually present in a limited space. This means that continuous members exposed to a fire would not always lead to high expansions or stresses, because the beam is just partially exposed to a fire.

5.2.4 Columns

Thermal expansions could also affect columns. The combination of damage to the external layers of the columns and the reduction of the Young's modulus result in a reduction of the stiffness of these elements. This already makes a column more sensitive to buckling under fire conditions than under a normal room temperature. However, due to thermal expansion, the axial forces of the column will become bigger. Furthermore, when the column is not heated from all sides, the column will bend due to the thermal expansion as a result of the temperature difference (ΔT_b) [67]. This means that due to the thermal expansions, the risk of buckling becomes much bigger. Especially in case of long, slender columns, this should be taken into account [62, 89].

Ghent, 1983

Linköping, 1996

Figure 36 | Shear failure in the column and at the top of the column due to axial restraint [91]

As was already told in paragraph 5.2.1, cast in-situ elements usually have a bigger amount of concrete cover than precast elements, which leads to a higher reserve capacity. This also holds for columns. However, precast concrete columns can be constructed as multi-storey elements with a constant cross-section (practically only applied in parking garages), while the cross-section dimensions of cast in-situ columns are more likely to change every several floors [89]. This causes a greater reserve capacity to the upper storey columns of a precast structure. Besides, continuous columns can offer a secondary stress path. A disadvantage, however, is that continuous columns will obstruct the thermal expansions of the horizontal members of the structure, causing large horizontal forces which could lead to shear failure such as the previous examples in Ghent and Linköping [62, 89].

5.2.5 Cooling phase effects

In addition to the thermal expansions caused by the heat, the cooling phase can influence the loadbearing capacity of a concrete structure as well. Surviving structural concrete elements which had expanded and sagged under the influence of heat, will begin to cool down and contract in the cooling phase [89]. This contraction pulls inwards on the adjacent structure. If concrete members had thermally induced compressive loads that were sufficient to cause plastic strains to develop, these members may experience tension when the contraction during the cooling phase reduces and possibly reverses the compressive loads on the adjoining members. In combination with the other effects of the thermal expansions, the strength reduction of steel and concrete, and spalling, this may cause stresses that exceed the normally expected level in concrete elements, especially for steel reinforcement details at the connections [89].

6 BUILDING STANDARDS

As was told in 4.6.1.3, the first Dutch concrete regulations, the "Gewapend Beton Voorschriften (GBV)" of 1912, already contained values for a minimum concrete cover of slabs, beams, and columns. The application of the cover was stimulated by the knowledge that the steel bars had to be protected from exposure to fire [72]. However, despite this early development concerning concrete structures exposed to fire, the GBV of 1962 (the last GBV which was published fifty years after the first one and which was used until the mid-seventies) did not contain calculation methods for concrete structures in case of a fire yet [72].

The next Dutch series of concrete regulations, known as the "Voorschriften Beton (VB)", which were published between 1974 and 1983, did not contain any fire calculation methods either [18]. The design of a concrete structure was completely determined according to the situation at room temperature. The regulations only stated that for concrete surfaces which are exposed to high temperatures for a long period, the concrete cover associated with an aggressive environment should be adopted. The safety control of structures in case of fire was only based on several prescribed measures, which followed from various fire tests. One should think at typical rules such as the already mentioned concrete cover, minimum dimensions of the cross-section, and the placement of the reinforcement.

In the eighties, a significant new development in the Dutch building regulations took place. Politicians had decided that there must be clear performance standards with unambiguous determination methods, which could be used to assess whether the performance requirements are met. These performance requirements were established in the Building Decree, as mentioned in paragraph 3.2.1. To comply with the principle of uniformity, the major operation "Building Decree and Standards" was launched [18]. All the standards, to which the Building Decree would refer, were revised drastically. This led to the "NEN 6720 Voorschriften Beton – Constructieve eisen en rekenmethoden (VBC 1990)" [92]. This standard formed (together with some other standards) the successor of the VB-series. In 1991, the VBC 1990 was supplemented with the NEN 6071, titled as "Determination by calculation of the fire resistance of building elements - Concrete structures" [92]. This was the first Dutch building standard, which contained calculation methods for concrete structures to show whether the fire resistance requirements were met. The simplest calculation method led to the roughest estimation of the bearing capacity in case of fire, while performing the most complex calculation was rewarded with the most accurate estimation [13].

Nowadays, one uses the European standards, called the "Eurocodes". Mainly based on these Eurocodes, the performance criteria concerning the structural design are assessed. In this process, the following steps are taken:

1. Consider a relevant fire scenario;
2. Choose an appropriate design fire;
3. Calculate the temperature distribution;
4. Calculate the effects of all the mechanical actions;
5. Verify the fire resistance.

These steps will be discussed one by one in the next paragraph.

6.1 Fire design procedure

6.1.1 Step 1: The design fire scenario's

The ***design fire scenario*** is defined in the NEN-EN 1991-1-2 as "a prescribed fire scenario on which the analysis is performed" [93]. A ***fire scenario*** is a qualitative description of the development of a fire in time, indicating which important events characterise the fire, and distinguishing the certain fire from other possible fires. Usually, these are the growth phase, the fire phase, and the decay phase of the fire development as mentioned in paragraph 2.2, with respect to the surrounding area and the installations which may influence the development of the fire [68]. The choice of a specific fire scenario should be based on a fire risk assessment, taking into account the possible ignition sources and the available fire suppression systems. The fire scenario should be defined by an expert, or could be prescribed by the National Annex of the NEN-EN 1991-1-2, which is not the case in the Netherlands [93].

Due to the large amount of factors which can determine the occurrence and the development of a fire, it is possible to set up an infinite amount of fire scenario's. Only the scenario's with substantial consequences, the "credible worst case scenario's", are taken into account [94].

6.1.2 Step 2: The design fire

The definition of a ***design fire*** is given in the NEN-EN 1991-1-2 as "a specific fire development, assumed for design and analysing purposes" [93]. In other words, a design fire is a fire model as described in paragraph 2.4. There, it was already explained that there are two groups of fire models, the nominal and the natural fire models. The nominal fire models only show the temperature development of a fire as a function of time, while natural fire models provide a more realistic approach of a fire by taking into account other physical parameters as well. According to the Eurocode, the standard fire can be assumed as the design fire for structures for which the government imposed requirements, unless stated otherwise [93]. The ***standard fire*** is a design fire, based on the standard fire curve, which is known as the most common nominal fire model.

The design fire should only be considered in one fire compartment of the building at a time, unless the design fire scenario mentions otherwise [93].

6.1.3 Step 3: Thermal calculation

After a fire model is chosen, the temperature of the structural elements as a function of time needs to be determined [94]. This can be done by using Annex A of NEN-EN 1992-1-2. This annex shows the calculated temperature distribution for slabs, beams, and columns with a siliceous aggregate, which have been exposed to a standard fire until the time when the gas temperature has reached the highest value [67]. These temperature distributions are conservative for most other types of aggregates. In addition to the use of this annex, the temperature distributions can be determined from tests or by calculation. Therefore, the net heat flux to these elements is used, along with the thermal material properties of the elements and of any protective layer [94]. The ***net heat flux*** is defined as "the energy absorbed by elements per unit of time and area", and should be determined as a function of the heat transfer by convection and radiation [93]. For the thermal calculation, it is necessary to take into account the location of the design fire with respect to the structural elements, as well as the way of exposure to the fire of the elements.

The period of time for which the thermal calculation should be performed, depends on the chosen design fire. In case of a nominal fire model, the calculation of the structural elements should be carried out for a prescribed period of time, without a decay phase. In the Netherlands, this specified period of time is included in the Building Decree. In case of a natural fire model, the thermal calculation should be performed for the complete duration of the fire [67, 93, 94].

The fire load density, which was described in paragraph 3.2.5.5, can be relevant for the determination of the temperature distribution as well [94]. For example, when is chosen for a compartment fire in step 2, it is required that the gas temperatures are determined based on physical parameters, with a consideration of at least the ventilation and the fire load density. This fire load density can be determined according to the NEN 6090.

6.1.4 Step 4: Mechanical calculation

By means of the mechanical calculation, the combined effects of all the mechanical actions during a fire are determined. For this calculation, the same period of time should be used as for the thermal calculation. The results of the mechanical calculation must be used to verify whether the fire resistance requirements are met. Usually, it is checked whether in each cross-section

$E_{d,fi}(t) \leq R_{d,fi}(t)$ for $t \leq t_{fi,req}$ [18, 93] (1)

in which

$E_{d,fi}(t)$ is the design value of the considered action-effect at the time t, including indirect actions

$R_{d,fi}(t)$ is the design value of the corresponding resistance in the fire situation at the time t

t is the considered period of time since the beginning of the fire

$t_{fi,req}$ is the required fire resistance in minutes (performance criterion R).

This can be done on a level of a structural element, a part of the structure, or the complete structure. For the monitoring of standard fire resistance requirements, an analysis of elements is generally sufficient. In this type of analysis, indirect fire loads does not need to be taken into consideration, except those which are the result of temperature gradients. ***Indirect fire loads*** are defined as "internal forces and moments, which are caused by thermal expansion" [93]. When a part of the structure or the complete structure is analyzed, the indirect fire loads have to be taken into account [18, 93].

6.1.4.1 Action-effects

A fire needs to be considered as an accidental situation. So to calculate the action load, the accidental load combination (equation 6.11b of NEN-EN 1990) is used. This equation reads:

$$(E_{d,fi,t} =) \sum_{j \geq 1} G_{k,j} + P_k + A_d + \psi_{x,1} Q_{k,1} + \sum_{i > 1} \psi_{2,i} Q_{k,i} \quad (2)$$

in which

$+$ can be read as "in combination with"

G_k is the permanent load (self-weight)

P_k is the characteristic value of a prestressing force

A_d is the dominant accidental force

$Q_{k,1}$ is the main variable load

$Q_{k,i}$ are the other variable loads.

In the Dutch National Annex of NEN-EN 1990, it is shown that the partial factors γ_G, γ_Q, and γ_P in the accidental load combination are equal to 1 [48]. This also holds for the partial factors of the materials concrete γ_c and steel γ_s. The value of the variable load $Q_{k,1}$ may be represented by the frequent value $\psi_{1,1}$ or the quasi-permanent value $\psi_{2,1}$. This choice is specified in the same National Annex. The Dutch National Annex states that only for the wind load in combination with the fire load concerning the assessment of the disproportionate damage according to the NEN-EN 1991-1-7, the factor $\psi_{1,1}$ is used [95]. In all other cases, the quasi permanent value $\psi_{2,1}$ is prescribed. Table NB2 – A1.1 of the National Annex shows that the value of $\psi_{2,1}$ for snow loads, rainwater loads, and temperature loads (which do not concern a fire), equals 0. This means that for a calculation of the fire resistance, these loads do not need to be taken into account. The value of $\psi_{2,1}$ for wind loads is also 0, but this load can be included as the main variable load with $\psi_{1,1}$ = 0.2, because for the wind the factor $\psi_{1,1}$ instead of $\psi_{2,1}$ should be used. Therefore, only the imposed loads in the buildings (which depend on the categories of use) and the wind load need to be included as variable loads in the calculation of the action-effect [15, 18, 48, 95].

The accidental force A_d refers to the indirect actions as a result of restrained external or internal deformations. Internal deformations which are restrained may cause tensile stresses in the center of a cross-section, while restrained external deformations can cause bending moments. This can lead to damage of the concrete, even in elements which are not directly exposed to a fire. In paragraph 4.5, this phenomenon has already been explained extensively. Because an analysis of elements (in which the indirect fire loads do not have to be taken into account) is generally sufficient for the monitoring of standard fire resistance requirements, the accidental force A_d is often equated to 0 [18].

If the indirect fire loads do not have to be considered explicitly, it is allowed to determine the action effects by load combinations at the time t = 0 [93]. The action-effect $E_{d,fi}$ can be calculated as a constant for the whole duration of the fire. In addition, the action-effects may be derived from those which are determined for the design and calculation at a normal temperature. So instead of equation (2), one could make use of [93]:

$$E_{d,fi,t} = E_{fi,d} = \eta_{fi} E_d \tag{3}$$

in which:

E_d is the design value of the action-effects of the application of the fundamental load combination

$E_{d,fi}$ is the corresponding constant design value in the fire situation

η_{fi} is the reduction factor, determined according to the fire design sections (NEN-EN 1992-1-2 in case of a concrete structure).

The NEN-EN 1992-1-2 provides formulas to determine the reduction factor, which depend on the chosen load combination. For load combination 6.10 holds [67]:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{4}$$

and for the load combinations 6.10a and 6.10b holds the smallest value of the following two expressions:

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{5}$$

$$\eta_{fi} = \frac{G_k + \psi_{fi} Q_{k,1}}{\xi \gamma_G G_k + \gamma_{Q,1} Q_{k,1}} \tag{6}$$

where

γ_G is the partial factor for a permanent load

$\gamma_{Q,1}$ is the partial factor of variable load 1

ψ_{fi} is the combination factor for frequent or quasi-permanent values

ξ is a reduction factor for unfavorable permanent loads

The load combination given by equation 2 clearly leads to action-effects $E_{d,fi}$ which are lower than the corresponding values E_d which hold for a permanent design situation. This is caused by the fact that in the last case $\gamma_G > 1, \gamma_Q > 1$, and $\gamma_P < 1$ [18]. In addition, the values of ψ_1 for buildings lie between 0.5 and 0.9, and the values of ψ_2 for buildings lie between 0.3 and 0.8. These last values should be compared to the values of ψ_0, which hold for the load combination of the permanent design situation and which lie between 0.7 and 1.0 [19, 67]. Due to these differences, the value of the reduction factor η_{fi} lies between 0.3 and 0.7, which is shown in Figure 37. It is therefore allowed to use a recommended value of η_{fi} = 0.7 as a safe simplification [19, 67].

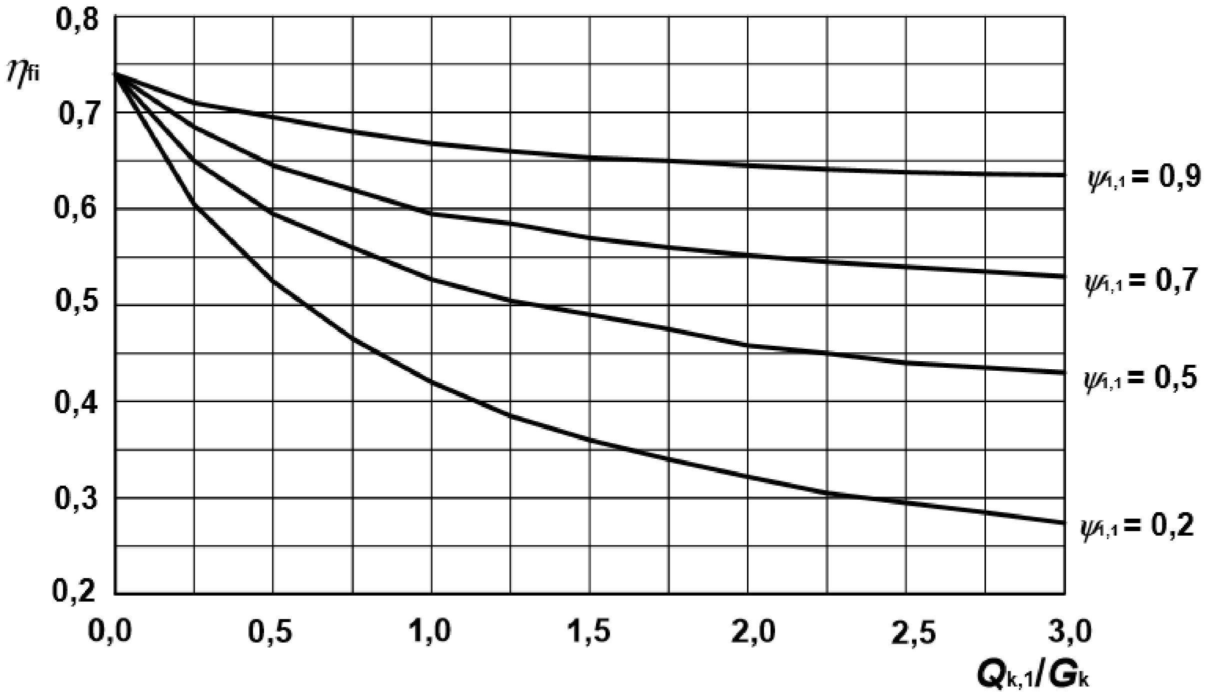


Figure 37 | Examples of the course of the reduction factor η_{fi} as a function of the ratio between the permanent load and the dominant variable load for different values of $\psi_{1,1}$, given by the NEN-EN 1992-1-2. Assumed is that $\gamma_G = 1.35$ and $\gamma_Q = 1.5$ [67].

6.1.5 Step 5: Verification of the fire resistance requirements

The last step of the fire design procedure contains the verification of the fire resistance requirements, according to equation 1. This verification can be performed in the following ways [13]:

- by using design solutions in terms of tabulated data;
- by using simplified calculation methods for specific elements;
- by using advanced calculation methods in which accurate thermal and mechanical analyses of elements, a part of the structure, or the complete structure are carried out .

6.1.5.1 Tabulated data

The NEN-EN 1992-1-2 contains tables which give minimum values for the cross-sectional dimensions and for the axis distances of the main bars for fire resistance values up to 240 minutes [13, 67]. These minimum dimensions are determined in such a way, that the use of these tables always leads to $E_{d,fi} \leq R_{d,fi}$. The values are based on equation 3 with a reduction factor of η_{fi} = 0.7, unless it is stated otherwise. The tables are only applicable for the verification of individual elements. An advantage of the use of the table values is that the designer can see very quickly whether the dimensions of the elements, which followed from a calculation at normal temperature, can lead to problems at high temperatures.

The tables have been developed on an empirical basis, general experience and theoretical evaluation of tests, based on the standard fire curve [18, 19]. The values are applicable to concrete with siliceous aggregates. If limestone or lightweight aggregates are used, the minimum cross-sectional dimensions can be reduced by 10% [67].

Explosive spalling for all environmental classes has been taken into account by the table values [18, 19]. This basically means that when the table values are used for the dimensions and axis distances of a concrete element, this element does not require further verification. The same holds for the verification of shear, torsion and anchorage.

The minimum axis distance (a) is defined as the distance from the axis of the main reinforcement (or other reinforcement that could be relevant in a certain failure mechanism) to the nearest concrete surface, and is related to the minimum concrete cover (c) [67]. This relationship is generally known as $a = c + \emptyset/2$, where $\emptyset$ is the diameter of the (main) reinforcement bar. The concrete cover of the reinforcement which lies in the tensile zone of the concrete element, was calculated based on the critical steel temperature θ_{cr} = 500 °C. ***The critical steel temperature*** is the temperature of the reinforcement at which the failure of the member is expected to occur under fire conditions for a given level of steel stress $\sigma_{s,fi}$ [18, 19]. This level of stress is based on [67]:

$$\sigma_{s,fi} = \frac{E_{d,fi}}{E_d} \cdot \frac{f_{yk}(20°C)}{\gamma_s} \cdot \frac{A_{s,req}}{A_{s,prov}} \tag{7}$$

where

$f_{yk}(20°C)$	is the characteristic yield strength of the reinforcement steel at normal temperature
$A_{s,req}$	is the required surface of reinforcement steel
$A_{s,prov}$	is the provided surface of reinforcement steel

If one suggests that $E_{d,fi} = 0.7\, E_d$ (according to the value of η_{fi}), $\gamma_s = 1.15$, and $A_{s,req} = A_{s,prov}$, the value of the steel stress will be $\sigma_{s,fi} = 0.6\, f_{yk}$. According to curve 1 in Figure 39, where the factors $k_s(\theta_{cr})$ and $k_p(\theta_{cr})$ are defined as $k_s(\theta_{cr}) = f_{yk}(\theta_{cr})/f_{yk}(20°C)$ and $k_p(\theta_{cr}) = f_{pk}(\theta_{cr})/f_{p0.1k}(20°C)$, this result corresponds to the critical steel temperature of 500 °C [68].

While one uses the minimum values according to the tables of the NEN-EN 1992-1-2, it is important to keep in mind that these minimum values are mainly required for the fire resistance. It is possible that these values are lower than the required values according to the NEN-EN 1992-1-1, which take into account other aspects as well [18, 19, 67].

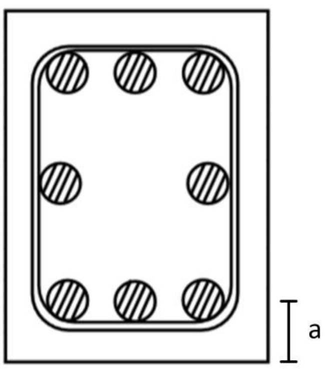


Figure 38 | The minimum axis distance *a* [18]

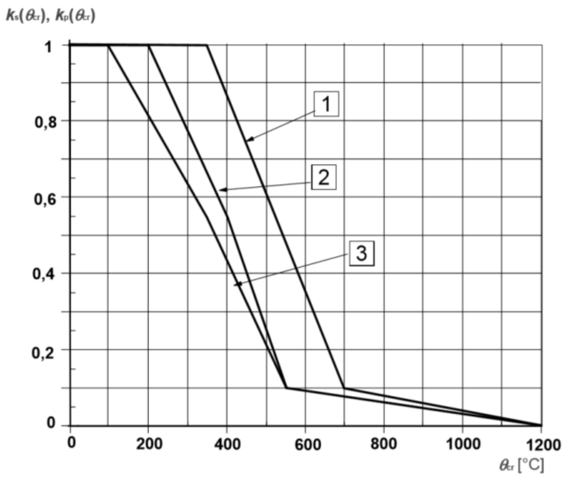


Figure 39 | Reference curves of the NEN-EN 1992-1-2 for the critical temperature of reinforcing and prestressing steel with the reduction factor $k_s(\theta_{cr})$ or $k_p(\theta_{cr})$ [67]

6.1.5.2 Simplified calculation methods
Simplified calculation methods are used to determine the ultimate load-bearing capacity of a heated cross-section and to compare the capacity with the relevant load combination. It shall be verified that the design action-effect for the fire situation $E_{d,fi}$ is less than or equals to the corresponding design resistance in the fire situation $R_{d,fi}$. These methods are mainly applied for individual elements, which means that indirect fire loads do not have to be considered [19].

Three methods that are often used, are the "500 °C isotherm method" and the "zone method", which both can be found in Annex B of the NEN-EN 1992-1-2, and the method of Annex E for the calculation of the resistance of beams and slabs subjected to fire [19, 67].

6.1.5.2.1 The 500 °C isotherm method
This method is applicable to a standard fire exposure and any other fire model, which cause similar temperature fields in the fire exposed member [18, 19]. The method is based on the assumptions that in a cross-section of an element, concrete with a temperature above 500 °C is damaged in such a way, that it does not contribute to the bearing capacity of the element anymore, while concrete below this temperature will still have its initial strength (the strength at a temperature of 20 °C). This leads to a reduced cross-section. Reinforcement bars that lie outside this reduced cross-section are still taken into account in the calculations, but with strength values which correspond to their actual temperature. With these assumptions, the calculation of the resistance of a cross-section in a fire situation is reduced to a calculation at normal temperature [67, 94].

6.1.5.2.2 The zone method
This method is more laborious, but more accurate, than the 500 °C isotherm method [19, 94]. In contrast to the 500 °C isotherm method, this method is applicable to the standard fire curve only. The method also works with a reduced cross-section, but this reduced cross-section is determined in a different manner. First of all, the cross-section is divided into several parallel zones with an equal thickness. Then the temperature in the middle of each zone is calculated and the corresponding reduction factors for the compressive strength are determined. Based on these reduction factors, the width of the damaged zone of the element is calculated, which leads to the reduced cross-section. From here, one could proceed with the calculation of the resistance with the reduced cross-section as in the 500 °C isotherm method [19, 67, 94].

6.1.5.2.3 Annex E
The calculation method of Annex E provides an extension to the use of the method with the tabulated data for (continuous) beams that are exposed to fire on three sides, and for slabs. The method determines the effect on the bending resistance for situations where the axis distance a to the bottom reinforcement is less than that required axis distance which is given by the tables. It can be used to justify a reduction of the axis distance which followed from these tables [19, 67].

6.1.5.3 Advanced calculation methods
Advanced calculation methods should provide a realistic calculation of structures exposed to fire. They may be used for an analysis on the level of a structural element, a part of the structure, or the complete structure [94]. In case of an analysis of a part of the structure or the complete structure, indirect fire loads have to be considered. Furthermore, the continuous changes of mechanical and thermal properties of the materials (concrete and steel), together with their influence on each other and on the complete structure, need to be taken into account in every case. Each potential failure

mode that has not been included in the advanced calculation method (such as insufficient rotational capacity or spalling), must be ruled out in advance by taking appropriate measures. Any fire curve could be used for these methods, as long as the material properties are known for the relevant temperature range [67].

6.1.6 Verification of the fire resistance requirements according to the former building standards NEN 6720 and NEN 6071

As has been discussed in the previous paragraphs, the verification of the fire resistance requirements nowadays can take place based on the table values or the calculation methods which are included in the EN 1992-1-2. However, a method which is not mentioned yet, is the determination of the fire resistance in an experimental way [16]. This can be done by using the NEN 6069. The NEN 6069, "Testing and classification of resistance to fire of building products and building elements", is a standard which provides methods for testing and classifying of the fire resistance of building components and building products in the Netherlands. The choice between assessing the fire resistance by calculation or in an experimental way, usually depends on the applied materials and the complexity of the structure. For non-standard construction materials, insulation systems, or new combinations of materials (for example, weight-saving elements in concrete, such as plastic balls or Styrofoam elements), there are no standardized calculation rules (yet). Neither is it already possible to calculate the deformation behavior of separating structures. In these cases, testing is the only way to determine the fire resistance. Besides, a calculation of a structure in which a lot of materials are combined, can become very expensive. However, for standard load-bearing elements such as floors and columns, it is generally advantageous to determine the fire resistance by using the calculation methods mentioned in the standards [16]. For this reason, the NEN 6069 is – and will not be – discussed in this thesis.

In the nineties, before the introduction of the Eurocode, there were also three ways to verify the fire resistance requirements, namely [96]:

- examination of the detailing provisions which were included in the NEN 6720 (VBC 1990);
- calculation of the fire resistance in accordance with the calculation methods which were mentioned in the NEN 6071;
- testing the building structure according to the NEN 6069.

The detailing provisions in the NEN 6720 consisted of several conditions and rules of thumb which were related to various structural parts. Besides, the NEN 6720 also contained tables with minimum reinforcement distances, just like the current tables in the EN 1992-1-2. Application of these detailing provisions was usually sufficient, just as the application of the tables in the EN 1992-1-2 is mostly enough in current situations.

In special cases, the calculation methods of the NEN 6071 could be used [13]. As was told in the introduction of this chapter, the NEN 6071 was the first Dutch building standard, which contained calculation methods for concrete structures to show whether the fire resistance requirements were met. The basics of these calculation methods do not differ significantly from the current methods in the EN 1992-1-2. In both standards, the material properties in relation to the ambient temperature are specified and data is given for the determination of the temperature in the concrete structure. However, the NEN 6071 is limited to a fire resistance requirement of 120 minutes, while the

Eurocode provides calculation methods which make it possible to examine a structure on a fire resistance up to 240 minutes [13]. Besides, the tabulated data in Annex A of the NEN 6071 which shows the minimum reinforcement distances (which are the same as the tables in the NEN 6720), is extended in the EN 1992-1-2 [13]. For example, Annex A of the NEN 6071 only shows one simple table for columns, while the EN 1992-1-2 give nine different tables where the columns are sorted by reinforcement ratios and first order moments. And finally, the EN 1992-1-2 has also added a couple of rules which hold for high-strength concrete [13].

So in short, the different ways of verification of the fire resistance requirements according to the current building standards are broadly in line with the methods which are given by the standards NEN 6071 and NEN 6720, but they gradually have taken more shape.

6.2 Summary of the current relevant building standards

In the description of the fire design procedure in the previous section, the building standards NEN 6090, NEN 6069, EN 1990, EN 1991, and EN 1992 (which is used instead of the former NEN 6071 and NEN 6720 nowadays) were mentioned. However, Section 2.2 of the Building Decree refers to another building standard. This is the NEN 8700 [26]. The NEN 8700, "Assessment of existing structures in case of reconstruction and disapproval", establishes the principles, rules of application and determination methods with regard to safety, serviceability and durability of renovations and existing structures [50]. The Building Decree refers to this standard to determine the level of the fire resistance requirements for alterations or renovations, and for the load combinations which have to be considered in case of the performance level of alteration or renovation and the performance level of existing buildings. Because this standard forms the basic principles for the structural assessment of existing buildings and is relevant for the performance level of alteration or renovation as was mentioned earlier in paragraph 3.2.7, this standard will be discussed in the remainder of this chapter.

Table 17 | Building standards which are referred to in Section 2.2 of the Building Decree [26]

Building standard	Required for
NEN-EN 1990 and NEN-EN 1991, or NEN 8700 and NEN 8701	the load combinations which have to be used in case of fire
NEN-EN 1992 NEN-EN 1993 NEN-EN 1994 NEN-EN 1995 NEN-EN 1996 NEN-EN 1999	the determination of the fire resistance in a computational way
NEN 6069	the determination of the fire resistance in an experimental way
NEN 6090	the determination of the permanent fire load density of the fire compartment

6.3 The NEN 8700

6.3.1 Introduction

An important starting point of the evaluation of existing structures, is that the assessment of existing structures will be based on the regulations in force at the moment of the assessment. In the Netherlands, these are the Eurocodes. However, it is allowed to apply certain reductions in safety margins or security for economic reasons. This prevents that an entire existing structure has to be adapted due to a number of slightly more stringent requirements.

Before the introduction of the NEN 8700, it was already possible to deviate from the severe requirements. According to the Building Decree of 2003, a municipality was allowed to grant an exemption (Article 1.11 (2)[97]). This authority disappeared since the introduction of the Building Decree of 2012. As was mentioned in paragraph 3.2.7, this adaption was carried out because of the fact that the government was not satisfied about the way this authority was used. The number of granted exemptions was very low, causing delays or impositions of requirement levels which were strictly speaking not necessary. That is why in the Building Decree of 2012, a specific requirement level for buildings which would be altered or renovated was implemented. This encourages to deviate from the requirements for new buildings, because a clear degree of deviation is provided [47]. For the same reason, it was decided to write a series of standards for the assessments of existing buildings.

This series of standards, the NEN 8700-series, is specifically intended to be used in conjunction with the series of standards EN 1990 up to EN 1999, to assess whether [50]:

- a renovation or alteration of an existing structure has a sufficient degree of sustainable safety and usability;
- an existing structure still has a certain (preselected) performance level with regard to sustainable safety and usability;
- an existing structure should be disapproved.

It contains the safety levels of alteration or renovation and disapproval, as well as the principles of the assessment of the structural safety. It does not contain elaborated methods of determination to establish the current or future safety levels. For these elaborations, reference is made to other documents of the NEN 8700 series, which are still in development at the moment of writing this thesis [50].

An important point to note is that the Building Decree refers to this standard to determine the level of the fire resistance requirements for alterations or renovations, and for the load combinations which have to be considered in case of the performance level of alteration or renovation and the performance level of existing buildings (as was mentioned in the former paragraph). The method of determination of the fire resistance can be found in the EN 1992. However, in case of the performance level of existing buildings, reference is made only to the NEN 6069 and not to the EN 1992. This means that if one uses the NEN 8700 in combination with the series of standards EN 1990 up to EN 1999 to determine the fire resistance concerning the performance level of existing buildings, this falls under the principle of equivalence. In these situations, as mentioned in paragraph 3.2.7.1 and 3.2.8, the authorities may impose higher requirements, based on the advice of the fire brigade.

6.3.2 Field of application of the NEN 8700

6.3.2.1 Alteration or renovation

Although the NEN 8700 is introduced for existing buildings, it does not mean that it has to be applied in every situation of alteration or renovation. According to the NEN 8700, the requirements for new buildings hold for alteration or renovation, unless 15 years has elapsed. Only after this period, derogations are permitted [50]. While applying these derogations, the standard assumes that one is guided by administrative and economic principles, which refers to the principle of proportionality as mentioned in paragraph 3.2.7. Consequences of the provisions which are to be implemented, should be weighed against the added value of the safety and the usability. The consequences may include necessary investments, but also a decrease of the usability, unreasonable considerations of the quality of the building, and the future plans. All these consequences must be proportional to the revenues in safety and usability. The standard also assumes that one does not consciously divide a large structural operation into a series of small operations, which are each in itself administratively and economically accountable, but not as a whole [50, 98].

To know whether one should use the NEN 8700, it is important to understand the term ***alteration or renovation***. The Building Decree defines this term as "totally or partially renew, change or enlarge a building". These forms of alteration or renovation are shown in Figure 40 [51]. For totally renewing a building, one should think of a situation in which a new building is constructed, after an existing building is demolished down to the foundation due to an incident (a). If a building is rebuilt after it is stripped to the shell, or when some parts of a building are replaced, one speaks of a partial renovation (b). Changing a building refers to adjustments to a building which does not influence the contours of a building, such as internal alterations or the placement of apartments in an office building (in case this operation requires structural measures)(c). In case of enlarging a building, the contours do change because the size of the building increases. Examples of enlargements are the construction of an extension or the addition of extra floors (d).

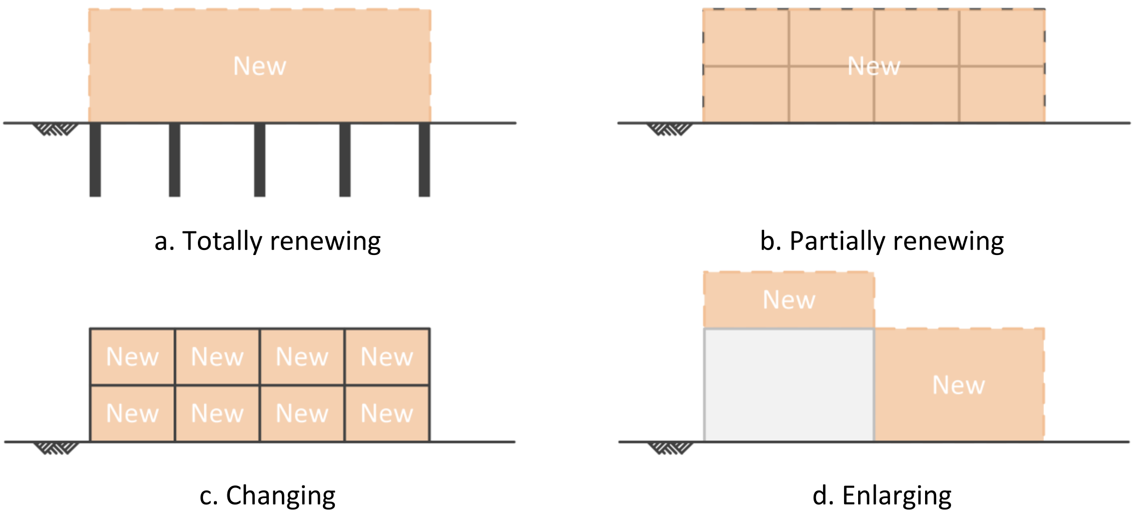


Figure 40 | Forms of alteration or renovation

The Building Decree mentions that to totally renew a building, the requirements for new buildings must be met. All other forms of alteration or renovation may comply with the requirement level of alteration or renovation. It does not matter whether an extension to an existing building is much bigger than the existing building itself: if one is not completely renewing a building and the first 15 years of the design life have passed, one is allowed to apply the NEN 8700 for the requirement level of alteration or renovation [26, 51].

According to Article 4 of the Housing Act, this requirement level only has to be applied on the parts that are being (re-)constructed (as was explained earlier in paragraph 3.2.7). If this was not the case, the level should be applied on the whole structure. This would lead to the advantage that the whole assessment procedure would be easier to implement. However, the consequence would be that structural parts which still fulfill the requirements of existing buildings, need to be repaired. This is why in the background report of the NEN 8700, also preference is given to apply the requirement level of alteration or renovation only on the parts that are being (re-)constructed [99].

6.3.2.2 Disapproval

The requirement level of existing buildings is the minimum requirement level, and is also known as the "level of disapproval" [26]. In paragraph 3.2.7, it has been described that a building should be disapproved when the requirements of this level are not met and that measures should be taken immediately. According to the NEN 8700, this only holds for existing buildings which are older than 15 years. If they are less than 15 years old, they should be assessed on the legally obtained level.

The NEN 8700 defines that a building is an ***existing building***, directly after the notification of completion of the construction. However, it is not intended that immediately after the commissioning of a building, this building only needs to meet the requirements for existing buildings. To prevent abuse of this definition, it is regulated by law that an environmental permit for the construction can be revoked when a planning application is incomplete or incorrect [50]. For example, one could think of a building which is used in another way as was mentioned in the planning application, shortly after the commissioning. Alteration or renovation could not directly be performed according a lower level of requirements, because of the minimum of 15 years in which a building needs to fulfill the requirements of a new building, which was mentioned in the previous paragraph [50].

6.3.2.3 Function change

Annex F.2 of the NEN 8700 describes how one should assess a building in case of a function change. This approach reads [50]:

- In case of a function change, the requirement level of new buildings counts as the target level. If the requirements of this level are met, no further action is required.
- If the requirements of this level are not met, and measures to still reach this level lead to disproportional high costs, then one should asses the building according to the requirements of the legally obtained level (which hold for the changed function).
- If the requirements of the legally obtained level are not met as well (and measures to still reach this level also lead to disproportional high costs), one could distinguish the following two situations:
    - If in case of the new function, the performance level of existing buildings can be met, no further action is necessary (unless the authority intervenes in accordance with section 13 of the Housing Act)
    - If structural measures are required because the demands of the performance level of existing buildings are not met, then these measures should fulfill the requirements of the level of alteration or renovation.

These principles are legally defined and discussed earlier in paragraph 3.2.

Annex F also mentions that if the legally obtained level is unknown, the building needs to fulfill at least the requirements of existing building. However, if the safety level is nearly above the level of disapproval, economic considerations and expectations about degradation could lead to direct repairs or other actions. And of course, the authority could still intervene based on section 13 of the Housing Act [46].

6.3.3 Safety levels

The risk of death due to an accident in the Netherlands (like falling down the stairs or a traffic accident) is about 10^{-4} per year [99]. The chance of becoming a victim of a structural calamity will not allowed to be higher. It is suggested that the maximum acceptable risk of becoming a victim of a structural failure equals 10^{-5} [99]. Because of the safety margins, most structures have more capacity than which is minimum required, causing a lower risk than the suggested risk of 10^{-5} [99].

For the purpose of reliability, ***consequence classes*** (CC) are defined by the assumed consequences of failure or malfunctioning of considered structures [98]. The NEN 1990 describes these consequence classes in qualitative terms with regard to mortal danger and economic damage. The classification into these consequence classes is a method for allowing moderate differentiation in the partial factors for loads and resistances.

The TNO background report of the NEN 8700 translated the qualitative terms of the consequence classes in risks of mortal danger (P_l), shown in Table 18 [99]. These risks are conditional, meaning that they are risks of situations where is assumed that a structure really fails. The risks relate to individuals who are regularly in or on a building; the number of people is not of importance in this approach. The TNO background report formulated a correlation between the risk of mortal danger and the risk of failure (P_g) as $P_g \cdot P_l < 10^{-5}$. With this correlation, the acceptable risk of failure could be calculated for each consequence class. These are 10^{-2}, $3{\cdot}10^{-4}$, and $3{\cdot}10^{-5}$ per year for consequence class 1, 2, and 3, respectively [99].

Table 18 | Consequence classes with qualitative and quantitative risks of mortal danger per year [98, 99]

Consequence class	Qualitative risk of mortal danger	Quantitative risk of mortal danger (P_l)	Examples of application
1A	Null	-	Agricultural buildings Greenhouses Light industrial buildings
1B	Very low	10^{-3}	Standard single-family houses Industrial buildings (one or two floors)

Consequence class	Qualitative risk of mortal danger	Quantitative risk of mortal danger (P_l)	Examples of application
2	Substantial	$3 \cdot 10^{-2}$	Housing Office buildings Public buildings Industrial buildings (three or more floors)
3	High	$3 \cdot 10^{-1}$	Highrise buildings Grandstands Exhibition spaces Concert halls Large public buildings

The safety level of a (part of a) structure could theoretically be defined as a risk of failure during a relevant time period. However, instead of working with these risks during the development of technical regulations, use is made of the ***reliability index β*** [99]. This index is in a direct relation with the risk of failure, which is shown in Table 19.

Table 19 | Relation between the reliability index and the probability of failure [99]

Reliability index β	Probability of failure P_g
1.0	0.16
2.0	0.023
3.0	0.0013
4.0	0.000032

The practical method to determine the desired safety level is based on the correct choice of the following (calibrated) variables [99]:

- the consequence class which includes the building type;
- the prescribed characteristic loads;
- the prescribed load factors γ_f and combination factors Ψ;
- the standardized calculation rules and the material properties;
- the prescribed material factors γ_m.

The load- and material factors are chosen in such a way, that the safety level expressed in β which corresponds to the concerned consequence class, is achieved [99].

The Eurocodes also allow the Member States to determine the safety level based on a probabilistic calculation under certain specified conditions. In the Netherlands, this probabilistic method was already recognized as the formal basis of designing in the past decades, which will remain that way in the future. However, this method is rarely used in practice, because it is too laborious and requires special knowledge [99].

6.3.3.1 Safety level of new buildings

The reliability indices for new buildings (β_n) are given by table B2 in the Eurocode EN 1990. These values are shown below in Table 20. In the Netherlands, the values of the fourth column appeared to be hardly feasible in practice when the wind loads are dominant. In such cases, lower values are applied which better reflect the reality. These values, given by Annex C of the EN 1990, are shown in the last column.

Table 20 | Reliability indices for new buildings with a design life of 50 years [99]

Consequence class	Consequences of failure		Wind not dominant	Wind dominant
	Risk of mortal danger	Risk of economic damage		
CC1	Null / very low	Small	$\beta_n = 3.3$	$\beta_n = 2.3$
CC2	Substantial	Substantial	$\beta_n = 3.8$	$\beta_n = 2.8$
CC3	High	High	$\beta_n = 4.3$	$\beta_n = 3.3$

All the reliability indices in the table are based on a design life of 50 years [99]. Based on economic motives, it is rational to use the same values for shorter periods. According to the relation between the reliability indices and the risk of failure, the risk of failure would increase in that case. This can be justified by the fact that an investment in the safety of a building is more profitable if one could benefit from it for a longer time.

However, in terms of human safety, one maintains a constant risk per year, regardless of the design life of a building. An increasing risk concerning human safety is simply impermissible. This leads to a higher β-value for shorter periods of time and a limit to the reduction of the period in which the reliability index can be kept constant. Based on the acceptable risks of failure per year which were mentioned earlier (which were 10^{-2}, $3 \cdot 10^{-4}$, and $3 \cdot 10^{-5}$ per year for consequence class 1, 2, and 3, respectively), the reliability indices can be expressed as a function of the considered reference period t in years in the following way [99]:

$$\beta_n = 2.3 - 1.10 \log t \quad \text{(CC1)} \tag{8a}$$
$$\beta_n = 3.4 - 0.75 \log t \quad \text{(CC2)} \tag{8b}$$
$$\beta_n = 4.0 - 0.60 \log t \quad \text{(CC3)} \tag{8c}$$

According to equation 8a and the values in Table 20, this criterion is never normative for CC1. But for CC2 and CC3, it could be decisive in case the wind is dominant. This is solved by the introduction of a so-called ***reference period*** apart from the design life. For the benefit of the structural safety, the design values of the loads and the strength should be determined by using this period. The minimum of the reference period is 15 years, which is based on the fact that after these 15 years, the criteria according to 8b and 8c could not rise above the values of Table 20. In other words, this is the mentioned limit from where the reliability index can be kept constant [99].

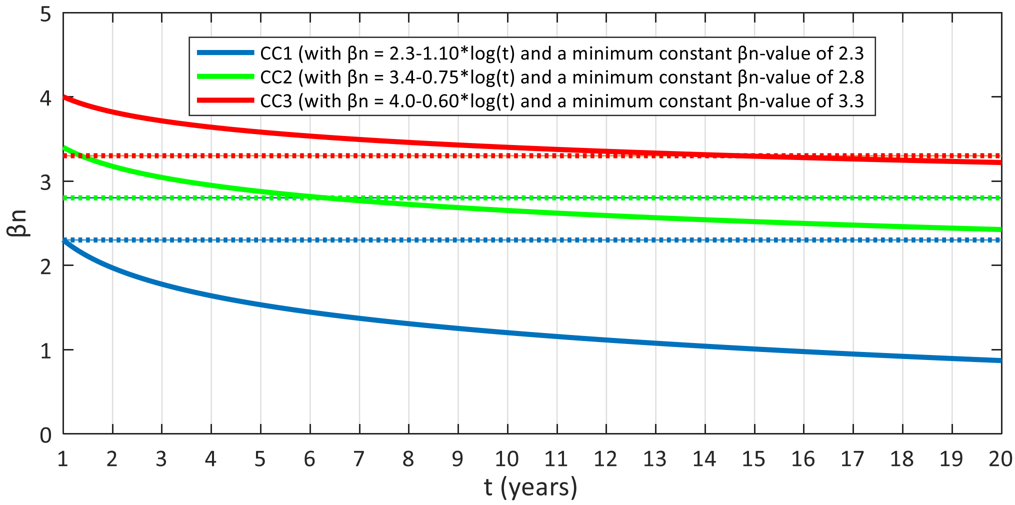


Figure 41 | Determination of the minimum reference period by studying the intersections of the β_n-equations and the minimum β_n values shown in Table 20

6.3.3.2 Safety level of existing buildings

The NEN 8700 applies lower reliability indices for existing buildings, because the assessment of existing buildings essentially differs from the assessment of new buildings on the following points [50]:

- Increasing the level of safety usually brings relatively more costs for existing buildings than buildings in the design phase;
- The period which the existing building has to maintain this safety level is often lower than the general design life of 50 years;
- For existing buildings, there is a possibility to find out more about the structure through measurements.

These aspects will be illustrated one by one in the following subparagraphs.

6.3.3.2.1 Economic aspect

What in fact already came forward in the previous paragraphs, is that one has two reasons to impose a reliability demand to a structure, namely, the economic motives and the human safety. The first consideration leads to an economic optimization of the sum of the cost of construction and the product from damage and risk of failure, while the other one seeks to limit the risk of mortal danger to a level that is significantly lower than the other risks which are faced by people in everyday life.

One should adhere to the strictest safety standards concerning these considerations. In case of existing building situations, one has much less structural possibilities than in case of new buildings [99]. Adjustments to improve the safety level are therefore very expensive. So in order to limit the costs, it is effective to apply a safety level as low as possible. However, the requirements concerning the risk of mortal danger are not lowered. This is why the human safety is nearly always the normative consideration for existing buildings, in contrast to new buildings.

Figure 42 shows the difference between the optimal reliability indices of new and existing buildings [99]. The minimization of the construction costs ($C_{construction}$) and the expected damage (P_fS) leads to a reliability index for new buildings of (for example) β_n = 3.8 [99]. The optimum of existing buildings will lie at a lower index (here β = 3.2), because alteration or renovation of an existing building to reach a higher safety level is much more difficult and expensive than a new building that only exists on paper. It also needs to be checked whether it is not more economical to maintain the existing situation and accept the higher risk. In other words, point A at β = 3.2 in figure Figure 42B should be lower than point B at β = 2.5. Because this is not the case, the situation of this graph implies that one should better not decide to alter or renovate the existing building.

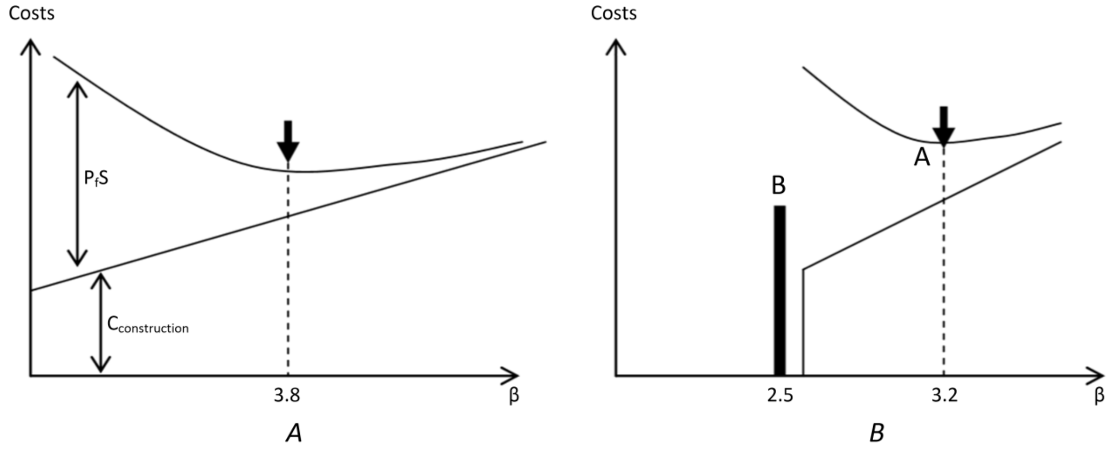


Figure 42 | Optimization of the construction and reparation costs [99]

6.3.3.2.2 Time aspect
For existing structures is often a shorter planning period applied than for new structures [99]. As was told in paragraph 6.3.3.1, a reliability index of 50 years which is used for a shorter period leads to an increasing risk of failure, which could be justified by the fact that an investment in the safety of a building is more profitable if one could benefit from it for a longer time. In this context, a shorter period of time could not be used as an argument for the reduction of the reliability index.

However, concerning the time aspect, the reduction can be explained in context of the lower risk of the occurrence of extremely high loads in a short period [50, 99]. The expected maximum values of variable loads can be reduced according to the shorter reference period, as mentioned in the NEN 8700. This reference period will be discussed in paragraph 6.3.3.5.

Aging also plays a role at the influence of the time aspect [50, 99]. The required safety must be met, including effects such as corrosion, carbonisation, and the wear and tear of materials. If the planned period of time is shorter, the spare capacity which is applied in order to take into account these aging-effects can therefore be smaller.

6.3.3.2.3 Data aspect
Experience has shown that during a realisation of a structural design, specifications can vary between certain boundaries [99]. One expresses this in terms of variation coefficients, which in turn have an impact on the safety factors that have to be applied. If the specifications which were used during the design of an existing building are still valid, the same design values can be used for the alteration or renovation of the building. However, in case of an existing building, one can determine these values more accurately through measurements. The values which follow from these measurements often lead to an increase of the designing strength, because several minimums for dimensions and material properties were used during the designing phase of the existing building.

It is also possible that the old specifications still exist, but there are reasons to doubt the quality of the implementation, or the old specifications are unknown. In this situation, one can apply the lowest possible strength, or perform measurements as well [50, 99].

Both situations imply that the design safety level is not just a fixed property of a structure. It depends on the available knowledge of the structure and the loads. This means that a lack of safety could be caused by a lack of knowledge, and that this lack of safety can be refuted by measurement values. Besides the application of measurements, this can also be done by applying more advanced calculations [50, 99].

6.3.3.3 Lower limit of existing building structures

Before the introduction of the first Building Decree, municipal instructions were mainly based on Article 307 of the model municipal building regulation (in Dutch: "Model-Bouwverordening"), called "Condition of a building" [99]. The criteria which were used for the lower limit of existing buildings were qualitative, such as insufficient maintenance or improper materials. The reliability indices for existing buildings could not be based on former regulations, in contrast to new buildings. For this reason, there is searched for principles by other means. The TNO background report of the NEN 8700 does not mention how this is done, but that the considerations result in $\beta_d \geq \beta_n - 1.5$ and $\beta_r = \beta_n - 0.5$, where β_d is the reliability index corresponding to the level of disapproval, β_n is the reliability index for new buildings, and β_r is the reliability index for buildings which are to be altered or renovated [99].

However, the report does state that the safety level which was realised with old regulations is taken into account in the determination of the lower limit of the level of safety. It is not reasonable that buildings which are still in a good condition are labelled as unsafe due to the introduction of new regulations. According to the report, the old safety level of new buildings ($\beta_{n,old}$) should be checked. The reliability index corresponding to the level of disapproval β_d should in any case not be higher than the value $\beta_{n,old}$, while for alterations or renovations it is reasonable to consider $\beta_d < \beta_r < \beta_{n,old}$ (which corresponds to the principle of the legally obtained level). Unfortunately, it did not seem to be possible to determine a value for $\beta_{n,old}$ [99].

So in short, the lower limits are roughly determined due to the fact that the safety levels which were formerly used are not known in quantitative terms. The use of the current limits should show whether these lead to more or less disapprovals. It is therefore conceivable that the NEN 8700 will be adapted in the future, based on new experience [99].

6.3.3.4 Remaining lifetime

The ***remaining lifetime*** of a structure is the remaining period during which a (part of a) structure has to meet a structural reliability level in accordance with the NEN 8700. This period equals the original design life minus the period in which the building is already in use, unless this value comes beneath the minimum value of 15 years. This minimum holds for buildings which are to be altered or renovated; for the assessment of disapproval, a remaining lifetime of 1 year should be assumed. In a remark in the NEN 8700 it is stated that instead of the minimum of 15 years, it would be better to assume a value of 30 years as lower limit in case of a renovation, but this remark is not motivated. A structural engineer, however, will be tend to choose for the shorter remaining lifetime, because this will lead to lower variable loads [50, 98].

6.3.3.5 Reference period

The ***reference period*** is a time period chosen and used to determine the values of the variable loads and (eventually) the accidental loads [50]. As was explained in paragraph 6.3.3.1, this period was introduced to prevent that the risk of human safety would become too high. For some variable loads, the EN 1991 contains rules to calculate the maximum values which correspond to this reference period which is shorter than the default of 50 years, such as snow or wind loads. When the EN 1991 does not give any rules for a specific load, the maximum values must be based on a general formula which is given in the NEN 8700. In case of alteration or renovation, the reference period must at least be equal to the remaining lifetime [50]. However, the term "remaining lifetime" should be clearly distinguished from "reference period", especially in case of a disapproval assessment. Here, the remaining lifetime amounts 1 year, but the reference period must still meet a minimum of 15 years. However, not every building is populated. It would be strange if one could not reduce the renovation costs by reducing the reference period because of a human safety requirement, while there are no persons in the building. For this reason, the consequence class 1 in the NEN 8700 got divided into 1A and 1B, where 1A refers to unpopulated buildings. Because the human safety risk in this class can be ignored, the minimum reference period for this class is 1 year instead of 15 [50, 98].

6.3.4 Verification and calculation of the limit states

Both for alteration or renovation and for the assessment of existing buildings, legal requirements are only taken for the ultimate limit state (ULS). However, the NEN 8700 states that for alteration or renovation, distinction must be made between ultimate limit states and serviceability limit states (SLS), in contrast to cases of the assessment of existing buildings. This is because the decision of disapproval of an existing building is mainly based on the risk of failure, and not on the usability of the building.

The EN 1990 distinguishes four different ultimate limit states which can lead to the loss of bearing capacity and are referenced by the NEN 8700 [48]:

- EQU loss of static equilibrium of the structure or a structural element, considered as a rigid body;
- STR failure or excessive deformation of the structure or structural elements (including the foundation), where the strength of the materials is governing;
- GEO failure or excessive deformation of the soil, where the strength of the soil is governing for the resistance ;
- FAT failure of the structure or structural elements because of fatigue;

Because this master research focuses on the fire resistance with respect to collapse, only the ultimate limit state STR will come up in the remaining part of this thesis.

The design and calculation situations for structures which are part of an alteration or renovation, should be distinguished as follows, taking into account the principle of proportionality [98]:

- permanent design and calculation situations, which refer to the conditions of normal use;
- temporary design and calculation situations, which refer to temporary conditions which are applicable to the structure during construction or renovation for example;

- accidental design and calculation situations, which refer to exceptional circumstances which are applicable to the structure or to which they are exposed, like fire, explosions, or local collapse.

The verification situations for existing structures which are assessed to verify whether the level of disapproval is not reached, should be distinguished in a slightly different way [50, 98]:

- permanent verification situations, which refer to the conditions of normal use;
- accidental verification situations, which refer to exceptional circumstances during the exposure of the building to a fire.

So there are no temporary verification situations for existing buildings which are assessed. This can be explained because this assessment does not involve construction or renovation operations. When the level of disapproval is not met, these operations are necessary, but they need to be carried out on the level of alteration or renovation.

Another difference can be seen between the accidental situations. These accidental situations are legally obliged to be considered since 1992 [99]. Before this year, there were no accidental situations included in any regulation, except fire. It was stated that local damages should not have catastrophic consequences, but it was not indicated how that should be achieved. Nowadays, economically acceptable solutions to accidental situations can usually be found for new buildings. However, this is much more complicated for existing buildings. Here, significant structural modifications can be necessary, if there is no secondary stress path. Given the small chance of accidental loads, it does not seem acceptable to require these modifications for large numbers of existing buildings. For this reason, the assessment of safety in relation to disapproval in terms of the accidental loads is limited to only the fire situation, and the safety for alteration or renovation in terms of these accidental loads are limited to the situations which were observed in the original design [50, 98, 99].

6.3.5 Load combinations

In case of permanent design situations of new buildings, to determine whether a limit state is exceeded, the representative values of the loads has to be multiplied by the partial load factor γ and a combination factor Ψ in the fundamental load combinations 6.10a and 6.10b, according to the EN 1990 (this has partly been covered in paragraph 6.1). The equations of these combinations read [48]:

6.10a: $$\sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} \Psi_{0,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (9a)$$

6.10b: $$\sum_{j\geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (9b)$$

The same approach is used for permanent verification situations of existing buildings according to the NEN 8700. The combination factors can be found in Annex A of the NEN 8700 and are shown in Table 21. These values correspond to the values given by the National Annex of EN 1990.

Table 21 | Ψ-factors for buildings, according to the NEN 8700 and the EN 1990 [50, 95]

Loads	Ψ_0	Ψ_1	Ψ_2
A. living spaces	0.4	0.5	0.3
B. office spaces	0.5	0.5	0.3
C. conference spaces	0.6/0.4[a]	0.7	0.6
D. shopping areas	0.4	0.7	0.6
E. storage areas	1.0	0.9	0.8
F. traffic areas (vehicle weight ≤ 30 kN)	0.7	0.7	0.6
G. traffic areas[b] (30 kN < vehicle weight ≤ 60 kN)	0.7	0.5	0.3
H. roofs	0	0	0
Snow load	0	0.2	0
Rainwater load	0	0	0
Wind load	0	0.2	0
Temperature load (no fire)	0	0.5	0

a The value of 0.6 holds for building parts which can be loaded by a crowd in case of an incident (escape routes or stairways); the value of 0.4 holds for all other cases

b A traffic area refers to an area where vehicles could ride, such as parking garages

However, for existing buildings, the partial factors are changed according to the discussed reliability indices [99]. This concerns the partial load factors, not the partial material factors. In Annex B of the EN 1990, it is stated that the partial material factors remain unchanged during an adaption of the target value of the safety level. The same method is used for the NEN 8700. In the partial load factors, the total effect of the reduction of the safety level, including the strength, has been discounted. For this reason, the partial material factors are not lowered [99].

The partial factors for new buildings according to the EN 1990 are shown in Table 22, while the adapted partial factors for alteration or renovation and the level of disapproval are shown in Table 23 and Table 24.

Table 22 | Partial load factors for the level of new buildings [95]

CC	Permanent and temporary design situations	Permanent loads		Governing variable load	Variable loads simultaneously with the governing variable load	
		Unfavorable $\gamma_{G,j,sup}$	Favorable $\gamma_{G,j,inf}$	$\gamma_{Q,1}$	Most important $\gamma_{Q,i}$	Other $\gamma_{Q,i}$
CC1	6.10a	1.2	0.9		1.35	1.35
	6.10b	1.1[a]	0.9	1.35		1.35
CC2	6.10a	1.35	0.9		1.5	1.5
	6.10b	1.2[a]	0.9	1.5		1.5
CC3	6.10a	1.5	0.9		1.65	1.65
	6.10b	1.3[a]	0.9	1.65		1.65

a This value is calculated with $\xi = 0.89$

Table 23 | Partial load factors for the level of alteration or renovation [50]

CC	Permanent verification situations	Permanent loads		Governing variable load (no wind)[a]	Governing variable load (wind)[a]
		Unfavorable $\gamma_{G,j,sup}$	Favorable $\gamma_{G,j,inf}$	$\gamma_{Q,1}$	$\gamma_{Q,1}$
CC1	6.10a	1.15	0.9	1.1	1.2
	6.10b	1.05	0.9	1.1	1.2
CC2	6.10a	1.3 (1.2)[b]	0.9	1.3	1.4
	6.10b	1.15	0.9	1.3	1.4
CC3	6.10a	1.4 (1.2)[b]	0.9	1.5	1.6 (1.5)[b]
	6.10b	1.25 (1.2)[b]	0.9	1.5	1.6 (1.5)[b]

a The last column of the table applies if the wind is the governing load for which the different β-values have been established (paragraph 6.3.3.1)

b The values between the brackets are only allowed to be used for buildings for which a license is granted according to the Building Decree of 2003 or before

Table 24 | Partial load factors for the level of disapproval [50]

CC	Permanent verification situations	Permanent loads		Governing variable load (no wind)[a]	Governing variable load (wind)[a]
		Unfavorable $\gamma_{G,j,sup}$	Favorable $\gamma_{G,j,inf}$	$\gamma_{Q,1}$	$\gamma_{Q,1}$
CC1	6.10a	1.1	0.9	1.05	1.1
	6.10b	1.0	0.9	1.05	1.1
CC2	6.10a	1.2	0.9	1.15	1.3
	6.10b	1.1	0.9	1.15	1.3
CC3	6.10a	1.3 (1.2)[b]	0.9	1.3	1.5
	6.10b	1.2	0.9	1.3	1.5

a The last column of the table applies if the wind is the governing load for which the different β-values have been established (paragraph 6.3.3.1)

b The values between the brackets are only allowed to be used for buildings for which a license is granted according to the Building Decree of 2003 or before

In case of accidental situations, equation 6.11b of the EN 1990 should be used, which was shown in paragraph 6.1.4.1. This paragraph also mentioned that all of the partial factors in this equation should be equal to 1.0. According to the NEN 8700, the same holds for existing buildings. So in principle, in case of a fire, the procedure of the accidental situation for alteration or renovation and assessment of existing buildings is the same for new buildings. However, if the existing buildings do not meet the minimum fire resistance requirements, it could be considered in consultation with the municipal to reduce the fire load [50, 98, 99].

Table 25 | Partial load factors for the accidental load combination [50, 95]

Accidental verification situations	Permanent loads		Governing variable load	Accidental
	Unfavorable $\gamma_{G,j,sup}$	Favorable $\gamma_{G,j,inf}$	$\gamma_{Q,1}$[a]	$\gamma_{Q,i}$
6.11b	1.0	1.0	1.0	1.0

a Ψ_1 only holds for wind combined to fire while assessing disproportional damage according to the EN 1991-1-7. All other cases uses Ψ_2.

6.3.6 Representative values of the loads

The representative values of the loads generally follow from the EN 1990 and the EN 1991. However, via the principle of equivalence which is recorded in the Building Decree (see paragraph 3.2.8), it is possible to determine these loads in a different way for existing buildings, for instance by means of measurements [98, 99].

The measurement of permanent loads is usually not a problem [99]. The magnitude of the permanent loads can be simply determined by means of metering and weighing. The dead weight for example can be established based on the measured dimensions and the average mass units. These dimensions could also be derived from old building plans.

Determining the extreme values of the variable loads or their effects (moments, forces and stresses) is harder [99]. They will not be able to be established by means of direct measurements. However, one can often determine momentary values, which can lead to extreme values by extrapolation. Because these values are based on the actual use of the building, instead of the most unfavorable scenarios within a broad definition of the intended use as is done for the nominal values in the EN 1991, this could lead to a reasonable reduction of the loads. Besides, the use of the reference period which is shorter than the default of 50 years as mentioned in paragraph 6.3.3.2.2 and 6.3.3.5 leads to a reduction as well. If the prescribed safety level still cannot be reached with these values, it can be decided to apply some other measures to reduce the occurring loads. In case of an existing building, one could think of the partial clearing of various departures. In such situations, the extreme loads are assumed to be known. In here, the reductions in relation with the shorter reference period are already processed, so the loads may not be reduced any further at this point [98, 99].

6.3.7 Representative values of materials

When by means of building plans or site inspections is examined what materials are used in existing buildings, the representative characteristics of these used materials should be adopted [50]. In principle, material properties can be derived from the currently used standard sheets for new buildings. However, a lot of the assessments methods mentioned in these standard sheets cannot be used for existing buildings. For example, core samples can still be used, but test cubes cannot be made anymore.

The representative values of the material properties which could not be determined by the standards for new buildings, may be established from former regulations, building plans or other documentation, in reliance to the principle of equivalence [99]. But if the new standards emphatically vary from the old standards for explicit reasons, the new standards should be maintained. One should take into account, however, that the current state of the materials could deviate from the different kinds of documentation, due to the deterioration of the concrete over the years (see paragraph 4.6) in combination with deviations following from an inaccurate execution.

If both the new and the old assessment methods are unsuitable or if one doubts the applicability, material properties can be determined by measurements to the structure [98, 99]. Various measurement methods are described in the material bounded sections of the current and the former standards. But even if the assessment methods for the determination of material properties of the material bounded regulations could be used, it could still be more practical to try to establish the properties in another way first. For example, instead of beginning with removing core samples from a concrete structure, it can possibly be sufficient to perform measurements on several places of the structure with a Schmidt Hammer.

Finally, when there are no usable methods found in the former and recent standards to establish representative values from the test results of a measurement, the values must be based on the statistical valuation in accordance with Annex D of the EN 1990 [98, 99].

6.4 Summary of the fire design procedure in case of existing buildings

According to this chapter, the fire design procedure for existing buildings can generally be completed in the same way as for new buildings. This procedure consists of five different steps, which are:

1. Consider a relevant fire scenario;
2. Choose an appropriate design fire;
3. Calculate the temperature distribution;
4. Calculate the effects of all the mechanical actions;
5. Verify the fire resistance.

However, one should keep in mind that the level of requirements for the fire resistance of existing buildings is lower than the requirement level of new buildings, and that the procedure should be based on the principles of the NEN 8700. These principles are mainly based on economic motives and the human safety, and differ in certain ways from the Eurocodes:

- The calculations are based on the remaining lifetime and a reference period instead of a design life;
- The partial load factors which should be used for calculations in the fundamental load combinations are lowered;
- Loads and material properties can be determined by measurements to the building.

The lower partial load factors do not influence the calculation of the strength in case of a fire though, because for this case, the accidental load combination is applied, in which all the partial factors are equal to 1.

6.5 Restrictions of the Eurocode

The tabulated data and the simplified calculation methods based on the standard fire curve in the Eurocode are mainly used to determine the fire resistance of individual elements. Paragraph 2.4.1 already stated that although nominal temperature-time curves are very easy to apply, they do not give a realistic view of a fire. These curves do not contain a single relationship to the characteristics of the building which is considered and assume a uniform temperature field inside the compartment, representing a fully-developed fire. Although this phase is very important for the assessment of the fire resistance, the growth phase of a fire can be relevant as well, since this is the only phase where evacuation is possible. Besides, the decay phase is also neglected. This means that the behaviour of the concrete while it is cooling down – slowly or abruptly due to extinguishing – is not taken into account, although both paragraph 4.1.1 and 5.2.5 showed that the cooling down of the concrete can cause additional damage. This additional damage is a case of economic damage, since people already left the building by that time.

In contrast to the tabulated data, the simplified calculation methods can also be used to consider the structural behaviour of a part of the structure, instead of only determine the fire resistance of an individual element. Besides, these methods can also be used in combination with natural fire models next to the standard fire curve. However, the temperature profiles in the Eurocode for the use of the simplified calculation methods are only developed for the standard fire curve, applied on individual elements. So using natural fire models for simplified calculation methods and consider the behaviour of a part of the structure with these methods is possible, but the Eurocode does not give any required information about this.

To get a realistic view of the fire behaviour of a building, the interaction between the members such as discussed in chapter 5 should be considered in the model. For this purpose, a global analysis of the entire structure is required, including the parts of the structure that both are and are not directly exposed to the fire. This can be done by using advanced calculation models as mentioned in paragraph 6.1.5.3. However, the Eurocode only contains the basic principles of these advanced models, and does not describe how they should be used. Because the lack of knowledge which was also discussed in paragraph 3.3, in combination with economic motives, advanced calculation methods are rarely applied and most of the fire resistance determinations are based on the strength of the cross-section of an individual element.

This strength of the cross-section refers to the permissible bending moments and axial forces of the individual concrete elements in case of a fire. Many different aspects which are mentioned in chapter 4, such as loss of bond strength and spalling, are barely covered in the Eurocode. The same holds for other failure modes such as shear and torsion, which are only treated in an informative annex. But most notable is the absence of the structural behaviour due to the most important phenomenon: thermal expansions. As was explained in chapter 5, these thermal expansions influence the behaviour of the structure as a whole. Practice has shown that in case of a concrete structure, the critical component is mainly formed by the structure as a whole, and rarely by an individual element. Therefore, it is important to consider both these thermal expansions and the structural consequences of these expansions. But although the Eurocode contains many remarks stating that these thermal expansions and consequences should be taken into account, it never explains how this should be done.

So for this reasons, it is interesting to get an idea of the magnitude of the influence of these thermal expansions. Therefore, this will be looked into by means of a case study in chapter 7. First, the fire resistance of a concrete T-beam will be checked by a simplified calculation method according to the Eurocode. Secondly, a rough computer model of the concrete member will be developed, to visualize the effects of the temperature distribution, after which the resulting thermal deformations and induced thermal forces will be discussed.

Table 26 | Summary of the alternative methods to verify the fire resistance [67]

	Tabulated data	Simplified calculation methods	Advanced calculation methods
Member analysis	✓	✓	✓
Analysis of parts of the structure	✗	✓	✓
Global structural analysis	✗	✗	✓

7 CASE STUDY: HOF VAN MAERLANT

7.1 Introduction

7.1.1 Project description

The Maerlant is an old school building in Brielle (the Netherlands) which will be reused for housing. The building has a monumental entrance and two lateral wings. Patrick Meerkerk, project manager at IOB, told about the plans of architect Joris Molenaar of Molenaar&Co, developer VolkerWessels Vastgoed BV, and engineering firm IOB, to reallocate the former classrooms for 36 apartments of various sizes, with respect to the historical value and structure of the building. By placing new staircases and elevator shafts at the site of the former toilet groups, the monumental staircase with the impressive stained-glass windows will remain intact. Further on, the 1920s building will be expanded by dormer windows in the attics of the main building and the lateral wings, balconies on the facades and a new transparent part of the building at the backside.

Entrance (photo by IOB)

Stained-glass windows of monumental staircase

Impression of the dormer windows, balconies, and new transparent part of the building (drawing by Molenaar&Co)

Figure 43 | The Maerlant in Brielle, the Netherlands

7.1.2 Measurements and assumptions

For this case study, measurements were made of a T-beam in the old library section of the building. This part of the building was chosen, because at this place, the beams were not covered with a false ceiling, so nothing had to be removed or demolished. Besides, most of the classrooms were inhabited as squat apartments.

Because there were no structural drawings available, the first things which were measured, were the dimensions of the main T-beams and the cross members, taking into account a plaster ceiling with a thickness of 100 mm. Before these measurements, holes were drilled in the attic floors to be able to place ventilation shafts. The drill cores revealed that the thickness of the concrete floor amounts 100 mm.

After measuring the dimensions of the beams, the cover of the main T-beams were measured with a digital reinforcement detector. The distance between the surface of the longitudinal reinforcement and the nearest concrete surface seemed to vary between 20 and 40 mm. However, some exceptions of 10 mm were found as well. Because these values were only found over very short lengths, it was concluded that these exceptions indicated the positions of the stirrups. For this reason, a cover depth of 10 mm will be used in the fire calculations, in combination with a diameter of 10 mm for the stirrups.

Figure 44 | The measured T-beams and cross beams in the old library section of the Maerlant

Besides the structural drawings, there were no reinforcement drawings as well. Neither was it already possible to determine the concrete strength. So for the reinforcement, an assumption is made, based on the bending moment in ultimate limit state and a steel strength of f_{yk} = 220 N/mm^2. For the concrete strength, a value of f_{ck} = 20 N/mm^2 is assumed. Because the beams are supported by masonry walls, the calculation is based on a model of a simply supported beam. The loads which are used for this calculation consist of the weight of the floor, the live load on the floor, the weight of the beams and a finishing layer. The weight of the crossbeams is included as a line load on the main T-beams. Both the assumption of the reinforcement and the summation of the loads can be found in Annex B.

Finally, the effective width of the T-beams is calculated, based on section 5.3.2.1 of the EN 1992-1-1. This calculation can be found in Annex B as well.

7.2 Calculation of flexural capacity and shear capacity of the T-beam

Based on the measured values and assumptions mentioned in the previous paragraph, the flexural capacity of a T-beam after fire exposure is checked.

Information resulting from the measurements and assumptions

Beam span	$l = 7\ m$
Centre-to-centre distance	$b = 3.74\ m$
Web width	$b_w = 340\ mm$
Beam depth	$h = 510\ mm$
Floor depth	$h_f = 100\ mm$
Effective flange width	$b_{eff} = 2420\ mm$
Compressive strength concrete	$f_{ck} = 20\ N/mm^2$
Tensile strength concrete	$f_{ctk,0.05} = 1.5\ N/mm^2$
Yield strength	$f_{yk} = 220\ N/mm^2$
Bar diameter of longitudinal reinforcement at bottom	$\varphi_{l,b} = 25\ mm$
Number of bars at bottom	$n_b = 6$
Total steel area	$A_{sl,b} = n_b \cdot \frac{1}{4} \varphi_{l,b}{}^2 = 2945\ mm^2$
Bar diameter of stirrups	$\varphi_s = 10\ mm$
Cover	$c = 10\ mm$
Vertical distance between reinforcement bars at bottom	$e = 32\ mm$
Axis distance of longitudinal reinforcement at bottom	$a = c + \varphi_s + \varphi_{l,b}/2 = 32.5\ mm$
Effective depth	$d = \dfrac{4 \cdot \left(h - c - \varphi_s - \frac{\varphi_{l,b}}{2}\right) + 2 \cdot \left(h - c - \varphi_s - \varphi_{l,b} - e - \frac{\varphi_{l,b}}{2}\right)}{6} = 459\ mm$
Dead load	$G = 18\ kN/m$
Live load	$Q = 8.5\ kN/m$

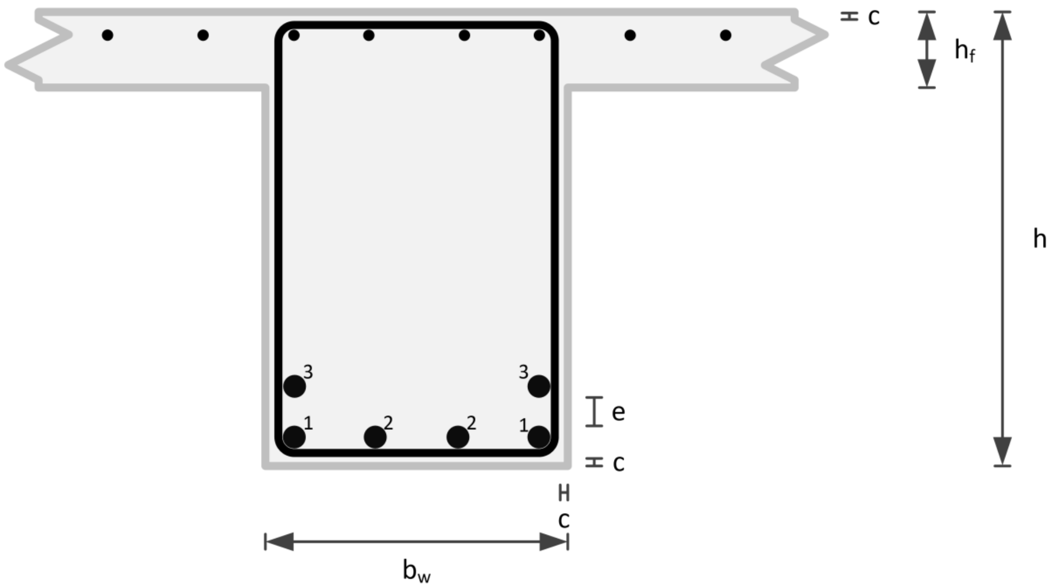


Figure 45 | Cross-section of the reinforced concrete T-beam

Calculations at room temperature

Design load, bending moment, and shear force

Design load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 1.35 \cdot G + 1.5 \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \frac{1}{8} q_d l^2 = 227\ kNm$
Shear force	$V_d = (q_d l)/2 = 130\ kN$

Design concrete strength and yield strength

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1.5 = 13.3\ N/mm^2$
Design concrete tensile strength	$f_{ctd,0.05} = f_{ctk,0.05}/\gamma_c = \frac{f_{ctk,0.05}}{1.5} = 1\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1.15 = 191.3\ N/mm^2$

Bending strength

Depth of compression zone	$x = A_{sl,b} f_{yd}/(0.85 f_{cd} b_{eff}) = 21\ mm$
Internal lever arm	$z = d - x/2 = 448\ mm$
Bending strength	$M_R = A_{sl,b} f_{yd} z = \mathbf{253\ kNm}$

$M_R > M_E$, so bending strength is OK.

Shear strength

According to the EN 1992-1-1, the shear capacity is calculated as:

Shear capacity $$V_{Rd,c} = (C_{Rd,c}k(100\rho_l f_{ck})^{\frac{1}{3}} + k_1\sigma_{cp})b_w d$$

with a minimum value of:

Shear capacity (min.) $$V_{Rd,c} = \left(\nu_{\min} + k_1\sigma_{cp}\right)b_w d$$

in which:

$$\nu_{\min} = 0.035k^{\frac{3}{2}}f_{ck}^{\frac{1}{2}}$$
$$k = 1 + \sqrt{\frac{200}{d}} \leq 2.0$$

The values of $C_{Rd,c}$ and k_1 are found in the Dutch National Annex of the EN 1992-1-1:

$$C_{Rd,c} = 0.18/\gamma_c = 0.18/1.5 = 0.12$$
$$k_1 = 0.15$$

The value of the compressive stress which follows from the axial force in the cross-section due to loading or prestressing, here indicated with σ_{cp}, is assumed to be 0 N/mm^2. Finally, the value of ρ_l follows from:

$$\rho_l = A_{sl,b}/(b_w d) = 2945/(340 \cdot 459) = 0.019$$

In this way, the following value for the shear capacity is found:

$$k = 1 + \sqrt{\frac{200}{459}} = 1.66\ (< 2.0)$$
$$\nu_{\min} = 0.035 \cdot 1.66^{\frac{3}{2}} \cdot 20^{\frac{1}{2}} = 0.33$$

Shear capacity (minimum) $$V_{Rd,c} = (0.33 + 0.15 \cdot 0) \cdot 340 \cdot 459 = 52 \cdot 10^3\ N$$

Shear capacity $$V_{Rd,c} = \left(0.12 \cdot 1.66 \cdot (100 \cdot 0.019 \cdot 20)^{\frac{1}{3}} + 0.15 \cdot 0\right) \cdot 340 \cdot 459$$
$$= \mathbf{104 \cdot 10^3\ N}$$

$$V_{Rd,c} < V_d \rightarrow not\ OK$$

This means that stirrups are required.

Calculations after fire exposure

Design load, bending moment, and shear force in case of a fire

Design load $$q_{fi} = G + \Psi_2 \cdot Q = G + 0.3 \cdot Q = 20.6\ kN/m$$

Elastic bending moment $$M_{E,fi} = \frac{1}{8}q_{fi}l^2 = \mathbf{126\ kNm}$$

Shear force $V_{fi} = (q_{fi}l)/2 = 72\ kN$

Design concrete strength and yield strength in case of a fire

Design concrete compressive strength $f_{cd} = f_{ck}/\gamma_c = f_{ck}/1 = 20\ N/mm^2$

Design yield strength $f_{yd} = f_{yk}/\gamma_s = f_{yk}/1 = 220\ N/mm^2$

Reduced yield strength after a fire exposure of 30 minutes

The reinforcement bars are divided in three groups, as is shown in Figure 45. The bars in the same group have the same temperature. The temperatures corresponding to the position of the centre of these reinforcement bars can be determined by using the design charts from Annex A of EN 1992-1-2. According to these charts, a fire exposure of 30 minutes leads to:

Bar group 1 $T_1 = 430\ °C$
Bar group 2 $T_2 = 240\ °C$
Bar group 3 $T_3 = 220\ °C$

Based on these temperatures, reduction factors have to be determined in order to calculate the reduced yield strength of the reinforcement bars. These factors can be determined by Table 3.2a of EN 1992-1-2, using linear interpolation. Assuming that the bars are hot-rolled, the reduction factors and the corresponding reduced yield strength values are:

Bar group 1 $k_{\theta,1} = 0.93 \rightarrow f_{yd,fi,1} = k_{\theta,1}f_{yd} = 205\ N/mm^2$
Bar group 2 $k_{\theta,2} = 1.00 \rightarrow f_{yd,fi,2} = k_{\theta,2}f_{yd} = 220\ N/mm^2$
Bar group 3 $k_{\theta,3} = 1.00 \rightarrow f_{yd,fi,3} = k_{\theta,3}f_{yd} = 220\ N/mm^2$

It appears that the yield strength of bar group 2 and 3 is not reduced. This corresponds to paragraph 4.4, which mentioned that reinforcement steel starts losing its strength, only when the temperature is above 300 °C.

To continue with the calculation of the bending strength of the beam in case of a fire, one single reduced yield strength value is applied, which is the mean value of all the reduced yield strength values of the individual bar groups. So for a fire exposure of 30 minutes, this value amounts:

Reduced yield strength $$f_{yd,fi} = \frac{2 \cdot f_{yd,fi,1} + 2 \cdot f_{yd,fi,2} + 2 \cdot f_{yd,fi,3}}{6} = 215\ N/mm^2$$

Bending strength after a fire exposure of 30 minutes

Based on the reduced yield strength of the reinforcement, the bending moment capacity is calculated as follows:

Depth of compression zone $x = A_{sl,b}f_{yd,fi}/(0.85 f_{cd}b_{eff}) = 15\ mm$
Internal lever arm $z = d - x/2 = 451\ mm$
Bending strength $M_{R,fi} = A_{sl,b}f_{yd,fi}z = \mathbf{285\ kNm}$

Since this value is much larger than the value of the calculated elastic bending moment of $M_{E,fi} = \mathbf{126\ kNm}$, the fire resistance of 30 minutes is clearly met.

Bending strength after a fire exposure of 60 and 90 minutes

When the beam is exposed to a fire for a longer period than 30 minutes, the reduction of the yield strength of the reinforcement steel increases. Applying this reduction on the same calculation method, a new bending strength value can be determined. For a fire exposure of 60 minutes, this leads to:

Bar group 1 $T_1 = 600\ °C$
Bar group 2 $T_2 = 460\ °C$
Bar group 3 $T_3 = 410\ °C$

Bar group 1 $k_{\theta,1} = 0.47 \rightarrow f_{yd,fi,1} = k_{\theta,1} f_{yd} = 103\ N/mm^2$
Bar group 2 $k_{\theta,2} = 0.87 \rightarrow f_{yd,fi,2} = k_{\theta,2} f_{yd} = 191\ N/mm^2$
Bar group 3 $k_{\theta,3} = 0{,}98 \rightarrow f_{yd,fi,3} = k_{\theta,3} f_{yd} = 216\ N/mm^2$

Reduced yield strength
$$f_{yd,fi} = \frac{2 \cdot f_{yd,fi,1} + 2 \cdot f_{yd,fi,2} + 2 \cdot f_{yd,fi,3}}{6} = 170\ N/mm^2$$

Depth of compression zone $x = A_{sl,b} f_{yd,fi} / (0.85 f_{cd} b_{eff}) = 12\ mm$
Internal lever arm $z = d - x/2 = 452\ mm$
Bending strength $M_{R,fi} = A_{sl,b} f_{yd,fi} z = \mathbf{227\ kNm}$

A fire exposure of 90 minutes results in:

Bar group 1 $T_1 = 740\ °C$
Bar group 2 $T_2 = 610\ °C$
Bar group 3 $T_3 = 550\ °C$

Bar group 1 $k_{\theta,1} = 0.18 \rightarrow f_{yd,fi,1} = k_{\theta,1} f_{yd} = 40\ N/mm^2$
Bar group 2 $k_{\theta,2} = 0.45 \rightarrow f_{yd,fi,2} = k_{\theta,2} f_{yd} = 99\ N/mm^2$
Bar group 3 $k_{\theta,3} = 0.63 \rightarrow f_{yd,fi,3} = k_{\theta,3} f_{yd} = 138\ N/mm^2$

Reduced yield strength
$$f_{yd,fi} = \frac{2 \cdot f_{yd,fi,1} + 2 \cdot f_{yd,fi,2} + 2 \cdot f_{yd,fi,3}}{6} = 92\ N/mm^2$$

Depth of compression zone $x = A_{sl,b} f_{yd,fi} / (0.85 f_{cd} b_{eff}) = 7\ mm$
Internal lever arm $z = d - x/2 = 455\ mm$
Bending strength $M_{R,fi} = A_{sl,b} f_{yd,fi} z = \mathbf{123\ kNm}$

From these results, it appears that besides a fire resistance of 30 minutes, a fire resistance of 60 minutes can be met as well, because $M_{R,fi} > M_{E,fi}$ also holds for this situation. After 90 minutes, the bending strength reaches a value that lies below the value of the calculated bending moment. However, the difference between the required bending strength of 126 kNm and the obtained value of 123 kNm is less than 2.5%. Because the calculation is based on several assumptions, a difference less than 2.5% does not have to be a reason for disapproval. This means that theoretically, the T-beam is not able to fulfil a fire resistance requirement of 90 minutes, but the beam could be accepted in practice.

It should be noticed that the calculations did not contain a reduction of the concrete strength. This can be explained by the fact that the compression zone is situated in the top of the floor, while the T-beam is heated from beneath. In this way, the temperature in the top of the floor will be much lower than 500 °C. This means that - according to the 500 °C isotherm method mentioned in paragraph 6.1.5.2.1 - the full strength of the concrete can be applied. Besides, the floor would not reach very high temperatures anyway, because the plaster ceiling with a thickness of 100 mm works as a heat insulating layer.

Shear strength after a fire exposure of 90 minutes

According to article 4.4 of EN 1992-1-2, a concrete element exposed to a fire does not have to be checked on shear, if the dimensions and axis distances are bigger or equal to the minimum dimensions that are given in tables. For a simply supported beam holds:

Table 27 | Minimum dimensions and axis distances for simply supported beams [67]

Fire resistance	Possible combinations of b_{min} and a					b_w
30 minutes	b_{min}	=	120	160[1]	200	80[3]
	a	=	20	15[1,2]	15[2]	
60 minutes	b_{min}	=	160	200[1]	300	100[3]
	a	=	35	30[1]	25	
90 minutes	b_{min}	=	200	300[1]	400	100[3]
	a	=	45	40[1]	35	

1 Generally holds that $a_{sd} = a + 10\ mm$, but this increase is not required for values which are higher than these b_{min} values.
2 In this situation, the concrete cover of EN 1992-1-1 is usually governing.
3 These minimum web widths must be applied according to the National Annex of the EN 1992-1-2

According to this table, the T-beam - which has a web width of 340 mm and an axis distance of 32.5 mm - only needs to be checked on shear for a fire resistance of 90 minutes, because in this case, the axis distance is smaller than the prescribed value in Table 27.

The shear capacity of the T-beam after a fire exposure of 90 minutes can generally be calculated in the same way as is done at room temperature, but one should take into account a reduction of the cross-section. It is assumed that the concrete with a temperature above 500 °C has no compressive strength and that the concrete below 500 °C has full compressive strength. According to the temperature profiles of Annex A of EN 1992-1-2, the depth of the 500 °C isotherm is $c_{fi} = 30\ mm$ after a fire exposure of 90 minutes. This leads to the following reduction:

Web width $\quad b_{w,fi} = 340 - 2 \cdot c_{fi} = 280\ mm$

The effective depth d starts at $510 - 459 = 51\ mm$ from the exposed surface. This value is bigger than the depth of the 500 °C isotherm, which means that the effective depth does not have to be reduced.

So with this reduced concrete section, the shear capacity of the T-beam after a fire exposure of 90 minutes will be:

$$k = 1 + \sqrt{\frac{200}{459}} = 1.66\ (< 2.0)$$

$$v_{\min} = 0.035 \cdot 1.66^{\frac{3}{2}} \cdot 20^{\frac{1}{2}} = 0.33$$

$$\rho_l = A_{sl,b}/(b_{w,fi}d) = 2945/(280 \cdot d) = 0.023$$

$$C_{R,fi,c} = 0.18/\gamma_c = 0.18/1 = 0.18$$

$$k_1 = 0.15$$

$$\sigma_{cp} = 0\ N/mm^2$$

Shear capacity (minimum) $V_{R,fi,c} = (0.33 + 0.15 \cdot 0) \cdot 280 \cdot 459 = 43 \cdot 10^3\ N$

Shear capacity

$$V_{R,fi,c} = \left(0.18 \cdot 1.66 \cdot (100 \cdot 0.023 \cdot 20)^{\frac{1}{3}} + 0.15 \cdot 0\right) \cdot 280 \cdot 459 = \mathbf{137 \cdot 10^3\ N}$$

This value is larger than the calculated shear force of 72 kN. So although the calculations at room temperature concluded that stirrups are required, these stirrups do not seem to be significant to withstand the shear force during a fire exposure. Besides, the shear capacity during fire exposure could even be higher if the imposed deformations are taken into account. These deformations can cause compressive stresses, resulting in a higher value for σ_{cp}.

However, when the shear forces would be higher and the shear reinforcement would play a significant role, one should take into account the temperature distribution. Just as with the longitudinal reinforcement, the temperature distribution mainly depends on the cover and the duration of the fire. One should be aware, though, that the stirrups run through several temperature zones, in contrast to the longitudinal reinforcement. In this way, the heat will be transferred to cooler temperature zones, which leads to the fact that the temperature of the stirrups will be lower than the temperature of the heated concrete. For this reason, one should determine a reference temperature [67]. Finally, it should be noted that eigen stresses could influence the shear capacity, because they lead to higher tensile stresses at the inside of a concrete element (as was mentioned earlier in paragraph 4.5).

7.3 Influence of thermal expansions on the T-beam

To get an idea of the influence of the thermal expansions on the T-beam, this beam is subjected to a temperature distribution, using modelling software of Technosoft. As was told in paragraph 4.5, a temperature distribution is generally divided into three components, which are the mean temperature ΔT_m, the temperature difference ΔT_b, and the eigen temperature ΔT_e. Because all these components represent a part of the total temperature difference, the term "temperature difference" for ΔT_b could be confusing. From now on, ΔT_m and ΔT_b will be mentioned as the "constant temperature difference ΔT_m" and the "linear temperature difference ΔT_b". In Annex C1, the values of the constant temperature differences and the linear temperature differences are determined for a fire exposure of 30, 60, and 90 minutes. These values are applied in the computer model.

In paragraph 4.5, it was mentioned that constant temperature differences could lead to axial compressive forces, depending on the degree of restraint. This can be expressed by the following formula [61]:

Axial compressive force, caused by the constant temperature difference $N_c(\Delta T_m) = \alpha_c(\Delta T_m) \cdot E_c A_c \cdot r_t$

in which α_c is the thermal coefficient of the concrete (which has an assumed value of $\alpha_c = 10^{-5}\ °C^{-1}$) and r_t is the translational degree of restraint. Linear temperature differences can cause moments, which depend on a degree of restraint as well:

Moment, caused by the linear temperature difference $M(\Delta T_b) = \kappa(\Delta T_b) \cdot E_c I \cdot r_r = \frac{\alpha_c \cdot \Delta T_b}{h} \cdot E_c I \cdot r_r$

in which h is the height of the beam and r_r is the rotational degree of restraint [61].

In case of a clamped T-beam, both the translational and the rotational degree of restraint are equal to 1 [61]. According to the model in Technosoft, this leads to the following axial compressive force and moment distributions after a fire exposure of 30 minutes:

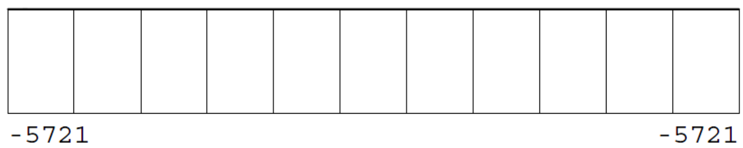


Figure 46 | Normal force of a clamped T-beam after a fire exposure of 30 minutes, caused by the constant temperature difference

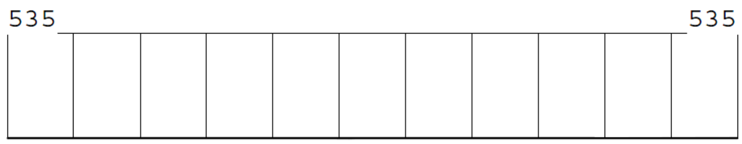


Figure 47 | Moment distribution of a clamped T-beam after a fire exposure of 30 minutes, caused by the linear temperature difference

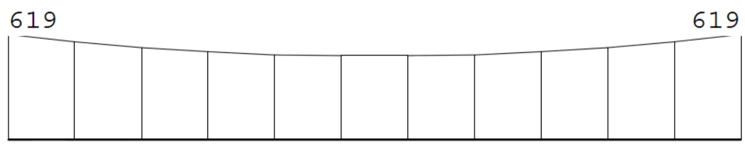


Figure 48 | Total moment distribution of a clamped T-beam after a fire exposure of 30 minutes

First of all, the maximum positive moment after a fire exposure of 30 minutes is 619 kNm. Based on the amount of reinforcement which is determined in Annex B4, the clamped T-beam could resist a maximum positive moment of 174 kNm after a fire exposure of 30 minutes (the corresponding calculations can be found in Annex C2). It is clear that this capacity is far below the value of 619 kNm. This means that flexural failure will occur. Secondly, the normal force which follows from the constant temperature difference after a fire exposure of 30 minutes is very high as well. A buckling check in Technosoft indicates that the clamped T-beam cannot resist this normal force either (see page 141-145).

When the T-beam is simply supported and free to deform, both degrees of restraint will be equal to 0 instead of 1 [61]. According to the mentioned formulas which present the axial compressive force caused by the constant temperature difference and the moment caused by the linear temperature difference, both the normal force and the moment will be 0.

However, the degrees of restraint of the T-beam in the Maerlant will lie between 0 and 1. Although the beam is calculated as a simply supported beam in paragraph 7.2, the rotations will be partially blocked because the beam is located between masonry walls (see Figure 49). For this reason, a rotational spring value of 2500 N/mm^2 is applied in the computer model. Besides, the masonry walls partially restrain the horizontal translation as well. This restraint is represented by a translational spring value in the computer model, based on the stiffness of the wall. The calculation of this spring value can be found in Annex C3.

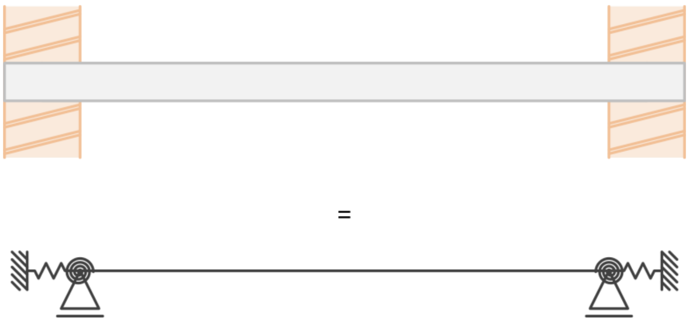


Figure 49 | Schematization of the T-beam in the Maerlant

Table 28 shows the moment distribution caused by the linear temperature differences $M(\Delta T_b)$, the moment distribution during a fire exclusive the moment caused by the linear temperature differences ($M_{E,fi}$), and the total moment distribution inclusive the moment caused by the linear temperature differences ($M_{E,fi,T}$) - all following from the model in Technosoft. The last column shows the values of the flexural capacity which are calculated in paragraph 7.2 and Annex C4. The table clearly shows that the thermal expansion would not lead to flexural failure. On the contrary, the thermal expansion would even increase the flexural capacity. The negative moment at the midspan reduces due to the thermal expansion, which means that the reinforcement in the bottom of the member can be heated to a higher temperature before flexural failure will occur.

Table 28 | Moment values according to the model in Technosoft, compared to the calculated moment capacity

Fire duration	$M(\Delta T_b)$ $[kNm]$	$M_{E,fi}$ $[kNm]$	$M_{E,fi,T}$ $[kNm]$	$M_{R,fi}$ $[kNm]$
30 minutes	19	-123 - 3	-104 - 22	-285 - 91
60 minutes	29	-123 - 3	-94 - 32	-227 - 88
90 minutes	36	-123 - 3	-87 - 39	-123 - 86

In the third and fourth column of Table 29, the other results of the Technosoft model are given, which are the horizontal translations and the normal forces, caused by the constant temperature differences. Each fire duration shows three different values, depending on the translational spring values which are based on the characteristic compressive strength of the masonry (see Annex C3). It is clear that the maximum normal force of 12.8 kN, which occurs at a stiff masonry wall, exposed to a fire for 90 minutes, is much lower than the normal force of 5721 kN of a clamped beam after a fire exposure of 30 minutes. This means that the beam could resist the thermal expansion caused by the constant temperature difference, just after a horizontal displacement of 3.76 mm. However, the question arises which consequences this displacement and normal force have on the masonry wall. For this reason, the following formula is used to calculate the stress in the outer fibre of the masonry wall:

Stress in outer fibre of the masonry wall

$$\sigma_m = -\frac{P}{A_m} + \frac{P \cdot u(\Delta T_m)}{W_m} + \frac{\frac{1}{4} \cdot N(\Delta T_m) \cdot h_m}{W_m}$$

in which P is the vertical load of the masonry wall. The values of the area A_m, moment of resistance W_m, and height h_m, can be found by using the dimensions given in Annex C3:

Area of masonry wall	$A_m = 220 \cdot 4840 = 1.065 \cdot 10^6\ mm^2$
Moment of resistance of masonry wall	$W_m = \frac{1}{6} \cdot 4840 \cdot 220^2 = 39.04 \cdot 10^6\ mm^3$
Height of masonry wall	$h_m = 8400\ mm$

The results which follow from this formula are shown in the last two columns of Table 29. For the fifth column, it is assumed that $P = 0\ kN$, while the sixth column is based on $P = 150\ kN$.

In EN 1996-1-1, a characteristic bending strength of $f_{xk1} = 0.1\ N/mm^2$ is given in case of failure in a plane parallel to the horizontal joint [100]. From Table 29, it becomes clear that when $P = 0\ kN$, this value is reached for all fire durations and wall stiffnesses. When $P = 150\ kN$, the stresses of the least rigid wall are lower than $f_{xk1} = 0.1\ N/mm^2$. However, the other walls still show higher values. The stiffest walls even reach values above $f_{xk2} = 0.4\ N/mm^2$, which is the maximum characteristic bending strength concerning failure in a plane perpendicular to the horizontal joint [100]. This means that it is likely that the thermal expansion leads to cracks in the masonry.

Table 29 | Consequences of the horizontal thermal expansion

Fire duration	$f_k\ [N/mm^2]$	$u(\Delta T_m)[mm]$	$N_c(\Delta T_m)[kN]$	$\sigma_{m,P=0\ kN}$ $[N/mm^2]$	$\sigma_{m,P=150\ kN}$ $[N/mm^2]$
30 minutes	3.1	1.75	1.89	0.10	0.03
	7.5	1.75	4.56	0.25	0.11
	10	1.75	6.1	0.33	0.19
60 minutes	3.1	2.83	3.06	0.16	0.03
	7.5	2.83	7.4	0.40	0.27
	10	2.83	9.9	0.53	0.40
90 minutes	3.1	3.67	3.97	0.21	0.09
	7.5	3.67	9.6	0.52	0.39
	10	3.67	12.8	0.69	0.56

7.4 Discussion of the results

The calculations according to the Eurocode, which did not take into account the imposed deformations, showed that the concrete T-beam has enough capacity to resist the moments and shear forces in case of a fire exposure of 30 and 60 minutes, but not enough to resist the moment in case of a fire exposure of 90 minutes. However, the difference between the occurring moment and the moment capacity in case of a fire exposure of 90 minutes is so low, that the beam could be accepted in practise.

The model in Technosoft, in which the T-beam was exposed to thermal expansions, demonstrated that these expansions could cause very high moments and normal forces. These consequences occurred at a fully clamped beam with no freedom to translate or rotate. One could think of such a situation in case of a beam between very stiff elevator shafts. However, in most cases, adjacent structural elements could deform, allowing the beam to partially expand. This leads to smaller moments and normal forces. For this reason, the T-beam between the masonry walls could resist the moments and normal forces, in contrast to the clamped beam. The moment capacity is even positively influenced by the thermal expansion. The negative moment at the midspan reduces due to the thermal expansion, which means that the reinforcement in the bottom of the member can be heated to a higher temperature before flexural failure will occur.

However, although the beam could resist the fire over 90 minutes, one should keep in mind that the masonry could be damaged. This would not lead to a structural collapse, but the cracks in the concrete could lead to a fire spread to other compartments or buildings.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Many vacant buildings that are reused are constructed in concrete. With concrete, one thought to have found a solution for the fire resistance of buildings. Today it appears that the fire resistance of concrete is not always obvious. Therefore, it is important to properly asses the fire resistance of existing buildings. The research carried out in this thesis covered the following main points concerning this assessment:

- **The fire resistance requirements in the Building Decree are unclear, which can hinder a correct application of these requirements and the use of the principle of equivalence.** Over the years, the fire resistance requirements are slightly adapted and supplemented. However, these requirements are still mainly based on principles which were derived in the early and mid-twentieth century. The background of these performance requirements are not explained in the Building Decree. They are not scientifically substantiated and many performance requirements are inconsistent and illogical. Besides, some performance requirements are conflicting with the functional requirement of the Building Decree.
- **The municipality and fire brigade have little knowledge concerning behaviour of concrete structures in case of a fire, which can lead to unmotivated high demands and troubles the use of the principle of equivalence.** Because there is often a deficiency of information to determine a legally obtained level for the fire resistance of an existing concrete building, fire resistance requirements of the level of existing buildings are applied. In this case, municipalities may ask for tougher demands based on advice of the fire brigade, as long as they are not higher than the requirements for new buildings. The lack of knowledge can lead to unmotivated high demands. Besides, if an engineer wants to apply the principle of equivalence, the suggested alternative solution should be approved by the municipality. Because of the lack of knowledge of the municipality and the fire brigade, it could be very difficult for an engineer to convince them of an alternative solution.
- **The temperature effects on reinforced concrete are very complex and it is hard to predict how different factors influence the fire resistance. In any way, the concrete cover and carbonation should at least be taken into account.** Several cases of fire damage are known, but because they depend on many different factors, it remains difficult to get an idea of the extent to which these factors influence the fire resistance of the concrete structure. Although it is known that the concrete cover (of which the thickness is often limited in an historic building) plays a significant role for the reduction of the strength of the reinforcement steel under fire conditions, it is much harder to indicate such an important factor for the cause of spalling. Many factors could be both advantageous and disadvantageous. However, this thesis stated that the corrosion products caused by carbonation can substantially increase the risk of spalling. So for this reason, a structural engineer should at least pay attention to corrosion to take into account the risk of spalling.

- **Thermal expansions can greatly influence the structural behaviour under fire conditions, but this does not necessarily have to be a disadvantage.** Thermal expansions can lead to moments and normal forces. The values of these moments and normal forces mainly depend on the size and shape of the concrete element, the support conditions, and the surrounding elements (acting as restraints). Moments and normal forces due to thermal expansions do not occur in case of simply supported unrestrained beams. The thermal expansion of a simply supported restrained beam causes a thrust force, which can act as a prestressing force. This force causes a positive moment which is beneficial to the concrete member which has to withstand a negative moment, and thus increases the fire resistance. In case of a continuous concrete member (which for example appears in the many monolithic structures of existing buildings), a moment redistribution occurs. This moment redistribution increases the fire resistance as well. However, when the moments and normal forces become too high, flexural failure or buckling can occur. A long beam clamped between two stiff concrete cores can be seen as one of the most unfavourable situations. Because the thermal expansions are totally blocked, the moments and normal forces would become extremely high. However, a long beam is rarely exposed to a fire over the full length, which means that the moments and normal forces in practise would not be that high.
- **The Eurocode only explains simple calculation methods, which does not take into account the structural behaviour of a concrete building in case of a fire. However, simple calculations can be used to get an idea of the structural behaviour and to identify possible bottlenecks.** Although the ways of verification of the fire resistance requirements have gradually taken more shape since the first calculation methods of 1991, the principles are still the same. The Eurocode mainly explains simple calculation methods which only concerns individual elements or small parts of the structure. However, because of the thermal expansions of concrete elements in case of a fire, it is important to look at the total structure instead of an individual element. The Eurocode does mention advanced calculation methods concerning the structure as a whole, as well as the importance of thermal expansions, but it does not give an explanation on how to use or determine it. The case study shows, though, that by using these simple calculation methods in combination with a modelling of the temperature distribution, a rough estimation of the fire resistance can be established.

Recommendations for practise

Based on the main points which followed from the research of this thesis, the following approach to assess the fire resistance of an existing concrete building is recommended:

1. **Determine the fire resistance requirements according to the Building Decree and contact other involved parties as soon as possible.** In this way, it is prevented that the differences between the knowledge of the municipality, the fire brigade, and the structural engineer could lead to a delay of the assessment, because of different interpretations of the performance requirements or an alternative solution according to the principle of equivalence. When these different interpretations will be brought up in a late stage of the fire resistance assessment, this could mean that the requirements which are applied would not be approved and the assessment has to be adapted.
2. **Determine the geometry of the structure, along with the material properties.** Besides the dimensions of the structure, the geometry of the structure concerns the support conditions, the reinforcement steel, and the thickness of the concrete cover as well. In addition to measurements and visual inspections, one could also use original design drawings, building licenses or historic building standards, but one should take into account possible deviations. The fire resistance of the concrete material is mainly established by fire tests. However, if these tests are not executed, at least pay attention to the degree of corrosion.
3. **Determine the load distribution in case of a fire.** For this purpose, apply the accidental load combination, in which all the partial factors are equal to 1.
4. **Determine and check the capacity of the concrete element(s) according to the Eurocode.** When applying the simple calculation methods, keep in mind the reduced concrete strength and yield strength in case of a fire.
5. **Model the concrete element(s) in a computer program and apply a temperature distribution.** This can give a view of the structural behaviour in the case of a fire and indicates the bottlenecks which can cause high moments and normal forces due to thermal stresses. Do not only pay attention to the possible consequences for the modelled concrete element(s) itself, but also to the consequences for the surrounding element(s). If it is possible to express these moments and normal forces in certain values, combine these values with the thermal loads determined in step 3 and check the capacity of the concrete element(s) as described in step 4.
6. **In case the fire resistance is not sufficient, consider a fire safety measure.** If this measure concerns a structural solution (such as an additional concrete cover or a fire-resistant coating), step 2 to 5 should be repeated, taking into account this structural adaption. For other solutions (such as the application of a sprinkler system), the principle of equivalence should be applied. This requires a clear description of the alternative solution for other involved parties.

Recommendations for further research

Finally, it can be concluded that for a better assessment of the fire resistance of existing concrete buildings in the future, the following topics remain important for further research:

- **The fire resistance of aged concrete.** In this thesis, some of the factors which influence the temperature effects are generally discussed. To be able to correctly predict the occurrence of the complex temperature effects, it is important to fully understand these phenomena. This still requires a large amount of experimental data, which need to be found by more specific researches.
- **The structural behaviour of concrete structures exposed to fire.** A rough idea of the consequences of the thermal expansions of concrete members is given in this research. However, an accurate calculation of these effects is not yet possible. This requires further research, where it should be wise to make a distinction between different types of structures (which was impracticable to do within the scope of this thesis). Besides, it is also important to look into the eigen stresses more deeply. Because these stresses do not cause moments or normal forces, they are not further discussed in this thesis. However, these stresses may be of interest in case of spalling or shear forces, due to the additional compression in the outer layer and the additional tension in the inner layer of the concrete.
- **Realistic calculation methods.** Based on the research into the fire resistance of aged concrete and the structural behaviour of concrete structures exposed to fire, the Eurocode could be supplemented with more detailed calculation methods. This could lead to more accurate calculations, resulting in more efficient measures.
- **Fire resistance requirements.** The current fire resistance requirements, mainly based on assumptions made in the twentieth century, are unclear and not always logical. If these requirements would be replaced by (scientifically based) requirements with a clear explanation, differences in interpretations could be avoided and the background of the requirements could be used for alternative solutions as well.

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ANNEX A

This annex shows a collection of photographs of an apartment at the Korianderstraat in Hoogvliet (in the Netherlands), which burnt out in November 2015. In April, structural engineer André Lankhof from IOB was asked to take a look at the apartment, in order to assess the structural damage caused by the fire. It mainly concerned a crack in the ceiling. However, this crack turned out to be a crack in the stucco, and not in the stony material of the loadbearing structure. There appeared to be no structural damage.

These photographs demonstrate the advantages of stony construction materials as were mentioned in chapter 4: while the timber window frames are completely burnt, the entire loadbearing structure is unharmed.

Timber window frame

Timber window frame

Bearing wall

Partition wall

Ceiling

Balcony (bottom)

Figure A1 | Fire damage of an apartment in Hoogvliet

ANNEX B

This Annex shows the rough calculations which were made to determine the loads on the T-beam in the Maerlant, as well as the effective width of the beam and the amount of the reinforcement steel.

B1. Loads

Measurements:

Beam span	$l = 7\ m$
Centre-to-centre distance	$b = 3.74\ m$
Web width	$b_w = 340\ mm$
Beam depth	$h = 510\ mm$
Floor depth	$h_f = 100\ mm$
Width of cross beams	$b_{cb} = 270\ mm$
Depth of cross beams	$h_{cb} = 210\ mm$

Assumptions:

Density of reinforced concrete	$\rho = 25\ kN/m^3$
Weight of finishing layer	$1\ kN/m^3$
Variable load (incl. separation walls)	$Q = 2.25\ kN/m^2$

Calculation of the total loads:

Weight of cross beams	$(2 \cdot (3.74 - 0.34) \cdot 0.27 \cdot 0.21 \cdot 25)/7$	$\approx 1.4\ kN/m$	+
Weight of floor	$3.74 \cdot 0.1 \cdot 25$	$\approx 9.4\ kN/m$	+
Weight of main beam	$0.34 \cdot (0.51 - 0.1) \cdot 25$	$\approx 3.5\ kN/m$	+
Finishing layer	$3.74 \cdot 1$	$\approx 3.7\ kN/m$	+
Total dead load	$\boldsymbol{G}$	$\boldsymbol{= 18\ kN/m}$	
Total variable load	$\boldsymbol{Q = 3.74 \cdot 2.25}$	$\boldsymbol{\approx 8.5\ kN/m}$	

It should be noted that the loads are established according to the current Eurocode, based on a residential function. This means that the loads which were originally applied for the design, could deviate from the values which are determined above.

B2. Effective width of the T-beam

The effective width of the T-beam is determined according to paragraph 5.3.2.1 of EN-1992-1-1. This article states that the effective flange width b_{eff} for a T-beam may be derived as:

$$b_{eff} = \sum b_{eff,i} + b_w \leq b$$

where

$$b_{eff,i} = 0.2 b_i + 0.1 l_0 \leq 0.2 l_0$$

and

$b_{eff,i} \leq b_i$.

For the current T-beam holds:

$b = 3740\ mm$
$b_i = b_1 = b_2 = (3740 - 340)/2 = 1700\ mm$
$l_0 = l = 7000\ mm$ (assuming a simply supported beam)
$b_{eff,i} = 0.2 \cdot 1700 + 0.1 \cdot 7000 = 1040\ mm$ $< 0.2 l_0 = 1400\ mm$
$< b_i = 1700\ mm$
$< 3740\ mm$

$\boldsymbol{b_{eff} = 2 \cdot 1700 + 340 = 2420\ mm}$*

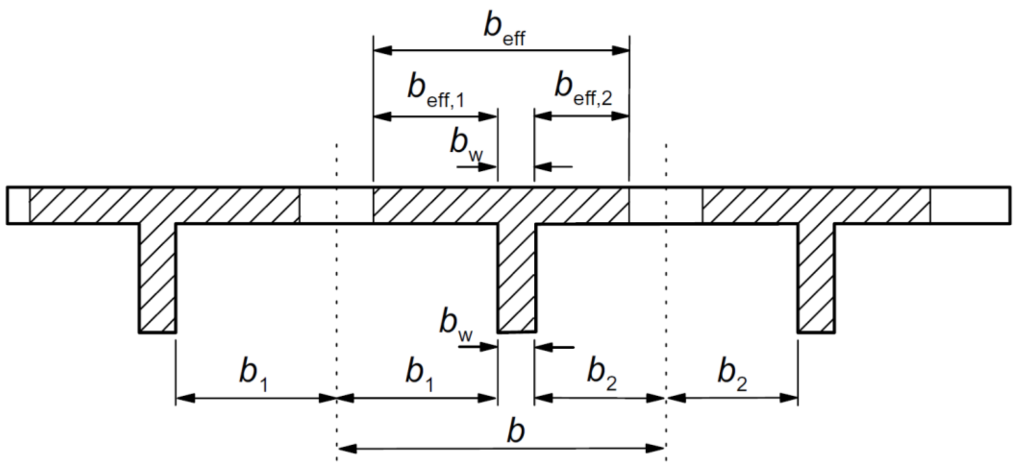


Figure B1 | Effective flange width parameters [74]

B3. Reinforcement of simply supported T-beam

Assumptions:

Bar diameter of longitudinal reinforcement at bottom	$\varphi_{l,b} = 25\ mm$
Bar diameter of longitudinal reinforcement at top	$\varphi_{l,t} = 12\ mm$
Yield strength	$f_{yk} = 220\ N/mm^2$
Concrete compressive strength	$f_{ck} = 20\ N/mm^2$
Amount of reinforcement in top of beam (at supports)	$A_{sl,t} = 1/3 \cdot A_{sl,b}$
Bar diameter of stirrups	$\varphi_s = 10\ mm$
Concrete cover	$c = 10\ mm$
Partial factor steel	$\gamma_s = 1.15$
Partial factor concrete	$\gamma_c = 1.15$
Partial factor permanent load	$\gamma_G = 1.35$
Partial factor variable load	$\gamma_Q = 1.5$

*For the convenience, this effective flange width is also used for a clamped beam in this thesis, although this support condition would lead to a lower effective flange width value.

Calculation of the required reinforcement:

Design value of total load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \frac{1}{8} q_d l^2 = 227\ kNm$
Effective depth	$d = h - c - \varphi_s - \varphi_{l,b}/2 = 478\ \ mm$
Required amount of reinforcement steel (bottom)	$A_{sl,b} = M_E/(d \cdot (f_{yk}/\gamma_s)) = 2482\ mm^2$ → $\mathbf{6Ø25}$
Required amount of reinforcement steel (top)	$A_{sl,t} = 1/3 \cdot A_{sl,b} = 827\ mm^2$ → $\mathbf{8Ø12}$

Check:

Total steel area	$A_{sl,b} = 6 \cdot (25/2)^2 \cdot \pi = 2945\ mm^2$
Depth of compression zone	$x = A_{sl,b}(f_{yk}/\gamma_s)/(0.85(f_{ck}/\gamma_c)b_{eff}) = 21\ mm$
Internal lever arm	$z = d - x/2 = 467\ mm$
Bending strength	$M_R = A_{sl,b}(f_{yk}/\gamma_s)z = 263\ kNm$

$M_R > M_E$, so the amount of reinforcement steel of 6Ø25 is OK.

B4. Reinforcement of clamped T-beam

Assumptions:

Bar diameter of longitudinal reinforcement at bottom	$\varphi_{l,b} = 20\ mm$
Bar diameter of longitudinal reinforcement at top	$\varphi_{l,t} = 12\ mm$
Yield strength	$f_{yk} = 220\ N/mm^2$
Concrete compressive strength	$f_{ck} = 20\ N/mm^2$
Bar diameter of stirrups	$\varphi_s = 10\ mm$
Concrete cover	$c = 10\ mm$
Partial factor steel	$\gamma_s = 1.15$
Partial factor concrete	$\gamma_c = 1.15$
Partial factor permanent load	$\gamma_G = 1.35$
Partial factor variable load	$\gamma_Q = 1.5$

Calculation of the required reinforcement at midspan:

Design value of total load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \frac{1}{24} q_d l^2 = 76\ kNm$
Effective depth	$d = h - c - \varphi_s - \varphi_{l,b}/2 = 480\ \ mm$
Required amount of reinforcement steel (bottom)	$M_E/(d \cdot (f_{yk}/\gamma_s)) = 828\ mm^2$ → $\mathbf{3Ø20}$

Check:

Total steel area	$A_{sl,b} = 3 \cdot (20/2)^2 \cdot \pi = 942\ mm^2$
Depth of compression zone	$x = A_{sl,b}(f_{yk}/\gamma_s)/(0.85(f_{ck}/\gamma_c)b_{eff}) = 7\ mm$
Internal lever arm	$z = d - x/2 = 477\ mm$
Bending strength	$M_R = A_{sl,b}(f_{yk}/\gamma_s)z = 86\ kNm$

$M_R > M_E$, so the amount of reinforcement steel of 3Ø20 is OK.

Calculation of the required reinforcement at support:

Design value of total load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \frac{1}{12} q_d l^2 = 151\ kNm$
Effective depth	$d = h - c - \varphi_s - \varphi_{l,t}/2 = 484\ \ mm$
Required amount of reinforcement steel (top)	$M_E/(d \cdot (f_{yk}/\gamma_s)) = 1630\ mm^2$ $\rightarrow \mathbf{15Ø12}$

Check:

Total steel area	$A_{sl,t} = 15 \cdot (12/2)^2 \cdot \pi = 1696\ mm^2$
Depth of compression zone	$x = A_{sl,t}(f_{yk}/\gamma_s)/(0.85(f_{ck}/\gamma_c)b_w) = 84\ mm$
Internal lever arm	$z = d - x/2 = 441\ mm$
Bending strength	$M_R = A_{sl,t}(f_{yk}/\gamma_s)z = 143\ kNm$

$M_R < M_E$, so the amount of reinforcement steel of 15Ø12 is not enough. The same calculations for an amount of 16Ø12 leads to $M_R = 152\ kNm$, which means $M_R > M_E$, so this amount of $\mathbf{16Ø12}$ can be used.

ANNEX C

This Annex shows the calculations which belong to paragraph 7.3

C1. Temperature distributions of the T-beam

To determine the temperature distributions of the T-beam, it is assumed that only the web of the beam will heat up. This assumption is based on the fact that the flange of the beam is covered with a plaster ceiling of 100 mm, which prevents high temperatures in the flange.

C1.1 Constant temperature difference

The calculation is based on the method of Wickström, which is also used for the temperature-time curves in EN 1991-1-2. The formulas of this method read [68]:

Temperature of exposed surface	$\Theta_m = \eta_m \Theta_g$
Temperature of the concrete	$\Theta_c = \eta_x \Theta_m$
Fire temperature	$\Theta_g = 20 + 345 \log(8t + 1)$

in which:

$$\eta_m = 1 - 0.0616 t_h^{-0.88}$$
$$\eta_x = 0.18 \ln(t_h / x^2) - 0.81$$

The symbol t indicates the time expressed in minutes, while the time expressed in hours is denoted by t_h. The cross-section of the web of the beam is divided in strips with a width of 20 mm where the temperature is above the assumed room temperature of 20 °C. The temperatures at the borders of these strips are calculated with the functions mentioned above. Subsequently, the average temperatures of all the strips are determined by summing up the values of both borders and divide this by two. All these average temperatures are lowered by 20 °C to get the average temperature differences. The average temperature differences are multiplied by the areas of the strips and added up. Finally, this value is divided by the total area of the cross-section of the T-beam (including the flange: A_c = 381400 mm^2) to get the constant temperature difference ΔT_m.

Table C1 | Determination of the constant temperature difference ΔT_m after a fire exposure of 30 minutes

Strip			Temperatures at borders [°C]			Average temperatures [°C]	Average temperature differences [°C]		Area [mm^2]	
0	-	20 mm	746	-	353	550	530	x	22400	+
20	-	40 mm	353	-	167	260	240	x	20800	+
40	-	60 mm	167	-	58	113	93	x	19200	+
60	-	80 mm	58	-	20	39	19	x	17600	+
							= 18984000 °Cmm^2			
ΔT_m	=			18984000 / 381400				=	**50 °C**	

Table C2 | Determination of the constant temperature difference ΔT_m after a fire exposure of 60 minutes

Strip	Temperatures at borders [°C]	Average temperatures [°C]	Average temperature differences [°C]		Area [mm^2]	
0 - 20 mm	887 - 531	709	689	x	22400	+
20 - 40 mm	531 - 309	420	400	x	20800	+
40 - 60 mm	309 - 180	245	225	x	19200	+
60 - 80 mm	180 - 88	134	114	x	17600	+
80 - 100 mm	88 - 20	54	34	x	16000	+
			= 30624000 °Cmm^2			
ΔT_m =		30624000 / 381400		=	**81 °C**	

Table C3 | Determination of the constant temperature difference ΔT_m after a fire exposure of 90 minutes

Strip	Temperatures at borders [°C]	Average temperatures [°C]	Average temperature differences [°C]		Area [mm^2]	
0 - 20 mm	963 - 646	804	784	x	22400	+
20 - 40 mm	646 - 406	526	506	x	20800	+
40 - 60 mm	406 - 266	336	316	x	19200	+
60 - 80 mm	266 - 166	216	196	x	17600	+
80 - 100 mm	166 - 88	127	107	x	16000	+
100 - 120 mm	88 - 25	57	37	x	14400	+
120 - 140 mm	25 - 20	23	3	x	12800	+
			= 39886400 °Cmm^2			
ΔT_m =		39886400 / 381400		=	**105 °C**	

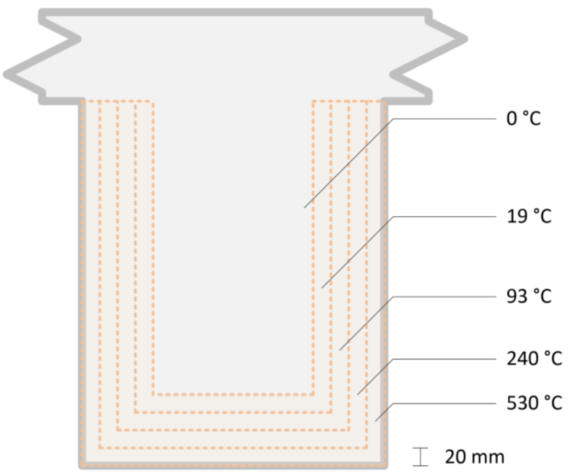


Figure C1 | The cross-section after a fire exposure of 30 minutes, divided in strips of 20 mm of which the average temperatures are shown

C1.2 Linear temperature difference

For the linear temperature distribution holds [61]:

$$\Delta T_b(x) = \frac{\Delta T_b}{h} \cdot x$$

in which:

$$\Delta T_b = \frac{h}{I} \int_{x_1}^{x_2} T(x) \cdot b(x) \cdot x \, dx$$

To apply these formulas on the T-beam, the web is divided in vertical strips over the full height of the beam (corresponding with the width of the strips defined in paragraph C1.1 and the remaining part of the cross-section as shown in Figure C2). A coordinate system with the x-axis on the half of the total height is applied. It is assumed that $b(x) = 1$ as a unit of width. For $T(x)$, use is made of the already calculated average temperatures of paragraph C1.1.

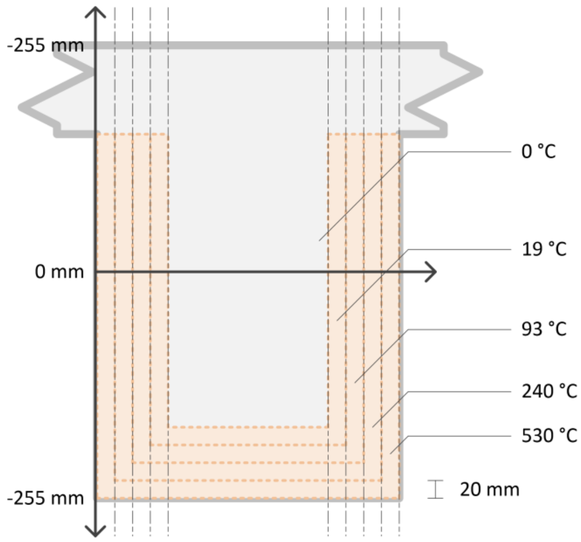


Figure C2 | The cross-section after a fire exposure of 30 minutes, divided in vertical strips of 20 mm of which the average temperatures are shown

After calculating the value of ΔT_b for each individual vertical strip, the average value of ΔT_b is determined by multiplying the individual values with the widths of the strips, summing up these values, and divide this by the total width of the web. With this value, the linear temperature can be expressed by using the first formula of this paragraph. However, this expression only holds for the temperature distribution of the web of the beam. If this expression is used in a computer program, the program will apply this distribution on the entire cross-section of the T-beam. This means that the flange will contribute to the thermal deflections as well, while this part of the cross-section does not contain any temperature differences.

To solve this problem, the temperature distribution is adapted in such a way that it causes the same moment for a T-beam as the original temperature distribution does for a rectangular cross-section. For this method, the following relations are applied:

Curvature $\kappa = \dfrac{\Delta T_b * \alpha_c}{h}$

Moment $M = EI\kappa$

Thermal strain $\varepsilon_{\Delta T} = \alpha_c \cdot \Delta T$

Thermal stress $\sigma_{\Delta T} = \varepsilon_{\Delta T} \cdot E$

With these relations, the following equation can be established (see Figure C3):

$$\begin{aligned}\frac{I\Delta T_b}{h} = \quad & (\Delta T_{top} - \frac{h_v}{\bar{y}}\Delta T_{top}) \cdot h_v \cdot b_{eff} \cdot (\bar{y} - \frac{1}{2}h_v) && + \\ & \frac{h_v}{\bar{y}}\Delta T_{top} \cdot \frac{1}{2}h_v \cdot b_{eff} \cdot (\bar{y} - \frac{1}{3}h_v) && + \\ & (\Delta T_{top} - \frac{h_v}{\bar{y}}\Delta T_{top}) \cdot \frac{1}{2}(\bar{y} - h_v) \cdot b \cdot \frac{2}{3}(\bar{y} - h_v) && + \\ & \frac{\bar{y}}{h_t - \bar{y}}\Delta T_{top} \cdot \frac{1}{2}(h - \bar{y}) \cdot b \cdot \frac{2}{3}(h - \bar{y}) &&\end{aligned}$$

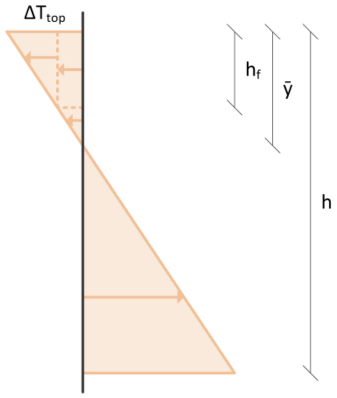


Figure C3 | Schematization which is used to determine the equation shown above

To solve this equation, $\bar{y}$ and I need to be calculated.

Web width $b_w = 340\ mm$
Beam depth $h = 510\ mm$
Floor depth $h_f = 100\ mm$
Effective flange width $b_{eff} = 2420\ mm$

Floor area $A_{c,f} = h_f b_{eff} = 242000\ mm^2$
Web area $A_{c,w} = (h - h_f) \cdot b_w = 139400\ mm^2$
Centroidal axis (top of beam is assumed as y = 0) $\bar{y} = \dfrac{A_{c,f} \cdot \left(\frac{h_f}{2}\right) + A_{c,w} \cdot \left(h_f + \frac{(h-h_f)}{2}\right)}{A_{c,f} + A_{c,w}} = 143\ mm$

Moment of inertia floor $I_f = \dfrac{1}{12} b_{eff} h_f^3 = 202 \cdot 10^6\ mm^4$

Moment of inertia web $I_w = \dfrac{1}{12} b_w (h - h_f)^3 = 1953 \cdot 10^6\ mm^4$

Total moment of inertia

$$I = I_f + A_{c,f}\left(\bar{y} - \frac{h_f}{2}\right)^2 + I_w + A_{c,w}\left(\bar{y} - \left(h_f + \frac{(h - h_f)}{2}\right)\right)^2$$
$$= 7905 \cdot 10^6\ mm^4$$

By assuming that the thermal expansion coefficient $\alpha_c = 10^{-5}\ °C^{-1}$, solving the equation leads to:

$$\Delta T_{top} \approx 0.13 \cdot \Delta T_b$$

For the temperature at the bottom holds:

$$\Delta T_{bottom} = \frac{h - \bar{y}}{\bar{y}} \Delta T_{top} \approx 0.34 \cdot \Delta T_b$$

These ratios can be used to determine the temperature distributions of the T-beam, based on the calculated temperature distribution of the web.

Table C4 | Determination of the linear temperature difference ΔT_b after a fire exposure of 30 minutes

Strip			ΔT_b per strip [°C]		Width [mm]	
0	-	20 mm	501	x	20	+
20	-	40 mm	293	x	20	+
40	-	60 mm	217	x	20	+
60	-	80 mm	193	x	20	+
80	-	260 mm	190	x	180	+
260	-	280 mm	193	x	20	+
280	-	300 mm	217	x	20	+
300	-	320 mm	293	x	20	+
320	-	340 mm	501	x	20	+
			= 82360 °Cmm			
$\boldsymbol{\Delta T_b}$	=		82360 / 340	=	**243 °C**	
$\boldsymbol{\Delta T_{top}}$	=		-0.13 · 243	=	**-32 °C**	
$\boldsymbol{\Delta T_{bottom}}$	=		0.34 · 243	=	**83 °C**	

Table C5 | Determination of the linear temperature difference ΔT_b after a fire exposure of 60 minutes

Strip			ΔT_b per strip [°C]		Width [mm]	
0	-	20 mm	652	x	20	+
20	-	40 mm	444	x	20	+
40	-	60 mm	354	x	20	+
60	-	80 mm	318	x	20	+
80	-	100 mm	306	x	20	+
100	-	260 mm	306	x	140	+
240	-	260 mm	306	x	20	+
260	-	280 mm	318	x	20	+
280	-	300 mm	354	x	20	+
300	-	320 mm	444	x	20	+
320	-	340 mm	652	x	20	+
			= 125800 °Cmm			
$\boldsymbol{\Delta T_b}$	=		125800 / 340	=	**370 °C**	
$\boldsymbol{\Delta T_{top}}$	=		-0.13 · 370	=	**-49 °C**	
$\boldsymbol{\Delta T_{bottom}}$	=		0.34 · 370	=	**126 °C**	

Table C6 | Determination of the linear temperature difference ΔT_b after a fire exposure of 90 minutes

Strip			ΔT_b per strip [°C]		Width [mm]	
0	-	20 mm	742	x	20	+
20	-	40 mm	542	x	20	+
40	-	60 mm	444	x	20	+
60	-	80 mm	405	x	20	+
80	-	100 mm	392	x	20	+
100	-	120 mm	392	x	20	+
120	-	140 mm	396	x	20	+
140	-	200 mm	397	x	60	+
200	-	220 mm	396	x	20	+
220	-	240 mm	392	x	20	+
240	-	260 mm	392	x	20	+
260	-	280 mm	405	x	20	+
280	-	300 mm	444	x	20	+
300	-	320 mm	542	x	20	+
320	-	340 mm	742	x	20	+
			= 156340 °Cmm			

ΔT_b	=	156340 / 340	=	**460 °C**
ΔT_{top}	=	-0.13 · 460	=	**-61 °C**
ΔT_{bottom}	=	0.34 · 460	=	**157 °C**

C2. Calculation of flexural capacity of a clamped T-beam

Information resulting from the measurements and assumptions

Beam span $l = 7\ m$
Centre-to-centre distance $b = 3.74\ m$
Web width $b_w = 340\ mm$
Beam depth $h = 510\ mm$
Floor depth $h_f = 100\ mm$
Effective flange width $b_{eff} = 2420\ mm$

Compressive strength concrete $f_{ck} = 20\ N/mm^2$
Yield strength $f_{yk} = 220\ N/mm^2$

Bar diameter of longitudinal reinforcement at bottom $\varphi_{l,b} = 20\ mm$
Number of bars at bottom $n_b = 3$
Total steel area at bottom $A_{sl,b} = n_b \cdot \frac{1}{4} \varphi_{l,b}{}^2 = 942\ mm^2$

Bar diameter of longitudinal reinforcement at top $\varphi_{l,t} = 12\ mm$
Number of bars at top $n_t = 16$
Total steel area at top $A_{sl,t} = n_t \cdot \frac{1}{4} \varphi_{l,t}{}^2 = 1810\ mm^2$

Bar diameter of stirrups $\varphi_s = 10\ mm$

Cover	$c = 20\ mm$
Effective depth at midspan	$d_m = h - c - \varphi_s - \varphi_{l,b}/2 = 480\ mm$
Effective depth at support	$d_s = h - c - \varphi_s - \varphi_{l,t}/2 = 484\ mm$
Dead load	$G = 18\ kN/m$
Live load	$Q = 8.5\ kN/m$

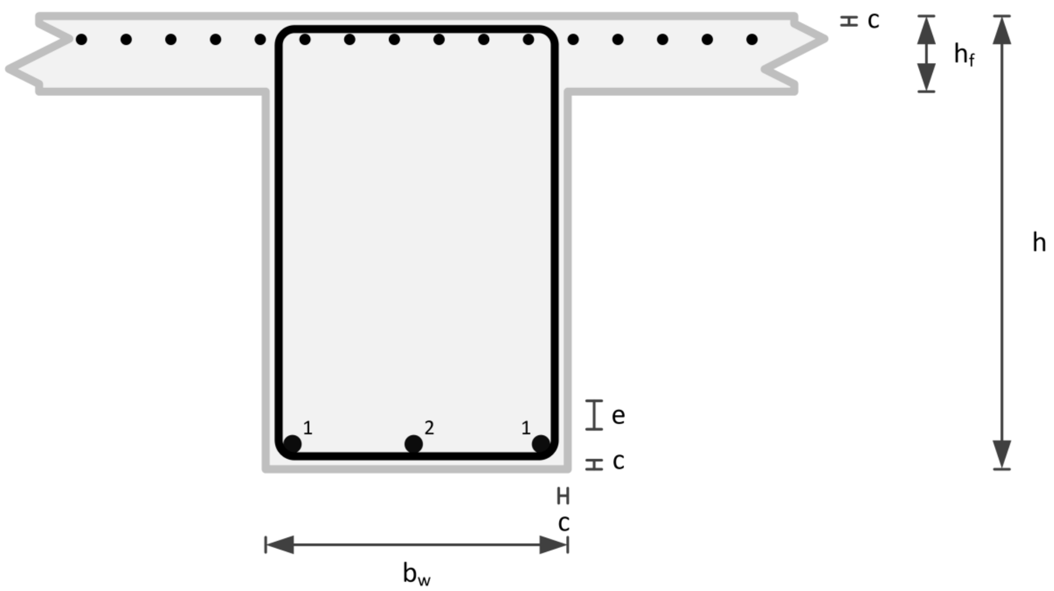


Figure C4 | Cross-section of the reinforced concrete T-beam which is clamped

C2.1 Calculations at midspan

Calculations at room temperature

Design load and bending moment

Design load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 1.35 \cdot G + 1.5 \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \dfrac{1}{24} q_d l^2 = 76\ kNm$

Design concrete strength and yield strength

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1.5 = 13.3\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1.15 = 191.3\ N/mm^2$

Bending strength

Depth of compression zone	$x = A_{sl,b} f_{yd}/(0.85 f_{cd} b_{eff}) = 6.6\ mm$
Internal lever arm	$z = d_m - x/2 = 477\ mm$
Bending strength	$M_R = A_{sl,b} f_{yd} z = \mathbf{86\ kNm}$

$M_R > M_E$, so bending strength is OK.

Calculations after fire exposure

Design load and bending moment in case of a fire

Design load $q_{fi} = G + \Psi_2 \cdot Q = G + 0.3 \cdot Q = 20.6\ kN/m$

Elastic bending moment $M_{E,fi} = \frac{1}{24} q_{fi} l^2 = \mathbf{42\ kNm}$

Design concrete strength and yield strength in case of a fire

Design concrete compressive strength $f_{cd} = f_{ck}/\gamma_c = f_{ck}/1 = 20\ N/mm^2$

Design yield strength $f_{yd} = f_{yk}/\gamma_s = f_{yk}/1 = 220\ N/mm^2$

Reduced yield strength after a fire exposure of 30 minutes

(See Figure C4 for the bar groups)

Bar group 1 $T_1 = 430\ °C$

Bar group 2 $T_2 = 240\ °C$

Bar group 1 $k_{\theta,1} = 0.93 \rightarrow f_{yd,fi,1} = k_{\theta,1} f_{yd} = 205\ N/mm^2$

Bar group 2 $k_{\theta,2} = 1.00 \rightarrow f_{yd,fi,2} = k_{\theta,2} f_{yd} = 220\ N/mm^2$

Reduced yield strength $f_{yd,fi} = \frac{2 \cdot f_{yd,fi,1} + 1 \cdot f_{yd,fi,2}}{3} = 210\ N/mm^2$

Bending strength after a fire exposure of 30 minutes

Depth of compression zone $x = A_{sl,b} f_{yd,fi}/(0.85 f_{cd} b_{eff}) = 5\ mm$

Internal lever arm $z = d_m - x/2 = 478\ mm$

Bending strength $M_{R,fi} = A_{sl,b} f_{yd,fi} z = \mathbf{94\ kNm}$

$M_{R,fi} > M_{E,fi} \rightarrow OK$

Reduced yield strength and bending strength after a fire exposure of 60 minutes

Bar group 1 $T_1 = 600\ °C$

Bar group 2 $T_2 = 460\ °C$

Bar group 1 $k_{\theta,1} = 0.47 \rightarrow f_{yd,fi,1} = k_{\theta,1} f_{yd} = 103\ N/mm^2$

Bar group 2 $k_{\theta,2} = 0.87 \rightarrow f_{yd,fi,2} = k_{\theta,2} f_{yd} = 191\ N/mm^2$

Reduced yield strength $f_{yd,fi} = \frac{2 \cdot f_{yd,fi,1} + 1 \cdot f_{yd,fi,2}}{3} = 133\ N/mm^2$

Depth of compression zone $x = A_{sl,b} f_{yd,fi}/(0.85 f_{cd} b_{eff}) = 3\ mm$

Internal lever arm $z = d_m - x/2 = 478\ mm$

Bending strength	$M_{R,fi} = A_{sl,b} f_{yd,fi} z = \mathbf{60\ kNm}$

$M_{R,fi} > M_{E,fi} \rightarrow OK$

Reduced yields strength and bending strength after a fire exposure of 90 minutes

Bar group 1	$T_1 = 740\ °C$
Bar group 2	$T_2 = 610\ °C$

Bar group 1	$k_{\theta,1} = 0.18 \rightarrow f_{yd,fi,1} = k_{\theta,1} f_{yd} = 40\ N/mm^2$
Bar group 2	$k_{\theta,2} = 0.45 \rightarrow f_{yd,fi,2} = k_{\theta,2} f_{yd} = 99\ N/mm^2$

Reduced yield strength	$f_{yd,fi} = \dfrac{2 \cdot f_{yd,fi,1} + 1 \cdot f_{yd,fi,2}}{3} = 59\ N/mm^2$

Depth of compression zone	$x = A_{sl,b} f_{yd,fi} / (0.85 f_{cd} b_{eff}) = 1\ mm$
Internal lever arm	$z = d_m - x/2 = 479\ mm$
Bending strength	$M_{R,fi} = A_{sl,b} f_{yd,fi} z = \mathbf{27\ kNm}$

$M_{R,fi} < M_{E,fi} \rightarrow NOT\ OK$

C2.2 Calculations at support

Calculations at room temperature

Design load and bending moment

Design load	$q_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 1.35 \cdot G + 1.5 \cdot Q = 37\ kN/m$
Elastic bending moment	$M_E = \dfrac{1}{12} q_d l^2 = 151\ kNm$

Design concrete strength and yield strength

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1.5 = 13.3\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1.15 = 191.3\ N/mm^2$

Bending strength

Depth of compression zone	$x = A_{sl,t} f_{yd} / (0.85 f_{cd} b_w) = 90\ mm$
Internal lever arm	$z = d_s - x/2 = 439\ mm$
Bending strength	$M_R = A_{sl,t} f_{yd} z = \mathbf{152\ kNm}$

$M_R > M_E$, so bending strength is OK.

Calculations after fire exposure

Design load and bending moment in case of a fire

Design load	$q_{fi} = G + \Psi_2 \cdot Q = G + 0.3 \cdot Q = 20.6\ kN/m$
Elastic bending moment	$M_{E,fi} = \frac{1}{12} q_{fi} l^2 = \mathbf{84\ kNm}$

Design concrete strength and yield strength in case of a fire

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1 = 20\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1 = 220\ N/mm^2$

Reduction of the cross-section after a fire exposure of 30 minutes

The top reinforcement bars lay at a depth of $h_f - c - \varphi_s - \varphi_{l,t} 2 = 74\ mm$ from the exposed concrete surface. According to Annex A of EN 1992-1-2, the temperature at this depth is below 200 °C after a fire exposure of 90 minutes. This means that the yield strength is not reduced. Besides, the temperature at this depth will even be lower when the plaster ceiling of 100 mm is taken into account. However, the concrete of the web of the beam will already reach temperatures above 500 °C after 30 minutes of fire exposure. So assuming that the concrete has no compressive strength above 500 °C and full compressive strength below 500 °C, the cross-section should be reduced, based on the depth of the 500 °C isotherm. This depth can be determined using annex A of EN 1992-1-2. For a fire exposure of 30 minutes, this will lead to:

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 10\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 320\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 474\ mm$

Bending strength after a fire exposure of 30 minutes

Depth of compression zone	$x = A_{sl,t} f_{yd} / (0.85 f_{cd} b_w) = 73\ mm$
Internal lever arm	$z = d_{s,fi} - x/2 = 437\ mm$
Bending strength	$M_{R,fi} = A_{sl,t} f_{yd} z = \mathbf{174\ kNm}$

$M_{R,fi} > M_{E,fi} \rightarrow OK$

Reduction of the cross-section after a fire exposure of 60 minutes

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 23\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 294\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 461\ mm$

Bending strength after a fire exposure of 60 minutes

Depth of compression zone	$x = A_{sl,t}f_{yd}/(0.85f_{cd}b_w) = 80\ mm$
Internal lever arm	$z = d_{s,fi} - x/2 = 421\ mm$
Bending strength	$M_{R,fi} = A_{sl,t}f_{yd}z = \mathbf{168\ kNm}$

$M_{R,fi} > M_{E,fi} \rightarrow OK$

Reduction of the cross-section after a fire exposure of 90 minutes

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 30\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 280\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 454\ mm$

Bending strength after a fire exposure of 90 minutes

Depth of compression zone	$x = A_{sl,t}f_{yd}/(0.85f_{cd}b_w) = 84\ mm$
Internal lever arm	$z = d_{s,fi} - x/2 = 412\ mm$
Bending strength	$M_{R,fi} = A_{sl,t}f_{yd}z = \mathbf{164\ kNm}$

$M_{R,fi} > M_{E,fi} \rightarrow OK$

C3. Modelling of the masonry wall

To determine the value of the translational spring which represents the translational restraint of the masonry wall, the wall is modelled as is shown in Figure C5. For this situation, the horizontal deflection of the wall can be expressed as:

Horizontal deflection

$$u = \frac{1}{48} \cdot \frac{Fh^3}{EI}$$

If it is assumed that $F = 1\ N$, the translational spring value can be calculated as:

Translational spring value

$$k_t = \frac{F}{u} = \frac{1}{u} = \frac{48EI}{h^3}$$

It is clear that before the translational spring value can be calculated, the moment of inertia and the Young's modulus have to be determined. The moment of inertia depends on the thickness and the effective width of the wall. It is assumed that the thickness of the wall amounts $t = 220\ mm$. The effective width of the wall is established according to the schematization shown in Figure C6. In this schematization, the slope between the floor and the load distribution of the masonry wall is assumed as 60° [100]. This means that the load of the T-beam with an effective width of 2420 mm will spread over a total width of $b = 2 \cdot (4200/\tan 60) + 2420 \approx 7620\ mm$. For the effective width of the masonry wall, the mean value of this total width and the effective width of the T-beam is used, which amounts $b_e = (7620 + 2420)/2 = 4840\ mm$ [100].

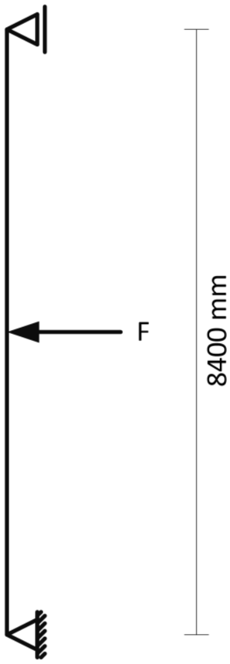


Figure C5 | Schematization of the masonry wall

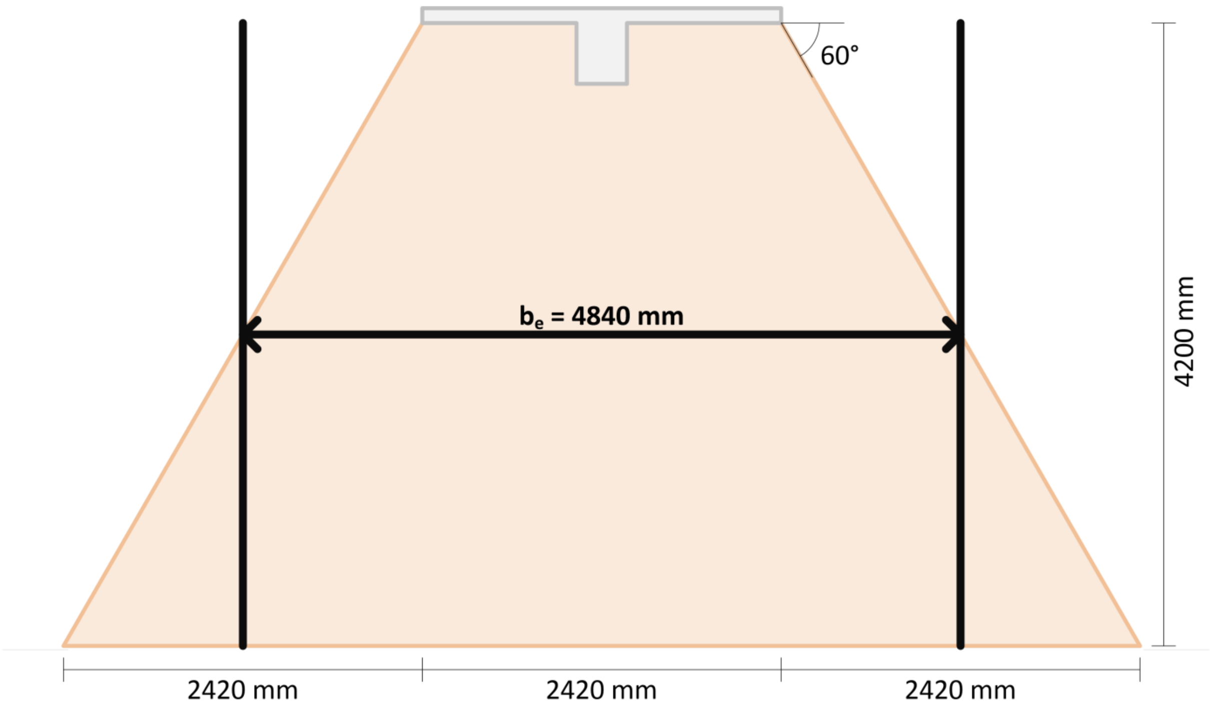


Figure C6 | Schematization of the load distribution of the masonry wall

According to EN 1996-1-1, a value of $E = K_E f_k$ for the Young's modulus may be used if the Young's modulus is not determined based on tests. For K_E, the standard EN 1996-1-1 recommends a value of 1000. For the characteristic compressive strength of the masonry, three different values are used: $f_k = 3.1\ N/mm^2$, $f_k = 7.5\ N/mm^2$, and $f_k = 10\ N/mm^2$. Based on these values and the relations mentioned above, the corresponding translational spring values are calculated, which are shown in Table C7.

Table C7 | The translational spring values, based on f_k, E, I, and h

$f_k\ [N/mm^2]$	$E\ [N/mm^2]$	$I\ [mm^4]$	$h\ [mm]$	$k_t\ [N/mm]$
3.1	3100	$4.30 \cdot 10^9$	8400	**1078**
7.5	7500	$4.30 \cdot 10^9$	8400	**2609**
10	10000	$4.30 \cdot 10^9$	8400	**3478**

C4. Calculation of flexural capacity of a T-beam (positive moments)

In paragraph 7.2, the flexural capacity that was calculated was based on the longitudinal reinforcement at the bottom of the T-beam, which should resist negative moments. However, the T-beam also contains longitudinal reinforcement at the top of the beam, which should resist positive moments. The amount of the top reinforcement was determined in Annex B3. In this Annex, the flexural capacity of this top reinforcement is calculated.

Information resulting from the measurements and assumptions

Beam span	$l = 7\ m$
Centre-to-centre distance	$b = 3.74\ m$
Web width	$b_w = 340\ mm$
Beam depth	$h = 510\ mm$
Floor depth	$h_f = 100\ mm$
Effective flange width	$b_{eff} = 2420\ mm$
Compressive strength concrete	$f_{ck} = 20\ N/mm^2$
Yield strength	$f_{yk} = 220\ N/mm^2$
Bar diameter of longitudinal reinforcement at top	$\varphi_{l,t} = 12\ mm$
Number of bars at top	$n_t = 8$
Total steel area at top	$A_{sl,t} = n_t \cdot \frac{1}{4}\varphi_{l,t}{}^2 = 905\ mm^2$
Bar diameter of stirrups	$\varphi_s = 10\ mm$
Cover	$c = 10\ mm$
Effective depth	$d = h - c - \varphi_s - \varphi_{l,t}/2 = 484\ mm$

Calculations at room temperature

Design concrete strength and yield strength

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1.5 = 13.3\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1.15 = 191.3\ N/mm^2$

Bending strength

Depth of compression zone	$x = A_{sl,t} f_{yd}/(0.85 f_{cd} b_w) = 45\ mm$
Internal lever arm	$z = d_s - x/2 = 462\ mm$
Bending strength	$M_R = A_{sl,t} f_{yd} z = \mathbf{80\ kNm}$

Calculations after fire exposure

Design concrete strength and yield strength in case of a fire

Design concrete compressive strength	$f_{cd} = f_{ck}/\gamma_c = f_{ck}/1 = 20\ N/mm^2$
Design yield strength	$f_{yd} = f_{yk}/\gamma_s = f_{yk}/1 = 220\ N/mm^2$

Reduction of the cross-section after a fire exposure of 30 minutes

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 10\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 320\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 474\ mm$

Bending strength after a fire exposure of 30 minutes

Depth of compression zone	$x = A_{sl,t}f_{yd}/(0.85f_{cd}b_w) = 37\ mm$
Internal lever arm	$z = d_{s,fi} - x/2 = 456\ mm$
Bending strength	$M_{R,fi} = A_{sl,t}f_{yd}z = \mathbf{91\ kNm}$

Reduction of the cross-section after a fire exposure of 60 minutes

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 23\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 294\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 461\ mm$

Bending strength after a fire exposure of 60 minutes

Depth of compression zone	$x = A_{sl,t}f_{yd}/(0.85f_{cd}b_w) = 40\ mm$
Internal lever arm	$z = d_{s,fi} - x/2 = 441\ mm$
Bending strength	$M_{R,fi} = A_{sl,t}f_{yd}z = \mathbf{88\ kNm}$

Reduction of the cross-section after a fire exposure of 90 minutes

Depth of the 500 °C isotherm (assuming one-dimensional heat transfer)	$c_{fi} = 30\ mm$
Reduced web width	$b_{w,fi} = b_w - 2c_{fi} = 280\ mm$
Reduced effective depth	$d_{s,fi} = d_s - c_{fi} = 454\ mm$

Bending strength after a fire exposure of 90 minutes

Depth of compression zone	$x = A_{sl,t}f_{yd}/(0.85f_{cd}b_w) = 42\ mm$

Internal lever arm $z = d_{s,fi} - x/2 = 433\ mm$

Bending strength $M_{R,fi} = A_{sl,t} f_{yd} z = \mathbf{86\ kNm}$

ANNEX D

This Annex shows the results of the models of the T-beam in Technosoft in the following order:

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (ingeklemd)
Dimensies: kN;m;rad (tenzij anders aangegeven)
Datum....: 19/09/2016
Bestand..: h:\afstuderen\iob map\aangepast\ingeklemd.rww

Rekenmodel.......: 1e-orde-elastisch.
Theorie voor de bepaling van de krachtsverdeling:
Geometrisch lineair.
Fysisch lineair.

Gunstige werking van de permanente belasting wordt automatisch verwerkt.

Toegepaste normen volgens Eurocode (CEN)

Belastingen	EN 1990:2002	C2:2010
	EN 1991-1-1:2002	C1:2009

MATERIALEN

Mt	Omschrijving	E-modulus[N/mm2]	S.M.	Pois.	Uitz. coëff
1	C20/25	30000	25.0	0.20	1.0000e-005

MATERIALEN vervolg

Mt	Omschrijving	Cement	Kruipfac.	Toeslag	Rho[kg/m3]
1	C20/25	N	3.01	Normaal	2400

PROFIELEN [mm]

Prof.	Omschrijving	Materiaal	Oppervlak	Traagheid	Vormf.
1	B*H 2420*510	1:C20/25	3.8140e+005	7.9059e+009	0.00

PROFIELEN vervolg [mm]

Prof.	Staaftype	Breedte	Hoogte	e	Type	b1	h1	b2	h2
1	0:Normaal	2420	510	366.8	5:T1	1040	410	1040	410

KNOPEN

Knoop	X	Z
1	0.000	0.000
2	7.000	0.000

STAVEN

St.	ki	kj	Profiel	Aansl.i	Aansl.j	Lengte	Opm.
1	1	2	1:B*H 2420*510	NDM	NDM	7.000	

BRANDGEGEVENS

Brand Nr.	Omschrijving	Eis [min]	Verhit. wijze	Profiel- volgend	Soort	P [1/m]	dikte [mm]

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (ingeklemd)

STAVEN - BRANDGEGEVENS

St.	Brandgegevens	Vervalt bij brand
1		nee

VASTE STEUNPUNTEN

Nr.	knoop	Kode XZR 1=vast 0=vrij	Hoek	Vervalt bij brand
1	1	111	0.00	nee
2	2	111	0.00	nee

BELASTINGGEVALLEN

B.G.	Omschrijving		Type	
1	Permanente belasting	EGZ=0.00	1	
2	Variabele belasting		2	Ver. bel. pers. ed. (p_rep)
3	Constante temperatuurverdeling [30]		24	Temperatuursverschillen
4	Lineaire temperatuurverdeling [30]		24	Temperatuursverschillen

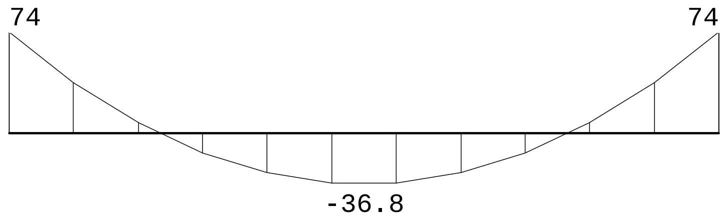
STAAFBELASTINGEN

B.G:1 Permanente belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-18.00	-18.00	0.000	0.000			

MOMENTEN

B.G:1 Permanente belasting



NORMAALKRACHTEN

B.G:1 Permanente belasting

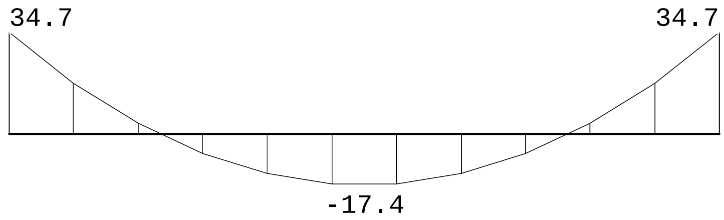
STAAFBELASTINGEN

B.G:2 Variabele belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-8.50	-8.50	0.000	0.000	0.7	0.5	0.3

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (ingeklemd)

MOMENTEN
B.G:2 Variabele belasting



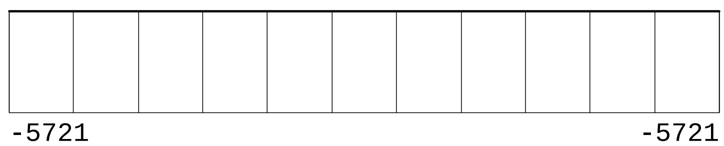
NORMAALKRACHTEN
B.G:2 Variabele belasting

TEMPERATUUR BELASTINGEN
B.G:3 Constante temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	50.000	50.000	0.7	0.5	0.3

MOMENTEN
B.G:3 Constante temperatuurverdeling [30]

NORMAALKRACHTEN
B.G:3 Constante temperatuurverdeling [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (ingeklemd)

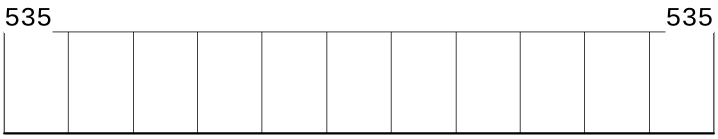
TEMPERATUUR BELASTINGEN

B.G:4 Lineaire temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-32.000	83.000	0.7	0.5	0.3

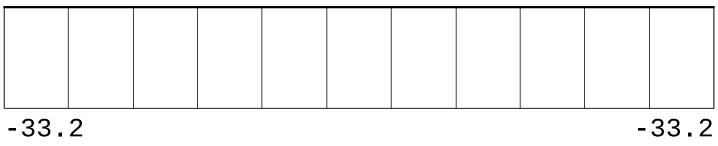
MOMENTEN

B.G:4 Lineaire temperatuurverdeling [30]



NORMAALKRACHTEN

B.G:4 Lineaire temperatuurverdeling [30]



BELASTINGCOMBINATIES

BC	Type						
1	Fund.	1.00 $G_{k,1}$	+ 0.30 $Q_{k,2}$				
2	Fund.	1.00 $G_{k,1}$	+ 0.30 $Q_{k,2}$	+	1.00 $Q_{k,3}$	+	1.00 $Q_{k,4}$

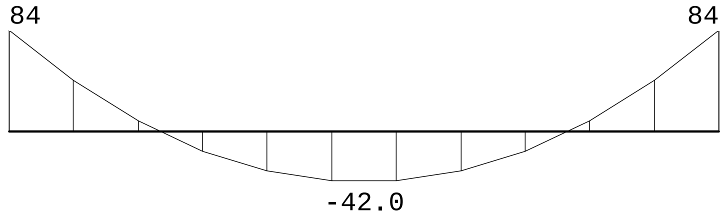
GUNSTIGE WERKING PERMANENTE BELASTINGEN

BC	Staven met gunstige werking
1	Alle staven de factor:1.00
2	Alle staven de factor:1.00

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (ingeklemd)

BELASTINGCOMBINATIE B.C:1 Brand

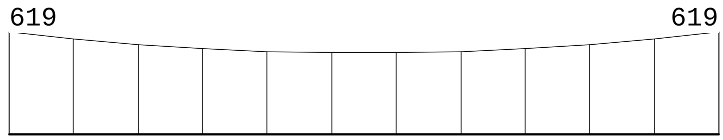
MOMENTEN
B.C:1 Brand



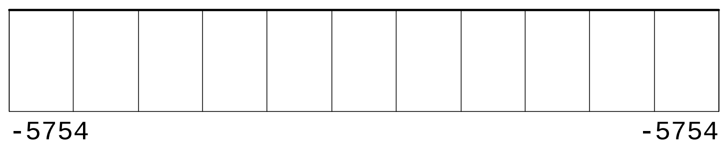
NORMAALKRACHTEN
B.C:1 Brand

BELASTINGCOMBINATIE B.C:2 Brand incl. th. uitzettingen

MOMENTEN
B.C:2 Brand incl. th. uitzettingen



NORMAALKRACHTEN
B.C:2 Brand incl. th. uitzettingen



Project	:	Case study "Hof van Maerlant"
Onderdeel	:	Uitknikken T-ligger met kniklengtefactor 0.5
Dimensies	:	kN;m;rad (tenzij anders aangegeven)
Datum	:	23/09/2016
Bestand	:	H:\Afstuderen\IOB map\Aangepast\T-ligger als kolom (factor 0,5).klw
Referentieperiode	:	50

Toegepaste normen volgens Eurocode met Nederlandse NB

Beton	NEN-EN 1992-1-1:2011(nl)	C2:2011(nl)	NB:2011(nl)

Geometrie

Type constructie		:	Kolom Rechthoekig Enkel excentrisch belast
Kolomafmeting in X/Y (=b*h)	[mm]	:	340 * 654
Kolomhoogte (L)	[mm]	:	7000
Belastingschema		:	Geschoord
Kniklengtefactor X		:	0.50
Pendelkolom		:	Nee

Belasting

			BG1	BG2	BG3	Maatgevend BC
Omschrijving belastinggeval		:				
Normaalkracht N Ek	[kN]	:	5721.00	0.00	0.00	5721.00
MEk,X boven	[kNm]	:	0.00	0.00	0.00	0.00
MEk,X onder	[kNm]	:	0.00	0.00	0.00	0.00
Belastingfactoren						
BC1	Fundamenteel	:	1.00	0.00	0.00	Maatgevend X

Beton en Wapening

Betonkwaliteit		:	C20/25	Prefab	:	Nee
Soort spanningsrekdiagram		:	Parabolisch - rechthoekig diagram			
Staalsoort		:	User	Symm.wapening	:	2-zijdig
f_{yk}	[N/mm^2]	:	220	ε_{uk} [‰]	:	2.5
Soort spanningsrekdiagram		:	Bi-lineair diagram met klimmende tak			
Basiswapening	[mm]	:	4 ø25	Bijlegw.[mm]	:	ø25, 25
Beugels	[mm]	:	ø10			

Betondekking

Milieu	:			XC1
Gestort tegen bestaand beton	:			Nee
Element met plaatgeometrie	:			Nee
Specifieke kwaliteitsbeheersing	:			Nee
Oneffen beton oppervlak	:			Nee
Ondergrond	:			Glad / N.v.t.
Constructieklasse	:			S4
Grootste korrel	:			31.5
Hoofdwapening	:			2de laag
Nominale dekking	:			30
Toegepaste dekking	:			40
Gelijkwaardige diameter	:			25
$C_{min,b}$ $C_{min,dur}$ ΔC_{dur}	:	25	15	0
C_{min} ΔC_{dev} C_{nom}	:	25	5	30

Project : Case study "Hof van Maerlant"
Onderdeel : Uitknikken T-ligger met kniklengtefactor 0.5

Betondekking

Beugel / Verdeelwapening	:			1ste laag
Nominale dekking	:			20
Toegepaste dekking	:			30
Gelijkwaardige diameter	:			10
$C_{min,b}$ $C_{min,dur}$ ΔC_{dur}	:	10	15	0
C_{min} ΔC_{dev} C_{nom}	:	15	5	20

Project : Case study "Hof van Maerlant"
Onderdeel : Uitknikken T-ligger met kniklengtefactor 0.5

Belastingcombinatie 1: (Fundamenteel)

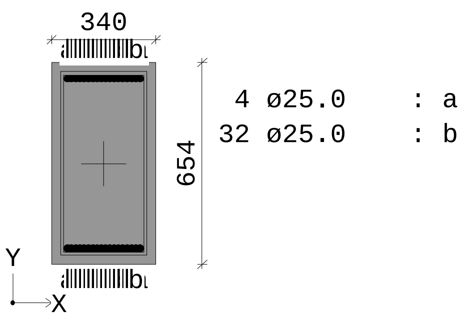
Berekende gegevens			X-as	Y-as	BC1
Beginexcentriciteit e_{02}	[mm]	:	0.0		
Beginexcentriciteit e_{01}	[mm]	:	0.0		
Excentriciteit e_i	[mm]	:	4.4		
Excentriciteit e_2	[mm]	:	0.9		
Totale excentriciteit e_t	[mm]	:	21.8		
Min. wapening art. 9.5.2(2)	[mm2]	:	2990.5		
Min. wap. art. 9.5.2(2)&(4)	[mm2]	:	201.1 = 4 ø8.0		
Min. wap. art. 7.3.2	[mm2]	:	0.0		
Totaal ber. wap. 1e/2e orde	[mm2]	:	16916.7		
Maatgevende wapening	[mm2]	:	16916.7		

Project : Case study "Hof van Maerlant"
Onderdeel : Uitknikken T-ligger met kniklengtefactor 0.5

Maatgevende belastingcombinatie 1: (Fundamenteel)

Gevonden wapening		basiswapening	X-as	Y-as
Bijlegcombinatie 1	17671 [mm2] :	4 ø25.0	32 ø25.0	

Grafische uitvoer bijlegcombinatie 1



Opmerkingen
[101] De berekende wapening is de totale wapening in de doorsnede.
[69] Bijlegcomb. 1 X-ri voldoet met -11.2 mm niet aan minimale tussenruimte 36.5 mm (art. 8.2(3))
[87] Bijlegcomb. 1 : wapeningspercentage 7.9 is te hoog: boven 4.0% (art. 9.5.2(3))
[113] Twee-zijdige wapening
[108] Gevonden wapening onverminderd toepassen over gehele kolomhoogte

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)
Dimensies: kN;m;rad (tenzij anders aangegeven)
Datum....: 19/09/2016
Bestand..: H:\Afstuderen\IOB map\Aangepast\Verend.rww

Rekenmodel.......: 1e-orde-elastisch.
Theorie voor de bepaling van de krachtsverdeling:
Geometrisch lineair.
Fysisch lineair.

Gunstige werking van de permanente belasting wordt automatisch verwerkt.

Toegepaste normen volgens Eurocode (CEN)

Belastingen	EN 1990:2002	C2:2010
	EN 1991-1-1:2002	C1:2009

MATERIALEN

Mt	Omschrijving	E-modulus[N/mm2]	S.M.	Pois.	Uitz. coëff
1	C20/25	30000	25.0	0.20	1.0000e-005

MATERIALEN vervolg

Mt	Omschrijving	Cement	Kruipfac.	Toeslag	Rho[kg/m3]
1	C20/25	N	3.01	Normaal	2400

PROFIELEN [mm]

Prof.	Omschrijving	Materiaal	Oppervlak	Traagheid	Vormf.
1	B*H 2420*510	1:C20/25	3.8140e+005	7.9059e+009	0.00

PROFIELEN vervolg [mm]

Prof.	Staaftype	Breedte	Hoogte	e	Type	b1	h1	b2	h2
1	0:Normaal	2420	510	366.8	5:T1	1040	410	1040	410

KNOPEN

Knoop	X	Z
1	0.000	0.000
2	7.000	0.000

STAVEN

St.	ki	kj	Profiel	Aansl.i	Aansl.j	Lengte	Opm.
1	1	2	1:B*H 2420*510	NDM	NDM	7.000	

BRANDGEGEVENS

Brand Nr.	Omschrijving	Eis [min]	Verhit. wijze	Profiel-volgend	Soort	P [1/m]	dikte [mm]

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

STAVEN - BRANDGEGEVENS

St. Brandgegevens	Vervalt bij brand
1	nee

VASTE STEUNPUNTEN

Nr.	knoop	Kode XZR 1=vast 0=vrij	Hoek	Vervalt bij brand
1	1	010	0.00	nee
2	2	010	0.00	nee

VEREN

Veer	Knoop	Richting	Hoek	Veerwaarde	Type	Ondergrens	Bovengrens	Vervalt bij brand
1	1	1:X-transl.	0.00	3.480e+003	Normaal	-1.000e+010	1.000e+010	nee
2	1	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee
3	2	1:X-transl.	0.00	3.480e+003	Normaal	-1.000e+010	1.000e+010	nee
4	2	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee

BELASTINGGEVALLEN

B.G.	Omschrijving		Type
1	Permanente belasting	EGZ=0.00	1
2	Variabele belasting		2 Ver. bel. pers. ed. (p_rep)
3	Constante temperatuurverdeling [30]		24 Temperatuursverschillen
4	Lineaire temperatuurverdeling [30]		24 Temperatuursverschillen
5	Constante temperatuurverdeling [60]		24 Temperatuursverschillen
6	Lineaire temperatuurverdeling [60]		24 Temperatuursverschillen
7	Constante temperatuurverdeling [90]		24 Temperatuursverschillen
8	Lineaire temperatuurverdeling [90]		24 Temperatuursverschillen

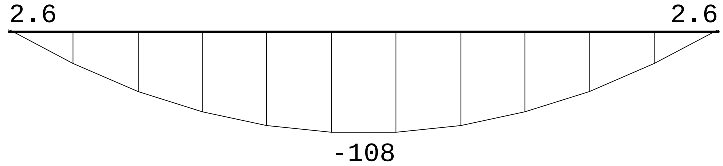
STAAFBELASTINGEN

B.G:1 Permanente belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-18.00	-18.00	0.000	0.000			

MOMENTEN

B.G:1 Permanente belasting



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN
B.G:1 Permanente belasting

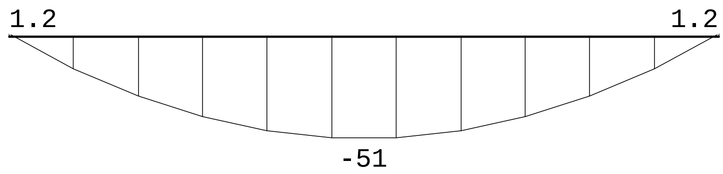
VERPLAATSINGEN [mm;rad]
B.G:1 Permanente belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00105
2	0.00	0.00	-0.00105

STAAFBELASTINGEN
B.G:2 Variabele belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-8.50	-8.50	0.000	0.000	0.7	0.5	0.3

MOMENTEN
B.G:2 Variabele belasting



NORMAALKRACHTEN
B.G:2 Variabele belasting

VERPLAATSINGEN [mm;rad]
B.G:2 Variabele belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00049
2	0.00	0.00	-0.00049

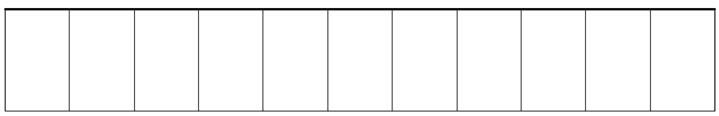
Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

TEMPERATUUR BELASTINGEN
B.G:3 Constante temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	50.000	50.000	0.7	0.5	0.3

MOMENTEN
B.G:3 Constante temperatuurverdeling [30]

NORMAALKRACHTEN
B.G:3 Constante temperatuurverdeling [30]



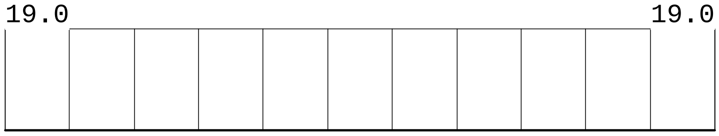
VERPLAATSINGEN [mm;rad]
B.G:3 Constante temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-1.75	0.00	0.00000
2	1.75	0.00	0.00000

TEMPERATUUR BELASTINGEN
B.G:4 Lineaire temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-32.000	83.000	0.7	0.5	0.3

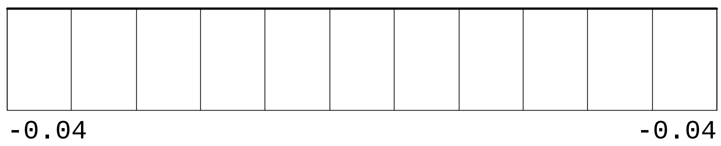
MOMENTEN
B.G:4 Lineaire temperatuurverdeling [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:4 Lineaire temperatuurverdeling [30]



VERPLAATSINGEN

[mm;rad] B.G:4 Lineaire temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.00761
2	0.01	0.00	-0.00761

TEMPERATUUR BELASTINGEN

B.G:5 Constante temperatuurverdeling [60]

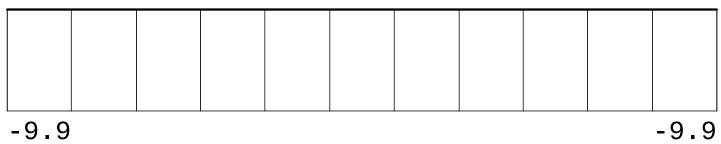
Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	81.000	81.000	0.7	0.5	0.3

MOMENTEN

B.G:5 Constante temperatuurverdeling [60]

NORMAALKRACHTEN

B.G:5 Constante temperatuurverdeling [60]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

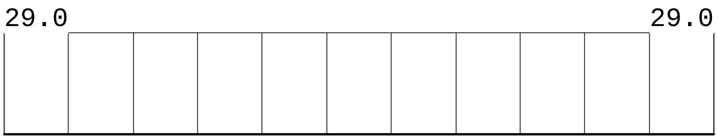
VERPLAATSINGEN [mm;rad] B.G:5 Constante temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-2.83	0.00	0.00000
2	2.83	0.00	0.00000

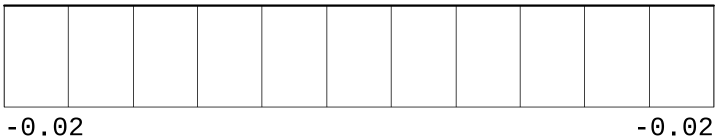
TEMPERATUUR BELASTINGEN B.G:6 Lineaire temperatuurverdeling [60]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-49.000	126.000	0.7	0.5	0.3

MOMENTEN B.G:6 Lineaire temperatuurverdeling [60]



NORMAALKRACHTEN B.G:6 Lineaire temperatuurverdeling [60]



VERPLAATSINGEN [mm;rad] B.G:6 Lineaire temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.00	0.00	0.01158
2	0.00	0.00	-0.01158

TEMPERATUUR BELASTINGEN B.G:7 Constante temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	105.000	105.000	0.7	0.5	0.3

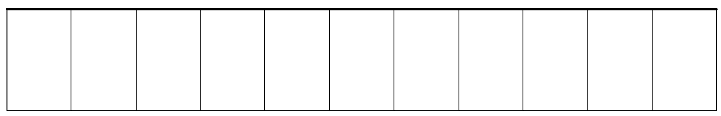
Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

MOMENTEN

B.G:7 Constante temperatuurverdeling [90]

NORMAALKRACHTEN

B.G:7 Constante temperatuurverdeling [90]



VERPLAATSINGEN [mm;rad]

B.G:7 Constante temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-3.67	0.00	0.00000
2	3.67	0.00	0.00000

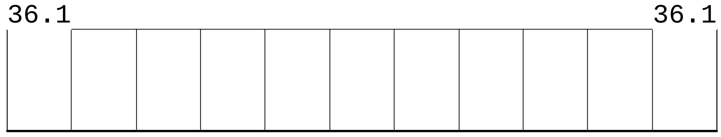
TEMPERATUUR BELASTINGEN

B.G:8 Lineaire temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-61.000	157.000	0.7	0.5	0.3

MOMENTEN

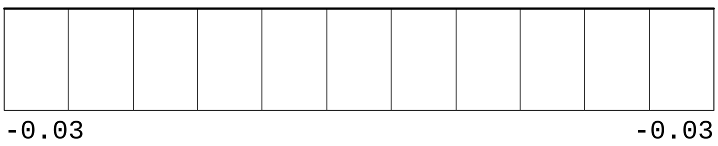
B.G:8 Lineaire temperatuurverdeling [90]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:8 Lineaire temperatuurverdeling [90]



VERPLAATSINGEN [mm;rad]

B.G:8 Lineaire temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.01443
2	0.01	0.00	-0.01443

BELASTINGCOMBINATIES

BC	Type							
1	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$				
2	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,3}$	+	1.00 $Q_{k,4}$
3	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,5}$	+	1.00 $Q_{k,6}$
4	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,7}$	+	1.00 $Q_{k,8}$

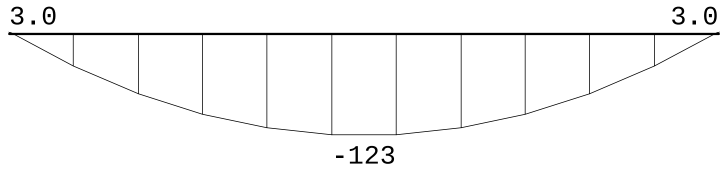
GUNSTIGE WERKING PERMANENTE BELASTINGEN

BC	Staven met gunstige werking
1	Alle staven de factor:1.00
2	Alle staven de factor:1.00
3	Alle staven de factor:1.00
4	Alle staven de factor:1.00

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:1 Brand

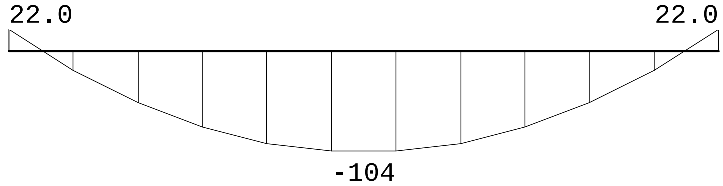
MOMENTEN B.C:1 Brand



NORMAALKRACHTEN B.C:1 Brand

BELASTINGCOMBINATIE B.C:2 Brand incl. th. uitzettingen [30]

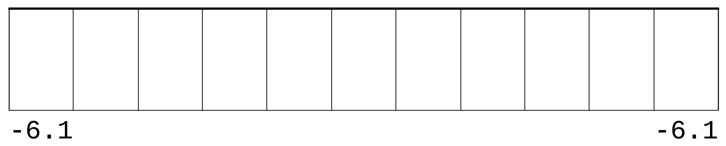
MOMENTEN B.C:2 Brand incl. th. uitzettingen [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

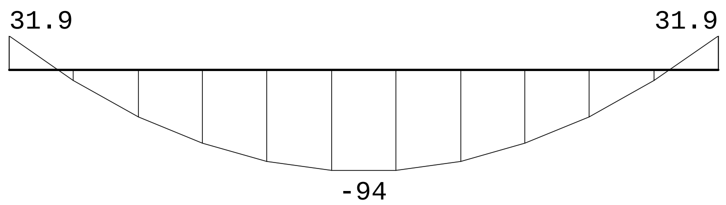
B.C:2 Brand incl. th. uitzettingen [30]



BELASTINGCOMBINATIE B.C:3 Brand incl. th. uitzettingen [60]

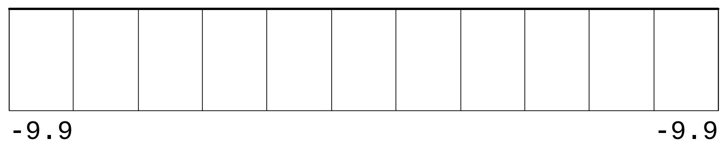
MOMENTEN

B.C:3 Brand incl. th. uitzettingen [60]



NORMAALKRACHTEN

B.C:3 Brand incl. th. uitzettingen [60]

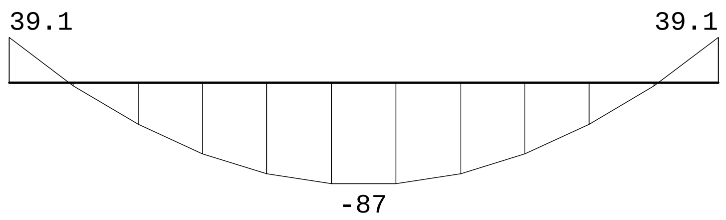


Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:4 Brand incl. th. uitzettingen [90]

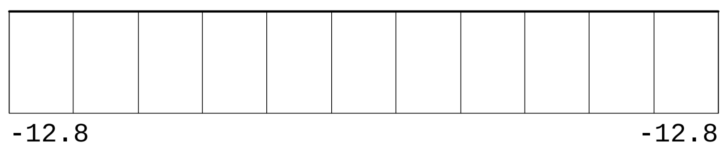
MOMENTEN

B.C:4 Brand incl. th. uitzettingen [90]



NORMAALKRACHTEN

B.C:4 Brand incl. th. uitzettingen [90]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)
Dimensies: kN;m;rad (tenzij anders aangegeven)
Datum....: 19/09/2016
Bestand..: H:\Afstuderen\IOB map\Aangepast\Verend.rww

Rekenmodel.......: 1e-orde-elastisch.
Theorie voor de bepaling van de krachtsverdeling:
Geometrisch lineair.
Fysisch lineair.

Gunstige werking van de permanente belasting wordt automatisch verwerkt.

Toegepaste normen volgens Eurocode (CEN)

Belastingen	EN 1990:2002	C2:2010
	EN 1991-1-1:2002	C1:2009

MATERIALEN

Mt	Omschrijving	E-modulus[N/mm2]	S.M.	Pois.	Uitz. coëff
1	C20/25	30000	25.0	0.20	1.0000e-005

MATERIALEN vervolg

Mt	Omschrijving	Cement	Kruipfac.	Toeslag	Rho[kg/m3]
1	C20/25	N	3.01	Normaal	2400

PROFIELEN [mm]

Prof.	Omschrijving	Materiaal	Oppervlak	Traagheid	Vormf.
1	B*H 2420*510	1:C20/25	3.8140e+005	7.9059e+009	0.00

PROFIELEN vervolg [mm]

Prof.	Staaftype	Breedte	Hoogte	e	Type	b1	h1	b2	h2
1	0:Normaal	2420	510	366.8	5:T1	1040	410	1040	410

KNOPEN

Knoop	X	Z
1	0.000	0.000
2	7.000	0.000

STAVEN

St.	ki	kj	Profiel	Aansl.i	Aansl.j	Lengte	Opm.
1	1	2	1:B*H 2420*510	NDM	NDM	7.000	

BRANDGEGEVENS

Brand Nr.	Omschrijving	Eis [min]	Verhit. wijze	Profiel- volgend	Soort	P [1/m]	dikte [mm]

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

STAVEN - BRANDGEGEVENS

St.	Brandgegevens	Vervalt bij brand
1		nee

VASTE STEUNPUNTEN

Nr.	knoop	Kode XZR 1=vast 0=vrij	Hoek	Vervalt bij brand
1	1	010	0.00	nee
2	2	010	0.00	nee

VEREN

Veer	Knoop	Richting	Hoek	Veerwaarde	Type	Ondergrens	Bovengrens	Vervalt bij brand
1	1	1:X-transl.	0.00	2.610e+003	Normaal	-1.000e+010	1.000e+010	nee
2	1	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee
3	2	1:X-transl.	0.00	2.610e+003	Normaal	-1.000e+010	1.000e+010	nee
4	2	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee

BELASTINGGEVALLEN

B.G.	Omschrijving		Type	
1	Permanente belasting	EGZ=0.00	1	
2	Variabele belasting		2	Ver. bel. pers. ed. (p_rep)
3	Constante temperatuurverdeling [30]		24	Temperatuursverschillen
4	Lineaire temperatuurverdeling [30]		24	Temperatuursverschillen
5	Constante temperatuurverdeling [60]		24	Temperatuursverschillen
6	Lineaire temperatuurverdeling [60]		24	Temperatuursverschillen
7	Constante temperatuurverdeling [90]		24	Temperatuursverschillen
8	Lineaire temperatuurverdeling [90]		24	Temperatuursverschillen

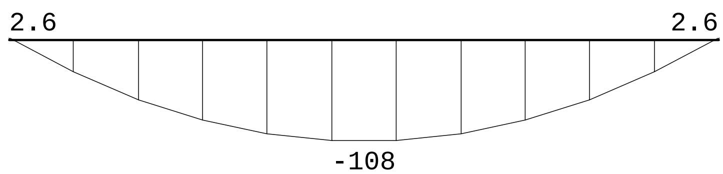
STAAFBELASTINGEN

B.G:1 Permanente belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-18.00	-18.00	0.000	0.000			

MOMENTEN

B.G:1 Permanente belasting



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:1 Permanente belasting

VERPLAATSINGEN [mm;rad]

B.G:1 Permanente belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00105
2	0.00	0.00	-0.00105

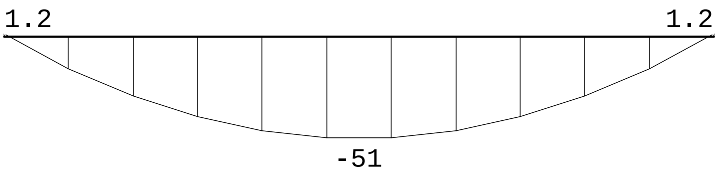
STAAFBELASTINGEN

B.G:2 Variabele belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-8.50	-8.50	0.000	0.000	0.7	0.5	0.3

MOMENTEN

B.G:2 Variabele belasting



NORMAALKRACHTEN

B.G:2 Variabele belasting

VERPLAATSINGEN [mm;rad]

B.G:2 Variabele belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00049
2	0.00	0.00	-0.00049

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

TEMPERATUUR BELASTINGEN

B.G:3 Constante temperatuurverdeling [30]

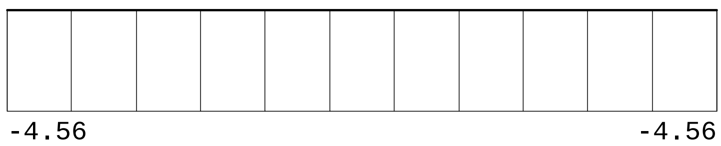
Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	50.000	50.000	0.7	0.5	0.3

MOMENTEN

B.G:3 Constante temperatuurverdeling [30]

NORMAALKRACHTEN

B.G:3 Constante temperatuurverdeling [30]



VERPLAATSINGEN [mm;rad]

B.G:3 Constante temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-1.75	0.00	0.00000
2	1.75	0.00	0.00000

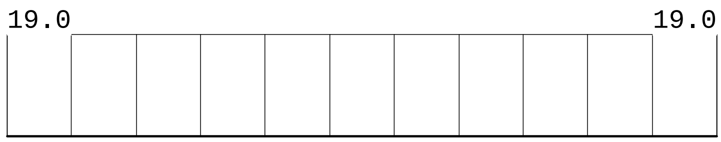
TEMPERATUUR BELASTINGEN

B.G:4 Lineaire temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-32.000	83.000	0.7	0.5	0.3

MOMENTEN

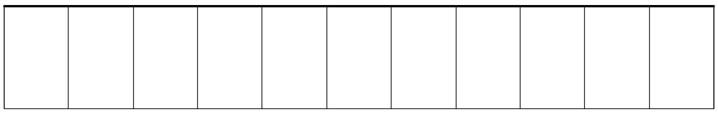
B.G:4 Lineaire temperatuurverdeling [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:4 Lineaire temperatuurverdeling [30]



VERPLAATSINGEN

[mm;rad] B.G:4 Lineaire temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.00761
2	0.01	0.00	-0.00761

TEMPERATUUR BELASTINGEN

B.G:5 Constante temperatuurverdeling [60]

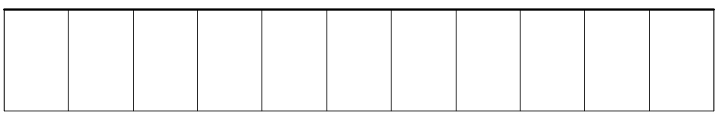
Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	81.000	81.000	0.7	0.5	0.3

MOMENTEN

B.G:5 Constante temperatuurverdeling [60]

NORMAALKRACHTEN

B.G:5 Constante temperatuurverdeling [60]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

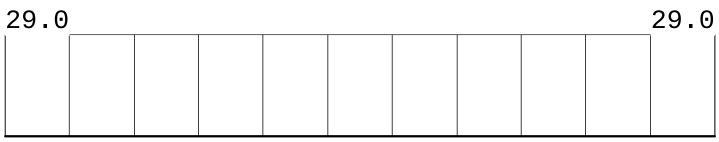
VERPLAATSINGEN [mm;rad] B.G:5 Constante temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-2.83	0.00	0.00000
2	2.83	0.00	0.00000

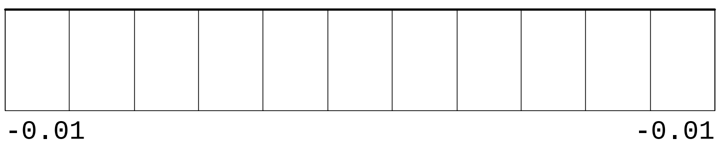
TEMPERATUUR BELASTINGEN B.G:6 Lineaire temperatuurverdeling [60]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-49.000	126.000	0.7	0.5	0.3

MOMENTEN B.G:6 Lineaire temperatuurverdeling [60]



NORMAALKRACHTEN B.G:6 Lineaire temperatuurverdeling [60]



VERPLAATSINGEN [mm;rad] B.G:6 Lineaire temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.00	0.00	0.01158
2	0.00	0.00	-0.01158

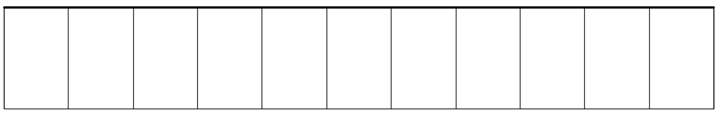
TEMPERATUUR BELASTINGEN B.G:7 Constante temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	105.000	105.000	0.7	0.5	0.3

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

MOMENTEN
B.G:7 Constante temperatuurverdeling [90]

NORMAALKRACHTEN
B.G:7 Constante temperatuurverdeling [90]



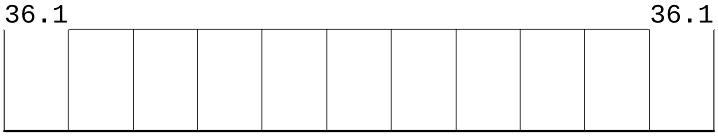
VERPLAATSINGEN
[mm;rad] B.G:7 Constante temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-3.67	0.00	0.00000
2	3.67	0.00	0.00000

TEMPERATUUR BELASTINGEN
B.G:8 Lineaire temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-61.000	157.000	0.7	0.5	0.3

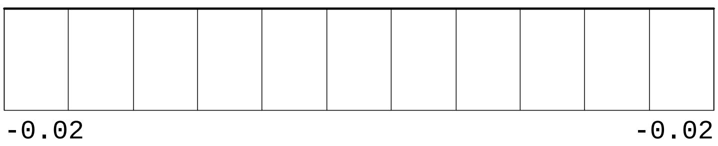
MOMENTEN
B.G:8 Lineaire temperatuurverdeling [90]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:8 Lineaire temperatuurverdeling [90]



VERPLAATSINGEN

[mm;rad] B.G:8 Lineaire temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.01443
2	0.01	0.00	-0.01443

BELASTINGCOMBINATIES

BC	Type							
1	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$				
2	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,3}$	+	1.00 $Q_{k,4}$
3	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,5}$	+	1.00 $Q_{k,6}$
4	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,7}$	+	1.00 $Q_{k,8}$

GUNSTIGE WERKING PERMANENTE BELASTINGEN

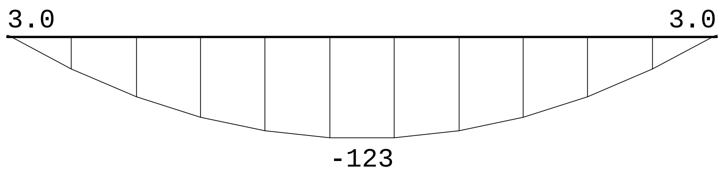
BC	Staven met gunstige werking
1	Alle staven de factor:1.00
2	Alle staven de factor:1.00
3	Alle staven de factor:1.00
4	Alle staven de factor:1.00

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:1 Brand

MOMENTEN

B.C:1 Brand



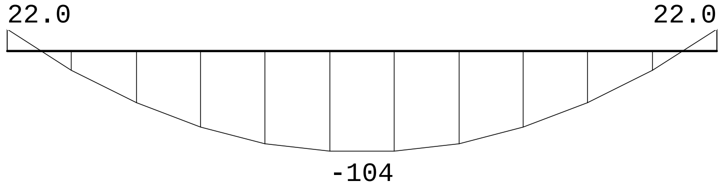
NORMAALKRACHTEN

B.C:1 Brand

BELASTINGCOMBINATIE B.C:2 Brand incl. th. uitzettingen [30]

MOMENTEN

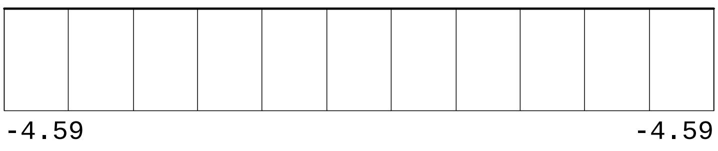
B.C:2 Brand incl. th. uitzettingen [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

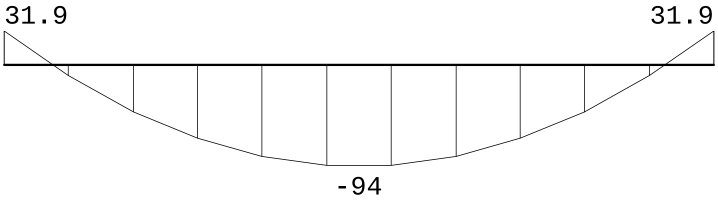
B.C:2 Brand incl. th. uitzettingen [30]



BELASTINGCOMBINATIE B.C:3 Brand incl. th. uitzettingen [60]

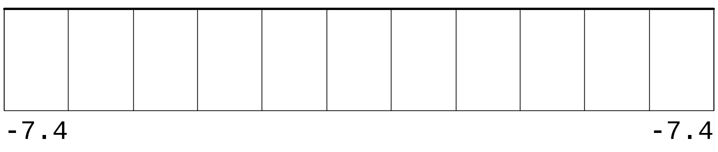
MOMENTEN

B.C:3 Brand incl. th. uitzettingen [60]



NORMAALKRACHTEN

B.C:3 Brand incl. th. uitzettingen [60]

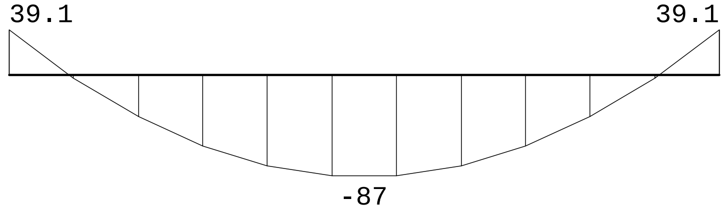


Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:4 Brand incl. th. uitzettingen [90]

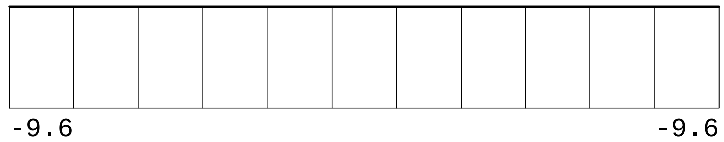
MOMENTEN

B.C:4 Brand incl. th. uitzettingen [90]



NORMAALKRACHTEN

B.C:4 Brand incl. th. uitzettingen [90]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)
Dimensies: kN;m;rad (tenzij anders aangegeven)
Datum....: 19/09/2016
Bestand..: H:\Afstuderen\IOB map\Aangepast\Verend.rww

Rekenmodel.......: 1e-orde-elastisch.
Theorie voor de bepaling van de krachtsverdeling:
Geometrisch lineair.
Fysisch lineair.

Gunstige werking van de permanente belasting wordt automatisch verwerkt.

Toegepaste normen volgens Eurocode (CEN)

Belastingen	EN 1990:2002	C2:2010
	EN 1991-1-1:2002	C1:2009

MATERIALEN

Mt	Omschrijving	E-modulus[N/mm2]	S.M.	Pois.	Uitz. coëff
1	C20/25	30000	25.0	0.20	1.0000e-005

MATERIALEN vervolg

Mt	Omschrijving	Cement	Kruipfac.	Toeslag	Rho[kg/m3]
1	C20/25	N	3.01	Normaal	2400

PROFIELEN [mm]

Prof.	Omschrijving	Materiaal	Oppervlak	Traagheid	Vormf.
1	B*H 2420*510	1:C20/25	3.8140e+005	7.9059e+009	0.00

PROFIELEN vervolg [mm]

Prof.	Staaftype	Breedte	Hoogte	e	Type	b1	h1	b2	h2
1	0:Normaal	2420	510	366.8	5:T1	1040	410	1040	410

KNOPEN

Knoop	X	Z
1	0.000	0.000
2	7.000	0.000

STAVEN

St.	ki	kj	Profiel	Aansl.i	Aansl.j	Lengte	Opm.
1	1	2	1:B*H 2420*510	NDM	NDM	7.000	

BRANDGEGEVENS

Brand Nr.	Omschrijving	Eis [min]	Verhit. wijze	Profiel-volgend	Soort	P [1/m]	dikte [mm]

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

STAVEN - BRANDGEGEVENS

St. Brandgegevens	Vervalt bij brand
1	nee

VASTE STEUNPUNTEN

Nr.	knoop	Kode XZR 1=vast 0=vrij	Hoek	Vervalt bij brand
1	1	010	0.00	nee
2	2	010	0.00	nee

VEREN

Veer	Knoop	Richting	Hoek	Veerwaarde	Type	Ondergrens	Bovengrens	Vervalt bij brand
1	1	1:X-transl.	0.00	1.080e+003	Normaal	-1.000e+010	1.000e+010	nee
2	1	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee
3	2	1:X-transl.	0.00	1.080e+003	Normaal	-1.000e+010	1.000e+010	nee
4	2	3:Rotatie	0.00	2.500e+003	Normaal	-1.000e+010	1.000e+010	nee

BELASTINGGEVALLEN

B.G.	Omschrijving		Type	
1	Permanente belasting	EGZ=0.00	1	
2	Variabele belasting		2	Ver. bel. pers. ed. (p_rep)
3	Constante temperatuurverdeling [30]		24	Temperatuursverschillen
4	Lineaire temperatuurverdeling [30]		24	Temperatuursverschillen
5	Constante temperatuurverdeling [60]		24	Temperatuursverschillen
6	Lineaire temperatuurverdeling [60]		24	Temperatuursverschillen
7	Constante temperatuurverdeling [90]		24	Temperatuursverschillen
8	Lineaire temperatuurverdeling [90]		24	Temperatuursverschillen

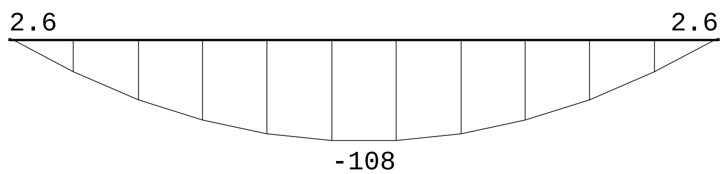
STAAFBELASTINGEN

B.G:1 Permanente belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-18.00	-18.00	0.000	0.000			

MOMENTEN

B.G:1 Permanente belasting



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN
B.G:1 Permanente belasting

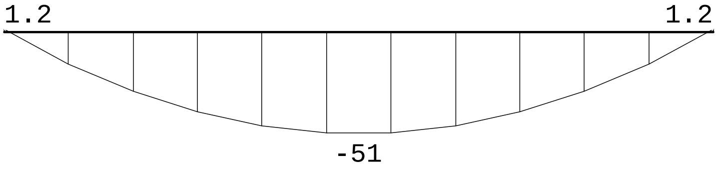
VERPLAATSINGEN [mm;rad]
B.G:1 Permanente belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00105
2	0.00	0.00	-0.00105

STAAFBELASTINGEN
B.G:2 Variabele belasting

Staaf	Type	q1/p/m	q2	A	B	ψ_0	ψ_1	ψ_2
1	1:QZLokaal	-8.50	-8.50	0.000	0.000	0.7	0.5	0.3

MOMENTEN
B.G:2 Variabele belasting



NORMAALKRACHTEN
B.G:2 Variabele belasting

VERPLAATSINGEN [mm;rad]
B.G:2 Variabele belasting

Kn.	X-verpl.	Z-verpl.	Rotatie
1	0.00	0.00	0.00049
2	0.00	0.00	-0.00049

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

TEMPERATUUR BELASTINGEN
B.G:3 Constante temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	50.000	50.000	0.7	0.5	0.3

MOMENTEN
B.G:3 Constante temperatuurverdeling [30]

NORMAALKRACHTEN
B.G:3 Constante temperatuurverdeling [30]

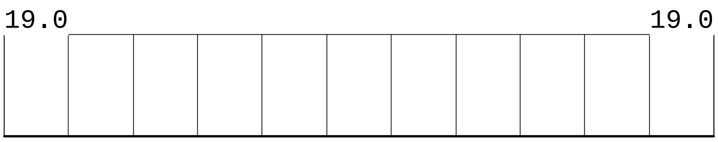
VERPLAATSINGEN [mm;rad]
B.G:3 Constante temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-1.75	0.00	0.00000
2	1.75	0.00	0.00000

TEMPERATUUR BELASTINGEN
B.G:4 Lineaire temperatuurverdeling [30]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-32.000	83.000	0.7	0.5	0.3

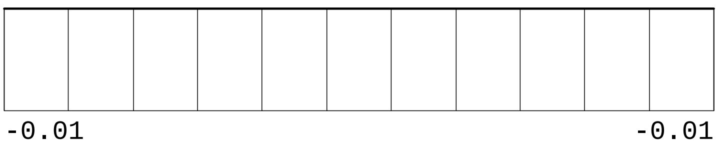
MOMENTEN
B.G:4 Lineaire temperatuurverdeling [30]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:4 Lineaire temperatuurverdeling [30]



VERPLAATSINGEN [mm;rad]

B.G:4 Lineaire temperatuurverdeling [30]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.00761
2	0.01	0.00	-0.00761

TEMPERATUUR BELASTINGEN

B.G:5 Constante temperatuurverdeling [60]

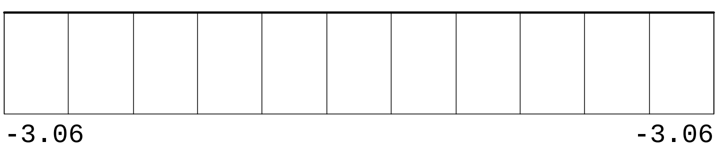
Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	81.000	81.000	0.7	0.5	0.3

MOMENTEN

B.G:5 Constante temperatuurverdeling [60]

NORMAALKRACHTEN

B.G:5 Constante temperatuurverdeling [60]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

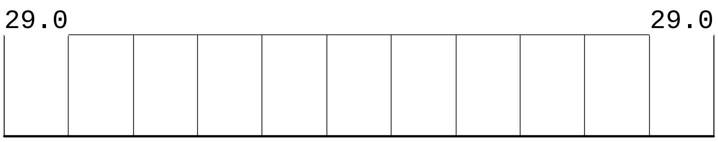
VERPLAATSINGEN [mm;rad] B.G:5 Constante temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-2.83	0.00	0.00000
2	2.83	0.00	0.00000

TEMPERATUUR BELASTINGEN B.G:6 Lineaire temperatuurverdeling [60]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-49.000	126.000	0.7	0.5	0.3

MOMENTEN B.G:6 Lineaire temperatuurverdeling [60]



NORMAALKRACHTEN B.G:6 Lineaire temperatuurverdeling [60]

VERPLAATSINGEN [mm;rad] B.G:6 Lineaire temperatuurverdeling [60]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.00	0.00	0.01158
2	0.00	0.00	-0.01158

TEMPERATUUR BELASTINGEN B.G:7 Constante temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	105.000	105.000	0.7	0.5	0.3

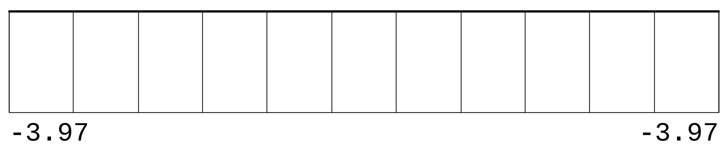
Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

MOMENTEN

B.G:7 Constante temperatuurverdeling [90]

NORMAALKRACHTEN

B.G:7 Constante temperatuurverdeling [90]



VERPLAATSINGEN [mm;rad]

B.G:7 Constante temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-3.67	0.00	0.00000
2	3.67	0.00	0.00000

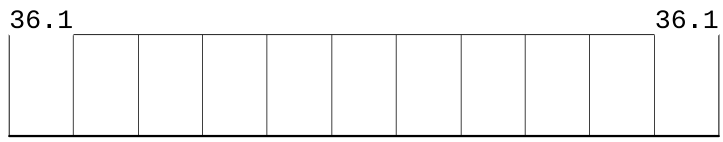
TEMPERATUUR BELASTINGEN

B.G:8 Lineaire temperatuurverdeling [90]

Last	Staaf	Temp.1	Temp.2	ψ_0	ψ_1	ψ_2
1	1	-61.000	157.000	0.7	0.5	0.3

MOMENTEN

B.G:8 Lineaire temperatuurverdeling [90]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN

B.G:8 Lineaire temperatuurverdeling [90]

VERPLAATSINGEN [mm;rad]

B.G:8 Lineaire temperatuurverdeling [90]

Kn.	X-verpl.	Z-verpl.	Rotatie
1	-0.01	0.00	0.01443
2	0.01	0.00	-0.01443

BELASTINGCOMBINATIES

BC	Type							
1	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$				
2	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,3}$	+	1.00 $Q_{k,4}$
3	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,5}$	+	1.00 $Q_{k,6}$
4	Fund.	1.00 $G_{k,1}$	+	0.30 $Q_{k,2}$	+	1.00 $Q_{k,7}$	+	1.00 $Q_{k,8}$

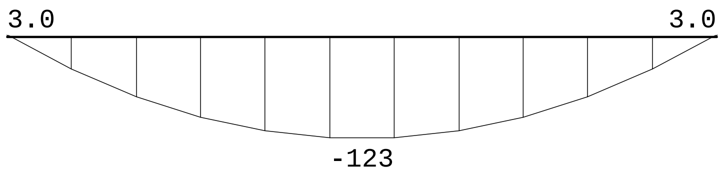
GUNSTIGE WERKING PERMANENTE BELASTINGEN

BC	Staven met gunstige werking
1	Alle staven de factor:1.00
2	Alle staven de factor:1.00
3	Alle staven de factor:1.00
4	Alle staven de factor:1.00

Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:1 Brand

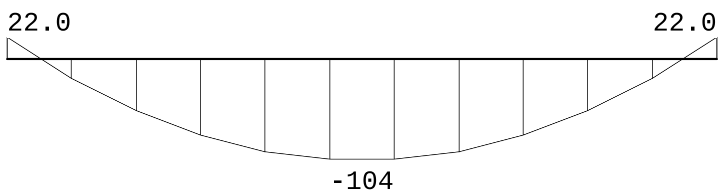
MOMENTEN B.C:1 Brand



NORMAALKRACHTEN B.C:1 Brand

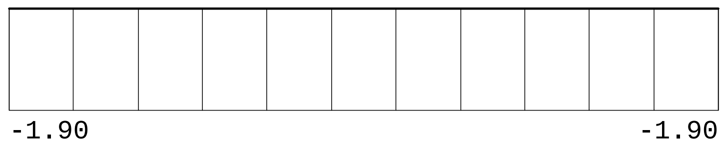
BELASTINGCOMBINATIE B.C:2 Brand incl. th. uitzettingen [30]

MOMENTEN B.C:2 Brand incl. th. uitzettingen [30]



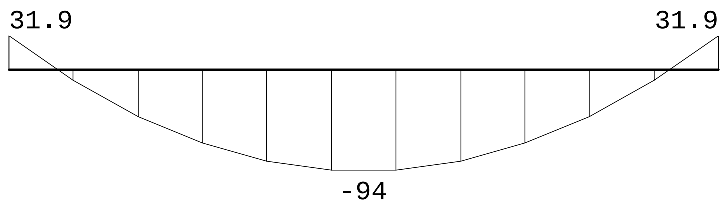
Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

NORMAALKRACHTEN
B.C:2 Brand incl. th. uitzettingen [30]

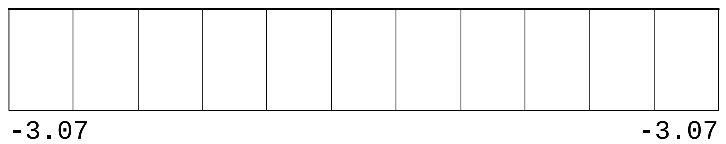


BELASTINGCOMBINATIE B.C:3 Brand incl. th. uitzettingen [60]

MOMENTEN
B.C:3 Brand incl. th. uitzettingen [60]



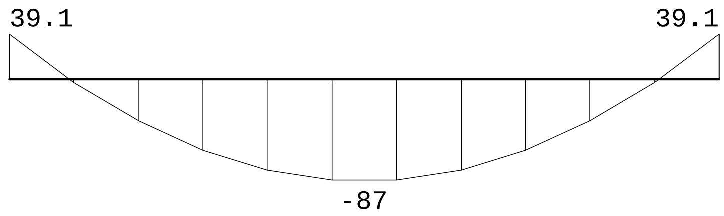
NORMAALKRACHTEN
B.C:3 Brand incl. th. uitzettingen [60]



Project..: Case study "Hof van Maerlant"
Onderdeel: Model T-ligger (verend)

BELASTINGCOMBINATIE B.C:4 Brand incl. th. uitzettingen [90]

MOMENTEN
B.C:4 Brand incl. th. uitzettingen [90]



NORMAALKRACHTEN
B.C:4 Brand incl. th. uitzettingen [90]