TU Delft Delft University of Technology

Fire safety design of a high-rise timber building

Fire safety design of a high-rise timber building

This thesis is written by Giulio Longhi[1] as partial fulfilment of his graduation project for the degree of Master of Sciences.

Graduation Committee:

Prof.Dr.Ir. J.W.G. van de Kuilen (Chairman)

Ir. A. Breunese

Drs. W.F. Gard

Ir. L.J.M. Houben

Ir. G.J.P. Ravenshorst

Ir. K.C. Terwel

Section of Steel and Timber Structures,
Structural Engineering,
Faculty of Civil Engineering and Geosciences,
Delft University of Technology,
April 2012.

[1] Email: giulio.longhi@gmail.com

Abstract

Timber structures have reached heights that were unthinkable decades ago. Good examples in Europe are the Limnologen project in Växjö (Sweden) and the Stadthaus apartment building in London (UK), which are 8- and 9-storeys high respectively. Thanks to their sustainability and their energy efficiency, timber buildings are getting more and more popular. The purpose of this study was to evaluate if timber buildings can be safely constructed even taller and reach heights of thirty storeys.

A detailed fire analysis has been carried out to determine if such a high building built with timber will meet the requirements of prescriptive building codes (Eurocode). Furthermore advanced calculations have been performed to evaluate the differences in the final charred depths with simplified models.

In order to address this answer, an ambient temperature design is performed to determine the thickness of the load bearing elements for the normal design, and the minimum required thickness required to withstand the load in case of fire. Then fire analyses were performed to determine the temperature fire curves in one room only. According to these fire curves different charred depths were determined.

The obtained results from thermal finite analysis show that charred depths proposed by advanced calculation methods are very different to the value given in the prescriptive codes. The charred depth according to natural fire curves was highly dependent on the initial fire load, on the relevant amount of charred structure accounted, and on the encapsulation method. For high fire loads the ISO (Standard temperature-time curve) value was underestimating the charring depth of about $100\ mm$ and the effect on the unaffected timber section was even greater.

The fire resistance of a timber element has to be evaluated according to the depth of the unaffected timber section (d_{100}). This depth should be determined by a thermal finite analysis based on the real fire curve, which, in turn, is obtained by fire analysis. It is believed that the use of a fire resistance concept based on an ISO fire exposure determined for an arbitrary time cannot be applied to buildings with combustible load bearing structure.

Finally, a possible solution, which has been already suggested by others sources in literature to overcome structural issues, is presented to overcome the major disadvantages related to high rise buildings. Those disadvantages are represented by a long egress time and by the inability to perform an external attack to the fire (at upper floors). It is believed that a different construction method, such as the three towers concept, could improve the overall fire safety of buildings allowing a faster evacuation and an external attack regardless the fire floor.

Acknowledgements

It would not have been possible for me to achieve the craved goal if I had not been surrounded by very good people who supported and encouraged me through the entire master program. Specially during the pasts months, when it has not been easy focussing on the thesis. First and foremost, I would deeply thank all my family who supported me for whichever decision I made, they made it possible for me studying abroad and giving me a huge opportunity to became finally a man, and an engineer.

Once, a close friend of mine told me a deeply true proverb. He said: "behind every great man, there is a great woman", I do not know if I am a great man, neither if I will ever be, but I am truly sure that my beloved girlfriend is a super great woman. She has been close to me in every moment with her love.

Of course, I have not been here alone all these years, I met some strangers, guys and girls who left their countries for a new adventure, as I did. In a while they become my best friends, some of them already left to going back their own country, some others are still here. They helped me a lot, they supported me, they made me laugh, but mostly, they were always there!

I would thank all the graduation committee members, who guided and helped me through the entire thesis with their patience and knowledge. In particular, I would like to thank the committee chairman, professor Van de Kuilen who, with his passion and enthusiasm, filled me with energy in every meeting. Moreover I really appreciated the availability and willingness to help me of all the committee members. Thousands of times I walked into their offices with unsolved problems and they always help me through, thanks!

Last but not least, I really have to thank all those people who, without even knowing who I was, decided to help me. Firstly, I am deeply grateful to Ms. Östman and Ms. Friquin from SP Trätek and from Norwegian University of Technology respectively. They provided me with an excellent material to get started with the thesis. Secondly, I would like professor Franssen and Mr. Gernay from the Université de Liège for their support regarding the finite element program SAFIR.

Giulio Longhi
Delft, 24th April, 2012

Index

1 Introduction

1.1 General introduction

Timber has been used as an excellent building material since the dawn of civilization. There are examples of old standing structures all around the World, from Norway to Japan. In Norway, there is the Urnes Stave Church which was built around the 12th century. It is one of the 29 churches which are still surviving. In Japan there is Hōryū-ji pagoda, where the wood used in the center pillar is estimated to have been felled in the 6th century, making it the oldest wooden buildings existing in the world.

Figure 1-1: Hōryū-Ji Pagoda

Figure 1-2: Urnes Stave Church, Norway

However to due to the poor knowledge and requirements about fire safety, during the past centuries entire cities have been damaged by fire, either by mistakes or by lighting. Examples are the great fire of Rome, 66 BC, the great fire of London, 17th century, and the great fire of San Francisco in the beginning of the 20th century. Fire has been always been a threat for poorly built timber structures. With the advent of the reinforced concrete, timber has been demoted for low rise timber frame houses.

Nowadays, with the growing interest in environmental issues, timber is having a renaissance. Timber is available in large areas, it is a renewable sources when it comes from sustainable forestry. Its production needs low emissions compared to other building materials and it acts as a carbon sink. Moreover timber building elements may be easily recycled. From an engineering point of view timber is a suitable building material for its properties as: strength to weight ratio, resistance to seismic load, thermal and acoustic insulation. However, the common misconception that a timber building, if designed correctly, will not be fire safe as a concrete building is hard to change.

The trend may change with the latest developments of timber products, represented by Cross Laminated Timber Panels. These panels show excellent mechanical properties and they show a very good fire behavior. It burs slowly and with a constant charring rate. Nevertheless due to the high prefabrication of elements the construction method is fast and efficient. Medium rise buildings have been already built with CLT panels. The next challenge is to go higher and determine if it is safe to build high-rise timber buildings.

1.2 Main objective and research questions

The main objective of this thesis is to evaluate the fire safety of a high rise timber building made of CLT panels and it can be summarized by the next question:

- **Is it possible to build a high rise building completely made of non-protected Cross Laminated Timber (CLT) panels?**

The main objective of this work is represented by a very wide field, and in order to answer the main question, the attention has been focussed on more detailed research questions which are related to building codes and to the material characteristics. The detailed research questions are:

- **Would it be possible to build a high rise timber building according to prescriptive fire regulations?**
- **Would it be possible to build a high rise timber building according to the material behaviour?**

To address the detailed research questions, an extensive literature review regarding building regulations, behaviour of timber in fires and fire safety engineering is carried out. Furthermore some interesting structures have been analysed. The literature study leads to more refined research questions:

- **Which are the most relevant regulations for timber high-rise buildings?**
- **Which advantages does a performance-based approach give instead of a prescriptive ones?**
- **Which are the protective measures which will ensure an adequate fire safety level in high-rise timber buildings?**
- **How should be taken into account the additional fire load due to the charring of structural combustible elements?**

1.3 Aspects not studied

It is believed that in unprotected timber compartment when a fire occurs the higher surfaces which are exposed first to the hot temperature will ignite before the bottom surfaces. This may lead to a faster growing stage of the fire due to the contribution of the charred depth to the fire load. However this effect has not been studied and a homogenous charring rate of the all surfaces has been assumed.

Another aspect which is recognized important but which is not studied here is represented by the effect of automatic suppression system activation on charring behaviour of timber elements. The reaction to fire for timber elements can be enhanced by wetting the surfaces using sprinkler systems. Spraying water on timber elements on fire is recognized effective because it has two effects, namely: it will reduce the combustion efficiency, slowing down the charring rate, and, the evaporated water will absorb energy from the fire, cooling down the temperatures. However it is believed that to only with real tests this question could have been answered.

1.4 Approach of the work

In order to answer the research questions, an extensive literature review is carried out to summarise the current knowledge about all the field of interest. Thus, the literature study has been subdivided in the next different subjects:

- Analysis of fire building codes. Determine if there is any obstacles in building codes which preclude the use of timber elements for high rise structures, and which solutions could solve it. Several building codes have been studied and compared, and the most important features have been used for the improving the fire safety of the fictional building.
- Behaviour of CLT panels on fire. First, the thermal behaviour of timber was studied, and a wide knowledge was obtained on the main parameters affecting the charring rate. Second, the CLT behaviour on fire was addressed.

- Fire safety engineering. This section provides a general knowledge on the basic measures to avoid human and material losses. Common fire safety measures were studied and their applicability to a fictional timber building was addressed. Then, the most effective ones have been implemented in the fictional building which will has to be designed.
- Case studies. Several buildings were studied in order to determine which protection measures were used to ensure an adequate fire safety level. These measures were then used in the fictional building.

Once the literature survey has been concluded, a fictional high-rise timber building has been designed according to a prescriptive approach. The approach is to perform two designs:

– A “cold design” has been carried out according to the fundamental design situation;
– A “hot design” or a fire design, has been carried out accordingly to **prescriptive fire methods**.

The cold design gives an estimation of the required thickness of the load bearing structure. This is not the main object of this work and therefore several assumptions were made in order to have more time to focus on the fire analysis. The fire design has been carried out in order to evaluate if the load bearing structure determined in the cold design will be able to withstand the action of a fire.

As it will be shown, it is believed that for timber structures a prescriptive design is not on the safe side. In order to compare the safety level between the prescriptive approach and the performance-based codes, advanced fire safety analyses are performed. Those analyses are focussed on evaluating the effect on the fire severity of the relevant part of the combustible structure. They are performed by the fire model OZone and by the software SAFIR developed at the University of Liege for the simulation of the behaviour of building structures subjected to fire (1).

The flow chart depicted below show the process followed in this work.

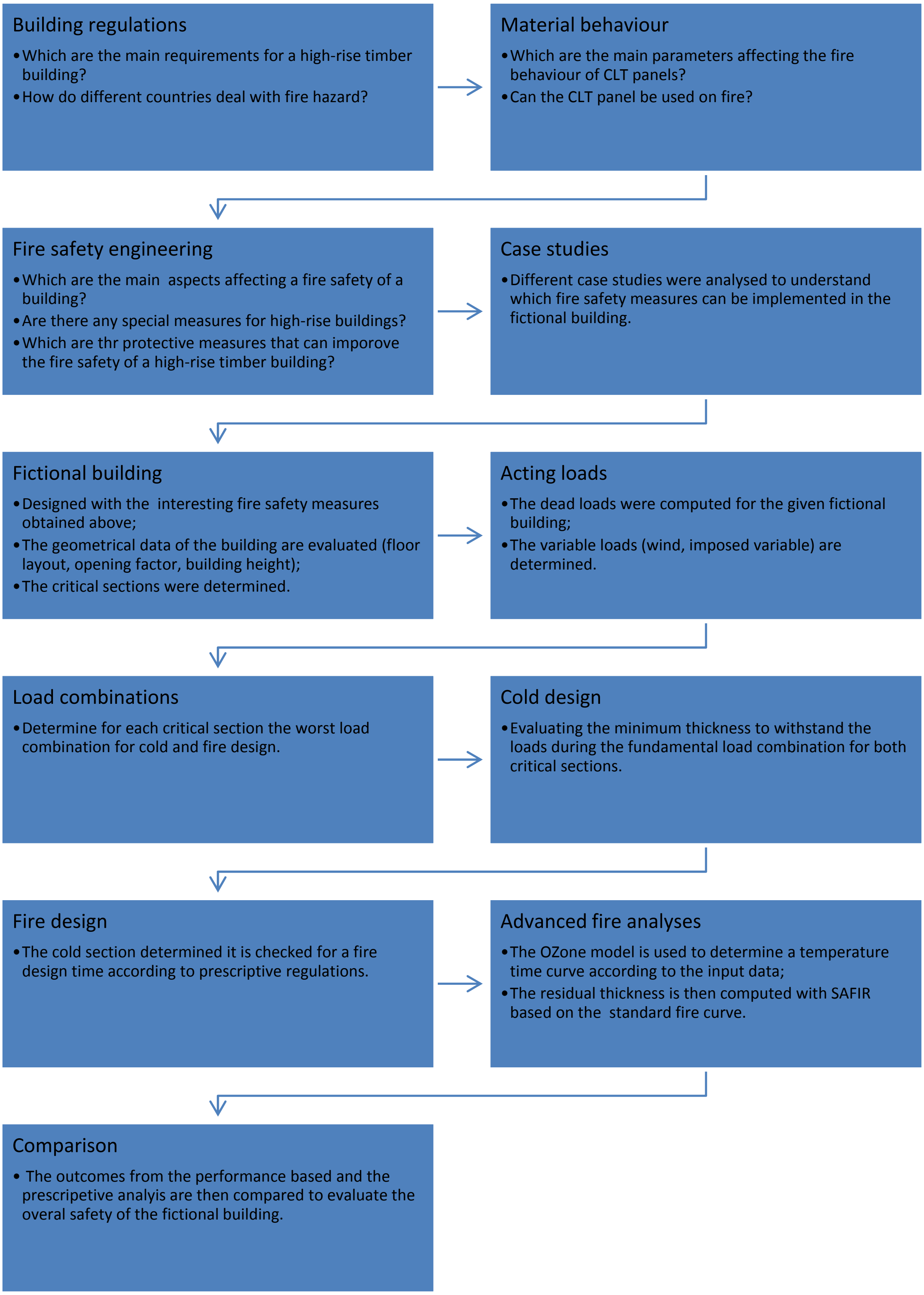


Figure 1-3: Approach of the work

1.5 Limitations

Performance based codes allow designers to use any fire safety strategy they wish, provided that adequate fire safety can be documented. Quantitative assessment should be used (either deterministic or probabilistic analysis) to evaluate the equivalent safety of the measures used. However, this is considered outside the scope of this thesis and therefore it is assumed that the combination of fire safety measures used in the fictitious building will provide an adequate safety.

The preliminary design for the fictional building is needed only to have an estimation of the dimensions needed to handle the basic load combination. Therefore only the effect of the vertical loads on the structure combined with the wind load has been studied. Second order effects, differential settlement of the building and accidental actions different from fire have been neglected. Also the connection design between the structural elements have been neglected. The preliminary designed building has not been checked for disproportional collapse neither for torsional effect.

Finally, the CLT panels used in the fictitious building have been exposed to one-side natural fire curve only, instead of two sides exposure. This is mainly due to the following reasons. First of all, it is assumed that the fire develops inside the room and then it may spread to the whole apartment. Second, only the effect of the additional fire load due to amount of structural timber which takes part in the fire is interesting for the purpose of this work. Last but not least, it was not possible to model more complex floor geometries, and having a simple room make it easier validate the outcomes with real scale tests.

2 Fire regulations

"If a builder build a house for someone,
and does not construct it properly,
and the house which he built fall in and kills its owner,
then that builder shall be put to death."
From: Code of Hammurabi, Article 229.

In this section different building regulations are studied to determine if there are any limitations in the use of timber as building material. If any obstacles are discovered, it will evaluated if it is possible to overcome these limitations, and how they can be solved. Being aware of the limitations present in building codes, a fictional building will be designed in the next sections fulfilling these requirements. Another important aspect studied here is represented by the different protection measures used in different countries. Indeed, not all the countries have the same prescriptions for a given issue, therefore different solutions are compared and the most effective will be adopted in the fictional building.

Fire safety regulations in different countries are based on the common strategies listed below (2):

- Stability in case of fire
- Limitation of spread of fire
- Escape routes
- Limitation of the development of fire

However there are substantial differences in the level of requirements. For example all the countries recognize that the risk of injury or death increases with the height of the building, which affects the ease of escape, the ease of fire-fighting, the rescue operations, and the consequences of any large scale collapse. However codes of different countries define different height range as show in Table 1.

Country	Height ranges (units are in meter)
Netherlands	< 7, < 13 or ≥ 13
England and Wales	Mostly ≤ 4.5 and > 4.5
Germany	≤ 7, ≤ 13, ≤ 22, separate section for taller building
Italy	From 12 to 24, from 25 to 32, from 33 to 54, from 54 to 80 and more than 80

Table 1: Different height ranges

A fire-fighting lift is required in UK for residential buildings with habitable areas higher than 18m a fire-fighting shaft with and in the recent amendments fire-fighting lifts are suggested for buildings over 11 meter in height (3). The UK is the only country which requires self-closing fire doors for all doors leading to the staircase of buildings higher than 3 storeys. The importance of self-closing doors is represented by an apartments building fire in Toronto, in 1992, where a 25-storey high building catch fire. The building was unsprinklered and it was made of reinforced concrete. The fire occurred in a flat at the 9th floor, the apartment occupants escaped leaving the entrance door open. In few minutes the smoke spread to all floors above the fire floor, but the fire was confined in the compartment of origin. The only one fatality occurred inside the elevator, when the elevator doors opened at the fire floor.

Only France, Belgium, England and Wales specified higher levels for fire safety of tall buildings. Only in the Netherlands further limitation in spread of smoke are present. Table 2 shows the required resistance times for different building heights.

[(1)]: for a fire load lower than 500Mj/m2, otherwise an increment of 30 minutes is applied.

	Single family house		Blocks of flats	
	2 storeys	3 storeys	8 storeys	15 storeys
Netherlands	30[(1)]	60[(1)]	90[(1)]	90[(1)]
England and Wales	30	60	90	120
France	15	30	60	90
Sweden	15	60	90	90

Table 2: Comparison of minimum fire resistance for structural elements (2)

Major differences have been found as the maximum number of allowed storeys and the amount of wood surfaces in facades element and in internal walls, especially in the escaping routes. For example prescriptive part of Finish regulations restricts use of wood in facades for two storeys buildings if sprinklers are not used (4).

In (5) national regulations for European and non-European countries (Japan, Canada, New Zealand, U.S.A.) are compared and the results are reported with maps. Several countries (Belgium, Greece, Netherlands, Norway, Spain, Sweden, New Zealand) do not have a limit for the number of storeys in timber buildings but a maximum of eight storeys is often used as a practical and economic solution when only CLT panels are used (6).

In Figure 2-1 the improvement in the maximum number of storeys for timber structures is shown.

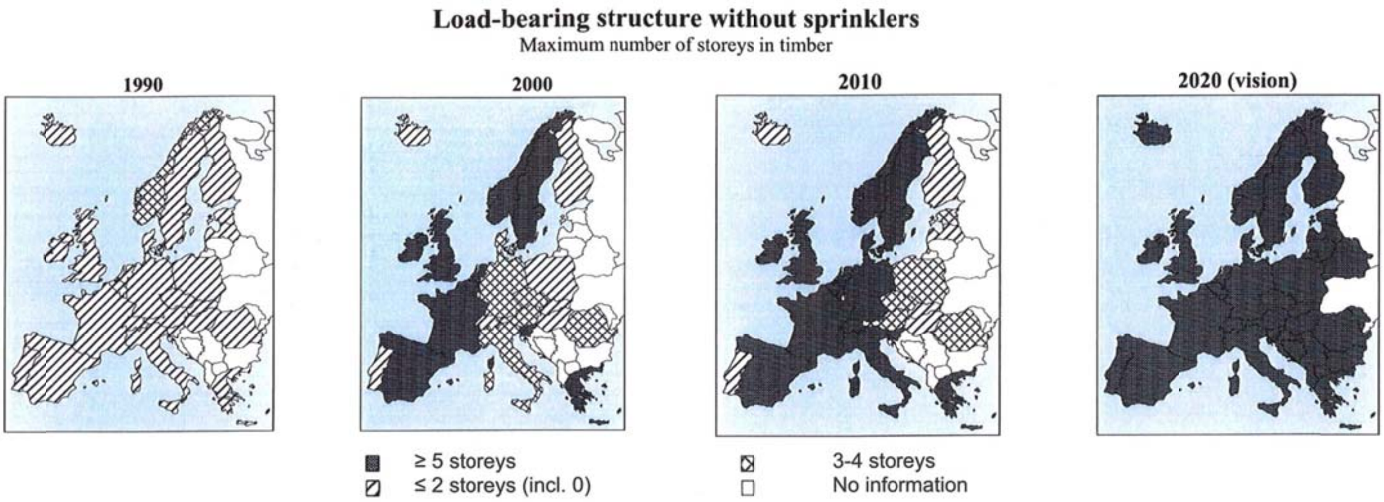


Figure 2-1: Changing in restrictions on Load-bearing structure without sprinklers (6)

In British Columbia, for instance, wood-frame residential buildings are allowed up to 6-storeys in height if the following restrictions are met (7):

- Maximum height of less than 18 m between the grade and the uppermost floor level of the top storey
- The building has to be sprinklered throughout
- Total building area is 1440 m² (5-storey) or 1200 m² (6-storey)

Data regarding cross laminated panels are not yet available in Canada, because it is a new European product which has drawn attention only in the past few years from the Canadian Wood Council, thanks to the Stadthaus building in London (8).

In New Zealand a new Building Code was introduced in 1992, it is a performance based building code which removed limitations in the maximum number of storeys for a timber building, provided it met certain performance criteria. The limit before 1992 was set as 3 storeys, but now allows construction of 5 storeys timber building (limitation due to the light frame construction method). Since then multi-storey timber buildings are performing well (9). For example, the code prescribes an increase in the escape open path length by 50%, 10% or 50%, when sprinkler devices, heat and smoke detectors are used respectively (from table 3.3 of Department of Building and Housing, 2010).

2.1 Fundamental requirements

Fire Safety Engineering (FSE) can be described as: "the application of scientific and engineering principles to the effects of fire in order to reduce the loss of life and damage to property by quantifying the risk and hazards involved and provide an optimal solution to the application of preventive or protective measures" (10). In buildings, fire safety engineering is based on the fundamental objects stated below:

- Safety of people (occupants, neighbours and rescue team);
- Reduce the property and financial losses;
- Environmental protection.

These three fundamental objects are fulfilled when the following functional requirements are met[2]:

- Load bearing capacity can be ensured for a specific period of time (fire design time < resistance time);
- The generation and spread of smoke within the construction is limited;
- The spread of fire to neighbouring buildings is limited;
- People in the building on fire can escape or being rescued safely;
- The safety of occupants and rescue service personnel is taken into account.

The functional requirements are provided by the following protection strategies:

- Detection;
- Suppression;
- Egress;
- Fire endurance.

It is impossible to achieve absolute safety, independently of the construction materials and systems involved (11). It is worth noting that the risk is proportional to the height of the building, in fact, it is reasonable to think that a fire in a single family house made of two storeys, has less negative results than a fire in a high-rise building. In low-rise structures, there are few people and they will need less time to evacuate the building. An eventual collapse of a low-rise building will not involve other buildings, while a collapse of a high-rise building will be not acceptable for the extreme expected losses which the accident will cause. High-rise buildings are characterised by higher number of occupants, long escaping routes and vertical movement of smoke inside the building, all those aspects will adversely affect the safety of people in case of fire.

2.2 Building regulations

Building regulations are aimed to bring the risk of a fire under an accepted and tolerated value from the society. There are two different ways to achieve the satisfactory level of safety which will in turn ensure that the fundamental requirements can be achieved. They are namely: prescriptive design and performance based design.

2.2.1 Prescriptive rules

The first design procedure is derived from our experience and it is based on prescriptive codes. The designer must follow well defined and stringent rules (or pre-accepted solutions), which will be certainly safe. Prescriptive code requirements are related to a certain fire design time, usually multiples of $30\ min$, depending on the fire risk of the building. Prescriptive rules have the following advantages: simple calculations are requested and fire safety experts are not needed. However, those features will be at the expense of an efficient use of materials and therefore on the overall economy of the project. Therefore prescriptive approach

[2] These fundamental objects and functional requirements are the bases for fire safety regulation all around Europe (1) (8).

works well for low-rise or traditional buildings, where the cost for fire protection engineers is higher than the cost for prescriptive solution.

Prescriptive codes, like the Italian building decree and the Bouwbesluit (Dutch building decree), give requirements that must be followed in order to ensure the occupants safety and avoiding collapse of buildings. Some of those requirements are material dependent and are limiting the use of comubustible material, regardless fire fighting measeures used. For example in the Dutch building decree it is stated (Article 2.93) that for buildings higer than 13 m only material class 2 are allowed on the façade and in escaping routes (which are defined as fire and smoke free). Dutch material class 2 is the equivalent of euroclass B or C[3]. The use of timber, as structural elements, is also affected by the amount of fire load in the compartment. In fact, there are articles which prescribes an increase in the fire resistance of 30 min when the fire load in a compartment is higher than 500 MJ/m^2.

2.2.1.1 Fire design time

Prescriptive codes impose a fire design time which have to be respected in order to ensure adeguate safety. If this resquirement will be fullfil no further analysis will be needed and the building will be considered safe. Fire design time of building components is usually expressed in multiples of 30 min, depending on the importance of the structure and the related risk. The time criterion should not be interpreted as the maximum fire endurance which can be handled by the building, but as a safety chain. Indeed, the fire design time is related to a standard curve, not to a real fire which can occur in the building. All the protective measures applied to the building must have the same resistance level, otherwise the entire safety of the building will be compromised. The fire design time is related to a temperature exposure according to the ISO fire curve which is different from a natural fire exposure. The standard temperature-time curve may be significantly different from a real fire. The time criterion should not be interpreted as an occupants escaping time or as a maximum time for a fire brigade intervention (12). However the Dutch building decree (Section 2.2, Article 2.9), prescribe a first fire design time which is independent of the building height, of the functional use of the building and of the amount of people that are in the building. During this time the ultimate limit state of the load-bearing structure of which collapse will render a smoke-free escape route unusable, is not exceeded during 30 minutes at the special combinations of actions that can occur in a fire. This requirement allows that in the event of a fire the structure can be left and searched during a reasonable period of time without the risk of collapse. Furthermore, a second fire design time is given and it is dependent of the building height and of the functional use of the building. For a residential structure with a habitable floor higher than 13 m above ground level, the fire resistance period with regard to collapse in minutes must be at least 120 min. During this time an ultimate limit state of a main load-bearing structure must not been exceeded. In the code it is not explain where the value of 120 min comes from, the main thinking behind this value has been founded in different literature sources ((2) (11) (13) (14) (15)).

One of the main objectives for fire safety is represented by preventing the collapse of the structure, avoiding property losses (13). This is achieved by active and passive control. Passive control refers to fire control by systems that are built into the structure, active systems are those systems which need to be activated in an event of fire, and both of them could fail. In order to achieve an adequate passive protection the structure must have sufficient fire resistance to prevent spread of fire and structural collapse. Adequate protection can be ensured when the fire resistance (property of the structural system) will be at least equal to the fire design time, which is explained below.

The fire design time is determined by: the importance of the building; the requirements of the owners or the insurance company; and the consequences of a structural collapse or spread of fire. The main factors influencing the fire design time are (15):

[3] Derived from fireretard.com (http://www.fireretard.com/main.php?pdf=5-3%20Euroclasses.pdf&langid=1&itemid=4&subitem=19&subsubitem=)

- Building geometry (height) and intended use (residential or office building[4]);
- Location of adjacent properties;
- Probability of a fire occurring;
- Fuel load distribution;
- Number, location and abilities of occupants;
- Proximity and likely response of the fire service;
- Available water supply;
- Building management practices that affect fire safety.

This explains why there are different fire design times for a two-storey detached house in the middle of nowhere and for a residential skyscraper in the city centre. In the detached house will be present only the family owner of the building which has to deal with a fire developed due to their actions, the escaping time is short and neighbouring building will not be affected by the fire.

Many small single storey buildings may be designed to protect the escape routes and to remain standing only long enough for the occupants to escape after that it is accepted that the fire will destroy the building. Alternatively, tall buildings, or buildings where people cannot escape easily, should be designed to prevent major spread of fire and structural collapse for a complete burnout of one or more fire compartments eventually without any intervention (One Meridian Plaza Fire). Probably if a timber structure is left on fire without any intervention it will not survive.

2.2.2 Performance based codes

Performance based codes are now being implemented in building codes, indeed they are more flexible and allow the designer to have more freedom to achieve a satisfactory fire safety. They do not prescribe any measures to reach the stipulated safety level and they are not focussed on single elements but they take into account the complete structure with regards to all the aspects involved (fire, compartments, fire safety measures, occupants) and their interactions.

When using performance based codes, fire safety goals are identified, which are related to the fire safety functional requirements. The functional requirements are the same as prescriptive design goals. These goals have to be accepted by the building owner and by the authorities for approval. Once those goals are approved, fire scenarios are established and probability of failure of each scenario is determined according to quantitative assessment. An example of quantitative assessment of with probabilistic analysis is represented by an event tree analysis. Event tree analysis can take into account both human behaviour and the reliability of fire protection system. An example of event tree analysis is given in the next figure.

[4] The intended use is important to establish the probability of awareness of occupants. Sleeping functions have been recognized as one of the most dangerous.

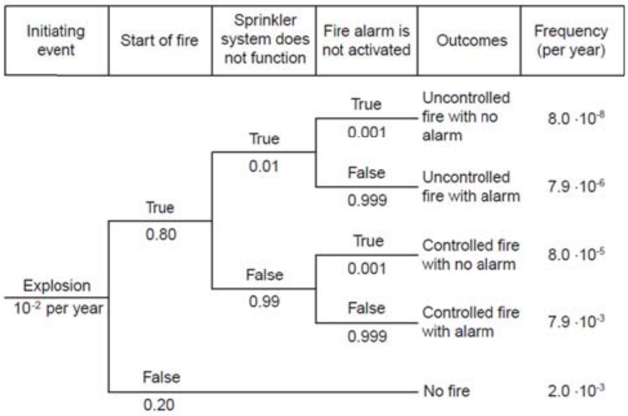


Figure 2-2: Event tree analysis[5]

However, evaluating the total fire risk according to the fire-fighting measures used with probabilistic analysis is utterly complicated (16) and it is believed being outside the scope of this work. Usually, performance based solutions meet or exceed the intent of prescriptive codes. However, it is believed that probabilistic analysis aimed to determine the probability of the outbreak of a severe fire in timber buildings, is outside the scope of this research. Therefore from now on the attention will be focussed only on fire measures, both, active and passive, neglecting their effectiveness and reliability.

2.3 National Building decree

2.3.1 Italian code

In the Italian Building Decree there are no direct limitations in use of wood (17). However, regarding the reaction to fire, the code state that in the escape routes and staircase no material with a reaction to fire class higher than zero (in Italy the reaction classes range from 0 to 5, the lower the non-combustible material) are allowed. Only for buildings class “a” or “b” (which means lower than 32 meter), material class 1 can be used. The Decree provides prescriptions for tall building (treated as higher than 80 meter) regardless the construction materials:

- Maximum compartment area of $2000\ m^2$;
- Maximum floor area for each staircase: $350\ m^2$, with a minimum of two;
- Helicopter deck to allow rescue must be present on top of the building and must be accessible by all the staircases;
- Each staircase and at least one elevator shaft must be smoke-proof;
- Staircase, elevator shaft, doors, structural members and compartment wall must have fire resistance (R,E,I) of 120 minutes.

In (5) is said that the maximum limit is 4 storeys, however with the last amendments of the Italian Building Decree, this limit was removed. This restriction in the use of wood was based on seismic considerations (18) (19). Currently, there are three major on-going projects in Italy regarding wooden structures, they regard social housing timber buildings, one high-rise and two low-rise, respectively one apartment building of 15 storeys in Milan[6] and a 6 and 4 storeys building in Florence[7].

[5] From http://www.ntnu.no/ross/slides/eta.pdf
[6] http://www.archisquare.it/primo-grattacielo-in-legno-milano-urbam-dante-o-benini-partners/
[7] Progetto Casa SPA, the first building, the smaller, will be ready on July 2011 (http://www.casaspa.it/stampa/Anno%202011/7%20maggio%202011/ludoteca.pdf), then the construction for the main

2.3.2 Dutch code

Analysing the standard, according with (20) and (5), no specific restrictions are present on the maximum number of storeys which can be realized with wood. A 6-storey wooden building was built in the city of Delft in 1994 (20).

The required minimum resistance times are given in Table 2 above. In the Dutch building decree limitations are present regarding the contribution to the spread of fire. Those requirements refer to the NEN 6065 and they are valid for all internal surfaces. Walls, ceilings, and floors in escapes routes must be class two (Euroclass B), while in England and Wales the upper surfaces of floors and stairs are not treated because they are considered insignificant to occupant safety.

The Dutch Building Decree does not have more stringent requirements for high rise buildings (higher than 70 meter) apart from smoke compartimentation of stairways. In Division 2.23 Article 2.208 of (21) it is stated that for a structure with a floor of habitable space higher than 70 meter, the articles for a low rise building apply with the only extra provision of smoke protection of stair case (22).

It is worth noting that only the Netherlands prescribe sub-fire compartments and smoke compartments. Another peculiarity of the Dutch building decree is the requirement regarding the smoke production of internal surfaces, especially walls and ceilings in the escaping routes. In other countries the smoke production is not specifically addressed because is commonly assumed that it will be affected by the limitation in spread of fire (see §4.1.1).

Last but not least, no provision is given for self-closing entrance doors for apartments which are mandatory in different European counties. Self-closing entrance doors enhance the protection of the corridor and then of the protected lobby, often occupants escaping from a fire leave doors open allowing a fast spread of fire inside the building.

Some other limitations present in the Bouwbesluit are represented by:

- Fire resistance period of $120\ min$ for buildings higher than 13 m with a permanent fire load higher than $500\ MJ/m^2$;
- Max compartment area of $1000\ m^2$;
- One protected staircase every $3500\ MJ$ of permanent fire load;
- Limitation regarding smoke density;
- Maximum escape route length.

2.3.2.1 Extensive analysis of Bouwbesluit

Fire safety aspects are treated in Chapter 2 of the Bouwbesluit (Dutch Building Decree). Chapter 2 deals with regulations concerning safety, in particular, fire regulations, are treated in the several divisions, each division is sub-divided in two group, namely: new and existing structures. In this work, only the relevant articles for a new residential building were studied. The divisions related to fire safety are listed below:

- Division 2.2 Strength in fire
- Division 2.11 Limiting the development of a fire hazard
- Division 2.12 Limiting the development of a fire
- Division 2.13 Limiting the propagation of a fire
- Division 2.14 Further limitation of the propagation of a fire
- Division 2.15 Limiting the development of smoke
- Division 2.16 Limiting the propagation of smoke
- Division 2.17 Escape from within a smoke compartment and a fire sub-compartment
- Division 2.18 Escape routes

one will start (http://www.casaspa.it/informazioni/stendardi_ExLonginotti.pdf). In English: http://www.casaspa.it/informazioni/posterCASASPA_ok.pdf

- Division 2.19 Layout of smoke-free escape routes
- Division 2.20 Preventing and limiting accidents in case of fire
- Division 2.21 Fire fighting
- Division 2.22 Large fire compartments
- Division 2.23 High and underground structures

The relevant articles which may affect the use of timber as construction material are given below, on the side a possible solution to overcome the restriction is presented.

Article 2.83:

Material used on the inside of a shaft, a duct or a channel with an internal diameter greater than 0.015 m2 and adjacent to more than one fire compartment is incombustible over a thickness of at least 0.01 m, measured perpendicularly to the inside. This does not apply if the shaft, the duct or the channel is situated inside and is exclusively intended for one or more toilet compartments or bathrooms situated above each other.

It can be solved encapsulating the shaft with non-combustible material such as gypsum panel.

Article 2.85:

A roof of a structure in which the functional unit is situated, is not flammable.

The roof structure can be built or can be protected by a non-combustible material.

Article 2.92

On a side not adjacent to the outside air, a structure element has a contribution to fire propagation that complies with the following Euroclasses:

- Fire and smoke free escape route, class C;
- Smoke free escape route, class C;
- Others, class D;

Using gypsum panels as internal finishing material in the escaping routes solves this problem. In other zones of the structure timber in sight can be used.

Article 2.93

1. On a side adjacent to the outside air, a structure element not being a door, a window, a frame or an equivalent structure element has a contribution to fire propagation that complies with the following Euroclasses:
    - Fire and smoke free escape route, class C;
    - Smoke free escape route, class C;
    - Others, class D;

As Article 2.92

2. On a side adjacent to the outside air, a part of a structure element that lies higher than 13 m above ground level, has a contribution to fire propagation that complies with Euroclass C.

No requirements concerning the distance from a neighbouring building are given here, (they are specified in Building Decree's WBDBO, according to ref. (14)*)*

3. On a side adjacent to the outside air, from the adjacent site up to a height of at least 2.5 m above it, a structure element of a structure of which a habitable space floor lies higher than 5 m above ground level, has a contribution to fire propagation that complies with Euroclass B.	*Solved using appropriated cladding.*
Article 2.94	
2. The upper side of a floor, a ramp or a staircase has a contribution to fire propagation that complies with the following Euroclass: • Fire and smoke free escape route, class C_{fl}; • Smoke free escape route, class C_{fl}; • Others, class D_{fl};	*Solved encapsulating the element.*
Article 2.126	
1. On a side adjacent to the interior air, a structure element has a smoke production with a smoke density not exceeding $10\ m^{-1}$ which is Euroclass s2.	*CLT panels meet this requirement.*
2. If a structure element, on a side adjacent to the interior air in an enclosed room through which a smoke-free escape route leads, has a contribution to fire propagation that complies with class C, but not with Euroclass B, on that side the structure element, in deviation of the first paragraph, has a smoke production with a smoke density not exceeding $2.2\ m^{-1}$.	*No conversion table is given to evaluate the Euroclass for a smoke density other than $10\ m^{-1}$.*
3. If a structure element, on a side adjacent to the interior air in an enclosed room through which a smoke-free escape route leads, has a contribution to fire propagation that complies with class B, on that side the structure element, in deviation of the first paragraph, has a smoke production with a smoke density determined according to NEN 6066 not exceeding $5.4\ m^{-1}$.	*No conversion table is given to evaluate the Euroclass for a smoke density other than $10\ m^{-1}$.*
Article 2.170	
1. The product of the permanent fire-load density and the net floor area of a protected staircase does not exceed $3500\ MJ$ per storey.	*Providing enough fire protective lining materials such as the protected timber will not be able to undergo to charring.*
2. An escape stair enclosure, other than a protected staircase, cannot directly be accessed from an enclosed room through which a smoke-free escape route leads, a toilet compartment, a lift shaft or a building services room, unless the product of the permanent fire-load density determined according to NEN 6090 and the total of the net floor areas of that escape stair enclosure, the enclosed room, the toilet compartment, the lift shaft and the building services room per storey, does not exceed $3500\ MJ$. This does not apply for a stair enclosure that complies with article 2.157, fifth paragraph.	*Same as above.*

2.3.3 Swiss code

In Switzerland, until 2005, new timber structures were allowed only with not more than two storeys, this requirement was based on the combustibility of wood. Now, Swiss fire regulations allow the use of timber in medium-rise structure (up to 6 storeys) (23).

	Storeys						
	One	Two	Three	Four	Five - Six	Seven - Eight	Tall Building[(1)]
Load-bearing structure	-	Design for normal temperature	R30	R60	R60/ EI30(nbb)[(2)(3)]	R60(nbb)[(2)]	R90(nbb)[(2)]
Separating elements	EI30	EI30	EI30	EI60	EI60/ EI30(nbb)[(2) (3)]	EI60(nbb)[(2)]	EI90(nbb)[(2)]

[(1)]: tall buildings are higher than 25 meter or with the top floor located higher than 22 meter from the level where fire brigade truck can stand.
[(2)]:nbb, non-combustible material
[(3)]:partial building encapsulation

Table 3: Swiss fire requirements

2.4 Standard for tall buildings

Standards for tall buildings are not developed yet, however they rely on the requirements for high structures with and increased resistance time to allow a complete evacuation of the occupants. A step forward is done in English regulations which request a fire fighting lift for structures over 30 meter since 1980s. This measure counteracts the effect of an impossible external attack to the fire. In fact, fire department ladders can only reach the 8th floor at most[8]. Other advantages in using fire fighter lifts are the quickness of response (preventing exhaust fire-fighters to become exhausted), and the avoidance of interferences between two counter flows in staircases due to the egress of occupants and the contemporary access of fire brigade (24).

The common requirements all around the World for tall buildings are:

- Refuge floors;
- Refuge areas along the staircases;
- Elevators used for evacuation;
- Fire fighting elevators;
- Emergency power;
- Protected lobbies;
- Vestibules;

It is important to realize that when elevators are used for evacuation issues, no reduction of the stair width is accounted.

[8] Fire ladder trucks can reach a height of 30 meter, http://www.iveco-magirus.de/?id=141&prod_id=110

2.5 Building regulations summary

There are no direct regulatory barriers to the use of wood in residential and non-residential construction throughout Europe (25). However, according to (26) there are many limitations in the use of wood as structural material, as follow:

- The main regulatory limitations are perceived to be fire and acoustic performance, particularly in multi-storey dwellings;
- In some countries there are regional differences in building regulations;
- There is a lack of codes and standards for many wood products;
- The use of Eurocodes is still limited, although it is increasing;
- There is uncertainty and lack of in-depth knowledge of building regulations relevant to the use of wood in construction;
- External use of wood and wood-based products is mainly limited by the height of the building and the distance between adjacent buildings;
- The maximum number of storeys permitted varies between countries;

A detail review on restrictions on different use of wood in residential buildings for some countries is reported in Table 4.

Building properties	Germany	Netherland	Italy	New Zealand	UK	Sweden
Load-bearing structure **with** sprinkler	3-5	(1)	-	-	6	(2)
Load-bearing structure **without** sprinkler	3-5	(1)	-	-	6	(2)
Facades **with** sprinkler (Wooded façade cladding)	3	5(3)	4	3	6	-
Facades **without** sprinkler (Wooded façade cladding)	3	5(3)	4	3	6	2
Wall and ceiling linings in **flats** without sprinkler (Surface linings of ordinary wood	-	-	4	-	2	2
Wall and ceiling linings in **escape routes** without sprinkler (Surface linings of ordinary wood)	Not allowed	Not allowed	Not allowed	Not allowed	Not allowed	Not allowed
Wall and ceiling linings in **flats** with surface linings of fire retardant treated wood	-	-	4	-	-	-
Wall and ceiling linings in **escape routes** with surface linings of fire retardant treated wood	Not allowed	-	4	-	-	-

(1): in practice 5 storeys (13 meter)
(2): 8 storeys are founded to be economically feasible
(3): no reference has been founded in Bouwbesluit

Table 4: Review on restrictions on using wood as building material in different countries, adjusted from (20)

In (5) major differences have been identified, both in terms of the number of storeys permitted, and in the type and amount of visible wood surface in interior and exterior applications. Generally, the maximum allowed number of storeys for load-bearing wooden structure is varying greatly, and it is not related to the use of automatic extinguishing devices (as sprinkler system). Only Iceland takes into account the presence of a sprinkler system improving the number of storeys from one to infinite.

In apartments the use of wood in walls and ceiling linings is widely allowed without any restriction, however the use of untreated timber in escape routes is not allowed in most of the countries studied (prohibited in 23 out of 29 countries). For example, Austria and Switzerland allow the wood finishing in escape routes for a maximum of two and one storeys respectively, but Romania and Greece have no limitations. The fire retardant treated wood is widely accepted in escaping routes. Softer limitations are present for the use of wood in floorings, permitting generally more storeys for both flats and escaping routes.

2.6 Summary

From the analysis of different building regulations it can be concluded that there are no stringent requirements on the use of timber as building material. Indeed, depending in which way the fundamental requirements have to be fulfilled, either prescriptive or performance-based, the design solution may be different. According to prescriptive codes, a fire design time of $120\ min$ has to be used, this alone will avoid a failure of the timber building. This value will be used for answering the main research question, therefore the hot design will be based on a fire resistance time of $120\ min$. Some other minor limitations are present with regards to generation and propagation of fire and smoke, however they can be met encapsulating the timber elements. The fire design according to prescriptive codes is carried out in §6.4.

Beside the building code requirements, this section shows that the height is the main factor which effects the safety of high rise building. Indeed the height of the building will affect the egress time, the vertical movement of smoke inside the building (stack effect), and the fire-fighters accessibility. In fact, the height affects both, the time needed to reach the fire floor and the possibility of an external attack. Indeed, fire fighters are able to climb one floor each minute for the first 10 floors, then due to physical fatigue they spend two minutes each floor, this is why the first seven storeys are called in jargon "the lucky 7"[9]. Fire truck ladders, instead, can reach at most the eighth floor (or $30\ m$), this will make impossible to attack the fire from the outside, nor rescuing entrapped people. Therefore, the height is a main concern for building regardless construction materials, examples are the TU Delft faculty of architecture, the One Meridian Plaza in Philadelphia, and the Windsor Tower in Madrid, where fires run out of control due to the inability to reach the fire floor from the outside. All those structures have been demolished because of doubts on their residual load bearing function.

Because the height of the fictional building is approximately $100\ m$, a failure of the main load bearing structure has to be avoided. A collapse of such high structure will cause immeasurable human and financial losses which cannot be accepted by the society. Therefore the main aim of this work is to guarantee that the fictional building will withstand a fire exposure without collapsing.

Contrary to prescriptive codes, a performance based design considers the effect of any fire safety measures used, but the overall fire safety of the measures used has to be determined with quantitative verification methods which are utterly complicated, and therefore are neglected in this study. Moreover the obtained solution has to be accepted by the authority. The main protection measures discovered in this section are listed below, and they will be part of the protective measures used in the fictional building.

- Self-closing entrance doors;
- Fire fighters elevators with protected lobbies;
- Rooftop helipads accessible from all the vertical escaping routes.

[9] From: http://highered.mcgraw-hill.com/sites/dl/free/0073382841/474847/Sample_Chapter14.pdf

3 Behaviour of timber and cross laminated timber panels on fire

As it has been shown in the previous chapter, there are no direct barriers to the use of timber as a building material. However some limitations are present due to its combustibility and its smoke production. In order to mitigate these aspects; a literature review is carried out on how timber behaves on fire. Some real scale tests on CLT panels have been founded in literature and they are treated below. The attention is then drawn to the charring rate and to the relevant factors that may affect it.

3.1 Thermal degradation and ignition

It's well known that wood is a combustible material, its behaviour on fire is characterized by the reaction, which is poor, and by the resistance to fire, which instead is good. The reaction to fire is represented by the initial response of a material to fire exposure (how much take part to the fire load), fire resistance indicates how the material will stand to the load regarding three main functions (R, E, I).

Thermal degradation of wood starts with the increase of temperature, this phenomenon is called pyrolysis. Pyrolysis happen when timber is exposed to the effect of fire, due to high temperatures, the wood undergoes to decomposition generating char and gases and reducing the density correspondingly.

Temp. range	Pyrolysis and combustion processes
100°C to 200°C	Water vapour is given off and eventually become charred. Gases are generated which consist 70% of incombustible carbon dioxide and 30% of combustible carbon monoxide.
200°C to 300°C	Water vapour, formic acid, acetic acids and glyoxal are given off, ignition is possible but difficult. Hemicellulose is pyrolyzed here. Carbon monoxide proportion increases. Lignin is pyrolyzed from 225 to 450°C, it is important to stress that the higher the content in lignin, the higher the residual char content.
300°C to 450°C	Vigorous production of combustion gases due to depolymerization of cellulose (carbon monoxide, methane…) diluted with carbon dioxide and water vapour. Residue is black fibrous char. Normally vigorous flaming occurs. If however the temperature is held below 500°C a thick layer of char builds up.
Above 450°C	The formed char layer undergoes to further oxidation until only ashes remain.

From (27) (28) (29)

Table 5: Effect of Temperature on wood

The necessary condition for ignition of timber is a flow of energy or a heat flux from a fire or other heated objects to the wood material to induce pyrolysis (28). Depending on the energy of the ignition objects or fire, two ignitions may occur:

- Flaming ignition of wood can occur without a pilot flame when timber is exposed to a temperature range of 400 to 500°C with a right combination of oxygen and combustible volatiles (according to (30) a temperature of 600°Cis required).
- Timber can be ignited also at lower temperatures (in the range of 300 to 400°C) if an ignition source is available.
- A third way to ignite wood is by means of a heat flux. It has been calculated by (28) that for wood products critical radiant flux is in the range of 10 and $13\ kW/m^2$ with a source of ignition, and $25\ kW/m^2$ for spontaneous ignition.
- Smouldering ignitions required longer time and a material which is unable to dissipate heat. Evidence of smouldering ignition is continuous smoking without presence of flames. It will produce dense and black smoke and it cannot be detected by heat sensors.

Factors affecting wood ignition are well known to everybody who has done a barbeque, wet wood is difficult to ignite, it needs more time and energy to start flaming, thin sections are easier to ignite than thick sections and light wood ignites faster than heavy wood (31). The ignitability of wood plays a major role for the fire reaction. Fire reaction of timber is poor, but it can be improved by retardant methods (§3.6) or by wetting the wood section.

3.2 Factors affecting the burning behaviour of timber

Charring rate is affected by density, moisture content and the external heat flux, but is not affected by the use of fire retardants (30). Dimensions of the specimen are also important; fire tests with timber beams exposed to fire on three sides clearly showed that the charring rate increases as soon as the residual cross-section is smaller than 40 – 60 mm (32).

The wide range of variability of charring rate is addressed in (28) (33) (27). They all identify moisture content as one of the major factors affecting charring rate together with density, as can be seen in Figure 3-1. According to (33) during laboratory experiments it was impossible to establish which of the following properties was more important, however for the purpose of this work it is believed that moisture content is the most interesting because it is the easiest wood properties which can be changed in the event of fire by fire-fighting. Panel properties are design dependent, and fire characteristics depend on the fire scenario. However according to (33) the charring behaviour is affected by the following factors:

- Wood properties
  - Density
  - Moisture content
  - Lignin content
  - Permeability
  - Grain direction
  - Char contraction factor
  - Char oxidation
- Panel properties
  - Thickness
  - Surface area
  - Orientation of the sample
- Fire characteristics
  - Thermal exposure
  - Oxygen concentration
  - Ventilation or opening factor

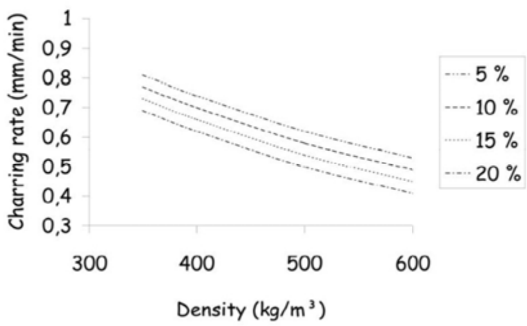


Figure 3-1: Charring rates as affected by density and moisture ratio (33)

3.2.1 Sample thickness

The size effect is important because it will affect the temperature distribution inside the sample. From a thermal point of view, a generic section, can be regarded as think or thick. Thermally thin sections ignite more quickly than a thermally thick material, and they have a linear temperature distribution, which means that when a thermally thin product is exposed to heat on one side, its opposite side heats up very close to the temperature of the exposed side by the time to ignition. Thermally thick specimens, on the other hand, have a non-linear thermal distribution over the cross section, which means that its core or the unexposed side, are still at ambient temperature (34) (33) (30).

In (32) a critical minimum dimension of the samples is set as 40-60mm. When a smaller section is used the core of the specimen will increase in temperature and heat will be stored, therefore a linear charring rate cannot be used anymore.

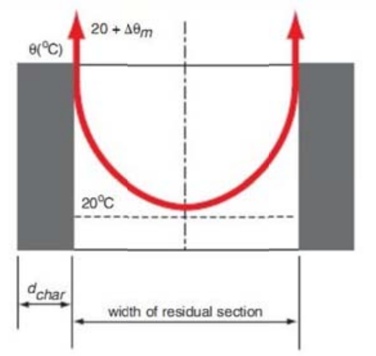


Figure 3-2: Temperature profile for thickness greater than the minimum, thermally thick sections

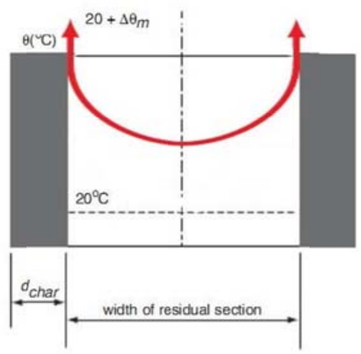


Figure 3-3: Temperature profile for thickness smaller than the minimum, thermally thin sections

3.2.2 Moisture content

Variation of Moisture Content (MC) below the Fibre Saturation Point (FSP) affects many mechanical properties at room temperature (28). Equilibrium Moisture Content (EMC) depends on the ambient temperature and the relative humidity, and for standard conditions (20°C at 65% of relative humidity (35) the EMC is 12%. Moisture content is a very important factor in timber fire behaviour (29), normally timber has moisture content between 8% and 15%. Therefore for a ton of wood from 80 to 150 kg of water have to evaporate. Water absorbs energy to evaporate from the fire, therefore it acts as a heat sink due its higher specific heat. In literature, it is well recognized that specimens with higher moisture content have the lower charring rate (28) (33) (27) (29). However, for a high saturated specimen the strength reduction is increased (34).

3.2.3 Density

Many researchers agree that the higher density will result in lower charring rate due to the presence of greater mass to degrade, however some experiments show that low density samples, pyrolyse slower due to their lower thermal conductivity, indeed they behave as insulators.

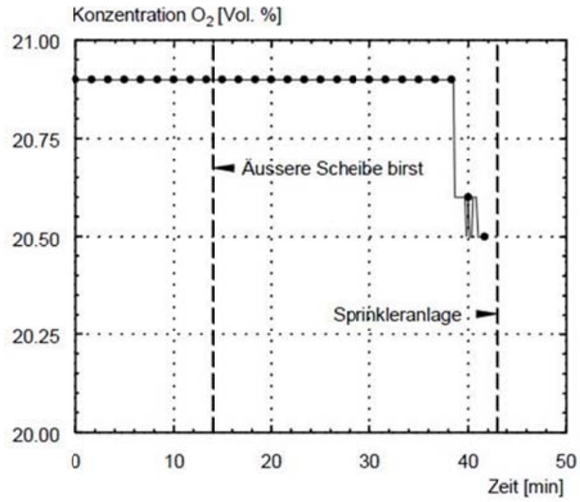
3.2.4 Opening factor

Opening factor (§7.1.2.1) is one of the most important parameters in the development of fire (together with: fire load density, thermal properties of boundary elements and rate of heat release) since it provides new oxygen to the fire. The effect of the opening factor is assumed to be a secondary effect, because it will affect the temperature-time curve first, then the charring rate.

3.2.5 Oxygen concentration

The oxygen concentration in the air is 21%, but in the event of fire, in the fire room, different level can be expected depending on the opening factor which affect how easily fresh air flows inside the compartment replacing the hot exhaust gasses. In (36) is stated that a reduction from 20% to 10% of oxygen concentration will generate a reduction of 20% of the mass loss rate and it is believed that it will also have the same effect on the charring rate.

However, full scale tests made by (37) show only a limited depletion of oxygen (from 20.9 to 20.5%) inside the fire room which will have no or little effect to the charring rate.



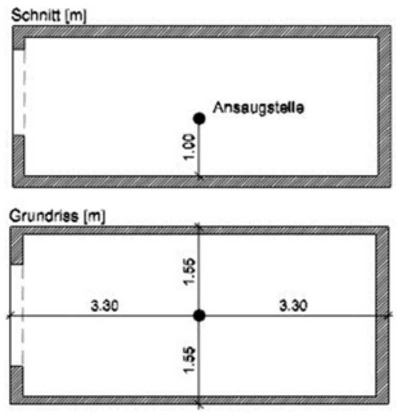


Figure 3-4: Oxygen depletion during fire test BU nbb (no gypsum protection)

3.3 Charring and fire resistance

3.3.1 Solid wood

Charring of timber could be seen as a generation of a protective layer to safeguard the virgin cross section. The charcoal layer has an extremely low thermal conductivity; therefore it is insulating the inner part of the wood which remains with its natural strength and stiffness (Figure 3-5). Indeed, the thermal conductivity of charcoal is one sixth of the pure solid timber. The temperature between the charcoal layer and the pyrolysis zone (called pyrolysis front) is exactly 288°C (34) but for simplicity it is assumed as 300°C. The charring rate corresponds to the propagation rate of the pyrolysis front (30).

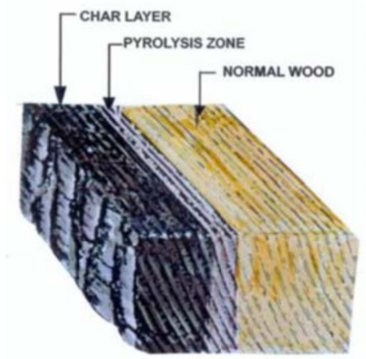


Figure 3-5: Illustration of charring of wood section

As can be seen in Figure 3-6 the temperature of timber element is affected only in the first 30 mm, therefore in this thickness there will be a reduction in strengths and stiffness. It is worth noticing that in (38) when a reduce cross section method is used a zero-strength layer 7 mm below the pyrolysis front is assumed, while in the New Zealand Code, for example, the residual section is assumed from the pyrolysis front, according to (39) this can lead to unsafe results. More recent developments (11) show that for CLT panels the 7 mm layers should be computed for each case.

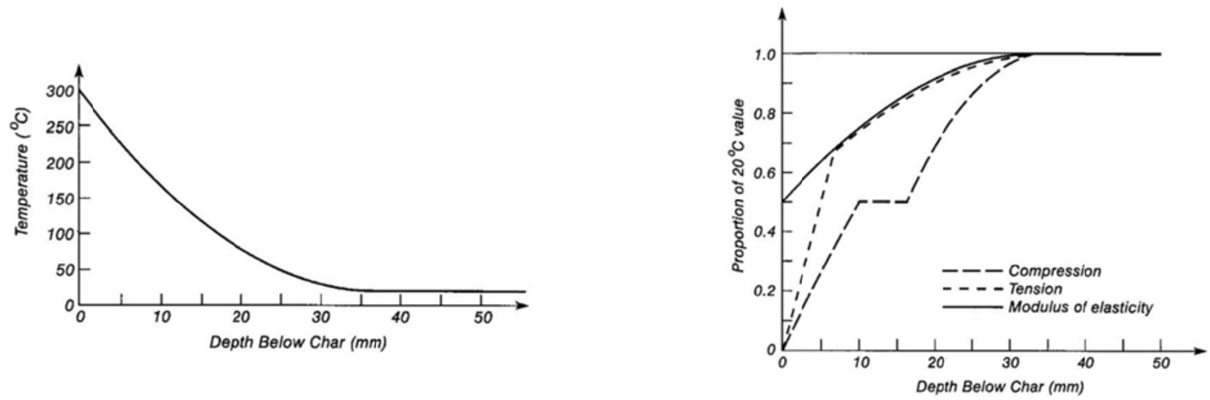


Figure 3-6: Temperature profile (left) and Residual properties (right)

Charring depth for solid woods is assumed varying linearly in standard fire tests. The charring rate may vary from 0.5 to 1.0 mm/min, data for solid and glued timber are given in Table 6.

	β_0	β_n
Softwood and beech	$[mm/min]$	
Glued laminated timber with a characteristic density bigger than 290 kg/m3	0.65	0.7
Solid timber with a characteristic density bigger of 290 kg/m3	0.65	0.8
Hardwood		
Solid or glued laminated timber with a characteristic density of 290 kg/m3	0.65	0.7
Solid or glued laminated timber with a characteristic density bigger than 450 kg/m3	0.50	0.55

Table 6: Design charring rates (characteristic and notational) from en 1995-1-2

The one dimensional charring rate β_0 is used for fire exposure on one side only or for a multi-side exposure taking into account the rounding of corners. However, for more than one side exposures, this method is cumbersome and for fire exposure on several sides the notational charring rate β_n is used. The latter values has a higher charring rate values which allows the use of an equivalent rectangular section.

3.3.2 Cross-Laminated Timber panels

Since there is little literature regarding CLT panels behaviour on fire, extensive scientific investigations and full scale tests have been carried out (33) (40) (41) (42).

3.3.2.1 Tests on panels

As shown by tests the adhesive used has an influence on the fire behaviour. Two adhesives were tested: the one component polyurethane glue (PU) and the melamine urea formaldehyde (MUF) glue. The latter (MUF) shows a better fire resistance against de-lamination (charred layer remains in place and does not fell off). The charring rate for MUF specimens was calculated as 0.60 mm/min, which is even lower than one dimensional charring rate given in (38) .

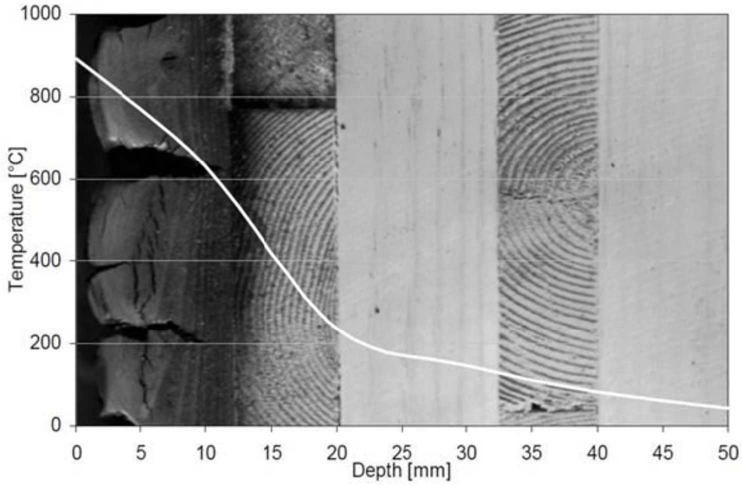


Figure 3-7: Step temperature gradient in the heated zone of the wood (33)

Whenever PU was used, the char layer fells off exposing the virgin wood to high temperature. In this test setup the thickness of the lamellas was relevant, indeed the thicker the lamellas, the better the behaviour. This is reasonable because the felling off of charcoal occurred less frequently during fire. The charring rate was increasing when a felling off of the charred layers occurred, because the fresh (unexposed) wood was suddenly exposed to high temperature without having an insulation layer. This speed slowed down after 25 mm, where the new charcoal layer protects the wood. It important to stress that the later the felling off occurred, the higher the increment of charring rate was, because the temperature in the furnace was higher. The charring rate vary greatly for different temperature time curves (33), indeed higher temperatures lead to higher charring rate, therefore the values given in (38) can be adopted only for standard or parametric fire curves described in (38). However, in this study, charring rates and thermal properties given in the Eurocode 5 are used for time-temperature curves which differ from the aforementioned curves.

Orientation of the lamellas seems to have little influence on the charring behaviour, however cross wise oriented panels show slower charring rate (40).

3.3.2.2 Full scale tests

3.3.2.2.1 Ivalsa fire test

The Ivalsa fire test, deeply described in (41), was carried out in 2006 and it was aimed to determine the fire resistance of a timber building exposed to a natural fire. For the test it was built a 3-storey high timber building made of CLT panels ($85\ mm$ thick for external walls and $142\ mm$ thick for internal wall and floor slab).

Figure 3-8: Full scale test after 40 minutes (flashover)

The fire room had the following dimensions $3.34x3.34x2.95\ m$. External walls were protected by two layers of gypsum plasterboard (type F – 12 mm, type A -12 mm) and by a 27 mm thick layer of mineral wool, internal walls had only one layer of gypsum type F of 12 mm. Two window openings were present, with the dimensions of $0.94x0.94\ m$ each, but only ¼ of the window width was open at the beginning of the test, leading to an opening factor $A_w\sqrt{H_0}/A_t \approx 0.0071\ m^{1/2}$. The windows broke after 20 and 30 min from ignition and flashover occurred after about 40 min. The movable fire load in the room was 7472 MJ, accounting the wooden floor with a participation factor of 50%, the total fire load was 8299 MJ, leading to a fire load density of 790 MJ/m^2. The wood net heat combustion used was 17.5 MJ/kg was assumed (neglecting the moisture content).

The temperature development was monitored at more than 100 locations (in the panel thickness, on the panel surface, in the fire room, outside the windows and in the adjacent rooms). The registered temperatures in the fire room and across the ceiling thickness are given below. During the test there was no fire spread in the other compartments, however after 53 minutes the door of the fire room started to fell off, leading smoke spreading in the adjacent room. Temperatures in the above and adjacent rooms were at normal values for the all duration of the test.

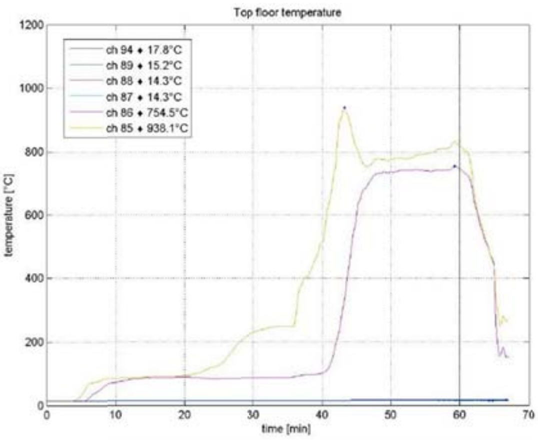


Figure 3-9: Temperatures across the ceiling thickness

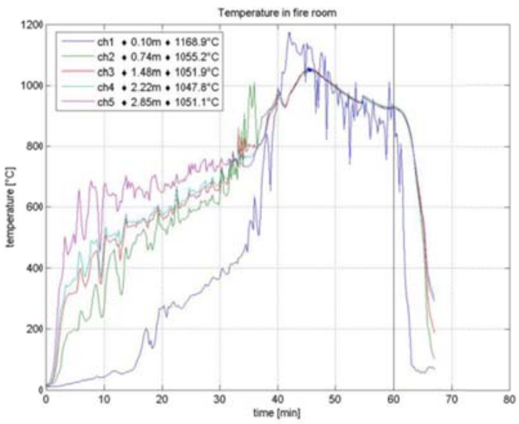


Figure 3-10: Fire room temperatures at different heights

The temperatures (Figure 3-10) grew slowly at the beginning of the test, until the failure of the windows, then flashover occurred in 10 minutes. The ceiling was protected by one layer of gypsum plasterboard type F (12.5 mm) and one layer of mineral wool (27 mm), and the maximum temperature reached in the interface between the mineral wool and the CLT floor panel (ch86 in Figure 3-9) was 754.5℃. The temperatures registered in the walls were less severe than the values obtained for the ceiling. The test was manually extinguished at 60 min from the beginning as already planned. The charred depths computed on the walls and on the ceiling were varying between 5 to 10 mm, but deeper values occurred locally.

3.3.2.2.2 Hakkanainen's fire tests

Real scale fire tests on a timber compartment were carried out in 2001 by Tuula Hakkanainen (43). Four room configurations were studied, depending on the protection setups. For the purpose of this work, only fire test 1 and 3 are deeply treated (marked in bolt in Table 7).

Test no.	Construction	Gypsum plasterboard protection	Time to flashover (min)	Temperature (°C) Front Typical	Front Max	Center Typical	Center Max	Rear Typical	Rear Max
1	**Heavy laminated timber - CLT**	**None**	**4:50**	**700**	**1050**	**700**	**1100**	**700**	**1050**
2	Heavy laminated timber - CLT	1 layer, type A	4:30	800	1050	750	1000	600	800
3	**Heavy laminated timber - CLT**	**2 layer, type A + F**	**6:00**	**1000**	**1100**	**950**	**1200**	**700**	**1150**
4	Wood studs with mineral wool insulation	2 layer, type A + F	6:10	1000	1200	1000	1200	750	1150

Table 7: Summary of tests carried out in (43)

The fire room was of $3.5\ m$ wide, $4.5\ m$ length and $2.5\ m$ high, with only one window opening (2.3 by $1.2\ m$) in the small side, no glass was installed in the opening, leading to an opening factor $A_w\sqrt{H_0}/A_t \approx 0.042\ m^{1/2}$. Four wooden cribs inside the room represented a fire load density of approximately $720\ MJ$, a net heat combustion of $15.5\ MJ/kg$ was assumed for wood with a moisture content of 10%.

In test 1, the gas temperature was approximately $700℃$ for the major part of the experiment. The relatively low temperature was due to insufficient ventilation. In addition to the fire load, the unprotected timber ceiling and walls of the room released pyrolysis gases from the very beginning of the test. Generating and heating up the large amount of pyrolysis gases consumed a lot of energy. Due to insufficient supply of oxygen, only a part of the gases could burn inside the room. The un-burnt gases flowed out of the window opening, causing intense combustion outside the room where oxygen was available. Towards the end of the experiment when most of the movable fire load had burnt, the temperature started to increase since the generation rate of pyrolysis gases was decreasing, and burning could take place in the fire room because more oxygen could enter the compartment. The same phenomena were seen also in test 2 in which the timber ceiling and walls were protected by a single layer of gypsum plasterboard. The gas temperature, however, was somewhat higher at approximately $800℃$. The gypsum plasterboards started to fall down approximately $13\ min$ after flashover allowing intense pyrolysis from the ceiling and the upper parts of the walls. The gas temperatures in tests 3 and 4 were considerably higher, approximately $1200℃$ at maximum. Two layers of gypsum plasterboards protected the timber structures for $25–30\ min$ after flashover. Thus, the ceiling and walls did not contribute to the fire load during the most intense burning of the wooden cribs and the particle board flooring. Consequently, the generation rate of pyrolysis gases was significantly lower than in tests 1 and 2. Burning took place mainly inside the room. It is noted, however, that these fires were also ventilation-controlled. In all experiments, the temperature was higher in the front than in the rear of the room. The wooden cribs positioned closer to the window opening burnt more severely than those in the rear due to the limited amount of oxygen. After the front cribs burnt out, burning of the rear cribs intensified and the temperature in the rear of the room increased.

	Time to flashover $[min:sec]$	End of the test Time $[min]$	Reason
Test 1	$4:50$	50	Excessive flaming
Test 3	$6:00$	169	Failure of ceiling-wall joint

Table 8: Flashover and end time of test 1 and 3

The temperatures obtained from test 1 and 3 are shown in Table 9. It is important to note the differences in the horizontal axis. Both the tests have the same initial fire load and opening factor, however test 3 reached higher temperature than test 1, this has been explained due to lack of oxygen inside the fire room in test 1 (the horizontal plateau in Figure 3-11 is due to large production of pyrolysis gasses).

The main conclusions from (43) are:

- Unprotected timber generates more pyrolysis gasses, which flew through the opening, and reaching oxygen caused intense combustion outside the compartment;
- Temperature inside the room was depending on the location respect to the opening, the closer, the higher;
- When gypsum plasterboards were used, higher temperature inside the room were reached because pyrolysis gasses production was less pronounced so oxygen could enter freely into the room and burn;
- Predicted time-temperature curves according EN 1991-1-2 parametric fire exposure computed with only the movable fire load and the floor (excluding walls and ceiling), were overestimating by 300-500°C the real temperature development for test 1, while the predicted temperature were closer for test 3.
- Gypsum plaster board delay significantly the onset of charring, by about 20 minutes, compared to unprotected timber.

Test 1:

- Heavy timber construction in sight;
- Insufficient ventilation due to the large amount of pyrolysis gasses produced by the timber structure;
- Vigorous flaming outside the opening;
- After 30 to 40 min the movable fire load was completely burnt, the temperature starts rise due to the lower production of pyrolysis gasses;
- Temperature in the fire room were 300-500°C lower than the predicted by parametric fire curve of EN1991-1-2;
- The maximum charred depth was between 29-41 mm at the end of the test (50 min);
- Early interruption of experiment to avoid burn-though and collapse of the timber structure.

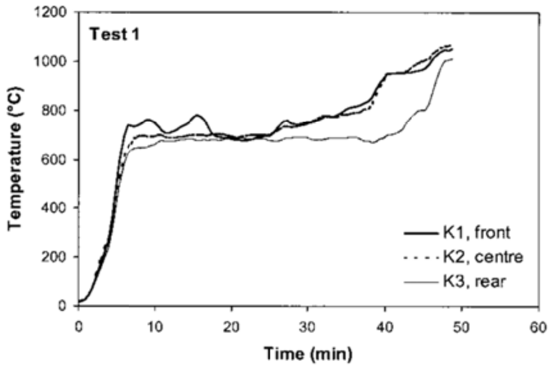


Figure 3-11: Room temperatures for test 1 (no gypsum)

Test 2:

- Heavy timber construction protected by a single layer of type A gypsum;
- The protective layer fell off 13 min after flashover;
- Charring started at 20 min from ignition;
- Early interruption of experiment to avoid burn-though and collapse of the timber structure;
- Temperature in the fire room were 300-500°C lower than the predicted by parametric fire curve of EN1991-1-2;
- The maximum charred depth was between 8-13 mm at the end of the test (46 min).

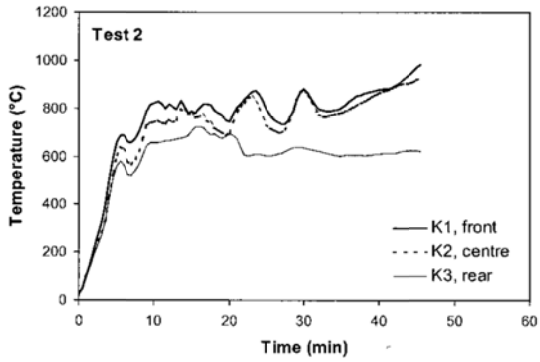


Figure 3-12: Room temperatures for test 2 (protected)

Test 3:

- Heavy timber construction protected by a one layer of type A gypsum and one layer of type F gypsum (close to the fire side);
- Charring started at 40 min from ignition, ;
- Burning took place mainly inside the fire room;
- Heat flux of flames from windows was maximum at 40 min;
- The temperature prediction of EN1991-1-2 was close to the measurements;
- The maximum charred depth was between 23-26 mm at the end of the test (169 min).

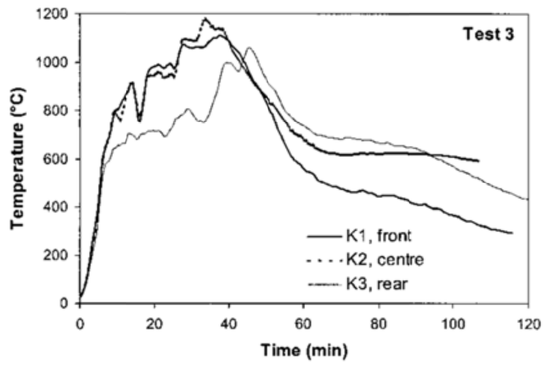


Figure 3-13: Room temperatures for test 3 (protected)

Table 9: Result of Tuula Hakkarainen. Note the different time scale of test 3.

3.3.2.2.3 Modular hotel units

Different fire tests were carried out to evaluate the effectiveness of active fire fighting systems, such as sprinkler and smoke detection system. The tests also studied the resistance of the load bearing function of the wooden structure. The module dimensions were $6.6\ m$ length, $3.1\ m$ wide and $2.8\ m$ high. Only one window was present, its dimensions were $1.5\ m$ high by $1.7\ m$ wide. With this dimensions, assuming the window completely open, the opening factor is $A_w\sqrt{H_0}/A_t \approx 0.0328\ m^{1/2}$. The total fire load density (calculated over the floor area) for the modules with non-combustible wall and ceiling linings varied between 363 and $366\ MJ/m^2$, and for the module with combustible wall and ceiling linings the total fire load density was approximately $855\ MJ/m^2$.

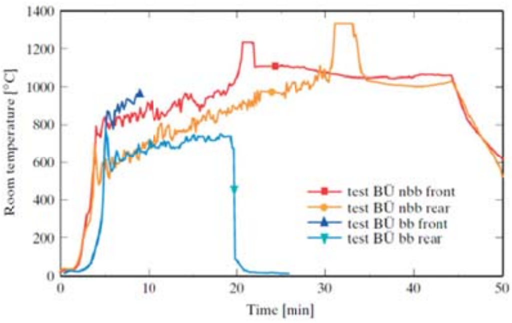
The three configurations studied are: unprotected compartment (*BÜ bb*), fire room protected with only one layer of gypsum (*BÜ nbb*) and protected compartment with 3 layers (*BÜ nbb demo*). During the test the modules were continuously weighed, the difference in weight was of about 8%, which is $900\ kg$. This weight loss was due to evaporation of water and degradation of charcoal (at temperature above 450°C). For the *BÜ nbb demo* test, $260\ Kg$ were lost by pyrolysing furnishings, $240\ kg$ were lost by floor pyrolysis and the remain part, $500\ Kg$, was due to water evaporation.

In all the tests carried out with activated sprinkler system, the fire was not able to reach flashover, moreover the sprinkler system was able to extinguish the fire before it could spread within the compartment. The automatic fire extinguishing system was activated between two and three minutes. The temperatures inside the fire room were varying between 50 and 200°C, avoiding the occurrence of flashover. When the sprinkler system was deactivated, for the module with combustible wall and ceiling linings flashover occurred after about four minutes. The flaming out of the window was much more severe for the combustible finishing module. For the modules with non-combustible wall and ceiling linings flashover occurred after approximately six to seven minutes (Table 10).

	Fire test		
	BÜ nbb	*BÜ bb*	*BÜ nbb demo*
Linings	Non –combustible	Combustible	Non –combustible
Layers	1 layer gypsum plasterboard	1 layer OSB panel	3 layers gypsum plasterboard on ceiling, 2 on walls
Flashover	$6:00$	$4:27$	$6:58$
End of the test	$44:15$	$18:53$	$59:01$
Reason	Interior glass layer of window of the upper module failed	Excessive flaming	Complete burnout of combustible material

Table 10: Main results from fire experiments (44)

During the tests, the temperatures were measured for different heights and for different locations in the fire room, their values are given in Figure 3-14.



Room temperatures measured on the ceiling in the front and in the rear of the lower modules for the fire tests BÜ nbb and BÜ bb (the temperature of the test BÜ bb front are not complete due to a loss of electrical power that occurred during the fire test after about 10 minutes). The test BÜ bb was stopped after 20 min from ignition due to excessive flaming.

Figure 3-14: Ceiling room temperatures in tests BU nbb and BU bb

The temperatures were higher close to the openings due to a high oxygen concentration than in the rear of the fire room, in accordance with values obtained by Hakkarainen in (43) and shown in §3.3.2.2.2. Further, no significant differences were observed in the temperature curves for the module with and without combustible wall and ceiling linings. This confirms that only a part of the pyrolysis gases released by the combustible wall and ceiling linings burnt inside the room. The unburnt gases flowed out of the window opening, causing intense combustion outside the module where oxygen was available in large quantities.

3.4 Wood properties at high temperatures

Mechanical properties of wood are affected by the temperature. As can be seen from Figure 3-6 above, high temperatures reduce mechanical properties of wood. For temperature rises below 100°C and, for rapid temperature changes, reduction effect is reversible. In the range of 7 to 38°C no significant reductions occur. When temperature rises more than 100°C permanent strength reductions occurs, and the magnitude of this effect depends on the moisture content, species of timber, level of radiant heat exposure, dimensions of the timber and exposure period (27) (28). An increase of wood moisture content generally leads to greater reductions (34) (45).

The thermal properties such as the thermal conductivity, the density and the heat capacity, are also influenced by the temperature (43) (46). Those three parameters affect the b-factor ($b = \sqrt{\lambda \rho c_p}$) of a boundary element which is one of the main factors for natural fire models. In order to simplify the analysis, the EN 1995-1-2 allows to use a constant value for b-factor calculated at 20°C (38). The thermal properties given in EN 1995-1-2 are apparent values determined for a fire exposure to a standard fire curve, ISO curve. Since these value are strongly related to the temperature it is not allowed to use the values proposed in EN 1995-1-2 for different fire exposure than the ISO fire curve. The apparent values proposed already take into account the effect of mass transfer. The thermal properties of timber are further explained in §7.3.5.1.

3.5 Smoke production

Smoke production during a fire, especially in the first phase, is a key feature. If significant amount of smoke is produced, occupants will not be able to leave the building safely. Smoke has two main effects; first, it will reduce the visibility, which in turn enhances the escaping time and makes escaping more difficult. Second, it is made of noxious and toxic substances (§ 4.1.1). Independently of what is burning, a fire which will reach flashover will generate large amount of toxic products as carbon monoxide, carbon dioxide and hydrogen chloride (28). The smoke generation aspect has been introduced in the European classes regarding reaction to the fire. The European classes go from A1, for non-combustible material, to F, for materials which are impossible to test[10]. Products in Euroclasses from A2 to D that generate little or no smoke will be classified S1;

[10] Interesting web site: http://www.rockwool.com/fire+safety/eu+standards/cen+classes

products with medium dense smoke S2; whereas materials generating large amounts of smoke - thus making escape difficult - will be classified S3.

3.6 Fire retardants treatments

Fire retardants treatments (FRT) affect the ignitibility of wood, slowing down the flame spread but they do not affect the charring rate (30). To achieve a better fire reaction the following techniques may be pursued:

i. changing the pyrolysis of wood, so the thermal degradation of cellulose will generate char and water (no flammable gases) reducing the production of combustible gases and thus decreasing the hear release rate;
ii. protecting the surface by isolating layers: delaying temperature rise by using intumescent coatings;
iii. slowing down ignition and burning by changing the thermal properties of the product. The easiest way to enhance fire resistance is to make the timber wet, this has two advantages:
    a. Water acts as a heat sink due its higher specific heat;
    b. Evaporating of water reduces the combustibility of the mixture of air and pyrolysis;
iv. reducing combustion by diluting pyrolysis gases, producing more non-combustible gases;
v. reducing combustion by inhibiting the chain reactions of burning, but they are not environmental friendly.

Most existing fire retardants are effective in reducing different wood reaction-to-fire parameters such as ignitability, heat release and flame spread. However they cannot make wood non-combustible (30). Many of FRT may have adverse effects on the other wood properties (mechanical strength may be reduced). Indeed, in (11), special attention is recommended when FRT woods has to be used as load-bearing structure, due to the strength reduction during the service life resulting from deterioration of the wood structure. This is founded also in (31), where it is stated that traditional fire retardants are water-based solutions, which are hygroscopic and cause high moisture contents, which, in turn, may produce undesirable effects such as corrosion, fungal decay and reduction of mechanical strength. For (13) fire-retardant chemicals do not significantly improve the fire resistance of timber members, because even though treated wood will not support combustion, it will continue to char if exposed to temperature of a fully-developed fire. Fire retardant treatments need to be maintained, increasing the costs and affecting the construction method because timber must be in sight in order to be painted.

3.7 Summary

According to the sources founded in literature, it can be concluded that timber has a good fire resistance but an extremely poor fire reaction. The fire reaction can be improved by fire retardants treatments, which, however, will not make it non-combustible. They may slow down the charring rate but they have major disadvantages like the reduction of mechanical strengths and the need of maintenance. These effects make these techniques not applicable to an high rise timber building. It is believed that a more effective way to reduce the production of smoke is represented by encapsulating the timber element.

The CLT panels shows a good charring behaviour, their charring rate is in the range of $0.6 \div 0.8\ mm/min$, depending on the adhesive used and the thickness of the lamellas. Delamination of the charred lamellas is a main concern to determine the charring rate. If the adhesive will fail and delamination occurs, higher charring rate are obtained.

As shown in the experiments cited above, the behaviour of CLT panels is strongly influenced by several factor which are listed below:

- The number of layers which affects the overall thickness of the panel, and the thickness of each layer;
- The adhesive used which affects the felling off of the charcoal layer;
- The orientation of the panels, vertical or horizontal, which affects the felling off of the charred layer;

- The use of protective material which affects the amount of pyrolysis gasses generated and in turn the maximum temperature (external or internal flaming);
- The amount of openings and the amount of fire load which affect the temperature-time curve;
- Close to the openings the temperature is higher, due larger availability of oxygen which allow burning of volatiles;
- With only passive fire protections it is possible to limit the spread of fire to the room of origin even for timber structures;
- The presence of sprinkler system prevents spreading of fire and the occurrence of flashover.

According to the real scale tests studied, a load bearing structure made of CLT panels can survive to a one-hour real fire exposure. The outcomes from those tests will be used to validate the results obtained from an advance fire analyses.

4 Fire safety engineering

This chapter deals with the main aspects which play an important role to fulfil one of the fundamental objects of fire safety engineering, namely the safety of people. However it is not possible to clearly distinct between measures for reducing human losses and measures for reducing structural or material losses, because they are greatly interconnected. In this section, the threads to inhabitants are analysed, then, fire protection measures are studied to overcome these hazards. The fire safety measures described here are commonly used for building with non-combustible structure but they can be also used in a high-rise timber building.

4.1 Safety of inhabitants

Worldwide, smoke and heat are recognized being the main hazard to life and it is widely accepted that when casualties occur, they occur at temperature much lower than temperature needed for structural collapse (15) (45) (47) (48) (49) (50) (51). Smoke consists of airborne, solid and liquid particulates and gasses that evolve when a material undergoes pyrolysis or combustion. Smoke presents potential hazards because it obscures vision and it contains noxious and toxic substances. The smoke hazard is sadly explained by one of worst accidents in the US fire hotel history, the MGM Grand Hotel fire, Las Vegas, 1980. The majority of the casualties, 85 people, died between the 20^{th} and the 25^{th} floors, while the fire floor was at the casino level, 4^{th} floor. At the 26^{th} floor there was enough heat to activate the sprinkler system ($t \geq 58$°C) (45). All of victims located in the high-rise tower (74 people) died due inhalation of carbon monoxide. Smoke and toxic fumes spread rapidly through the air ventilation systems, the stairways and the tower's elevator hoist ways (47). 619 people were transported to hospitals, 20 helicopters were involved for the evacuation of about 300 guests from rooftop and 12 from balconies (52). In Canada, fire statistics show that the majority of fire deaths occur in residential buildings. This is mainly due to the fact that Canadians spend two-thirds of their time at home and, for a significant portion of that time they are sleeping, thus a greater risk due to decrease of awareness. It is also shown that in the last two decades fire deaths have dropped due to increased use of smoke detectors, education programs, improvement in electrical and heating systems and changing in life-style habits (48).

To reduce the risk of life due to smoke and heat, there are two approaches: limiting the production of smoke and control the spread of smoke. Some other measures as smoke exhaust vents, early detection devices, and sprinkler systems can also reduce the life threat. It is believed that in an unprotected timber building it is almost impossible limiting the production of smoke. Indeed according to European norms timber elements are classified as D-s2,d0, meaning that they produce medium dense smoke without dropping of burning items. The designer must be aware of this material characteristic and account strategies to mitigate this negative and dangerous effect.

4.1.1 Smoke hazard

Toxicity of combustion products is a main concern, and it is not related to the burning material. Fire victims are often not touched by flames but died as a result of exposure to smoke, toxic gases, or oxygen depletion. Whichever fire reaches flashover will generate a large amount of smoke and dangerous levels of carbon monoxide independent of what is burning. Carbon monoxide is particularly dangerous for health because it is colourless, odourless, and tasteless, but highly toxic (Table 11). It can combine to the haemoglobin in the blood, producing carboxyhemoglobin, which is ineffective for delivering oxygen to bodily tissues. Carboxyhemoglobin in concentration higher than 50% in the blood is lethal (28). The generation of combustible and toxic carbon monoxide is strongly dependent on ventilation: well ventilated combustion produces considerably less CO (less than $10\ g/kg$ of burning material) than oxygen controlled burning where CO production is of the order of $100\ g/kg$ of burning material (30). The second toxic product of fires is carbon dioxide, which is toxic at high concentration and it will increase the breathing rate (at 5% of concentration breathing rate is increased by 3 times), enhancing the absorption of toxic compound (53).

CO Concentration	Symptoms
0.1 ppm	None - Natural atmosphere level
0.5 to 5 ppm	None - Average level in homes
35 ppm (0.0035%)	Headache and dizziness within six to eight hours of constant exposure
400 ppm (0.04%)	Frontal headache within one to two hours
1,600 ppm (0.16%)	Headache, tachycardia, dizziness, and nausea within 20 min; death in less than 2 hours
3,200 ppm (0.32%)	Headache, dizziness and nausea in five to ten minutes. Death within 30 minutes.
6,400 ppm (0.64%)	Headache and dizziness in one to two minutes. Convulsions, respiratory arrest, and death in less than 20 minutes.
12,800 ppm (1.28%)	Unconsciousness after 2-3 breaths. Death in less than three minutes.

Table 11: Carbon monoxide effects on human health, adapted from (54)

4.1.2 Smoke control

During evacuation it is essential that a smoke-free layer is ensured. A clear visibility of 2.5 or 3 meter above the floor is generally needed (10).

The main driving forces (45) that cause smoke movement through the building are:

- Stack effect, due to thermal differences between the inside of the building and the outside. When the inside temperature is higher than outside, normal stack effect will occur. Smoke movement can be dominated by stack effect, in case of reverse stack effect the buoyancy forces can be so great to overwhelm the downward movement of smoke.

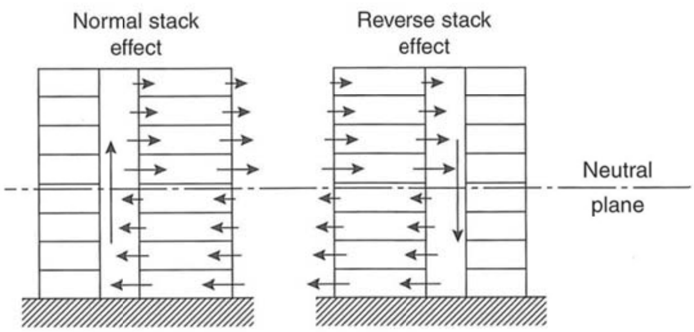


Figure 4-1: Air movement due to stack effect and reverse stack effect

- Buoyancy, due to the high temperature of smoke which reduces its density. This overpressure in the fire room force the hot smoke to move through any leakage path, for example doors, ceilings and service installations. As soon as the smoke travels away from the fire, its temperature drops down and consequently this effect decreases with distance from the fire.
- Expansion, due to the high temperature. This is noticed in presence of a window, where two counter flows will be present, hot smoke going out, and cool air coming in. The ratio of volumetric flows can be simply expressed as a ratio of absolute temperatures.

$$\frac{Q_{out}}{Q_{in}} = \frac{T_{out}}{T_{in}}$$

For compartment with openings the pressure difference across these openings is negligible. However, for tightly sealed compartment this overpressure may be important.

- Wind has the pronounced effect on smoke movement within a building, especially for loosely constructed buildings, or for buildings with open doors or windows. In case of fire, windows will break when flashover occurs. If broken windows are on the leeward side of the building, the negative pressure caused by the wind vents out the smoke form the compartment. This may reduce the smoke movement throughout the building. Otherwise, if the broken windows are on the windward side, smoke will be forced inside the building.

- HVAC systems, they can create significant smoke migration. Therefore, it is essential that HVAC systems have been designed to prevent smoke propagation and control spread of fire.

To allow a safe evacuation of the occupants and a smoke-free staging area for fire fighters pressurized stairwells are designed. This concept has a major disadvantage, if doors are left open or too many doors are open simultaneously, the system may not work properly allowing smoke to enter the compartment. The importance of self-closing doors is then highlighted. Elevator shafts have to be designed in order to control smoke spread, and special provisions have to be used to use the elevators for fire evacuation.

4.2 Fire safety in high rise buildings

Fire Safety Engineering in high rise building is a main concern due to different reasons:

- fire fighters action is limited to an internal attack, fire department ladders cannot reach floors above the 8th floor (30 meter);
- longer escaping time;
- significant amount of people to evacuate;
- smoke movement inside the building;
- presence of both, fire fighters and people, in the stair case;
- change of entrapped occupants in storeys above the fire floor.

Fire safety design for high rise building has to consider the structure as a total system considering all aspects of architecture, structure, fire protection, mechanical and electrical system (55).

A comparison between the acceptance levels for active fire measures in different countries is given in Table 12. From the table it can be seen that already twenty years ago the positive contribution of sprinkler systems was widely accepted, but still today its reliance is argued and some building codes do not recognize its benefits.

Countries		Switzerland	Italy	Canada	U.K.	Netherlands	Sweden	Norway
Acceptance of alternative concepts/active measures versus passive measures	Yes	✓						
	Some limitations			✓		✓		✓
	Strong limitations		✓		✓		✓	
	Not admitted							
Alternative concept allow for a major reductions of fire resistance requirements in connection with:	Autom. Sprinkler systems	✓	(✓)	✓	In some cases	✓	(✓)	(✓)
	Autom. Detection systems	✓		✓				
	Work fire brigade 24/24	✓						(✓)
	Heat and smoke vents							
Alternative concept allow for a major increases in the size of fire compartment in connection with:	Autom. Sprinkler systems	✓	✓	✓	✓	✓	✓	✓
	Autom. Detection systems	✓		✓	Doubling size	In some cases	✓	✓
	Work fire brigade 24/24	✓				In some cases	In some cases	
	Heat and smoke vents	✓				✓	✓	

(✓) = reduction 30 min fire resistance requirements ✓ = accepted

Source: (51).

Table 12: Building regulations

4.2.1 Sprinkler system

It is widly acceped that sprinkler systems can reduce the risk to life and significantly reduce the degree of damage caused by fire (56). Sprinkler systems fight fires more effectively than any other existing technology (57). Sprinklers provide improved capabilities to extinguishing a fire in its initial stages (28). One example of this is One Meridian Plaza fire, in 1991. When a 38 story building burned out of control until the fire reached the only floor equipped with automatic extinguishing devices (30th), only ten sprinkler heads were activated (58).

The fire claimed three fire fighter's lives, and eight floors were completely destroyed. Those sprinklers extinguished a blaze that had defeated the efforts of the Philadelphia Fire Department. It is believed that they are the basic fire safety measure that should always being installed in every building, regardless the height.

One of the difficulties in assessment of fire safety concepts is the possibility of failure of the system. If a building is designed assuming the effectiveness of the active system, some trade off can be applied, but since no system is 100% reliable, when the system will fail the structure will be exposed to a fire without adequate protection. The reliability of sprinkler system is studied in (59), where it is stated that the sprinkler systems operated properly in 91% of fires with an efficiency of 96%.

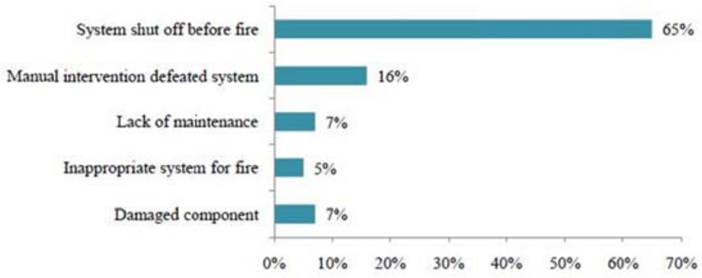


Figure 4-2: Reasons When Sprinklers Fail to Operate, 2005-2009

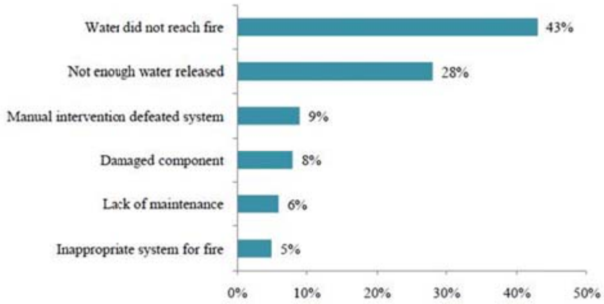


Figure 4-3:Reasons When Sprinklers Are Ineffective, 2005-2009

Koffel carried out an extensive research regarding reliability of sprinkler system, studying the NFPA report (60). The NFPA statistics determined a sprinkler reliability of 96%. His conclusion was that the value given by the NFPA is overstating the reliability of sprinkler systems and a more reasonable value of 90% is proposed, in accordance with British Standards.

Nystedt studied the effectiveness of Sprinkler systems based on data from (61), and he stated that sprinklers provide additional safety to the fire spread, and they are effective in reducing the risk of death (53). When wet pipe sprinklers were installed the death rate decreases of 83% related to non-sprinklered building. Sprinkler systems are designed to control the fire, to cooling down fire gasses, and wetting the surrounding material to avoid further spreading. However in most cases this target is exceeded and the fire is suppressed. Good examples are represented by full scale tests carried out on modular hotel units, where a compartment with and without combustible linings was exposed to a fire. In all the tests, the sprinkler system managed to extinguish the fire. Those tests are explained in deep in §3.3.2.2.3.

The effect on tenability conditions due sprinklers activation is analysed in different tests carried out by Purser, Williams, Sekizawa and Schonberg and cited in (53). The main findings are:

- Fires were rapidly extinguished before they could growth enough to produce excessive amount of heat and smoke;
- Peak temperature was greatly reduced, from more than 500°C to 125°C;
- Carbon monoxide concentration was generally reduced, but for tests performed by Sekizawa, an increase was noted. This can be explained by reduced combustion efficiency;
- Oxygen concentration was found to be at much higher level;
- The visibility in the fire room in the early stage was reduced, due to a well-stirred smoke layer (this is the major disadvantage);

The results of different tests are summarized below.

	Variable	Non-sprinklered	Sprinklered
Large room fire	Peak temperature	Exceeded 400°C	125°C
	Peak carbon monoxide	30000 ppm	700 ppm
	Minimum oxygen	2 %	19 %
Bed room fire	Peak temperature	Exceeded 500°C	110°C
	Peak carbon monoxide	4000 ppm	1000 ppm
	Minimum oxygen	12 %	19 %
Time top loss consciousness		1.5 min	6 min
Time to death		2.0 min	-

Table 13: Test results for sprinklered and non-sprinklered compartment

4.2.2 Evacuation procedures

Evacuation procedures are mainly divided in two strategies:

- Simultaneous evacuation: all occupants are evacuated at the same time, it needs wider escape stairs and it may overwhelm the pressurization system capacity in the staircase when doors are keep open for long time; or
- Phased evacuation: only occupants who are more exposed to an elevated risk are evacuated initially. This strategy requires more time to achieve a total evacuation.

In the occurrence of fire, especially in high populated structures, a phased evacuation mode is used (55) (62). This evacuation mode has two stages, the first consists of relocating occupants which are close to the fire hazard (fire floor, floor below and floor above), to the ground floor or several floors below in temporary refuge areas, waiting for further instructions. If the fire cannot be contained a stage two evacuation will be called and the whole tower will be evacuated.

In very tall buildings, for instance, since descending the stairs from floors over 100 meter is beyond the physical ability of ordinary people (63), occupants are asked to reach the sky lobbies and from there, use the shuttle lifts to the ground. This is done in the Eureka Place Tower, a 88-storeys residential building, in Australia. It is divided in three vertical evacuation zones, from ground floor to 23rd floor, from 24th to 53rd floor, and from 54th to 88th floor. Transfer areas are placed on levels 24 and 54 where express elevator to ground floor are present. In case of fire, the vertical zone where the fire is located, would evacuate by stair until the closer transfer floor, and there take the express elevator (64).

Different opinions regarding evacuation for residential buildings have been founded in literature. Nowadays there is the tendency to adopt the “stay in place” defence technique, which means that occupants do not have to leave the building but they have to stay in their flats and defend in place. Only people within the fire compartment will be evacuated. People in the remaining part of the building are deliberately not made aware of the fire incident (65) (66) (67). It is believed that in residential buildings it is safer not to evacuate the building because only people who experienced the fire will quickly get out, while people who hear the alarm will take long time before evacuating. People will not able to quickly evacuate the building because they will lose time getting dressed, taking care of relatives and trying to collect their personal belongings. A delayed evacuation could be fatal due to the amount of smoke and heat that have been produced by the fire. The stay in place defence seems to be very useful in residential building with elderly or impaired people who cannot safely escape by their ones. The stay-in-place defence however has several restrictions, like:

- It should be adopted only for buildings made of non-combustible materials;
- Only for residential building with more than 6 storeys;
- Self-closer door are needed for all entry doors;
- A central alarm system is required together with a voice communication system.

4.2.3 Helicopters for rooftop rescue

Helicopters may have huge advantages in saving life from high rise buildings, provided that people can reach the roof. This is in agreement with the Italian building code, which prescribes a helicopter deck on structures higher than 80 meter. After the event of 9/11 the FDNY intends to buy helicopters provide possibility to safe entrapped people and improve fire-fighting[11]. According to Deputy Fire Chief Don Anthony[12], helicopters proved their worth during the First Interstate Bank fire, in 1988, "I really think fire helicopters were critical on this fire, and I think if we had had hundreds of people on the roof, they could have effected a tremendous number of rescues." Helicopters of NYPD and NYFD saved several people from the helipad (7 people out of 50) (68). Also in the MGM Grand Hotel fire, in Las Vegas, helicopters provided their best. 20 helicopters rescued about 300 people from the rooftop (52). However, Best stated that the major problems with the use of helicopters involved communications, rotor noise, and rotor wash. Rotor noise and rotor wash were special problems due to the use of two Air Force Heavy Lift Aircraft, which hovering above the hotel, caused so severe noise at ground floor that fire officers had problem using their radio. In (55) and (63), is stated that most of the casualties from the World Trade Centre attack, perished above the impact floors, Table 14. Discussion has been arisen regarding the possibility of saving more human life allowing helicopter rescue[13].

	North Tower (WTC1)	***South Tower (WTC2)***
Impact floor	from 93rd to 98th	from 78th to 84th
Dead		
Total	1434	599
Above impact floor	1360	595
Below impact floor	72	4
Survivors		
Above impact floor	none	4 people
Below impact floor	99% over 4000 people	almost all[(1)]

[(1)]: adjusted from (63)

Table 14: World Trade Centre attack

4.2.4 Fire and smoke proof elevators

Fire and smoke proof elevators may be used for evacuate the building and for reach more quickly the fire floor. The use of elevators during an emergency is fully addressed in several sources (62) (24) (64). Fire-fighters lifts[14] are required for building higher than 30 meter in over 12 counties and they are defined as: protected elevators with a dedicated fire-proof lobbies on each floor, associated with a stairway fitted with a standpipe, housed in a 2 hours protected shaft. From each fire shaft must be possible to reach every point of the floor with 60 meter of hose. The fire protected lobby could be used as refugee area for the occupants (impaired people will have to wait being rescued here) and as a safe place where fire services can establish hose lines in order to attack the fire. It is important to install smoke detection system in the protected lobbies to avoid casualties. Another important aspect is to allow hoses to pass through doors without compromising their smoke tightness. Ingress of smoke in the lobby could compromise the elevator use (66), to overcome this problem, fire door with small openings on the bottom side should be used.

[11] http://imgsrv.1010wins.com/image/wins/UserFiles/File/FDNYHelicopterPresentation.pdf
[12] http://www.lafire.com/famous_fires/1988-0504_1stInterstateFire/050488_0788gv_AirOps-Roy.htm
[13] http://www.fpp.co.uk/online/01/10/WTC_Helicopters.html
[14] The best example of fire-fighter lift or protected lift is described in European Union elevator standard EN 81-72 (63)

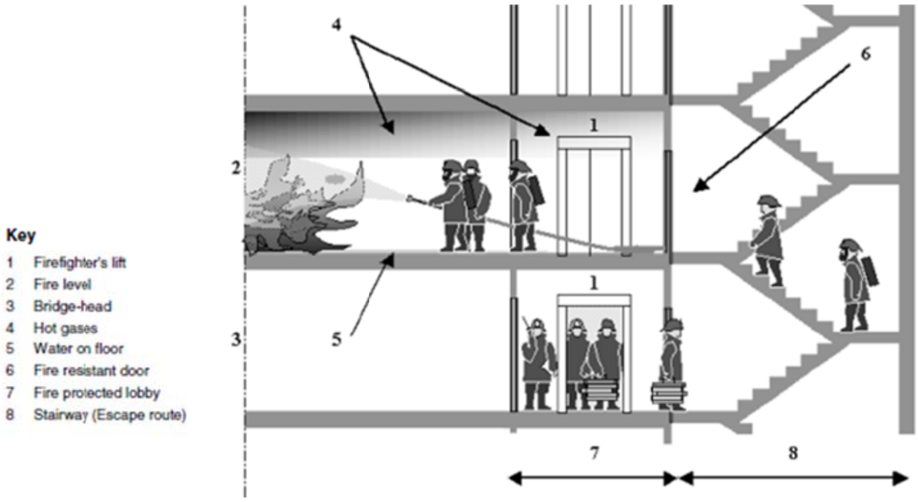


Figure 4-4: Use of fire-fighting elevators

In Toronto, in 1992, in a 25-storey high apartments building (unsprinklered and reinforced concrete construction), a fire occurred at the 9th floor. The apartment occupants escaped leaving the entrance door open. In few minutes the smoke spread to all floors above the fire floor, but the fire was confined in the compartment of origin. The only one fatality occurred inside the elevator, when the elevator doors opened at the fire floor (48). Another casualty due to the use of elevators in fires is represented by a maintenance employee who took a service elevator to investigate the source of alarm at the twelfth floor of the First International Bank building. He died when the elevator doors opened onto a burning lobby (68). He was the only casualty.

Elevators are capable to reduce the travel time for fire brigade, reduce the physical fatigue of fire-fighters who have to climb several storeys with heavy gear and equipment, loosing energy which may be used for fire-fighting (68). Using fire-lifts, fire fighters can only reach two or three floors below the fire floor, as stated in different standard operational procedure, and after having established a hose line, climb to the fire floor (64).

Elevators can reduce also the escaping time and avoid unpleasant congestion in the staircases as shown in Table 15, where four typology of building were tested. The minimum time was met when both the system were working together.

				Evacuation Time [min]		
Building	***Floors***	***Stairs/Elevators***	***Total Population***	***Stairs only***	***lift only***	***Stairs + Lifts***
Hoffman	13	2 / 2 group of 5	3506	14.9	24.3	11.2
White Flint	18	2 / 1 group of 4	1425	14.3	28.6	12.0
Jackson	36	2 / 3 group of 6	3021	23.1	16.5	12.8
GSA	7	6 / 6 group of 2	3621	7.0	17.0	6.3

Table 15: Comparison of evacuation time (64)

In Table 16: Guidance for the use of elevators , a guidance for the use of elevators is given. The study regarding the usefulness of elevators has been addressed by (65).

Building height	***Building use***	***Elevators used in evacuation***
Up to 50 storeys	Offices	++
	Hotel	+
	Residential	+
	Public space	++
50-100 storeys	Offices	++
	Hotel	++
	Residential	++
	Public space	+++
70-100 storeys	Offices	++
	Hotel	++
	Residential	+++
	Public space	+++
Over 100 storeys	Offices	+++
	Hotel	+++
	Residential	+++
	Public space	+++

+	Elevators considered to be of limited benefit
++	Elevators considered useful to support evacuation
+++	Elevators considered essential component of evacuation

Table 16: Guidance for the use of elevators (65)

In (69) a survey, about how to improve the design of tall buildings, has been carried out from architects and civil engineers in Singapore. Below the findings concerning the purpose of this work are given:

i. Fire engineering
    a. Fire fighting systems should be decentralized and placed at different locations of the building;
    b. A separate fire-fighter staircase should be used;
ii. Means of escape
    a. Providing exit signs with lighting at floor level;
    b. Fire refuge floors and rescue floors are considered relevant, they have to be designed to be completely fire proof;
    c. Sky bridges should be used to allow faster evacuation, but objections regarding the possible collapse due to overload were raised. This is denied in (24). A second disadvantage may be the travel of smoke from a tower to another.
    d. Additional staircases in tall building should be provided specially in the lower floors where large number of occupants are expected;
    e. Rooftop helipads could increase the safety of tall buildings;
    f. The concept of fire-proof elevators would not enhance the safety. It can be useful to evacuate the handicapped, the elderly and children. The main disadvantages are:
        i. Supply of electricity;
        ii. Means of escape if lifts fail;
        iii. Large crowds waiting at lift lobbies may cause chaos during evacuation;
    g. Escape chutes are very fast and they would not take up much room.

4.2.5 Refuge zones

Refuge zones can be divided into two classes, refuge areas and refuge floors. Refuge areas, as described in (70), are small standing areas along a safe path which provide an opportunity for slow moving occupants to rest without slowing down the movement of others, preventing congestions, and they are requested every 3 storeys. Refuge floors, instead, are designated as fire-free and smoke-free floors, where occupants can be safely accommodated for a required time. Their aim is to provide enough safety for people who are located in the higher part of the building, because for them could be too dangerous escaping from the fire floor using the staircases. Refuge floors should be fully sprinklered and connected to both evacuation elevators and stair cores. They must be separated by 120 minutes fire resistance compartment. In addiction smoke extraction and fresh air inlet are requested (65) (24).

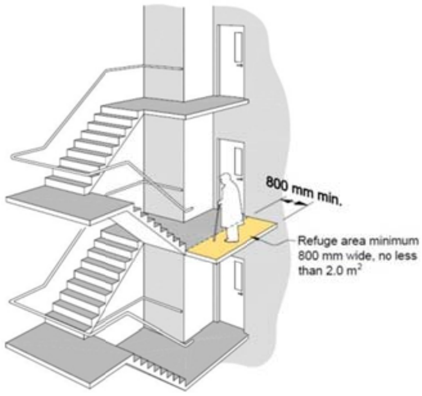


Figure 4-5: Refuge areas (70)

4.3 Summary

"Increased height brings extra risk, both in the time needed for escape and the difficulties posed to the fire service in attempting to assist evacuation, effect rescue or fight fires" (71).

Only some of fire protective measures treated in the literature have been addressed in this chapter. It is believed that they alone will provide an adequate level of fire safety and there is no need to investigate different solutions. Therefore, in this study it is assumed that the designed fictional building is furnished with the protective measures presented below.

- Fire-fighters elevators with fire protected lobbies equipped with dry risers are the main protection measure to ensure a quick and an effective fire fighter action;
- Sprinkler systems are very effective measures in the early stages of fires, they usually manage to extinguish the fire before it reaches flashover. Concerns arise about their reliability and about the tenability conditions during their action;
- Rooftop helipads improve the safety of the building because they allow fire-fighters enter the building from the top and most important they may rescue entrapped people if a fire breaks out above the higher sky garden;
- Refugee areas and fire protected lobbies must be provided to allow impaired people to wait for being rescued and to allow slow moving people to rest without slowing down other people;

The most important conclusion of this chapter is represented by the need to teach inhabitants must how to deal with fires. Indeed, it is strongly believed that without a proper fire safety education of inhabitants all the possible fire safety measures are useless (66). Skilled people may have the same effect of an in-site fire brigade allowing a reduction of fire load density.

5 Case studies

In this section the attention is draw on several projects which are of particular interest because they have some features that have been recognized vital for the design of a high-rise timber building. For each project a brief introduction is given, and then the main interesting aspects are then highlighted.

5.1 Island tower sky club, Fukuoka City, Japan

The Island Tower Sky Club is a super high-rise residential building constructed in Fukuoka City, Kyushu, Japan (Figure 5-1). The project is intended as a pioneering venture and a leading urban model by adopting inventively the most advanced building technology to create an incomparable building form (slender symbol tower) and an unsurpassed residential space (excellent lightning and gorgeous view) in an incomparable and sustainable “base-isolated 3-tower connected structure”. The structure is made high-strength reinforced concrete building and it is $145.3\ m$ tall with 42 stories above the ground. The building is conceived with three similar slender (1:7) towers, instead of one wide tower, to provide the apartments with excellent day lightning. Each tower has $20\ x\ 20\ m$ square floor plan and total floor area is $61296\ m^2$. Sprinklers are installed in all apartments and a couple of evacuation routes are secured through the sky gardens. The particularity of this building is represented by the presence of three aerial gardens which connect the three slender towers. The three sky gardens have the main function of improving the lateral and torsional stiffness of the building against seismic and wind load (72).

Sky gardens may play an important role in improving the safety of a high rise timber building. The sky gardens could be used by occupants of the tower “A” affected by fire, to flee crossing the sky garden to towers “B” and “C” which are assumed unexposed to fire. In this way people will reach in a shorter time a safe place, furthermore the width of staircases could be reduced because less people will have to walk down the complete height of the building, saving floor area for bigger apartments. In fact the width of the escaping routes is determined by the amount of occupants that have to use it (70). Last but not least, the sky gardens will allow fire brigade to perform an external attack to whichever height of the building standing on either, the gardens or the balconies of the adjacent tower.

Figure 5-1: Island City Sky Club

5.2 Petronas Twin Towers, Kuala Lumpur, Malaysia

The Petronas Twin Towers, with their height of $452\ m$, were the highest buildings in the 1996. They have been the tallest building in the World until 2004. The most important aspect which is interesting for this study is the sky-bridge between the two towers on the 41st and the 42nd floor. The bridge is $170\ m$ above the ground and $58\ m$ long, weighing $750\ tons$. The peculiarity of this building is that the skybridge acts as a safety device, in the event of a fire or other emergency in one tower, occupants can evacuate by crossing the skybridge to the other tower. A total evacuation triggered by a bomb hoax on September 12, 2001 showed that the bridge would not be useful if both towers need to be emptied simultaneously. Indeed the evacuation was a disaster, occupants from both towers tried to cross simultaneously the sky-bridge to reach the adjacent tower, they jammed inside the sky-bridge for several hours (24). Occupants from the lower part did not have any problem evacuating the building. Thus a review for the evacuation procedure was made, taking into account the unlikely event of simultaneously evacuation of both towers. In an evacuation drill, with the upgraded procedure which allows the use of shuttle lifts in the same tower, a total evacuation was reached in only 32 minutes (73).

Figure 5-2: Petronas Twin Towers

5.3 Cenni Street, Milan, Italy

A building complex composed by four 8-storeys timber buildings in Milan has already been designed. It is part of a social housing project. With a total height of $24.4\ m$ the building will become the highest timber building in Italy. According to Italian building decree the building is in class B (depending only on the height), which does not prescribe severe limitations. Only a load bearing structure with a fire resistance of $60\ min$ is requested. The fire protection is provided by gypsum encapsulation of timber elements. Due to the construction material, an improvement of fire safety measures has been requested by the fire department in order to provide the building permit. Such improvement includes hose reels on every floor, and fire resistant front doors (REI60). Stairwells are not designed to be smoke free and no fire fighting lift is used. No residential sprinklers and no automatic fire detection system were requested by the fire brigade. It is worth noting that there is another timber project going on in Milan, which envisages the construction of a high-rise building of 15 storeys, but unfortunately no detailed data has been founded at the time of writing this report.

This project is considered important because it stresses the importance of fire fighting devices as portable extinguisher or hose reels. Indeed, occupants must be taught how to act in case of fire in order to start fight the fire in the early stage. Unfortunately designers cannot only rely on skilled people.

5.4 Murray Grove, London, UK

The Stadthaus apartment building in Murray Grove is the current highest timber building in the World. It is made with solid timber panels used as walls and floors. Only the ground floor is made of cast in situ concrete, while the eight storeys above completely made out of CLT panels. External walls and central core provide all vertical load-bearing and lateral resistance. The tower is a cellular structure with apartments in a honeycomb pattern around a central core. The lift-core and stairwells are independent structures within the building and are isolated from surrounding core walls and perimeter which provide the overall lateral stability of the structure. In the figures below the timber element dimensions and the isolation detail can be established. Fire safety recommendations in UK for residential units prescribe that separations within flats should have half-an-hour's integrity, between flats one hour integrity and, between flats and principle vertical circulation two hours integrity.

A platform construction configuration was used, which means they set each floor on the walls below, and then another storey of walls was raised and so on up the top floor (Figure 6-4). This ensures a fast erection but could lead to side effects, such as crushing of perpendicular fibres. The eight-storey timber structure was assembled in eight weeks. The entire nine-storey structure was up in nine weeks.

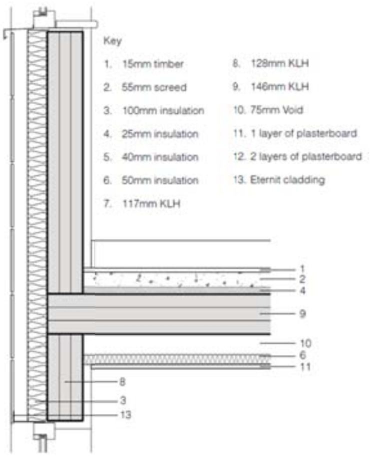


Figure 5-3: Element's dimensions in Murray Grove timber building

Figure 5-4: Murray Grove

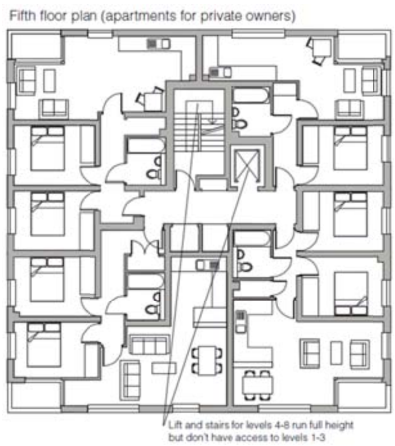


Figure 5-5: Floor layout for private owners

This project is interesting to understand how loads are transferred through the structure. In a CLT structure each wall has a load bearing function; the structural elements cannot be divided as for high-rise steel and concrete building, where the two materials have different structural functions. Designing with CLT is almost making a structure with shoeboxes, one on top of each other. Wherever possible, floor panels are designed to span in two directions or to cantilever if a support is removed. This will avoid disproportional collapse. For a fire safety point of view, no active measure was used; the designer used gypsum plasterboard layers to achieve $90\ min$ of fire resistance.

The engineering firm Techniker Ltd. which designed the Murray Grove building studied also the maximum height that can be reached by a CLT building (74). It is stated that if the platform construction method, currently being used, is unmodified then a straightforward point block reaches 15 storeys with economic wall thickness. If the bearing points are strengthened locally then two or three extra storeys might be added. A new joining method has been presented which allows having a continuum vertical load path without stress the horizontal panel in the perpendicular direction. The current materials now in use allow reaching 25 storeys high building without an inner concrete core, the P/Δ effect becomes important. In order to get higher, a concrete central core offers a conventional bracing provisions then the outer edge of the building can be opened out with simple vertical load bearing components.

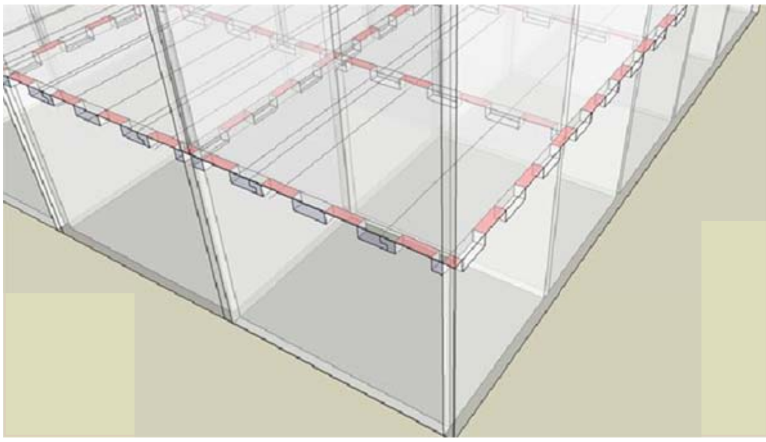


Figure 5-6: Techniker proposed connection

5.5 Limnologen, Växjö, Sweden

The Limnologen project in Växjö consists of four 8-storey residential buildings made of CLT panels, at the time of construction they were the tallest timber building in the World (75). As for Murray Grove, the bottom floor is made of concrete, mainly due to the increased self-weight. The concrete first floor facilitates the anchoring of the above storeys, in fact, in order to handle the lift-up as a result of wind loading, 48 tension rods have been mounted in every building. These tension rods are anchored in the concrete of the first floor, and extend all the way up to the top floor. In this way the force is transferred from the storeys down to the foundation. This design means that load transferring connectors between the wall elements are not needed. The tension rods must be re-tightened after some time due to relaxation in the steel, creep deformations in the woods and due to possible drying of the wood. The load bearing structure consists of CLT elements for both walls and floors. Traditional timber framed walls are used in separating walls (in between apartments). All exterior walls are parts of the load bearing system but some of the vertical loads are also taken by interior walls.

Figure 5-7: Floor layout of Limnologen project

Figure 5-8: Limnologen building complex (by Kirsi Jarnero)

The stabilising system consists of exterior walls, floors and apartment-separating walls. The horizontal loads are transferred by the floors, acting as stiff plates, to the top of the walls. In some parts of the buildings, glulam columns and beams have added to the load bearing system in order to reduce deformations.

Regarding fire safety design the complete buildings are equipped with residential sprinklers. This is not needed according to the Swedish legislation, but it has made it possible to use designs that would otherwise not been possible. An example is represented by the south façade which is made of entirely of wood, by the vertical distance between the windows on the north-west façade which has been minimised and by the wooden surface of the CLT-slabs of the balconies which are visible from below. These changings in design are possible since it can be shown that the total fire safety of the building sufficient. The Swedish requirements of the legislation on fire safety are independent of the material used in the load bearing structure. Since the buildings are of more than three storeys high they are classified in class BR1, a class having the highest requirements. The apartments are separate fire cells, and are according to the Swedish building code designed in class EI60, the only exception being a pram storage room on the first floor which is classified as EI30.

According to the general principles used in Sweden the presence of sprinkler system in Sweden can allow for relaxations in four alternatives[15]:

– Combustible façade cladding up to eight storeys;
– Decreased requirements on surface linings in apartments in multi-storey buildings, down to class D-s1,d0;
– Decreased requirements on fire spread through windows in the same building;
– Increased walking distance in escape routes.

The Limnologen project is believed to be interesting for two main aspects, the former is the complex geometry which can be realized with prefabricated timber panels, the latter is represented by the relaxations offered by the installed sprinkler system.

[15] Obtained by a private communication with Ms. Birgit Östman from SP Trätek, Wood Technology, Technical Research Institute of Sweden.

5.6 Dock Tower, feasibility study

A feasibility study for a tall timber building of 120 meter has been conduct for the project “Dock Tower”, showing that a mixed structure of concrete and Cross Laminated Timber panels can be built reaching high fire safety levels. The structure was designed with the central core and four staircase made of concrete, a concrete slab was adopted every three floors to prevent spread of fire through the building façade.

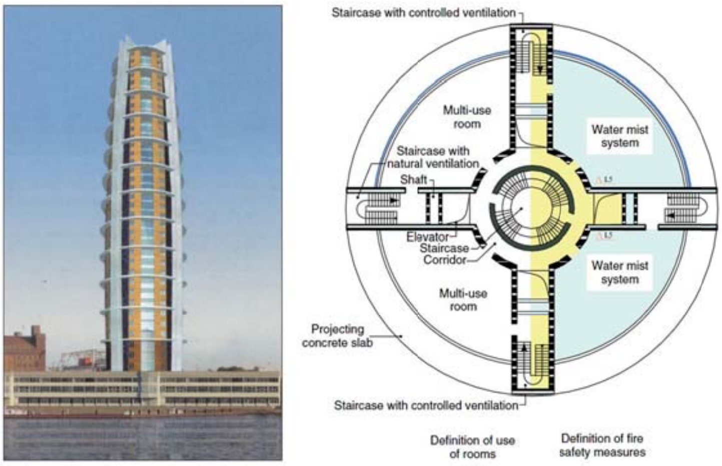


Figure 5-9: Technical feasibility of high-rise timber structure, Dock tower

5.7 Summary

Each building studied in this chapter has some particular features which will be used in the fictional building which will be design in the next section. First of all, for a timber high rise building it is believed being of paramount importance the three tower concept. Indeed, as already determined from the regulations review, the height of the building plays a major role in an even of a fire. The three towers concept allows a faster evacuation of the affected tower, and it will guarantee an external attack to the fire independently of the fire floor. Limitation on the external attack is the main disadvantage of high rise building, regardless the construction material. In fact it is believed that the fire-fighters standing either on the sky gardens or on the adjacent towers will be able to reach the fire with conventional fire apparatus (several deluge guns may be installed as well). It is important to note that an external attack has to be preferred instead of an internal attack. An internal attack to a fire is really dangerous and exhausting for fire fighters, who have to shift every $30\ min$ in order to rest and change oxygen cylinder. Moreover adopting the connected towers concept, fire fighter elevators may be unnecessary because occupants and fire service personnel could use the elevators in the un-threatened towers. Other advantages of the three towers concept are represented by reducing of drift at the top floors by 40%, as experienced in Island City Sky Club (72). Sky bridges could also reduce the P-Delta effect.

Other interesting protection measures founded in the case studies and applied to the fictional building are represented by fire resistance front doors, by fire extinguishers, and by hose reels. However these features rely on the inhabitant fire-fighting skill which is difficult to predict. Indeed it is believed that all the inhabitants should be taught how to behave in an event of a fire and how to start fighting a fire. Furthermore the building will be fully sprinklered. Sprinklers systems have a high redundancy which will reduce the risk of a fire. When they are used a relaxation may be accounted on others fire protective measure.

If the building design based on active protection measures will not accepted by the authority it is possible to use gypsum plasterboard to protect the load bearing structure. However the use of non-combustible linings has been discarded because of aesthetical reasons. Indeed, the main research question is focussed on the non-protected use of CLT panels.

6 Design of a high rise timber building

"Non-combustible does not mean the same as resistive. If non-combustible steel columns, girders, are not covered with fire retarding material, they cannot resist fire and can collapse quickly when they are heated by fire" (76).

A simplified structural design of a high rise timber building is carried out in order to answer the main research question. Although it has been shown in the previous chapter how it is important the three towers concept, in this chapter only one tower is studied. From a structural point of view the sky gardens and the two others towers are neglected. This analysis aims to determine the element sections needed in the so called "cold design" which will serve as a starting point to evaluate the fire resistance of the building. The cold design is designed according to the fundamental load combination given in Eurocode 1990. The building layout has to meet also the Dutch daylight entry requirements. When the cold design has been carried out, a fire design is performed according to prescriptive rules determined in the previous chapters. In this chapter, the load bearing structure will be checked, according to prescriptive methods, for a standard fire exposure of 120 min.

6.1 Fictional building

As already explained in the previous sections, the three towers concept is believed being the main feature which will allow the design of a high rise timber building providing an adequate fire safety level. Indeed it will provide more redundancy against an out-of-control fire and a faster escape. The possibility of carrying out an external attack to the fire is ensured by adjacent buildings and by the sky gardens, while the faster escape is ensured by connecting the buildings. The two sky gardens are located at the 10th and 20th floor respectively. In this study the sky gardens represent only a fire protective measure, therefore they are assumed having no structural function. Thus, the design of the sky gardens is not addressed. The fictional building is then represented by one tower only.

The first input data about the fictional building is the number of storeys needed; it has been set as 30 storeys. In order to determine the total height of the building the function of the building has to be known, and it is assumed being a residential building. The chosen function affects also the fire safety regulations. The height of the residential building is determined by the formula from the Council of Tall Buildings and Urban Habitat, and it is shown below.

$$H_{residential} = 3.1s + 7.75 + 1.55\frac{s}{30} = 102.5\,m \cong 105\,m \tag{1}$$

A total building height of 105 m is used. With the adjusted building height each floor is 3.5 m high. This value is higher than the average residential floor height in The Netherlands (up to 3.1 m). However the former value is used in the further analysis because it is believed that it will allow more freedom in the floor layout, indeed it will be able to accommodate a floor slab with deep beams to increase the floor span (Figure 6-10). The regulations which have been taken into account pre-dimensioning the building are based on the amount of daylight entry. Two requirements have to be met:

- Opening ratio of façade 30%;
- Window's area over floor area ($\geq 10\%$).

A simplified analysis is evaluated to estimate the minimum thickness required to support the building. It is assumed that the wind load is taken by the effective area of the outer walls, which will be loaded in tension and compression only. Only a small part of the outer walls has to carry the load from the floor slabs. It is reasonable to think that the floor load introduced in the section will be spread to the all effective cross section, reducing the maximum stress at the bottom wall. However, this analysis neglects redistribution of stresses in the section. One section only of the internal walls is checked, it is section 3, this section is the most loaded because has the biggest supported floor area. The inner walls are assumed providing vertical load bearing function only. If the horizontal deflection will exceed the maximum allowable value a more accurate analysis should be performed taking into account all the structural members and their connections.

6.1.1 Structural system

For steel and concrete high-rise building the slenderness ratio used is around 1:8, but for timber buildings the ratio of 1:4 should be preferred due to different material properties. Using the latter slenderness ratio, a stabilizing structure of 26 m wide is needed. A central core with the former dimensions is not considered to be economic, therefore it is used a stability system composed by external walls and the inner core (tube in tube structure). The external walls have large openings to meet the day light entrance requirements. The structure is depicted in Figure 6-1 and it has the following dimensions:

- Total height of the structure 105 meters (30 storeys x 3.5 m)
- External wall 26 m wide, square floor plan;
- Inner core of 8 by 8 meters, centred on the floor.

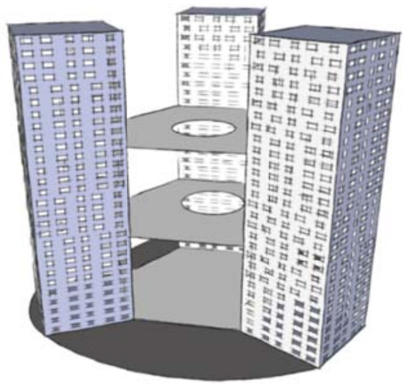


Figure 6-1: Three building concept

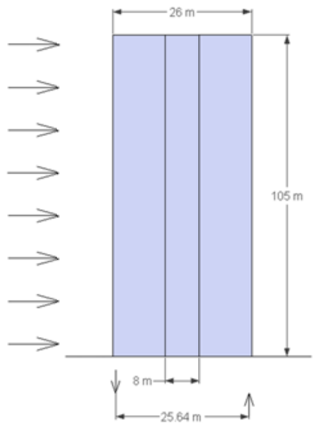


Figure 6-2: Side view of the building with wind load and restoring forces

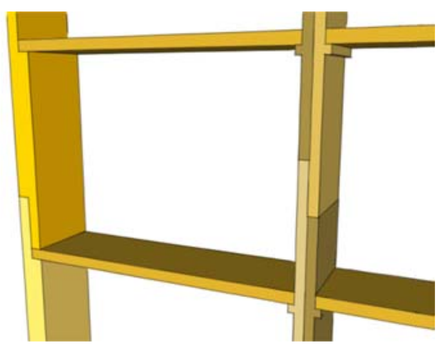


Figure 6-3: Connection proposed

The tube in tube structural system uses an external tube element which handle the vertical forced due to gravity and wind load (the vertical wind forces are those arose from the overturning wind moment, Figure 6-2), combined with a massif central core which provides stiffness against horizontal loads. Therefore the inner core is assumed carrying all the shear stresses plus part of the vertical load, while the external tube (the façade) is assumed carrying the vertical wind forces and part of the vertical load. The floor slabs and the inner partitions are assumed to be stiff enough to transfer the wind load from the windward side to the leeward side.

Due to the wind load on the façade, a lift-up of the external wall could happen if the structure is not heavy enough to balance those forces. If this will be the case, additional hold-down fixings may be needed. However, since for the weight analysis the dead load of the sky gardens is neglected, could be expected that they will improve the restoring moment. Studying a single tower, an uplift force is occurring at the base of the building and hold down provisions are needed. To carry the vertical tensile forces steel bars may be integrated in the panels, but they are expensive and they need to be adjusted during the life time of the structure due to creep and shrinkage. Due to the required thickness of the walls, each wall may be composed by two panels and overlapped to allow tensile forces to carried (Figure 6-3).

A second problem should arise if a non-symmetrical inner core is used. Unfortunately this is the case because the inner core has to provide openings for the residential units. This will lead to torsional issues of the building because the shear centre of the structure will shift from the centre of gravity. This behaviour has been neglected because it is believed that the three tower concept is able to improve the torsional behaviour of each building. However a more accurate analysis on the torsional issue is needed. However in this study, a single building is analysed neglecting the torsional behaviour.

6.1.2 Structural details

As already treated in (77), the main governing design aspects regarding CLT structure are represented by:

- Horizontal drift of top storeys;
- Bearing resistance of floors panels, crushing of perpendicular fibres;
- Fire safety endurance.

In the present study only the last point is deeply analysed, however following the recommendations obtained from literature it is believed that the first two items of the list will be ensured by the following design consideration:

- The structure will be stabilized by the outer walls together with the central core, inner walls and separating walls;
- If the shear forces at ground level will result too high, the bottom storeys could be made out of concrete. This will allow to handle large forces due to the vertical loads (as wind and gravity loads), and due to the horizontal load (shear force due to wind load on façade);
- There are two principal methods to connect floor slabs to wall panels in CLT structures: the platform frame and the balloon solutions. The former is used to speed up the construction (Figure 6-4); the floor slab is placed directly on top of the wall panel below. However the big disadvantage of this method is represented by the high compression forces perpendicular to the grain in the bearing area of the floor slab. The latter, Figure 6-5, is adopted to avoid high compression forces perpendicular to the grain in the floor slab. In this way, the floor slab lies on an edge beam called corbel, which can be already fixed on shop. This construction method should lead to slower construction speeds, but it is believed that is the only option to reach the building height. A new developed connection method has been discovered by Techniker firm, they proposed the connection given in Figure 5-6 which allow a fast erection, avoiding the perpendicular crushing of fibres (74).

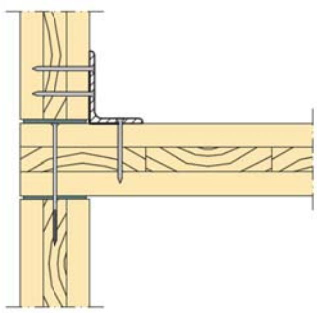


Figure 6-4: Platform frame connection from (77)

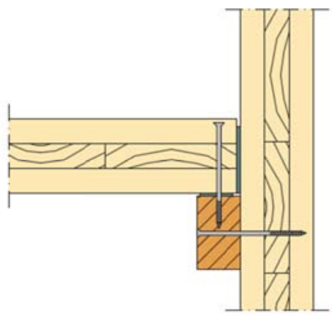


Figure 6-5: Balloon frame connection, from (77)

6.1.3 Floor plan

Differently from steel and concrete buildings, for CLT buildings the arrangement of the inner walls has the same importance as the stabilizing structure. In CLT structures the imposed loads are carried by all the walls according to the supported floor area. The arrangement of inner walls and separating walls has to meet the daylight entry requirements for flats. After several iterations it has been chosen the floor layout depicted in Figure 6-6. It is assumed that all the 30 storeys have the same floor layout.

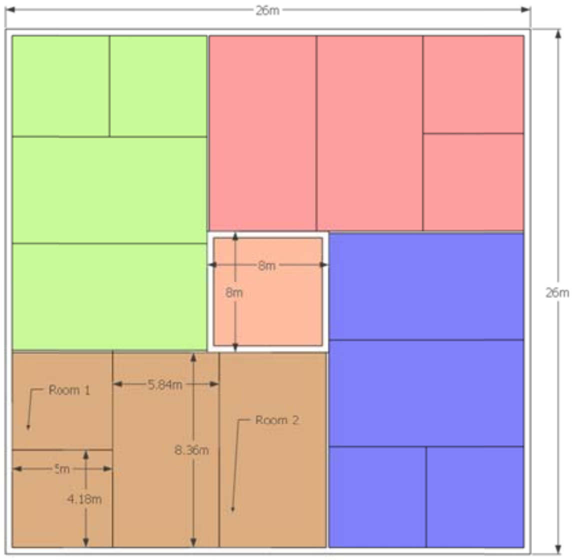


Figure 6-6: Concept building floor layout

Because the floor plan used is not symmetrical, torsional effects may affect the building but they are neglected in this work. However they can be solved mirroring the floor plan each storey, but then problems due to the vertical load distributions arise (due to the discontinuity of the main load bearing structure). The torsional effect of each building could be reduced by connecting the towers together.

The main disadvantage of my preliminary floor layout is the difficulty to achieve a good resistance against disproportional collapse; this is due to the simplicity of the design. A good detailing of the elements can improve the disproportional collapse problem. Further analyses to avoid disproportional collapse are needed.

6.1.3.1 Daylight entry

In this section the floor layout described above is checked regarding daylight availability. Each apartment is composed by two couple of rooms, two small and two big (Figure 6-6).

1st Room - small

$b_{window} = 2.5\ m$	$b_{floor} = 5\ m$	$b_{facade} = 4.18\ m$
$h_{window} = 2.1\ m$	$h_{floor} = 4.18\ m$	$h_{facade} = 3.5\ m$
$A_{window} = 5.25\ m$	$A_{floor} = 20.9\ m$	$A_{facade} = 5.14.63\ m$
	$\frac{A_{window}}{A_{floor}} = 25\%$	$\frac{A_{window}}{A_{facade}} = 35\%$

2nd Room - big

$b_{window} = 2 \cdot 2m$	$b_{floor} = 5.84\ m$	$b_{facade} = 4.18\ m$
$h_{window} = 2.1\ m$	$h_{floor} = 8.36\ m$	$h_{facade} = 3.5\ m$
$A_{window} = 8.4\ m$	$A_{floor} = 48.82\ m$	$A_{facade} = 5.14.63\ m$
	$\frac{A_{window}}{A_{floor}} = 17\%$	$\frac{A_{window}}{A_{facade}} = 41\%$

If the amount of daylight entry should not suffice, higher windows may be used without compromising the effective width of the external wall. The effective width of the external wall is 50% of the total building width ($b_{building,eff} = 26 \cdot 50\% = 13\ m$). Increasing the height of windows may reduce the bending stiffness of the side walls, however the contribution of the side walls to the stability was neglected from the calculations. In particular there are no floor slabs supported by a wall element which has a window opening, this will avoid stresses too high in the wall effective section.

A different floor layout can be chosen to allow a more freedom in designing the inner area of the building, glulam beams can be used spanning from the exterior wall to the central core as it is done in steel-concrete high rise buildings. However the latter design may be interesting for an office building which prefers large open space.

6.1.3.2 Interesting sections

Proved that the opening factor used meets the daylight regulations, the next step is to assume how the structure carries the loads. In the simplified analysis, the external façade is assumed carrying all the loads due to the wind load, while the inner central core is carrying only the shear forces due to wind action. The loads applied to the floor (self-weight and imposed load) are taken by the vertical load bearing structure according to the supported floor area. In this arrangement only a small section of external wall has to carry also the loads due to the floors. All the loads applied to the floor are taken by the inner walls, apart section 1 (depicted in Figure 6-8).

The contribution to the overall building stability of the inner walls and the external walls parallel to the direction of the wind load are not taken into account in the simplified analysis. Also locally peak stresses in the stiffest part of the building, known as shear lag effect, is neglected.

Sections	***Actions***	***Supported floor width*** $[m]$	***Floor area*** $[m^2]$	***wall section*** $[mm\ x\ mm]$	
I	Self-weight Wind Imposed loads	2.09	10.45	$5000\ x\ 360$	Critical
II	Self-weight Wind	–	–	$920\ x\ 360$	
III	Self-weight Imposed loads	5.84	48.83	$8360\ x\ 360$	Critical
IV	Self-weight Imposed loads	2.09	10.45	$5000\ x\ 360$	
V	Self-weight Iposed loads	4.18	20.9	$5000\ x\ 360$	
VI	Self-weight Wind force (tension)	4.5	96.75	$26000\ x\ 360$	Critical

Table 17: Interesting sections

The sections named in Table 17 can be seen in the Figure 6-7 and Figure 6-8.

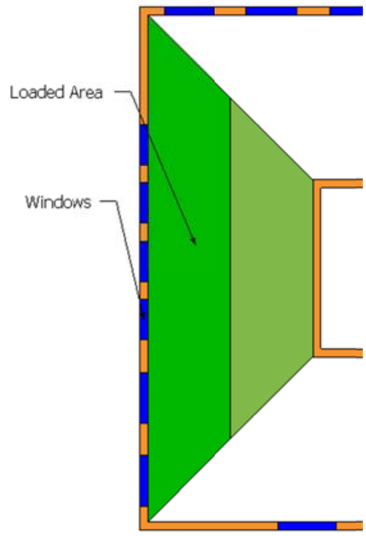


Figure 6-7: Section 6

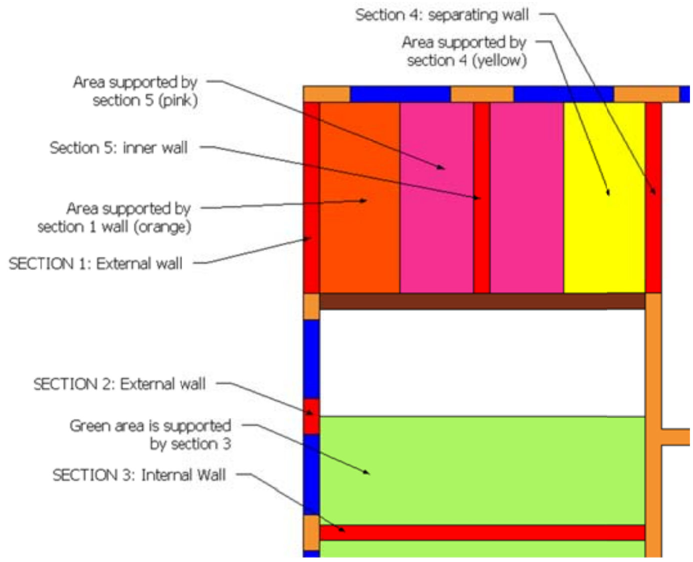


Figure 6-8: Interesting sections from 1 to 5

6.1.4 Loads

The following load analysis is carried out to determine loads which are acting on each interesting section. The following loads are considered:

- Dead load;
- Imposed load;
- Wind load.

Second order effects are not computed directly, but they are introduced by a magnification factor (value of 1.3) in the load combination analysis. This magnification factor take into account the $P/Delta$ effect, differential settlement of foundations and the dynamic wind load.

6.1.4.1 Dead Load

Dead loads of the structural elements are derived by the technical drawings founded in dataholtz.com. The floor thickness was derived by a previous thesis project in accordance with design chart by Finnforest for a simply supported beam with a span of 6 m and an imposed load of $2\ kN/m^2$ (77). The wall thickness was derived by several iterations, here the definitive thickness of 360 mm is used. For the purpose of this work the roof load is assumed to be the same as the floor load. Where the CLT floor slabs span parallel to external walls, it is assumed that these walls carry no floor loads, the floors are assumed one-way spanning.

6.1.4.1.1 Floor

The protected floor construction is assumed composed by the following elements.

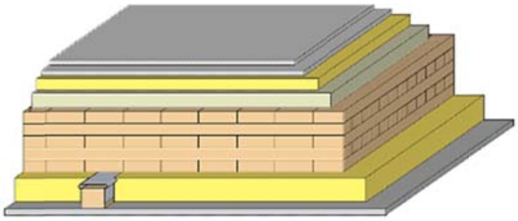


Figure 6-9: Floor slab section

[mm]	*From top to bottom:*
25	Dry screed;
30	Impact sound absorbing subflooring;
40	Fill
-	Trickling protection
186	Solid wood panel
50	Glass wool
2x12.5	gypsum plasterboards

With this configuration the dead load of the floor is $g_k = 2.128\ kN/m^2$. The loads due to the wood battens are neglected from the calculations. However a different floor layout may be used to improve vibrations resistance or to increase the span length inside a flat. Floor slab width deep beams could be used, this arrangement has been used in the Limnologen project and it is depicted in the next picture. Glulam beams may be used as well.

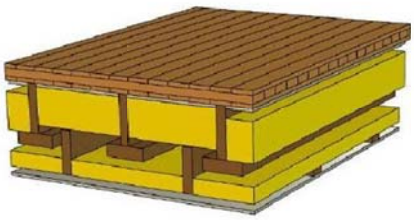


Figure 6-10: Limnologen floor element, please note the floor slab with the T-beams

6.1.4.1.2 External walls

In order to have a better estimation of the total dead load of the building, it is assumed that the thickness is changing linearly with the height. The thickness of the CLT panel in the walls is reduced with the height of the building. As preliminary calculation, bottom thicknesses of $t_{ext,bott} = 360\ mm$ and a top thickness of $t_{ext,top} = 220\ mm$ are assumed. The top thickness is not checked in this work, however it is believed being widely enough to handle the loads on top. Standard wall elements are made up with $95\ mm$ of CLT elements.

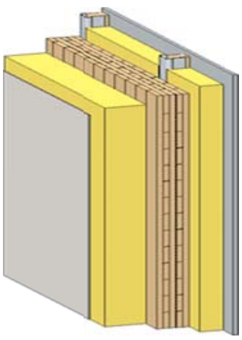


Figure 6-11: External wall layout

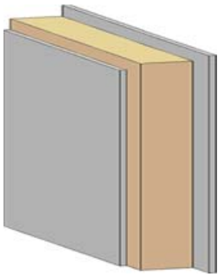
[mm]	*From outside (left) to inside (right):*
15	Plaster
100	Multilayer wood wool composite board plaster substitute
360*	Solid wood panel
50	Mineral wool
2x12.5	Gypsum fiber board high temp

*: the thickness of CLT panel is reduced up to $220\ mm$ at top floor

With this configuration the dead load of the bottom wall is $g_{k,bot,ext} = 2.344\ kN/m^2$, while the weight at the top is $g_{k,top,ext} = 1.644\ kN/m^2$. The averaged value is $g_{k,ext} = 1.994\ kN/m^2$.

6.1.4.1.3 Inner wall

Inner walls are subjected to a fire safety resistance of at least 60 min in order to prevent spread of fire across the apartment. However, since they are part of the main load bearing structure they have to resist for 120 min. The thickness of inner walls is reducing linearly with the height of the building up to $t_{int,top} = 220\ mm$.



[mm]	***From outside to inside:***
2x12.5	Gypsum fiber board high temp
360*	Solid wood panel
2x12.5	Gypsum fiber board high temp

*: the thickness of CLT panel is reduced up to 220 mm at top floor

Figure 6-12: Inner wall layout

With this configuration the dead load of the bottom wall is $g_{k,bot,int} = 2.250\ kN/m^2$, while the weight at the top is $g_{k,top,int} = 1.550\ kN/m^2$. The averaged value is $g_{k,int} = 1.900\ kN/m^2$.

6.1.4.2 Variable actions

Variable actions are defined as: “actions that do not remain monotonic and may vary with time”, and they are represented by imposed loading, wind, snow, and thermal loading.

For the purpose of the analysis, only wind and imposed load are taken into account, and they are discussed below.

6.1.4.2.1 Imposed variable load

Imposed loads on buildings are those arising from occupancy inside the apartments and therefore are distributed on the entire floor (they are given in EN1991-1-1). They are represented by:

– Normal use by persons;
– Furniture and moveable objects;
– Vehicles;
– Expecting rare events, such as concentrations of persons or of furniture, or the moving or stacking of objects which may occur during reorganization or redecoration.

In the next table the values given in the Eurocode are presented.

Category	Specific Use		Dutch National Annex $q_k\ [kN/m^2]$	Dutch National Annex $Q_k [kN]$
A	Areas for domestic and residential activities	floors	1.75	3.0
		stairs	2.0	3.0
		balconies	2.5	3.0
		corridors	2.0	3.0
B	Office areas	floors	2.5	3.0
		corridors	3.0	3.0

Table 18: Imposed variable loads on buildings
(EN1991-1-1 table 6.1, table 6.2 and National Annex)

Imposed load acting on a wall are computed according to the supported floor area according to Figure 6-8. Since the uniform distributed load produces the greater load the point load has been disregarded.

6.1.4.2.2 Wind load

Wind load on skyscrapers is very important since it can lead to axial tension in the load bearing structure, thus a special provisions to prevent up-lift of wall elements is needed (as seen in Limnologen project and (78)). There is an entire Eurocode which deals with wind load on structure, but it seems not reasonable to spend time for computing a perfect wind load value. A straightforward shortcut is then used, the wind pressure value it is assumed from a former thesis project (79). This is possible because the buildings have almost the same dimensions. This value is higher than wind load founded in the Dutch national annex but it is in agreement with the value proposed in (78). According to §6.1.1, the external walls are assumed carrying the vertical forces due to the bending moment, while the central core will carry the shear force. The wind pressure is dependent on the height of the building, the higher the stronger the wind pressure. In the next picture are shown the equivalent wind pressure used in this study id given (red line) together with the wind pressure determined in (79) depicted in blue, and with the wind pressure adjusted for the concept building height (green line).

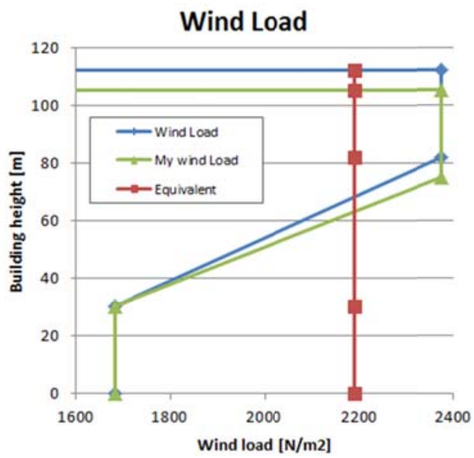


Figure 6-13: Wind pressure calculation

The wind pressure values depicted in Figure 6-13 are given in Table 19.

From 0 to 30 m	1682 $N/m2$
From 30 to 75 m	Linearly changing from 1682 $N/m2$ to 2374 $N/m2$
From 75 to 105 m	2374 $N/m2$

Table 19: Wind pressure values

The equivalent wind load is computed as the equally distributed load which generates the same bending moment of the real wind load. Its value is given here (characteristic):

$$q_{eq} = 2.19\ kN/m^2 \tag{2}$$

Total shear force acting at the base of the building:

$$V = q_{eq} \cdot L \cdot H_{building} \cong 5979\ kN \tag{3}$$

Total bending moment acting at the base of the building:

$$M = \frac{(q_{eq} \cdot L) \cdot H_{building}{}^{2}}{2} \cong 313882\ kNm \tag{4}$$

The overturning moment generated by the wind pressure has to be resisted by the compression of the leeward external wall, and by tension on the windward external wall. The absolute value of the vertical force in the external walls:

$$F_{vert,ext,walls} = \frac{M}{L_{building} - t_{ext,walls}} = \frac{313882}{26 - 0.36} \cong 12242\ kN \tag{5}$$

The $F_{vert,ext,walls}$ is assumed on 26 m of wall, but the effective section has to be reduced to account the openings for window, therefore the effective width which can be used to handle the force is reduced to:

$$\mathrm{A}_{eff,ext,wall} = 26 \cdot 50\% = 13\ m \tag{6}$$

The stresses in the effective cross section are derived by the ratio between (5) and (6), however it is just an approximation since local peak stresses could be present due to the openings.

$$F_{wind,ext,wall} = \frac{12242\ kN}{13\ m} = 942\ \mathrm{k}N/m' \tag{7}$$

The force (7) is computed at the bottom of the building.

6.1.4.3 Loads combination

For each design the governing load combination is determined according to Eurocodes. In each section (§6.3.1 and §6.4.1) the combination of actions for the fundamental combination (cold design) and for accidental design situation are computed accordingly with formulas 6.10 and 6.11 of EN 1990:2002 respectively.

The wind load for both the analysis is multiplied by a magnification factor of 1.3 which accounts second order effects and the dynamic wind load.

Redistribution of forces in the wall section is neglected, therefore the force is assumed remain in the critical section, this will lead to an overestimation of the stresses in the interesting section.

For the calculation of Vertical uplift force is considered applied to the all façade, therefore a different simplified structure is assumed, where all the load are taken by the façade and inner core, neglecting the contribution of inner partitions. In this way the maximum uplift force in the external wall element can be computed (Figure 6-7).

6.1.4.4 Load summary

In this section the representative values of loads are summarized.

Imposed distributed variable load	$q_k = 1.75\ kN/m^2$
Imposed variable point load	$Q_k = 3\ kN$
Vertical wind force on external wall	$F_{vert,ext,walls} = \pm 12242\ kN$
Floor dead weight	$g_{k,floor} = 2.13\ kN/m^2$
Average dead weight of external wall	$g_{k,ext,wall} = 2.00\ kN/m^2$
Average dead weight of internal wall	$g_{k,int,wall} = 1.90\ kN/m^2$

Table 20: Review on representative values acting

6.2 CLT design

6.2.1 Mechanically jointed beams

The CLT panel is a non-homogeneous material and the different boards orientation plays an important role, in fact when the different layers of the CLT panels are divided from the neutral axis by transverse lamellas, an effective inertia of the panel has to be computed. In literature are present two methods for evaluating the effective inertia of the CLT panel.

The first method, the composite theory (after Blass), it is a simplified method which can be used for span to depth ratios of at least 30 because it neglects the effect of shear deformations. It uses the gross section properties multiplied by a factor k that takes into account the different laminate build-ups of the panels. It is a conservative method.

The second method, used in the Eurocodes and adopted in this study, is based on the theory of mechanically jointed beams. This method is founded to be more accurate because it takes into account the effect of shear deformation of perpendicular laminate layers. It can be applied for whichever span to depth ratio. This method is more cumbersome because for each section the effective bending stiffness of the layers oriented parallel to the panel grain has to be computed. The shear deformation of cross layers is taken into account using a reduction factor γ (flexibility factor) which reduces the effective second moment of area of the longitudinal layers separated from the neutral axis by the cross layers. The expression s_i/K_i in formula B.5 of EN1995-1-1 is replaced by $t_{\bar{\imath}}/(G_{R,mean} \cdot b)$. It is shown in the formula below.

$$\gamma = \left(1 + \frac{\pi^2 E_i \cdot A_i \cdot t_{\bar{\imath}}}{l^2 \cdot G_{R,mean} \cdot b}\right)^{-1} \tag{8}$$

Where:

$t_{\bar{\imath}}$	Is thickness of the cross layers between the parallel layer i and the neutral axis
l	Is the decisive width between supports, assumed as the floor height 3500 mm
b	Width of the panel
E_i	Is the $E_{0,mean}$
A_i	Area of the parallel layer i
$G_{R,mean}$	Rolling shear value

It should be noted that the effective moments of inertia I_{eff} depends on the width between supports of the panels. The shorter the width between supports, the greater the proportion of shear deformations. Thus the percentage reduction of the moments of inertia will increase reducing the span.

6.2.2 Material strength

CLT panels are made out of C24 timber boards, the mechanical properties are determined according to European technical approval by the producer. The material properties given in Table 21 are derived from KLH product sheet, and from EN 338.

Load applied perpendicular to facing grain (floor slab)	
Modulus of elasticity	
Parallel to the direction of the panel grain $E_{0,mean}$	$12000\ MPa$
Parallel to the direction of the panel grain $E_{0,05}$	$7400\ MPa$
Perpendicular to the direction of the panel grain $E_{90,mean}$	$370\ MPa$
Shear modulus	
Parallel to the direction of the panel grain G_{mean}	$690\ MPa$
Perpendicular to the direction of the panel grain, roll shear module $G_{R,mean}$	$50\ MPa$
Bending strength	
Parallel to the direction of the panel grain $f_{m,k}$	$24\ MPa$
Tensile strength	
perpendicular to the direction of the panel grain $f_{t,90,k}$	$0.12\ MPa$
Compressive strength	
perpendicular to the direction of the panel grain $f_{c,90,k}$	$2.7\ MPa$
Shear strength	
Parallel to the direction of the panel grain $f_{v,k}$	$2.7\ MPa$
Perpendicular to the direction of the panel grain, roll shear module $f_{R,v,k}$	$1.5\ MPa$
Load applied in the plane of facing grain (wall panel)	
Modulus of elasticity	
Parallel to the direction of the panel grain $E_{0,mean}$	$12000\ MPa$
Shear modulus	
Parallel to the direction of the panel grain G_{mean}	$250\ MPa$
Bending strength	
Parallel to the direction of the panel grain $f_{m,k}$	$23\ MPa$
Tensile strength	
Parallel to the direction of the panel grain $f_{t,0,k}$	$16.5\ MPa$
Compressive strength	
Parallel to the direction of the panel grain $f_{c,0,k}$	$24\ MPa$
Concentrated, parallel to the direction of the panel grain $f_{c,0,k}$	$30\ MPa$
Shear strength	
Parallel to the direction of the panel grain $f_{v,k}$ (from EN 338)	$4\ MPa$
Parallel to the direction of the panel grain $f_{v,k}$	$5.2\ MPa$

Table 21: Mechanical properties of CLT panel

6.3 Cold design

The normal situation, the so called cold design, at normal temperature, is the basis for the fire design. Actually the fire design should be accounted in the cold design from the beginning, in order to avoid a trial and error procedure. In this work the cold design is needed to provide a minimum thickness of the structural elements which will ensure a $120\ min$ resistance time according to the Dutch building decree (§2.2.1.1).

6.3.1 Load combination

The normal situation, the so called cold design, at normal temperature, deals with different safety factors from the accidental design in order to provide proper safety. The following formula is given in EN1990 (6.10a) and it represents the combination of action for a persistent design situations (or fundamental combination) .

$$\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{9}$$

Where:

$G_{k,j}$: Design value for permanent action
$Q_{k,1}$: Design value for leading variable action
$Q_{k,i}$: Design value for accompanying variable action
$\gamma_{G,j}$: Partial factor for permanent action $j = 1.35$
$\gamma_{Q,i}$: Partial factor for variable action $i = 1.50$
ψ_0: Factor for combination value of a variable action $= 0.70$
ψ_1: Factor for frequent value of a variable action $= 0.50$
ψ_2: Factor for quasi-permanent value of a variable action $= 0.30$

For the cold design two different situations has to be checked compression and tension. The former analysis is carried out to respect to the actual layout of the building. Each wall is loaded by the supported area on top and it transfers this force to the wall elements beneath without any redistribution of stresses with adjacent elements. This assumption will lead to an overestimation of the compression force in the elements at the bottom, therefore it is used for compression analysis only.

For tension, a simplified structure is assumed and it is depicted in the next picture (Figure 6-7). In this analysis only the dead load and the wind force are taken into account, the dead load is favourable ($\gamma_{G,j} = 0.9$). For the tension analysis all the wall is assumed mobilized (50% of façade width). The governing load combinations are given in Table 22.

	Actions	Section	
LC1	Self-weight Wind Imposed loads		Maximize the compression force in the external wall. Section 1, $5.00\ m$ wide Leading variable action: wind action Short term loading, $k_{mod,short} = 0.9$ $E_{d,1} = 1.35\ G_k + 1.5\ Q_{k,wind} + 1.5 \cdot 0.7\ Q_{k,imp.load} = 12731\ kN$
LC2	Self-weight Imposed loads	III	Maximize the compression force in the internal wall. Section 3, $8.36\ m$ wide Short term loading, $k_{mod,med} = 0.8$ $E_{d,2} = 1.35\ G_k + 1.5\ Q_{k,imp} = 10456\ kN$
LC3	Self-weight Wind force (tension)	VI	Maximize the tension force in the external wall. Short term loading, $k_{mod,short} = 0.9$ $E_{d,3} = 0.9\ G_k - 1.5\ Q_{k,wind} = -9360\ kN$

Table 22: Governing load combinations

6.3.2 Cold strength

The characteristic values of the mechanical properties have to be reduced by the safety factor and the by k modification factor in order to obtain the design values for the cold design situation.

$$f_{d,short} = k_{mod,short} \frac{f_k}{\gamma_M} = \frac{0.9 \cdot f_k}{1,25} = 0.72\ f_k \tag{10}$$

The design strengths used in the cold design are:

$$f_{m,d,short} = f_{c,0,d,short} = 0.72 \cdot 24 = 17.28\ MPa \tag{11}$$

And:

$$f_{c,0,d,med} = k_{mod,med} \frac{f_{c,0,k}}{\gamma_M} = \frac{0.8 \cdot f_{c,0,k}}{1,25} = 0.64 \cdot 24 = 15.36\ MPa \tag{12}$$

The design shear strength for short term loading is:

$$f_{v,d,short} = k_{mod,short} \frac{f_{v,k}}{\gamma_M} = \frac{0.9 \cdot 4}{1,25} = 2.88\ MPa \tag{13}$$

The tensile strength used in LC3 is:

$$f_{t,0,d,short} = k_{mod,short} \frac{f_{t,0,k}}{\gamma_M} = \frac{0.9 \cdot 16.5}{1,25} = 11.88\ MPa \tag{14}$$

6.3.3 Section properties

Due to the inhomogeneity of the CLT panel, the effective values for the area, the inertia, the radius of gyration have to be computed related to the effective section of the panel. The panel used here is not given in the table sheet from the producer due to its large depth, however it will not be a problem to produce this panel thickness because depth up to $500\ mm$ are feasible upon request. The section of the panel used is

$360\ mm$ depth and it is depicted in Figure 6-14. The layers go from 1 (top) to 9 (bottom) and they are $40\ mm$ each.

Figure 6-14: CLT 360 mm section, 9 layers 40 mm each

The gamma factor for the transverse layers is (formula (8)):

$$\gamma_{CLT,360} = \left(1 + \frac{\pi^2 \cdot 11000 \cdot 40000 \cdot 40}{3500^2 \cdot 50 \cdot 1000}\right)^{-1} = 0.7790 \tag{15}$$

The effective inertia of the panel is computed by the following formula, where $\gamma_{1,2,8,9} = \gamma_{CLT,360}$ and $\gamma_{4,5,6} = 1$. The third and seventh layers, which are transverse, are omitted.

$$\mathrm{I}_{ef} = \sum_{i=1}^{9} I_i + \gamma_i A_i z_i = 2.66 \cdot 10^9\ mm^4 \tag{16}$$

In Table 23, all the section properties used in the further cold analysis are given.

Panel total thickness	$t_{panel} = 360\ mm$
Parallel thickness	$t_{netto} = 280\ mm$
Neutral axis (from top)	$y = 180\ mm$
Inertia effective	$\mathrm{I}_{ef} = 2.66 \cdot 10^9\ mm^4/m'$
Section modulus	$\mathrm{W}_{ef} = 1.47 \cdot 10^7\ mm^3/m'$
Radius of gyration	$\rho = 97.43\ mm$
E modulus panel	$E_{0,panel} = 7520\ MPa$
G modulus panel	$G_{panel} = 536\ MPa$

Table 23: Cold geometrical properties

6.3.4 Buckling reduction

The slenderness must be taken into account according to §6.3.2 of EN 1995-1-1-:2004. In the cold design situation, the floor slabs are assumed to be infinitely stiff respect to the wall element, therefore the sway mechanism of the outer wall will be prevented by the system composed by the central core with the floor slabs. With these assumptions, the wall section is treated with both ends pinned; the effective length is, in turn, equal to the system length.

The flexural bucking has to be checked with regard to the following governing equation (6.23 of Eurocode 5) where only the first two terms are present (only in plane bending):

$$\frac{\sigma_{c,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{17}$$

Sstem length	$h_{floor} = 3500\ mm$
Effective length	$l_{ef} = 3500\ mm$
Inerti effective	$\mathrm{I}_{ef} = 2.61 \cdot 10^9\ mm^4/m'$
Radius of gyration	$\rho = 96.53\ mm$
Slenderness ratio	$\lambda_y = 36.25$
Buckling reduction	$\boldsymbol{k_{c,y} = 0.94\ MPa}$

Table 24: Buckling reduction factor – cold design

6.3.5 Ultimate limit stage

6.3.5.1 Section 1 – LC1

According to the Eurocode, the external walls have to be checked for the simultaneously presence of compression and bending (formula 6.23 of EN 1995-1-1-:2004). The external wall between two floors is assumed as a simply supported beam of one span only (this assumption will lead to highest bending stresses in the wall section), and the uniform distributed design load is:

$$q_{wind,m,d} = 2.19\ kN/m^2 \cdot 1.5 \cdot 26\ m\ \cdot 1.3 = 111\ kN/m \tag{18}$$

Where the values 1.5 and 1.3 are safety factor and magnification factor respectively. A magnification factor is used to account second order effects and dynamic wind loading which have been neglected by the load analysis. The stresses on the effective wall section are:

$$\sigma_{m,y,d}{=}\frac{q_{wind,m,d} \cdot {h_{floor}}^2}{8} \cdot \frac{1}{W} = \frac{111 \cdot 3500^2}{8} \cdot \frac{1}{1.47 \cdot 10^7 \cdot 5} = 2.31\ N/mm^2 \tag{19}$$

The wall element of section 1 has the following vertical force (calculated in §6.3.1) and effective area:

$$\begin{aligned} E_{d,1} &= 12731\ kN \\ A_{effective,1} &= 1.4 \cdot 10^6\ mm^2 \end{aligned} \tag{20}$$

The cold unity check is satisfied, as can be see below:

$$\frac{9.09}{0.94 \cdot 17.28} + \frac{2.31}{17.28} = 0.56 + 0.13 = 0.69 \leq 1 \tag{21}$$

6.3.5.2 Section 3 – LC2

Here the leading variable load is represented by the imposed load; therefore the k factor is 0.8.

$$E_{d,2} = 10456\ kN \tag{22}$$

The section 2 is characterized by:

$$A_{effective,2} = 23.40 \cdot 10^5\ mm^2 \tag{23}$$

The cold unity check is satisfied, as can be see below:

$$\frac{4.47}{0.94 \cdot 15.36} = 0.31 \leq 1 \tag{24}$$

6.3.5.3 Section 6 – LC3

A simplified building model has been assumed to compute the maximum tensile force occurring in the windward side. The maximum tensile force in the external wall is $E_{d,3} = 9360\ kN$. This is applied to the netto area of the external wall, which is represented by the vertical layers without the window openings.

$$A_{netto,ext,wall} = (26 \cdot 50\% - 0.36) \cdot 0.28 = 3.539 \cdot 10^6\ mm^2 \tag{25}$$

$$\sigma_t = 2.65\ MPa \tag{26}$$

Unity check:

$$\frac{2.65}{11.88} = 0.23 \leq 1 \tag{27}$$

The connection between panels has to be designed to handle the tensile force.

6.3.5.4 Compression perpendicular to the grain

In the beginning of this work the platform frame connection was disregarded because of the high perpendicular stresses in the floor slab. This assumption is verified here.

$$\sigma_{max} = 9.04\ MPa \tag{28}$$

The maximum allowable stress in the fibre is:

$$f_{c,90,d} = \frac{f_{c,90,k} \cdot k_{mod}}{\gamma_m} = \frac{2.7 \cdot 0.9}{1.25} = 1.29\ MPa \tag{29}$$

As can be seen, the applied loads are more than 4 times bigger than the design compression strength perpendicular to the fibre. This will justify the use of the balloon frame connection, or the mixed connection.

6.3.5.5 Shear stress in the inner core

The central core is assumed carrying all the horizontal loads due to wind action (5979 kN). It is made from a CLT 360 mm panel, which has two layers of 40 mm each which are perpendicular to the direction of the panel grain, therefore, the shear calculations are made out of an equivalent effective section of 280 mm thick, with only seven parallel layers.

The shear stresses are checked in the neutral axis of the structure (which is due to the symmetry of the building the most loaded section) with the Jourasky formula.

$$\tau = \frac{V \cdot S}{b \cdot I} \tag{30}$$

Both the first and the second moment of area, are computed regarding the effective section. Their values are:

$$\begin{gathered} S = 1.44 \cdot 10^{10} mm^3 \\ I = 1.06 \cdot 10^{14} mm^3 \\ V_k = 5979\ kN \\ V_d = V_k \cdot 1.5 = 8970\ kN \\ b_{netto} = 560\ mm \end{gathered} \tag{31}$$

With those values the unity check is:

$$\frac{\tau_d}{f_{v,d,EN338}} = \frac{2.17}{2.88} = 0.75 \leq 1 \tag{32}$$

6.3.5.6 Serviceability limit stage

In this section the maximum deflection has to be verified. According to the Dutch national annex of Eurocodes, a maximum deflection on top has to be:

$$u < u_{max} = \frac{H_{building}}{500} \tag{33}$$

However this value is further reduced to account unforeseen events. The final deflection should be smaller than:

$$u < u_{max} = \frac{H_{building}}{750} = 140\ mm \tag{34}$$

The horizontal displacement on top of the building is computed with the following formula (formula 7.41 of (80)):

$$u = \frac{ql^2}{GA}\left(-\frac{1 + \alpha l \sinh(\alpha l)}{(\alpha l)^2 \cosh(\alpha l)} + \frac{\cosh(\alpha x) + \alpha l \sinh(l - x)}{(\alpha l)^2 \cosh(\alpha l)} + \frac{x}{l} - \frac{x^2}{2l^2}\right) \tag{35}$$

Where:

$$\alpha = \sqrt[2]{\frac{GA}{EI}} \tag{36}$$

$$q = q_{eq} \cdot 26\ m = 56.94\ N/mm$$

The x-axis is oriented according to Figure 6-15 below.

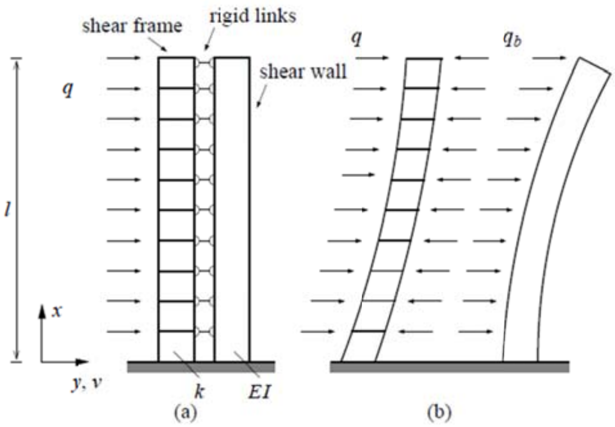


Figure 6-15: Shear beam-bending beam system

It is important to explain how the values for the E-modulus and the G-modulus were determined. In the deflection analysis, the effective values on the total thickness of the panel were used. The effective E and G-modulus were determined from the vertical layers of the panel only (neglecting the horizontal oriented layers), then this value was divided by the I_{panel} and A_{panel}.

$$E_{panel} = \frac{E \cdot I_{ef}}{I_{panel}} = 7520\ MPa \tag{37}$$

$$G_{panel} = \frac{G \cdot A_{ef}}{A_{panel}} = 536\ MPa \tag{38}$$

With these values, the bending stiffness and the shear stiffness are:

$$\begin{gathered} EI = 1.16 \cdot 10^{19}\ Nmm^2 \\ GA = 6.45 \cdot 10^9\ N \end{gathered} \tag{39}$$

The total deflection on top of the building (at $x = l$) is computed with formula (35) with values presented from formula (36) to (39).

$$u_{top} = 23\ mm \tag{40}$$

6.4 Fire design

6.4.1 Load combination

Accidental actions are classified according to EN 1995-1-1 table 2.2 as instantaneous actions. However in timber fire safety design, the attention is drawn to the structure's ability to resist the applied load after the fire event. Therefore the loads applied to the structure are not instantaneous, but they are assumed ad short-term loading (less than a week) because the damaged section has to stay in place until the original load-bearing strength is restored.

Simultaneously of different actions (variable action due to imposed load, wind or snow) should be introduced with the formula below. Decreasing of structure dead weight due to the progression of fire cannot be taken into account, according to EN 1991-1-1 § 4.2.

$$\sum G_{k,j} + A_d + \psi_{1,1} or\ \psi_{2,1} Q_{k,1} + \sum \psi_{2,i} Q_{k,i} \tag{41}$$

A_d, which is the design accidental action is normally taken equal to zero, because in timber structures the thermal elongation actions during the fire event can be neglected. This can be done because timber has an extremely low thermal elongation coefficient.

The use of the quasi-permanent value $\psi_{2,1}$ is recommended in 4.3.1 of EN1991-1-2 but its use is subjected to the national annex, however in a different source a $\psi_{1,1}$is founded. For the following analysis, the latter value is used.

In the Eurocode a second approach is also described, it is the simplified rule which allows determining the action in the fire event from the ones determined in the cold design (4.3.2 of EN1991-1-2).

$$\eta = \frac{\sum G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \sum \psi_{2,i} Q_{k,i}}{\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i}} \tag{42}$$

η is therefore calculated by the ratio between the fundamental and the accidental design situation, in a conservative way, η is simplify to a value of 0.6 (35). As described by the η-factor, in the event of fire, the loads can be reduced by 60%, this is a positive effect since reducing loads a smaller section will be needed.

	Actions	Section	
ALC1	Self-weight Wind Imposed loads	I	Maximize the compression force in the external wall. Section 1, $5.00\ m$ wide Leading variable action: wind action $E_{d,fi,1} = 1.00\ G_k + 0.5\ Q_{k,wind} + 0.3\ Q_{k,imp.load} = 5385\ kN$
ALC2	Self-weight Imposedlads	III	Maximize the compression force in the internal wall. Section 3, $8.36\ m$ wide $E_{d,fi,2} = 1.00\ G_k + 0.5\ Q_{k,imp} = 6069\ kN$
ALC3	Self-weight Wind force (tension)	VI	Maximize the tension force in the external wall. Section 6, 13 m wide (without openings) $E_{d,fi,3} = 1.0\ G_k - 0.5\ Q_{k,wind} = 3883\ kN$

Table 25: Governing accidental load combinations

As can be seen from load combination 3 in Table 25, in the accidental design situation no tension is occurring in any section of the building.

6.4.2 Material resistance in fire

The characteristic strength values given in §6.3.2, during the fire action are magnified, this is possible because fire is considered as an exceptional event. The design values are founded by the following relation.

$$f_{d,fi} = k_{mod,fi} \frac{f_{20}}{\gamma_{M,fi}} \tag{43}$$

$$f_{20} = k_{fi}\ f_k \tag{44}$$

Where:

f_k	Is the characteristic strength (5% fractile)
$f_{d,fi}$	Is the design strength in fire
f_{20}	Is the 20% fractile of a strength property at normal temperature
$k_{mod,fi}$	Is the modification factor for fire = 1, from §6.5.1 of (11)
$\gamma_{M,fi}$	Is the partial safety factor for timber in fire $= 1.0$ (EN1995-1-2)
k_{fi}	Is given for glued-laminated timber is 1.15

As can be seen:

$$f_{d,fi} = 1\ \frac{1.15\ f_k}{1} = 1.15\ f_k \tag{45}$$

The strength on fire is 159% higher than the strength for short term loading in the cold design.
The bending strength is:

$$f_{m,d,fi} = f_{c,0,d,fi} = 1.15\ \cdot 24 = 27.6\ MPa \tag{46}$$

and:

$$f_{t,0,d,fi} = 1.15 \cdot 16.5 = 19.97\ MPa \tag{47}$$

The design shear strength is:

$$f_{v,d,fi} = 1.15 \cdot 4 = 4.6\ MPa \tag{48}$$

6.4.3 Section properties

The residual section which is assumed having full strength can be computed with the simplified method proposed in EC5, §4.2.2, and shown below.

$$d_{ef} = d_{char} + k_0 d_0 \tag{49}$$

Where:

$d_{char,n}$ Is the charring depth, that can be computed as:

one dimensional charring rate, $\beta_0 \cdot t$, where $\beta_0 = 0.65\ mm/min$

notional charring rate (effect of corner rounding), $\beta_n \cdot t$, where $\beta_0 = 0.70\ mm/min$

k_0 is a factor on time exposure

d_0 is the depth at which unaffected timber can be founded, $= 7\ mm$;

The affected depth after a 120 min fire exposure is:

$$d_{ef} = 0.7 \cdot 120 + 7 = 91\ mm \tag{50}$$

The gamma factor (15) for fire design is the same as cold design since it is not affected by the progression of the char depth.

The effective inertia of the panel is computed by the following formula, where $\gamma_{1,2} = \gamma_{CLT,360}$ and $\gamma_{4,5,6} = 1$. In the fire design layer number 8 and 9 are destroyed by fire, the third and seventh layers, which are transverse, are omitted as before.

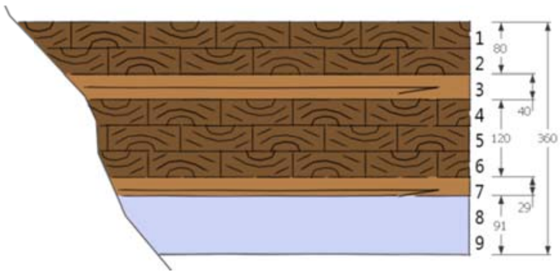


Figure 6-16: CLT 360 mm residual section, a depth of 91 mm is affected by fire

$$\mathrm{I}_{ef,fi} = \sum_{i=1}^{9} I_i + \gamma_i A_i z_i = 1.05 \cdot 10^9\ mm^4 \tag{51}$$

The section properties used in the further fire analysis are given in Table 26.

Panel total thickness	$t_{panel,fi} = 269\ mm$
Parallel thickness	$t_{netto,fi} = 200\ mm$
Neutral axis (from top)	$y = 124\ mm$
Inertia effective	$\mathrm{I}_{ef,fi} = 1.05 \cdot 10^9\ mm^4/m'$
Section modulus (compression fibre)	$\mathrm{W}_{ef,fi} = 8.47 \cdot 10^6\ mm^3/m'$
Radius of gyration	$\rho_{fi} = 72.43\ mm$
E modulus panel	$E_{0,panel,fi} = 7116\ MPa$
G modulus panel	$G_{panel,fi} = 513\ MPa$

Table 26: Section properties for fire analysis

6.4.4 Buckling reduction

As already done for the cold design, the slenderness of the reduced external wall is computed. In the fire design situation, it is assumed that the stability elements may collapse during fire, therefore here the system length is assumed as twice the cold length, taking into account the failure of a floor slab. As for the cold design the equation 6.23 of Eurocode 5 can be simplified as:

$$\frac{\sigma_{c,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{52}$$

In the fire situation case only the first two terms are present.

System length	$h_{system} = 2 \cdot h_{floor} = 7000\ mm$
Effective length	$l_{ef,fi} = 7000\ mm$
Inertia effective	$\mathrm{I}_{ef,fi} = 1.05 \cdot 10^9\ mm^4/m'$
Radius of gyration	$\rho_{fi} = 72.43\ mm$
Slenderness ratio	$\lambda_{y,fi} = 96.64$
Buckling reduction	$\boldsymbol{k_{c,y,fi} = 0.31\ MPa}$

Table 27: Buckling reduction during fire

6.4.5 Ultimate limit stage

6.4.5.1 Section 1 – ALC1

Section 1 in Figure 6-8 is checked for the simultaneous action of bending moment and compressive force as for the LC1. The external wall is now assumed stabilized by the floor slab every two storeys (according to §6.4.4) and it is assumed as a simply supported beam of one span only (this assumption will lead to highest bending stresses in the wall section), and the uniform distributed design load is:

$$q_{wind,m,d,fi} = 2.19\ kN/m^2 \cdot 0.5 \cdot 26\ m\ \cdot 1.3 = 37\ kN/m \tag{53}$$

Where the values 0.5 and 1.3 are safety factor and magnification factor respectively. The stresses on the effective wall section are:

$$\sigma_{m,y,d,fi}=\frac{q_{wind,m,d,fi}\cdot(2h_{floor})^2}{8}\cdot\frac{1}{W}=\frac{37\cdot 7000^2}{8}\cdot\frac{1}{8.47\cdot 10^6\cdot 5}=5.35\ N/mm^2 \tag{54}$$

The wall element of section 1 has the following vertical force (calculated in §6.3.1) and effective area:

$$E_{d,fi,1}\ =5385\ kN$$
$$A_{effective,fi,1}\ =1.0\cdot 10^6\ mm^2 \tag{55}$$

The cold unity check is satisfied, as can be see below:

$$\frac{5.38}{0.31\cdot 27.6}+\frac{5.35}{27.6}=0.62+0.19=0.81\leq 1 \tag{56}$$

6.4.5.2 Section 3 – ALC2

The loads acting in this load combination are the imposed load and the dead.

$$E_{d,fi,2}\ =6069\ kN \tag{57}$$

The section 2 is characterized by:

$$A_{effective,fi,2}\ =16.70\cdot 10^5\ mm^2 \tag{58}$$

The cold unity check is satisfied, as can be see below:

$$\frac{3.63}{0.31\cdot 27.6}=0.42\leq 1 \tag{59}$$

6.4.5.3 Section 6 – ALC3

A simplified building model has been assumed to compute the maximum tensile force occurring in the windward side. However in the accidental load combination no tensile force is occurring in the wall. A compression force of $E_{d,fi,3}\ =3883\ kN$ is founded. This load combination is less severe of ALC1 described in §6.4.5.1, therefore it is not further analysed.

6.5 Comparison between cold and fire design

In the next tables the results from the cold and fire design are summarized. As it is shown, according to prescriptive design rules it is possible to build the structure. Indeed, a CLT section of 360 mm can withstand the applied loads. The unity check values are referred to three interesting sections of the buildings and they are not representative of the real stresses which occur in the real structure. In order to evaluate the real stresses should be carried out a finite element analysis taking into account the inner load bearing walls and the slip modulus of the connections.

			Factorized load[16]									Unity check	
			Cold design	Fire design	Load duration	f_d	$f_{d,fi}$	$k_{c,y}$	$\sigma_{c,d}$	$\sigma_{m,d}$	Cold design	Fire design
			kN	kN		N/mm2	N/mm2					
Section I	LC1	$1.35\ G_k + 1.5\ Q_{k,wind} + 1.5 \cdot 0.7\ Q_{k,imp.load}$	12731		Short[17]	17.28		0.94	9.09	2.31	69%	
	ALC1	$G_k + 0.5\ Q_{k,wind} + 0.3\ Q_{k,imp.load}$		5384	Fire[18]		27.6	0.31	5.38	5.35		81%
Section III	LC2	$1.35\ G_k + 1.5\ Q_{k,imp.load}$	10456		Medium[19]	15.36		0.94	4.47	-	31%	
	ALC2	$G_k + 0.5\ Q_{k,imp.load}$		6068	Fire		27.6	0.31	3.63	-		42%
Section VI	LC3	$0.9\ G_k - 1.5\ Q_{k,wind}$	-9813		Short	11.88[20]			2.70	-	23%	
	ALC3	$1.0\ G_k - 0.5\ Q_{k,wind}$		3379	Fire		27.6		1.30	-		N/A

Table 28: Utilization ratio of different interesting sections

In agreement to the performed calculations following prescriptive codes, the section of 360 mm, determined in the cold design, is able to withstand a 120 min of ISO fire curve without being protected by gypsum plasterboard. The original section is reduced by 91 mm, and the residual section is 269 mm thick. Indeed the residual fulfils the requirements, furthermore the idea of adding some extra sacrificial thickness on the load bearing elements is useless in this case because the concept building has a strong reduction of loads during the fire event.

		LOADS	STRENGTH	AREA	BUCKLING REDUCTION
		positive	positive	negative	negative
		$\frac{E_{d,fi}}{E_d}$	$\frac{f_{d,fi}}{f_d}$	$\frac{A_{fi}}{A}$	$\frac{k_{c,y,fi}}{k_{c,y}}$
LC1	Section I	42%	160%	71%	31%
LC2	Section III	58%	180%	71%	31%
LC3	Section VI[21]	34%	232%	71%	N.A.

Table 29: Percentage differences between cold and fire design

[16] Negative values represent tensile forces
[17] Short term loading has a $k_{mod,short} = 0.9$
[18] In the fire analysis the k_{mod} is taken as 1
[19] Medium term loading has a $k_{mod,medium} = 0.8$
[20] This value is $f_{t,0,d}$
[21] There is a change from tension (cold design) to compression (fire design).

6.6 Summary

A feasibility study for a 30 storeys high rise timber building has been carried out. The building height was $105\ m$, with a section of 26 by $26\ m$. A tube in tube structural system has been assumed to carry the loads. The loads which have been taken into account were: the dead weight of the building elements, the imposed residential loads, and a wind load pressure of $2.19\ kN/m^2$. In the design a magnification factor has been used in order to take into account: second order effects, differential settlement of foundations and dynamic wind load. According to calculations a minimum wall section of $360\ mm$ is needed. The former section has been designed to withstand a standard fire exposure of $120\ min$ according to prescriptive fire regulations. The chosen section thickness has low utilization ratios, therefore a thinner section of $320\ mm$ was also checked. The thinner section met all the requirements apart the fire check for the critical section 1 (Figure 6-8,Table 17), which is 1.11%. However, if a different load path is assumed this utilization ratio fells at 0.73%.

According to prescriptive building codes the building is fire safe and it can be built. However, it is strongly believed that designing a high-rise timber building according to a fire design of only two hours is not safe. Doubts arise on the philosophy behind the prescriptive requirements and in their applicability to timber structures. Indeed, prescriptive regulations have been determined for non-combustible structures, where it is assumed that the main load bearing structure is not being destroyed by fire action. In a prescriptive approach it is also believed that as soon as the fire has consumed the entire movable fire load it has been extinguished, or it has been moved to another compartment. With timber compartment in sight it cannot happen, as long as there is the load bearing structure there is enough fire load to make up the fire. The fire will eventually go out only when the entire fire load will be consumed, eventually leading to a structural failure.

The aforementioned doubts lead to the need of deeper analyses on a reasonable fire growth in timber compartments. In order to evaluate a more realistic fire behaviour fire advanced analyses will be evaluated in the next chapter.

7 Advanced fire analysis

"Mankind really differentiated from animals only when they became master of the fire" Catherine Perlès.

It has been show in the previous chapter that it is possible to design the fictional building according to prescriptive regulations. Prescriptive methods rely on the standard fire curve which is one of the notional fire curves. It is believed that the philosophy behind prescriptive regulations should not be applied to buildings with a combustible load bearing structure. In a timber compartment the fire will not probably go out until the entire load bearing structure is consumed. This is in contrast with the main assumption of prescriptive codes which states that if the structure is designed for a given fire design time, it will withstand the fire even if it is left unattended.

Advanced fire analyses have been carried out in this chapter in order to determine if the section thickness (determined following prescriptive rules) will be able to withstand a more reasonable fire behaviour than the ISO fire curve. Thus, the main purpose of this section is to evaluate the differences between the charred depths obtained by standard fire exposure (according to Eurocodes) and the values obtained from advance calculation methods (performance-based codes).

Advanced fire analyses are performed according to the fictional floor layout previously determined. They take into account different amounts of initial fire load densities, and different encapsulation configurations (with and without exposed timber structure). However, in literature no model has been developed for timber compartments; indeed, all the models have been obtained for non-combustible structures. It is believed that this is a main limitation because in a compartment with timber structure in sight there is a mutual influence between the charring rate of the structural elements and the heat release rate of the fire. This may affect the quality of the results.

When advanced fire analysis are performed, it is necessary to take into account the "relevant part of the timber structure" which participates to the fire. This requirement is given in Eurocode 5; however no guidance on how to consider the charred thickness of the structural elements is given in the code. In the literature only one source has been founded which takes into account 50% of the fire load due to the charred timber floor. Unfortunately it has not been explained why only the 50% has been added to the initial fire load. Therefore the second objective of this section is to determine how different percentage values of the charred structural timber elements affect the fire design. The charred depth of the structure is also called additional fire load.

Finally, the obtained results were compared to real scale tests founded in literature (41) (43) (44).

7.1 Fire thermal models

"A very important part of the structural fire design of timber structures is to determine the temperature development in the fire compartment" (33).

According to EN 1991-1-2, the fire load resistance of a load bearing structure can be analysed with two fire thermal models, namely: by nominal fire curves, suitable for hand calculations; and by natural fire curves, which are more accurate and in turn more cumbersome. Natural fires may be evaluated by simplified models, such as the parametric fire exposure given in EN 1991-1-2, or by advanced fire models. Advanced fire models can be carried out by zone models or by field models. Field models represent cutting edge of fire protection engineering, they are complex computational fluid mechanic software which require a large computing time, but they can be used for complex geometry. For the purpose of this study, the attention is drawn mainly to advanced fire models, represented by zone models (OZone model), and to simple calculation rules, represented by the standard temperature-time curves. They are shown in bold in Table 30.

FIRE THERMAL MODELS

EN 1991-1-2	Nominal temperature-time curves (Prescriptive rules)		***Standard temperature-time curve (ISO)***
			External fire curve
			Hydrocarbon curve
	Natural fires (Performance based rules)	Simplified fire models	Compartment fires (Parametric fires)
			Localized fires
		Advanced fire models	***Zone models (OZone)***
			Field models (CFD)

Table 30: Fire thermal models

Nominal temperature-time curves, like the ISO fire curve, do not have a decay stage and the temperature in the compartment is assumed growing for ever. Natural fires, instead, are characterized by a more realistic fire behaviour composed by a growing stage, a fully developed stage, and a decay stage; this behaviour is depicted in Figure 7-1.

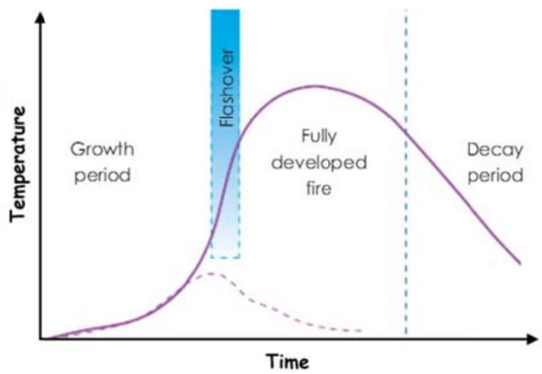
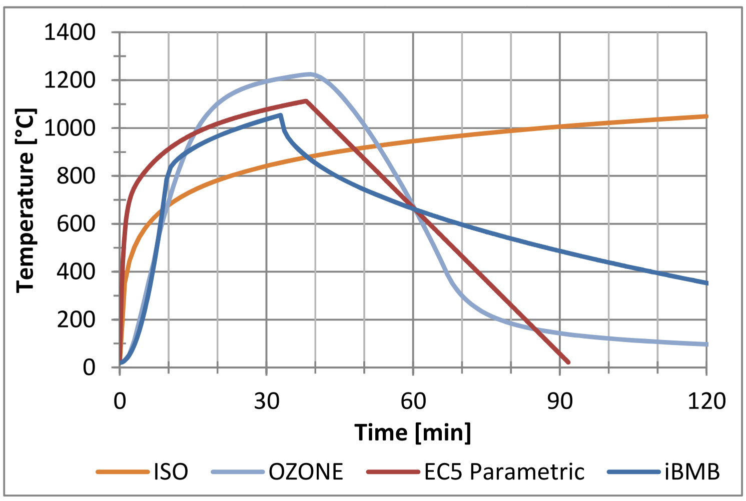


Figure 7-1: Fire growth

In the next diagram several fire curves are compared, it is important to note that the three natural fire curves were determined for the same parameters (opening factor, fire load density and b-factor).



Different natural fire curves compared to the ISO fire exposure (orange). Mind all the parametric curves have the same values for openings $\left(O_f = 0.0245\ m^{1/2}\right)$*, fire load density* $\left(q_{f,d} = 334\ MJ/m^2\right)$*, and b-factor* $\left(b = 322.5\ J/m^2 s^{1/2} K\right)$*.*

Figure 7-2: Different fire curves

The curves depicted in Figure 7-2 are:

1. ISO fire model (orange line), nominal-time curve, only function of time;
2. Parametric fire model from EN1991-1-2 (red line), depending on opening factor, thermal properties of boundaries, and fire load density. This is a natural fire model which uses a simplified fire model, called compartment fire;
3. iBMB fire curve (blue line), parametric fire model, depending on the opening factor, on the rate of heat release, on thermal properties of boundaries, and on the fire load density. This is a natural fire model which uses a simplified fire model, called compartment fire;
4. OZone fire model (sky-blue line), computed by OZone software, this fire model can take into account all the aforementioned parameters; furthermore it can consider the presence of horizontal openings, breaking of windows. The main feature is represented by the automatic transition from a two zones model to one zone only (at occurrence of flashover).

The first three fire models are computed manually, while the OZone fire curve is derived by the program.

7.1.1 ISO fire curve

The standard time-temperature curve in the Eurocodes (also known as ISO 834) which is depicted as the orange line in Figure 7-2. It is determined by the following formula:

$$T_g = 20 + 345\, log_{10}(8\, t + 1) \tag{60}$$

It is an empirical model, and as it can be seen it is only function of the time given in hours. The growing phase of the fire is neglected. It is very conservative for slow growing fires, while for fast growing fires is only a little conservative. The curve does not consider the fire load density, the opening factor, and the cooling stage of a fire (33). Using the ISO 834 curve in many cases result in a simple design calculation which are on the safe side, but, on the other hand, it will cause unsatisfactory need for fire protective measures which entails economic and aesthetical disadvantages (81).

7.1.2 Parametric fires

Parametric fires are empirical model which are tailored depending on three main parameters: opening factor, fire load density and thermal properties of boundaries. They also take into account a decay stage of the fire which starts when 70% of the fire load is consumed. In literature four parametric fire models have been founded: the Swedish curves; the Standard and Parametric curves from the Eurocode; and the iBMB curve, developed in Germany. A review on the factors which affect those models is given in Table 31. All of them were developed for non-combustible compartment. They only take into account the movable. According to sources in the literature, the iBMB fire model together with the “Swedish fire curves” were evaluated to give the best outcome for a real fire in a compartment with exposed timber structures (33). However since computing a parametric fire curve for each iteration is rather cumbersome, the fire exposure used in the further analysis is determined with the two-zone model OZone which has a more user friendly interface and is a more accurate model.

	Standard and parametric fire curves			
	Swedish curve	EC5 - ISO	EC5 - Parametric	iBMB
Fire load density	Yes	No	Yes	Yes
Area and height of ventilation openings	Yes(1)	No	Yes (1)	Yes(1)
Thermal proprieties of the compartment	Yes	No	Yes	Yes
Heat balance of compartment	Yes	No	No	No
Radiation through openings	Yes	No	No	No
Heat transfer though boundaries	Yes	No	No	No
Replaces of hot gasses by cold air	Yes	No	No	No
Cooling stage	Yes	No	Yes	Yes

(1) Only vertical openings

Table 31: Main factors affecting fire curves

7.1.2.1 Opening factor

Particular attention is paid to the opening factor which can affect both, the maximum temperature, and the fire duration. It influences the temperature-time curve as shown in Figure 7-4. However, lower temperatures may be reached when the openings in a compartment are too large. Because hot smoke may escape easily from the compartment, and colder air may go inside (82), this effect is shown in Figure 7-3.

Most of the fires, with small or medium openings, are ventilation controlled fires.

In the Eurocode, according to the parametric fire curve, the opening factor is defined as:

$$O = F_v = \frac{A_v\sqrt{H_{eq}}}{A_t} \tag{61}$$

It has to be limited to the range of $0.02 \leq O \leq 0.20$.

According to (13) the opening factor also plays a major role in the decay stage, because large openings will allow rapid heat loss from the compartment by both convection and radiation.

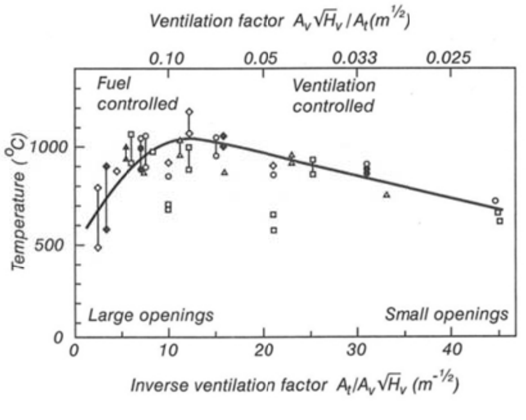


Figure 7-3: Opening factor (13)

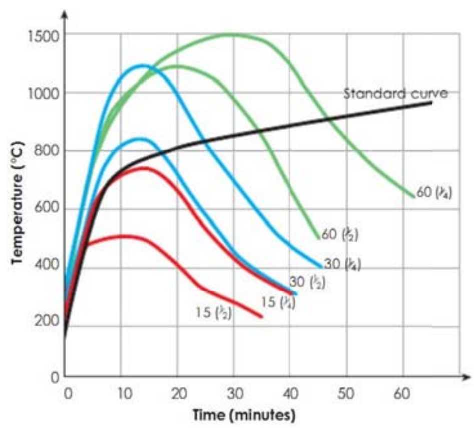


Figure 7-4: Time-temperature curves based on different opening factors and different load density (kg(opening factor))

The effect of horizontal openings is neglected in all the calculation model, however in (83) it is recognized that the presence of ceiling openings allow combustion products to exit the ceiling opening while cool air enters from the window, increasing the ventilation to the fire. An approximated method to take into account the presence of horizontal opening is briefly explained in (13) and (10), both of these studies refer to the research done by Magnusson and Thelandersson.

7.1.2.2 Eurocode parametric curve

Parametric curve in Eurocode has been derived to give a good approximation to the burning behaviour of the Swedish curve (13), which in turn was derived by Magnusson and Thelandersson (81). The parametric curve describe only the fully developed phase of the fire without considering the growing phase (as for the standard curve), but it accounts factors for the fire load density, compartment boundaries and ventilation area (Figure 7-5). However, it neglects the effect of horizontal openings, as all available models (83) (13).

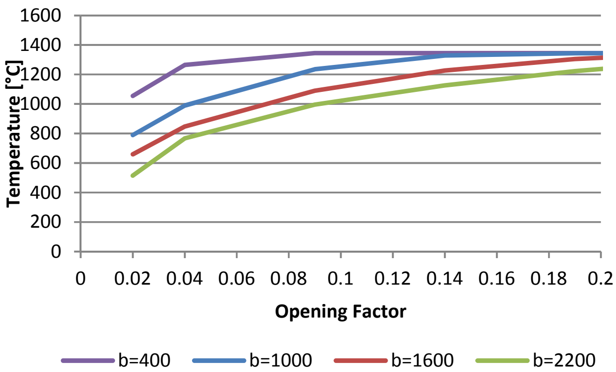


Figure 7-5: Parametric fire curves from EC5 depending on the thermal boundaries and the opening factor (gypsum plaster board $b = 410\ Ws^{1/2}/m^2K$, normal weight concrete $b = 1900\ Ws^{1/2}/m^2K$)

Parametric fire curve from Eurocode can be used for combustible structure, provided that the relevant part of the combustible structure is added to the fire load density. To do that iterations are required (33).

Feasey and Buchanam proposed modifications to the Eurocode parametric curve since they argued that the time-temperature outcome does not fit well with real fires (83). Those modifications are:

- Changing the reference value (from 1160 to $1900\ J/m^2 s^{1/2} K$) of the thermal properties ($\sqrt{k\rho c_p}$);
- Remove the lower limit of the thermal properties ($\sqrt{k\rho c_p}$);
- Changing the formula for the decay phase.

Only the second proposed modification has been adopted in the current version of the Eurocode. Indeed, the lower value for b-factor has been changed to $100\ Ws^{1/2}/m^2K$.

7.1.2.3 iBMB parametric curve

The iBMB parametric curve considers the actual boundary conditions of the fire compartment such as fire load, ventilation conditions, geometry and thermal properties. The peculiarity of this curve is that it is directly derived from the heat release rate. The iBMB model can take into account the effect of breaking of windows (increasing ventilation factor) and sprinkler systems activation (81). It may also include a progressive burning of the fire load or a spreading of fire to other fire cell. In Figure 7-6 the heat release rate and the temperature-time curve are shown respectively with black and blue lines, while in Figure 7-7 it is shown the rate of heat release for a progressive spreading of fire is given.

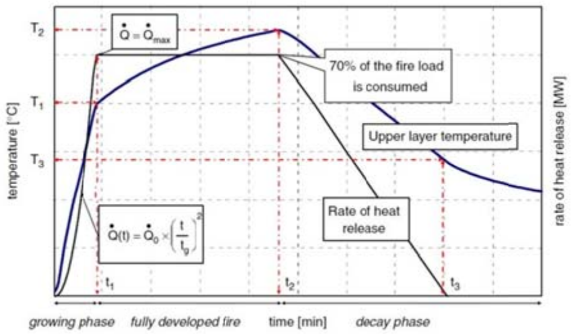


Figure 7-6: iBMB parametric curve and HRR (81)

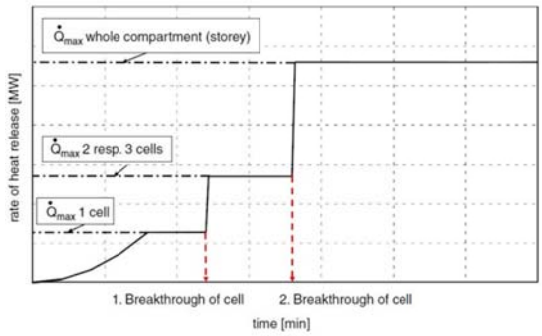


Figure 7-7: Development of RHH during spreading of fire through the compartment (qualitative) (81)

7.1.3 Zone models

Zone models are simple computer models that divide the considered fire compartments into separate zones, where the condition in each zone is assumed to be uniform. The simplest model is a one-zone model for post-flashover fires, in which the conditions within the compartment are assumed to be uniform and represented by a single temperature. The early one-zone models were developed in 1970s. Since then, zone models have gone through major development to multi-zones and multi-compartment for modelling localized and pre-flashover fires.

The theoretical background of zone models is the conservation of mass and energy in fire compartments. Basically, the models take into account of rate of heat release of combustible materials, fire plume, mass flow, smoke movement and gas temperatures. They rely on some assumptions concerning the physics of fire behaviour and smoke movement suggested by experimental observation of real fires in compartments. The zone models also model the fire compartments in more detail, compared to that for parametric fires and time equivalence methods. The geometry of compartments, as well as the dimensions and locations of openings, can be modelled easily (84).

The most common zone models will split a room into two zones, an upper hot zone and a lower cold zone. Some zone models, such as OZone, include the possibility to switch from a two-zone model to a one-zone model when the required conditions are reached (occurrence of flashover). Their application to the structural fire engineering is fixed only to the determination of the gas temperature.

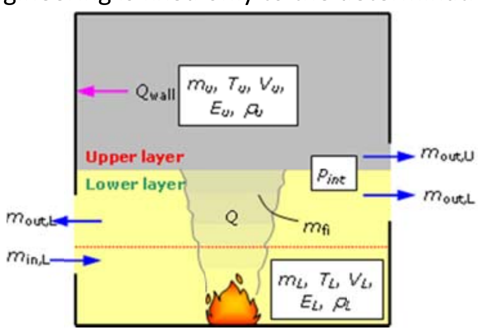


Figure 7-8: Two zones temperature – Pre-flashover

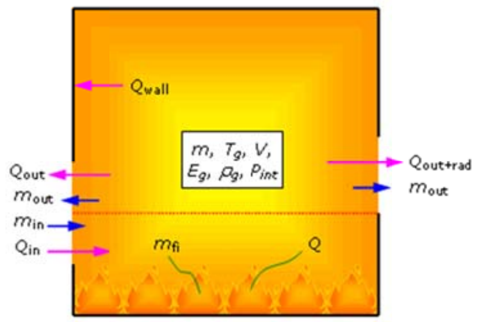


Figure 7-9: One zone temperature – Post-flashover

7.1.4 Timber compartment fire model

It believed that it is time to develop a fire thermal model for in sight timber compartment. The Swedish curves and the iBMB model are considered having a good approximation for a compartment with large amount of timber materials but however they were not developed for compartments completely made of combustible material. A fire model for a combustible compartment should be able to evaluate the mutual influence between the RHR curve and the temperature time curve. In a timber compartment it is believed that as soon as the movable fire load is ignited, it will start heating up the timber elements, and eventually when the fire room temperature is enough, the timber structure will start burning. When burning of the timber structure occurs, the relative released heat has to be added to the rate of heat release curve (RHR) of the movable fire load step by step. Non-combustible fire models cannot evaluate this effect. This has to main effects:

- Iterations are needed in order to evaluate the real amount of timber structure that takes part into the fire;
- The RHR curve is affected only in the fully developed stage, in fact growing stage and the decay stage are independent from the fire load.

The new fire model for timber compartments should compute step by step the RHR curve, the fire curve, and from the fire curve the model should evaluate if the timber structure is undergoing to charring. If the structure is charred, the amount of fire load due to the charred depth should be added to the next step of RHR curve. The suggested fire model is depicted in Figure 7-10.

However the suggested model has some limitations. Indeed, the horizontal plateau in the RHR graph is determined by the amount of openings (if the fire is ventilation controlled), or by the amount of fire load (if the fire is fuel controlled). This maximum value cannot be exceeded.

In this work it was not possible to directly change the RHR curve due to the program used and the suggested model could not be applied.

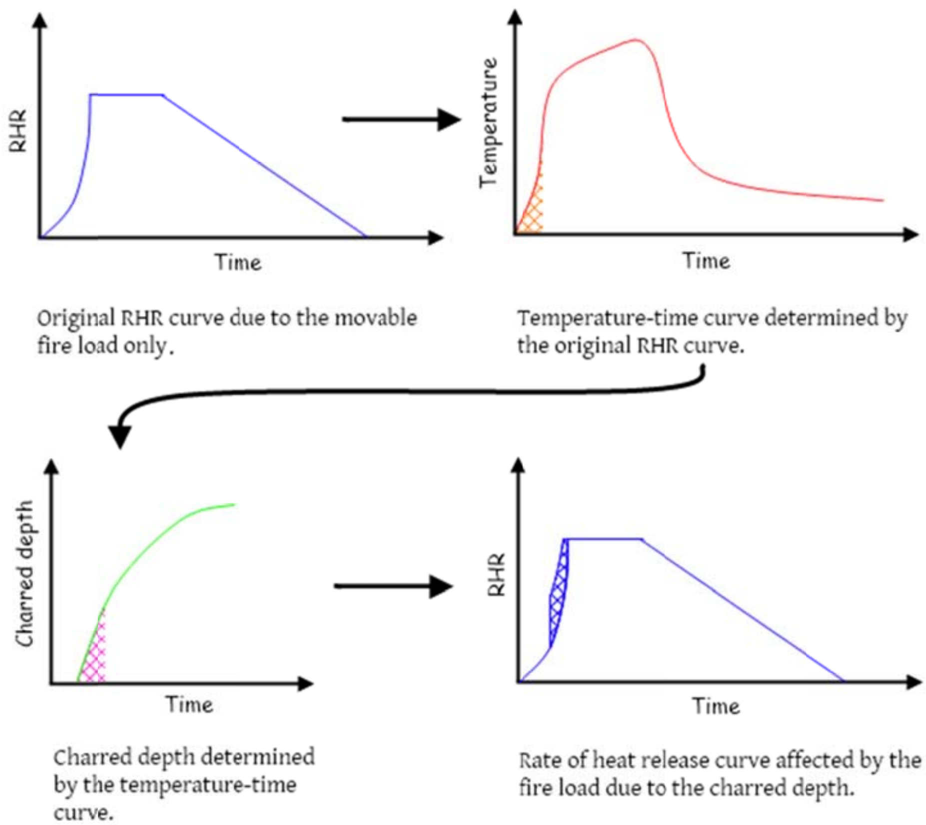


Figure 7-10: Suggested fire model in unprotected timber compartment

7.2 Design procedure for mechanical resistance

In the Eurocode 1995-1-2 some design procedures for mechanical resistance are described. Two simplified rules and one advance method are explained. They are listed below and treated in the next sections.

- Reduced cross-section method;
- Reduced properties method;
- Advanced calculation method.

7.2.1 Reduced cross-section method

The reduced cross-sectional method is based on the reduction of the initial cross-section by the effective charring depth determined accordingly to nominal temperature-time curves (standard fire curve). The residual section is easy to compute; in fact it is determined by the following formula:

$$d_{ef} = d_{100°C} = \langle \beta_0 or\ \beta_n \rangle \cdot t_{ISO} + k_0 d_0 \qquad (62)$$

For this reason it is suitable for hand calculations. The charring rate for timber elements exposed to the standard fire curve is well known, and it is only linearly related to the fire exposure time. An extra thickness of 7 mm is added to the obtained charred depth in order to find the unaffected timber section (with temperature lower than 100°C). The simplified rules give formulas also for taking into account the presence of protective materials, such as gypsum plasterboard or other sacrificial timber panel. In the graph below the charred depths

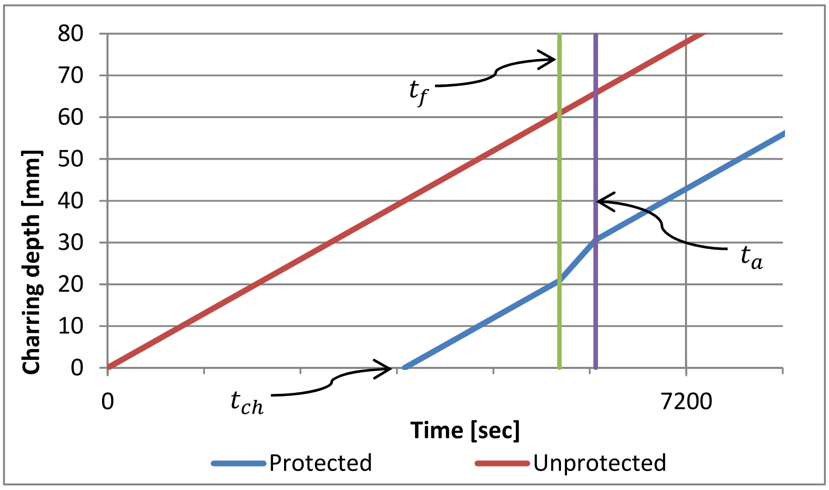


Figure 7-11: Charred depths according to standard fire curve (ISO)

Charring depths according to standard temperature-time curve for protected and unprotected configuration. The red line is determine for the unprotect timber element, while the blue line has a protective layer of 30mm of gypsum plasterboard type F.

The charring depth at 120 min for the element in sight is 78.0 mm, while for the protected element is 42.8 mm.

The beside graph has been determined by values computed in the next sections

7.2.1.1 Unprotect

When an unprotect timber element is exposed to an ISO fire exposure, according to EN 1995-1-2, it will char under a constant rate, therefore it is very easy to determine the charred depth. In order to determine the unaffected depth, the notational or the one-side charring rate has to be multiplied by the fire resistance time and then a depth of *7 mm* has to be added to account the heated timber depth.

$$d_{100°C} = \langle \beta_0 or\ \beta_n \rangle \cdot t + 7mm \qquad (63)$$

For a fire resistance of 120 min with one-side fire exposure, the unaffected timber section is founded at:

$$d_{100°C} = 0.65 \cdot 120 + 7 = 85\ mm \qquad (64)$$

The charred depth according with the simplified calculation method is given in Figure 7-11.

7.2.1.2 Protect

When a timber element is protected by a 30 mm of gypsum plasterboard type F, and it is exposed to an ISO fire exposure the charring rate will be affected by the presence of the protective layer as shown in Figure 7-11. The delay of start charring is represented by t_{ch}. After the time t_{ch} the timber starts charring at slower rate that the unprotected configuration until a complete failure of protective cladding occurs, at t_f. After the failure time, timber element which is now unprotected, is exposed to high temperature and its charring rate is twice the original value (β_n or β_0). The increased charring rate is used until the timber element does not “build” enough protective charred depth (determined by the code as 25 mm) which is represented in Figure 7-11 by t_a. The values for t_{ch}, t_f and t_a are determined according to calculation rules given in EN1995-1-2 and shown below. For gypsum plasterboard of type A or F, the time of start of charring t_{ch} should be taken as:

$$t_{ch} = 2.8 \cdot h_p - 14 = 61.6\ min \qquad \text{(3.11) of EN1995-1-2}$$

Where h_p is the thickness of the panel, in mm, which is determined in 3.4.3.3(4) as:

$$h_p = 15\ mm + 15mm \cdot 0.8 = 27\ mm \qquad \text{(3.11) of EN1995-1-2}$$

The failure time of claddings made of gypsum plasterboard F should be determined with respect to pull-out failure of fasteners due to insufficient penetration length into un-burnt wood (C.2.3(5)).

$$t_f = t_{ch} + \frac{l_f - l_{a,min} - h_p}{k_s k_2 k_j k_n \beta_0} = 61.6 + \frac{66 - 10 - 27}{1.1 \cdot 0.8529 \cdot 1 \cdot 1.5 \cdot 0.65} = 93.3\ min \qquad \text{(C.9) of EN1995-1-2}$$

The time limit t_a, for $t_{ch} < t_f$, should be taken as:

$$t_a = \frac{25 - (t_f - t_{ch}) k_{2,a} \beta_n}{k_3 \beta_n} + t_f = \frac{25 - (93.3 - 61.6)0.73 \cdot 0.65}{2 \cdot 0.65} + 93.3 = 101\ min \qquad \text{(3.9) of EN1995-1-2}$$

Where:

$$k_{2,a} = 0.73 \qquad \text{(3.7) of EN1995-1-2}$$

$$k_3 = 2 \qquad \text{3.4.3.2(4) of EN1995-1-2}$$

7.2.2 Reduced properties method

The reduced properties method is based on the reduction of the mechanical strength of the material. This method only applies to rectangular softwood cross-sections exposed to fire on three or four sides, and round cross-sections exposed to fire along their whole perimeter. It does not apply to timber panels used as walls or floor exposed to fire on one side only. The residual cross-section is determined by reducing the cross-section with the calculated total charring depth. It is assumed that the residual cross-section has reduced properties. The reduced properties can be found by multiplying the bending strength, compressive strength, tensile strength and modulus of elasticity, respectively, for fresh wood with a modification factor, $k_{mod,fi}$. The modification factor takes into account the reduction in strength and stiffness properties at elevated temperatures. The residual load-bearing capacity of the residual cross-section is calculated based on the reduced properties of the wood at elevated temperatures. However this method is banned in the Dutch national annex of the Eurocode, it is does not apply also in UK. Since it cannot be applied to CLT panels and it is also banned by the national annex, this method is omitted from the further analysis (85).

7.2.3 Advanced calculation methods

Advance calculation methods may be used for individual members, part of a structure or for entire structures. They can be applied for determining the charring depth, the development and distribution of the temperature within structural members and structural behaviour of the entire structure or of any part of it. In this work, advanced analyses are carried out to evaluate the charring depth and the temperature distribution only. Advanced calculation methods for determination of the mechanical resistance of a structure exposed to fire shall be based on the theory of heat transfer and they should take into account the variation of thermal properties of the material with temperature. However in this study the mechanical resistance is determined by effective cross section determined by finite thermal analysis. The thermal response model should take into account the variation of thermal properties of the material with temperature. The temperature dependent thermal properties for timber elements are only derived for standard fire exposure. When different fire curves are used the results may not lead to a reliable approximation.

7.3 Process

The process of advanced fire analyses carried out in this work starts with the fiction building previously designed which has been used to determine the geometrical properties of the fire room. Then, the initial fire load density was determined in accordance with the Eurocode. The initial fire load density is also called movable fire load. Two initial fire load densities have been considered depending on the fire safety measure used. The fire room has been entered in the zone model OZone. The first iteration runs with only the movable fire load. The temperature-time curve obtained is used as input data in SAFIR and the charring depth can be determined. The additional fire load related to the charred depth is then added to the initial fire load, together they represent the fire load for the second iteration. It has to be noted that the additional fire load has been accounted in different percentages, from A (100%) to G (5). A new iteration is performed by OZone with the new fire load composed of the movable and the additional fire load. A new fire curve is then determined and it is used in SAFIR. It is worth nothing that the fire curve in SAFIR is applied to the original thickness of the timber element. the additional fire load determined out of the charred depth of the second iteration is then added to the initial fire load density again. The process then continues until a good convergence is determined. A sketch of the process is given below.

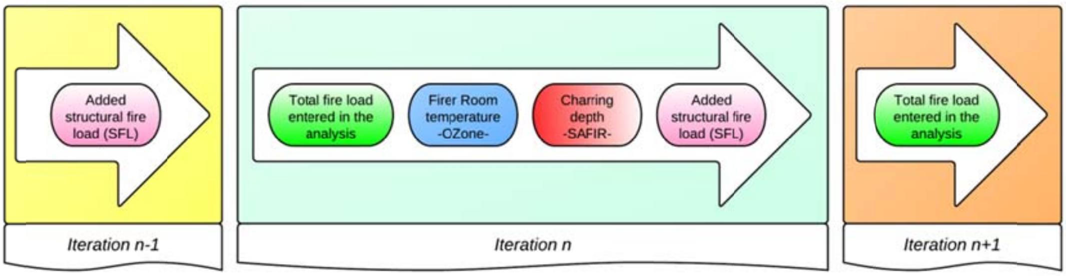


Figure 7-12: Detailed view of steps involved in each iteration

The different percentages studied are depicted in the next figure.

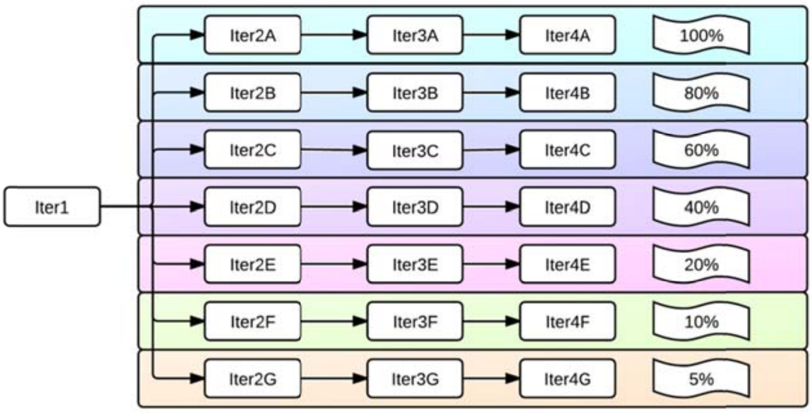


Figure 7-13: Process of this study, form the initial fire load (Iter1) to the different percentages studied

In order to perform the advanced calculations explained above the following assumptions have been adopted:

- The thermal timber properties given in EN 1995-1-2 are used for fire exposures different from the standard fire curve although they have been validated only for fire exposure according to the standard or parametric fire curve in the Eurocode. This may lead to unrealistic values, but however they are the only thermal temperature dependent values that have been founded in literature;
- The thermal properties of boundaries presented in (43) were used in this study. As the Eurocode suggests, the thermal properties, conductivity, density and specific heat, shall be used as temperature related, however it is not clear if the program assume this behaviour. To take this into account, in (43), averaged values are computed for all the thermal properties and they are used here;
- The charring depth has been assumed constant along all the boundaries of the fire room, however it is strongly dependent on the time-temperature curve, as it has been noted in the Ivalsa fire test (41) and in Tuula Hakkarainen tests (43);
- The net calorific values for timber at equilibrium moisture content (12%) has been computed according to the formula E.4 of EN1991-1-2, and its value was $15.1\ MJ/kg$. The net calorific value at equilibrium moisture content has been used also in (43) and in (83) (86). However in some other studies (41) the dry net calorific value has been used.

In this analysis three different scenarios have been studied, they are listed below:

- Low and high fire load densities, only for unprotected room;
- Protected and unprotected fire room with high fire load density;
- ISO fire exposure of 120 min according to the building codes, the charring depth and the unaffected timber depth are then computed by simplified hand calculations and with thermal FEM.

The unprotected fire room configuration has been checked for different initial fire load densities, the low design fire load density with a value of $328.7\ MJ/m^2$, and the high design fire load density with a value of $932.8\ MJ/m^2$. Since each iteration was relatively fast, it was possible to analyse different amounts of relevant combustible structure. Different percentages (5%, 10%, 20%, 40%, 60%, 80% and 100%) of the structural fire load were added to the initial movable fire load for each iteration. The analysis has been interrupted at the 4th iteration due to excessive fire load which leaded to a time temperature curve which was too high. The temperature-time curve and the fire load for the 4th iteration have been computed but it was not possible to evaluate the respective charring depth due to temperatures exceeding the temperature range of material data.

In the protected fire room configuration the timber structural elements were protected by a double layer of gypsum plasterboard type F (improved stability at high temperature). The thickness of the protective layer has been assumed as 30 mm. Usually the second layer, which is protected by the first, is a regular gypsum plasterboard, however to simplify the analysis in the FEM program only one protective material has been used. The design movable fire load $(q_{f,d})$ at the begin of the analysis was assumed as 917 MJ/m^2, instead of 932.7 MJ/m^2. The used value was a bit smaller than the value used in the unprotected configuration, however it is believed that a difference of 1.62% will not compromise the results. The finite element analysis with SAFIR for the protected timber elements were more cumbersome than unprotect analysis due to the small time step needed for the presence of gypsum plasterboard. In this analysis a time step of one second was used, to achieve a good outcome, according to the program's user manual. Therefore only few percentages of additional fire loads were analysed for each iteration, namely 5% (G), 20% (E), 60% (C) and 100% (A). The analysis was stopped at the 5th iteration due to an excessive fire load which leaded to a too high time temperature curve. The temperature-time curve and the fire load for the 5th iteration have been computed but it was not possible to evaluate the respective charring depth due to temperatures exceeding the temperature range of material data.

7.3.1 Fire room

The fire room has been assumed as the 2nd room in the concept building designed in §6.1.3 (the biggest), therefore the room and windows dimensions are fixed. The dimensions used in OZone are shown in Figure 7-14. It is assumed that the fire starts in the aforementioned room (which is not a fire compartment), and only when failure of separating element will occur there will be fire propagation to the whole compartment. In this study only the charring depth of the structural walls in the fire room has been studied.

Two room configurations have been analysed, namely: the protected and unprotected room configuration. In the protected configuration a total thickness of 30 mm of gypsum plasterboard type F has been applied on the timber surfaces. Both the room configurations have a concrete layer of 5 cm thick on top of the floor. The exposed timber surface is represented by the ceiling area together with walls area without the windows area, and its value is 134.54 m^2.

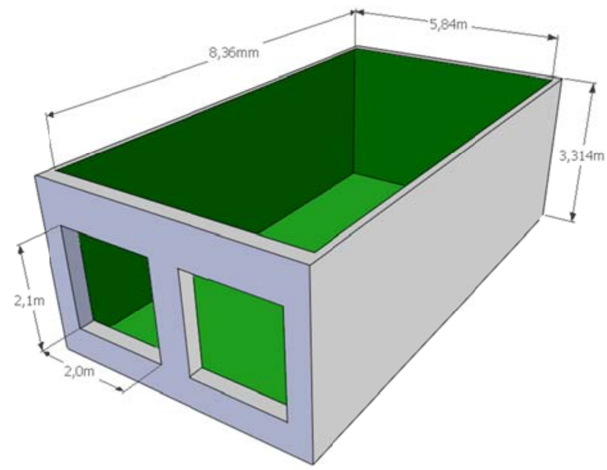


Figure 7-14: Fire room dimensions – 2nd room

Room dimensions :

$$b = 5.84\ m$$
$$l = 8.36\ m$$
$$h = 3.314\ m$$

Window dimensions :

$$b = 2.00\ m$$
$$h = 2.10\ m$$

According to Figure 7-14, the fire room is characterised by the following parameters.

Opening factor: $$\mathrm{O} = \frac{\mathrm{A}_v\sqrt{\mathrm{h}_{eq}}}{\mathrm{A}_t} = 0.0635\ m^{1/2}$$

Combustible surface: $$\mathrm{A}_{walls} - \mathrm{A}_v + \mathrm{A}_{ceiling} = 134.54\ m^2$$

7.3.2 b-factor

The values presented in Table 35 and in Table 37 are used for describing the timber and the gypsum behaviour respectively. These values are used in the finite element program SAFIR. With these temperature dependent values it will be possible to evaluate the real charring depth according to the fire exposure.

According to Eurocodes, the thermal properties of boundaries may be taken as temperature dependent, however the program does not allow changing the b-factor according to the temperature development, thus, as it has been done in (43), an averaged value over the range from 200°C to 800°C is determined. The obtained values are used as input data in OZone, and they are given in Table 32.

	Thermal conductivity, $\boldsymbol{\lambda}$ $[W/(mK)]$		Density, $\boldsymbol{\rho}$ $[kg/m^3]$		Specific heat, $\boldsymbol{c}$ $[J/(kgK)]$		$\boldsymbol{b} = \sqrt{\boldsymbol{\lambda\rho c}}$ $[J/(m^2K\sqrt{s})]$	
	Average	Average at $\mathbf{20}$°C[a]	Average	Average at $\mathbf{20}$°C[a]	Average	Average at $\mathbf{20}$°C[a]	Average	Average at $\mathbf{20}$°C[a]
Gypsum	0.16	0.24	640	460	1600	1000	350	430
Wood	0.14	0.14	240	470	1800	1700	240	330

[a]: Values from different sources at ambient temperature of 20°C

Table 32: Thermal properties of materials averaged over temperature range of $\mathbf{20-800}$°C and at normal temperature, from (43).

7.3.3 Fire length

While for notional fire curves the fire design time is given by the codes, for natural fire curve the entire length of the fire has to be considered. Indeed resistance times are set in 2.4(4) of EN 1991-1-2 and in EN 1995-1-2 clause 2.1.3, where it is stated that: “the load bearing function should be maintained during the complete duration of the fire including the decay phase, or a specific period of time”. Nominal temperature-time curves (standard, external, and hydrocarbon) do not include the cooling phase. Only natural fire models (compartment fires, localised fires and advanced fire model) take into account also the cooling stage. In this study, the “full duration of a fire” has been regarded as the maximum depth reached by the 100°C isotherm line.

7.3.4 Fire load density

The fire load density is determined as the ratio between the total fire load and the floor area, but in some fire models, such as the “Swedish curves” the fire load density is defined as the ration between the total fire load and the boundary surfaces. Since this study follows the formulas prescribed in Eurocodes, the former density is adopted.

$$q_{f,d} = \frac{Q_{f,d}}{A_{floor}} \tag{65}$$

The total fire load density for a combustible compartment is represented by the movable fire load, due to furniture, and by the relevant part of the structure which takes part to the fire. The former is given by codes (annex E of EN1991-1-2), while the latter is a bit more cumbersome to compute. Only the fire initial fire load is affected by the fire safety measures used. They are deeply explained in the next sections.

7.3.4.1 Movable fire load

The characteristic movable fire load density, $q_{f,k}$, for a dwelling occupancy is given in Table E.4 of EN 1991-1-2, its value is $948\ MJ/m^2$. This value represent the starting point of the analysis, indeed, it will determine the design fire load density which in turn will affect the temperature-time, then from the fire curve it can be computed the charred depth of the timber structure. According to annex E of EN 1991-1-2, the design movable fire load density is affected by the fire safety measures used, as shown below (formula E.1).

$$q_{f,d} = q_{f,k} \cdot m \cdot \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \tag{66}$$

Where:

$q_{f,k} =$ characteristic fire load density over floor area $= 948\ MJ/m^2$
$m =$ combustion factor $= 0.8$
$\delta_{q1} =$ Danger of fire activation (area related) $= 1.23$
$\delta_{q2} =$ Danger of fire activation (occupancy related) $= 1.0$
$\delta_n =$ function of active fire fighting measures used

In the next analysis two movable fire load densities are determined, according to the active fire fighting measures used. The fire load density is determined by the typology of fire fighting measures used, a high initial fire load density (HFLD) takes into account only the basic fire fighting measures ($\delta_{n8}, \delta_{n9}, \delta_{n10}$), while a low fire load density (LFLD) takes into account additional active fire fighting measures ($\delta_{n1}, \delta_{n4}, \delta_{n7}$). The fire safety measures used and their effect on the design fire load density is shown in the table below.

		LFLD	HFLD
δ_{n1}:	Automatic water extinguishing system:	0.61	1
δ_{n4}:	automatic detection by smoke:	0.73	1
δ_{n7}:	off-site fire brigade:	0.78	1
δ_{n8}:	Safe access route:	1	1
δ_{n9}:	Fire fighting devices:	1	1
δ_{n10}:	smoke exhaust system:	1	1
Active fire fighting measures factor (δ_n):	$\prod_{i=1}^{10} \delta_{ni} =$	0.3473	1
Design value of fire load density $[MJ/m^2]$:	$q_{f,d} =$	324	932.8

Table 33: Fire load densities studied

However, doubts may arise on the proposed value when sprinkler systems are used. Sprinkler system show a very good reliability, they may work or not work. In the case they work properly, even if they are design to slower the fire, they will extinguish the fire before it will reach flashover. If sprinkler systems fails, the outcome will be the same as they were not installed. Reducing the fire load density by approximately 40% is a probabilistic approach to account the reduced possibility of an out-of-control fire.

7.3.4.2 Relevant part of the structural element

All the combustible parts of the construction which char have to be taken into account when determining the additional fire load due to the structure. In this study, different percentages of the additional structural fire load have been evaluated, namely: 100%, 80%, 60%, 40%, 20%, 10%, and 5%.

Only one source has been founded in the literature which takes into account the additional fire load due to the combustible structure. For the calculation of the added fire load only the 50% of the wooden floor has been accounted. The walls were protected by two layers of gypsum plasterboard (F + A) and they were assumed non-participating to the fire. However the charred depth on the walls was measured after the fire test and its value was around 5 to $10\ mm$. This charred depth has been neglected. In the test report it is not explained why only the 50% of the wooden floor has been accounted or why the contribution of the walls has been neglected. The 50% percent of the total floor thickness represented 11% of total amount of fire load accounted (41).

The obtained additional fire load density is added to the initial fire load for each iteration. The structural fire load due to the charred timber structure is based on the d_{300}. It is assumed that all the boundary elements (apart the concrete floor) have the same charred depth. with this assumption the additional fire load can be computed with the following relation.

$$q_{f,d,structure} = d_{300} \cdot A_t \cdot \gamma \cdot H_{u,wet} \cdot \chi \cdot 10^{-3} = d_{300} \cdot 682.602144\ \mathrm{MJ}/mm \qquad (67)$$

Where:

$A_t =$ boundaries area, without openings and concrete floor $= 134.54\ m^2$

$\chi =$ combustion factor $= 0.8$

$\gamma =$ timber density $= 420\ \ kg/m^3$

$H_{u,wet} =$ net calorific value for wet wood $= 15.1\ \ \mathrm{MJ}/kg$

The $300℃$ isotherm line is derived by a FEM calculation (SAFIR) based on the temperature time curve obtained from OZone (Figure 7-15). The graphic processor DIAMOND gives a good user friendly interface which is needed to read the output file from SAFIR (Figure 7-16).

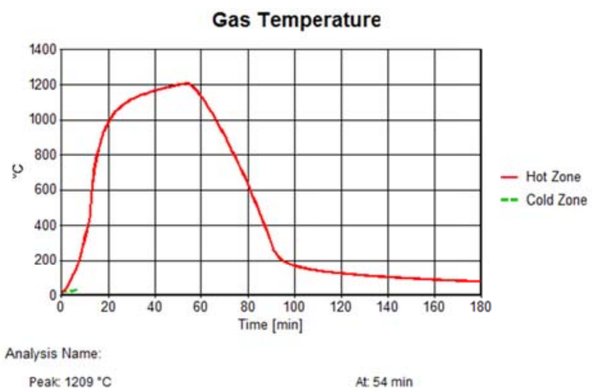


Figure 7-15: Fire analysis outcome obtained by OZone

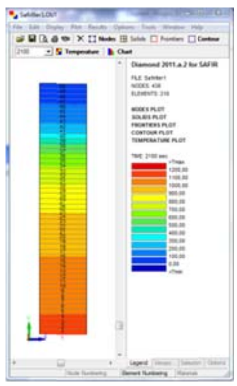


Figure 7-16: Thermal finite element analysis result, from SAFIR and shown by Diamond

7.3.5 Thermal properties for finite element analysis

In this section, a summary is given of all the thermal properties used in the finite thermal analysis. Different sources have been founded in literature.

7.3.5.1 Timber

General data related to CLT derived from the references studied.

Product name:	Cross Laminated Timber CLT, X-Lam		
Wood species:	Spruce		
Grade:	C 24 according to EN 338, higher strength class shall be produced		
Bonding adhesive:	Formaldehyde-free adhesive		
Moisture content:	12%		
Density:	According to EC5 $\rho_{mean} = 420\ kg/m^3$ (EN 338)		
Thermal conductivity:	$\lambda = 0.13\ W/(m\ K)$		
Specific heat:	$c_p = 1600\ J/(kg\ K)$		
Reaction to fire:	Euroclass D-s2, d0		
Resistance to fire:	Charring rate according to:		
	KLH product sheet:	$0.67\ mm/min$ for charring of top layer only	$0.76\ mm/min$ at charring of more layers
	EC-1995-1-2:	$\beta_n = 0.70\ mm/min$	$\beta_0 = 0.65\ mm/min$ *(used in the analysis)*

Table 34: Timber general data

Thermal calculations are carried according to the conductive model, based on the heat transfer differential equation. This model uses temperature dependent values proposed in the EC5 for thermal conductivity, specific heat and density. They are shown in the next table. The material WOODEC5 defined in SAFIR is based on these values.

Temperature (°C)	Conductivity (λ)	Specific heat (c_p)	Density (ρ)
20	0.120	1530	500.00
99	0.133	1770	500.00
99	0.133	13600	500.00
120	0.136	13580	447.00
120	0.136	2120	447.00
200	0.150	2000	447.00
250	0.123	1620	415.71
300	0.097	710	339.72
350	0.070	850	232.44
400	0.077	1000	169.86
500	0.090	1200	147.51
600	0.176	1400	125.16
800	0.350	1650	116.22
1200	1.500	1650	0.00

Table 35: Timber material properties

It is important to understand that those values are apparent values in order to take into account increased heat transfer due to shrinkage crack above 500°C, the increase in heat transfer is due to convection and radiation. Specific heat capacity values also include the energy necessary to evaporate the water, which explains the presence of higher values between 99 and 120°C (87). The values given above shall be used only for fire exposure related to fire curves in EN 1991-1-2 (standard or parametric). Different thermal histories from the prescribed values from EC1 could not be accurately predicted because timber behaviour on fire is strongly related to the maximum temperature reached and to the temperature development. During the decay phase, glowing combustion of char layer causing higher surfaces temperatures than the gas temperature in the compartment has an effect on the temperature development in the timber element (88). Different specific heat values should be used when the water content of the timber specimens is different from 12%, as proposed in (89).

7.3.5.2 Gypsum

There are different qualities of gypsum panels commonly used for linings, and they have major differences. In this section only gypsum panels used for fire protection are studied, they are:

- Gypsum Plasterboard (or GP) Type A: they are regular common boards and contain a porous gypsum core with no reinforcement except the paper-laminated surface. They are usually applied behind a fire resistant plasterboard;
- Gypsum Plasterboard (or GP) Type F: they show improved core cohesion at high temperatures by adding other materials to the core such as glass fibres and fillers. Due to these added materials type F gypsum panels will maintain integrity longer than type A. Gypsum plasterboards of type X commonly used in North America also have improved core cohesion at high temperatures and may be considered similar to gypsum plasterboards of type F.
- Gypsum Fibreboard (or GF): boards with added cellulose fibres in the core to improve core cohesion at high temperatures, they usually have a higher density in comparison to gypsum plasterboards of type A and F (Table 36) and may even exhibit a better fire performance than gypsum plasterboards of type F.

Thermal properties of gypsum panels have been extensively studied in different sources founded in literature. Below the main consideration are given (according to (90)).

- The length of the horizontal plateau at about $100℃$ was more or less the same for all the boards studied (GP-F, GP-A, GF), it was concluded then the water content in each board was about the same.
- The addition of fibres and fillers to the gypsum core did not significantly change the thermal behaviour of the boards. However, the reinforcement of the gypsum core with fibres and fillers generally improves the stability and the mechanical properties (shrinkage, cracking, ablation, falling-off) of the boards after complete dehydration.
- No significant difference was observed with regard to the thermal behaviour between gypsum fibreboards (higher density) and gypsum plasterboards. Thus, the density does not seem to be a relevant parameter to describe the thermal behaviour of gypsum boards.
- The main parameter is the water content and therefore the thickness of the board.

The last point has been recognized also by Buchanan (86). Gypsum plasterboard is a good construction material for fire-resisting protection because there is approximately 21% by weight of chemically bound water in gypsum crystals. Thus, a large amount of heat is absorbed by gypsum board during dehydration (horizontal plateau), which slows down the heat transfer from the fire to wood studs or joists.

One of the main difficulties encountered during this work was to determine gypsum thermal properties to use in the finite thermal analysis. Different sources have been founded in literature with a considerable scatter, the values founded are summarized in the Table 36 below.

		Properties at $20℃$		
		Density	Thermal conductivity	Specific heat
Source	Product name	ρ $[\ kg/m^3]$	λ $[W/(m\ K)]$	c_p $[J/(kg\ K)]$
EN 12524	Gypsum plasterboard	930	0.25	1000
EN 12524	Gypsum insulating board	600	0.18	1000
T. Hakkarainen, (43)	Gypsum plasterboard	760	0.24	1000
A. Frangi et al., (90)	Gypsum plasterboard F	853	–	–
A. Frangi et al., (90)	Gypsum plasterboard A	908	–	–
A. Frangi et al., (90)	Gypsum fibreboard	1186	–	–
SAFIR	C_GYPSUM	732	–	–
SAFIR	X_GYPSUM	648	–	–
Knauf[22]	Gypsum A	760	0.20	–
Knauf	Gypsum F	800	0.20	–
Knauf	Fibreboard	~767	0.23	–

Table 36: Scatter in gypsum properties

In the analysis below, the data founded in (11) and given in Table 37 are used because they are more accurate and temperature dependent. The values given in Table 37 are effective values, therefore they already take into account the effect of mass transfer and cracks formation.

[22] http://www.knaufinsulation.com/en

Temp. $[\,°\mathrm{C}]$	*Thermal conductivity* $[W/(m\,K)]$	*specific heat* $[J/(kg\,K)]$	*density* $[\,kg/m^3]$
0	0.40	960	825.0
20	0.40	960	825.0
70	0.27	960	825.0
100	0.13	960	825.0
130	**0.13**	**14900**	**764.0**
140	0.13	25200	744.2
150	0.13	21700	723.5
170	0.13	960	683.1
600	0.13	960	682.3
720	0.33	4359	681.5
750	0.38	960	640.2
1000	0.80	960	640.2
1200	2.37	960	640.2

Table 37: Temperature-dependent thermal properties of gypsum fibreboard, from (11)

The dry density of gypsum is assumed as the value at $130°\mathrm{C}$, with this assumption, the water content of the board is $61\ kg$. In a prescriptive approach, tabulated values shall be used to ensuring good fire protection. For instance, a minimum required thickness is given in (91) and it is shown below. According to those values there is no difference between gypsum plasterboard and gypsum fibreboard, as determined also in (90).

Fire resistance requirement	*Lining material*	*Lining thickness required* $[mm]$ *CLT* $> 85\ mm$	*CLT* $> 115\ mm$
$30\ min$	Gypsum plasterboard	12.5	9.5
	Gypsum fibreboard	10	10
$60\ min$	Gypsum plasterboard	20	15
	Gypsum fibreboard	20	15
$90\ min$	Gypsum plasterboard	15+15	15+15
	Gypsum fibreboard	15+15	15+15

Table 38: Required thickness for a given resistance class

7.4 Validation of the model

Before proceeding with finite thermal analyses, the material properties have being checked in order to ensure a reasonable outcome. Thermal analysis are carried out with the finite element program SAFIR developed at the University of Liege for the simulation of the behaviour of building structures subjected to fire. To validate the material behaviour, the outcomes from the SAFIR analysis were compared with the real test results cited in (92) and obtained in (93) In the series of fire tests spruce timber specimens were subjected to the standard fire curve, ISO-834, to one side only. The specimens had a moisture content of about 12%. Temperatures were measured in a depth of $6, 18, 30$ and $42\ mm$ from the surface exposed to fire.

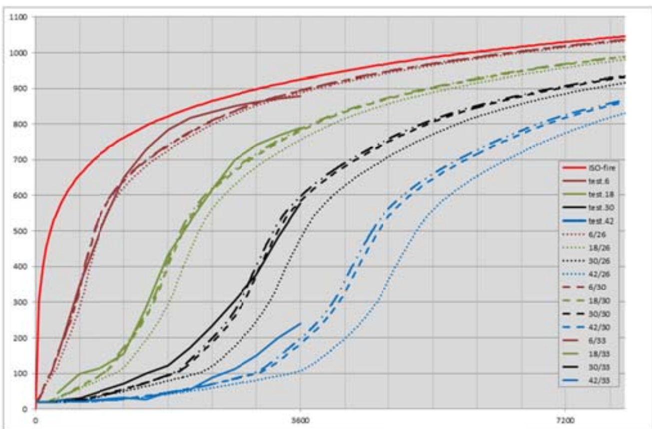
7.4.1 Timber model

Three material properties were compared to evaluate which gives the closest fit to the real tests, they are shown below. All the material behaviours are temperature dependent. Validating the model, a discrepancy in the user manual of SAFIR was founded. According to the manual the moisture content for a user defined material properties, should be given in kg/m^3, however as can be seen from Table 39 and Figure 7-17 below, the curve determined according to the manual (curve “test 26”) did not represent well the real test results. Curve 26 was not even close to curve “test 33”, which in theory has the same material properties. Changing the moisture content form kg/m^3 to a percentage value, it is achieved a closer fit with the experiment (curve “test 33”).

	test 33	test 26	test 30
Timber properties	WOODEC5	USER defined	USER defined
	timber properties in accordance with Eurocode 5 already present in SAFI	w in accordance with SAFIR thermal user guide	w guessed by the author
water content, w	12 %	53 kg/m^3	12 %
line	dashed – dotted	dotted	dashed

Table 39: Different timber thermal properties used for validate the model

It is important to note that the material properties from test 33, test 26 and test 30 were exactly the same, the only difference was the moisture content value. According to SAFIR manual the moisture content for the already known material WOODEC5 must be in percentage, while for an used defined material the moisture content should be introduced in kg/m^3. The comparison between the finite analysis and the test results is given in Figure 7-17.



Time [°C] **Figure 7-17: Validating the timber model**

Although the tests 30 and 33 gives good agreement with the real tests, there is a strong limitation that is present in the use of material data given from EN 1995. The Eurocode's values shall be used only with fire exposure to a standard or to a parametric fire curve, because the charring behaviour of wood it is strongly dependent on temperature development and maximum value reached. Being aware of this limitation, the material WOODEC5 defined in SAFIR will be used during this study. It will be used also with fire exposures different from the standard fire curve, because thermal properties of wood exposed to different fire curves do not exist in literature.

7.4.2 Gypsum plasterboard

Konig et al. performed a large amount of tests regarding timber frame members covered by one or two layers of standard or fire resistant gypsum plasterboards. All the experiments are carefully described in (94). Mineral wool was used to fill the cavities between the studs, only gypsum layers were shielding the timber studs from fire exposure. Specimens were exposed to a standard fire curve.

The following features were examined:

1. Joint configurations of linings, namely the location of the gap between two different panels;
2. Protective ability of regular gypsum plasterboard type A, called GN;
3. Protection ability of gypsum plasterboard type F, with improved resistance at high temperatures, called GF;

The studied studs had dimensions of $45\text{x}145\ mm$. In the figures below are shown the graphs for different linings configuration. When one single layer of gypsum plasterboard was used, the resistance time was $20\ min$ for GN, and $28\ min$ for GF. If two layers ware used there was a greater protection time and the charring started at $63\ min$. It is reasonable to think that due to the small dimensions of the stud and the presence of cavities, a CLT wall protected with the same lining material will achieve better charring rate.

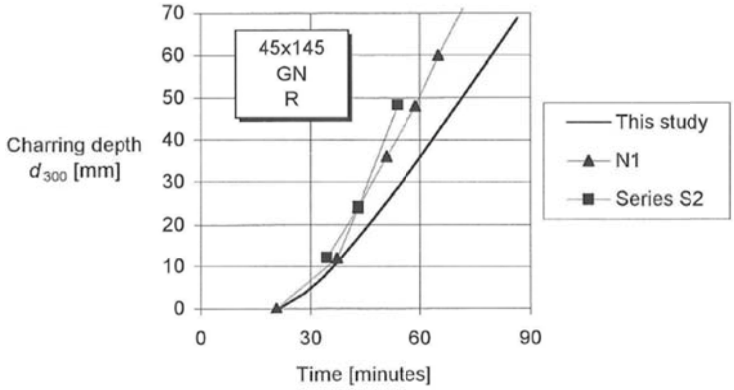


Figure 7-18: Charring depth versus time - Comparison of calculated values with test results for one layer of gypsum plasterboard GN, failure time 20 min – From (94)

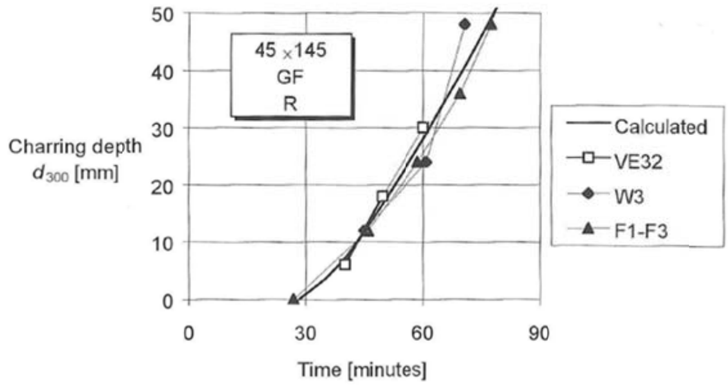


Figure 7-19: Charring depth versus time - Comparison of calculated values with test results for one layer of gypsum plasterboard GF, failure time 28 min - From (94)

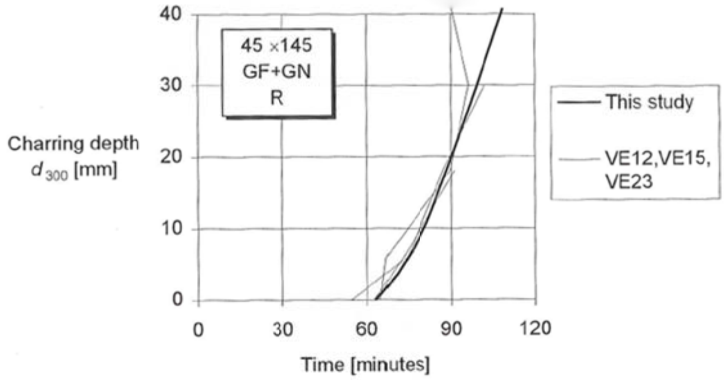


Figure 7-20: Charring depth versus time - Comparison of calculated values with test results for two layers of gypsum plasterboard GN+GF, failure time 64 min – From (94)

7.5 Results

In this section, all the results obtained by the finite thermal analysis are shown. It is important to note that since several software are used in the analysis (OZone, SAFIR and DIAMOND) the analysis outcome data are extremely voluminous, therefore only the final interesting values (isotherm depths) are extrapolated and plotted in the graphs below. Furthermore the results for each iteration are given as two tabulated text files, the first one, obtained by OZone is a fire curve, the second one is the SAFIR outcome which has to be open with a graphic processor (DIAMOND).

7.5.1 Different percentage of additional fire load

Different percentages of additional fire load due to the combustible structure have been studied. In the Eurocodes and in literature no reference has been founded on how consider the relevant part of timber structures exposed to fire action. In this section different percentages of additional fire load were exanimated, for each iteration the fire load due to the structure was accounted for 5%, 10%, 20%, 40%, 60%, 80% and 100%. The initial fire load density is taken as $932.8\ MJ/m^2$, with this initial fire load, a charred depth is obtained. The additional fire load due to the former iteration is determined from the charred depth. This value is then accounted in the total fire load density in the next iteration depending on the percentage value studied.

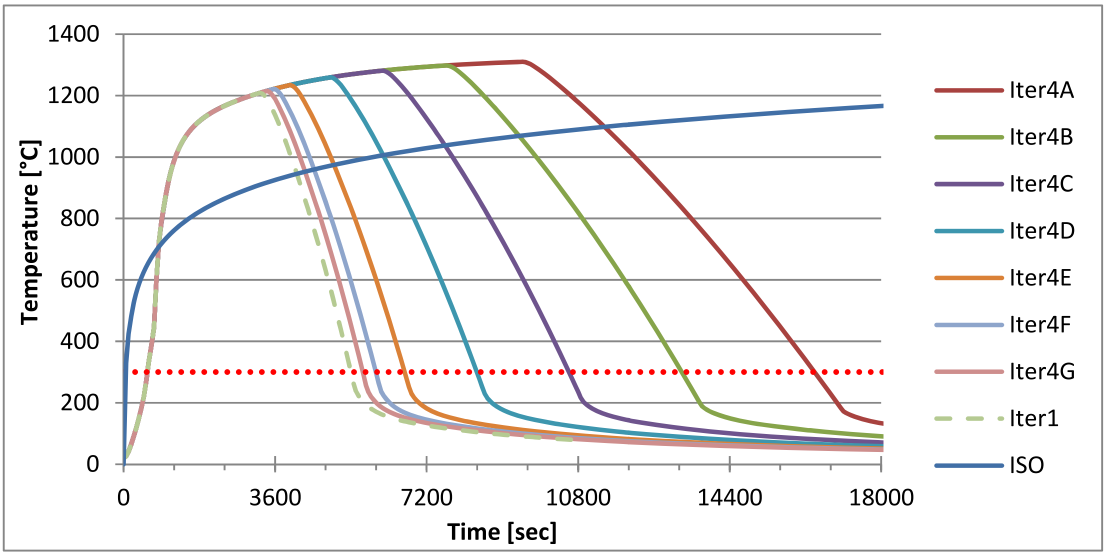


Figure 7-21: Time temperature curves for the 1st and for the 4th iterations

As depicted in Figure 7-21, if the complete amount of the charred depth is accounted in the total fire load after 4 iterations, it will lead to very long fire periods, while if the 5% is accounted (curve Iter4G) the fire curve is close to the original one. To compare the fire severity of the curves above, it is assumed that the fire severity is function of the area between the fire curve and the 300°C isotherm, multiplied by the maximum temperature reached. This is a rough method which is based to the equivalent fire severity model proposed by Ingberg (1928) and modified by the author. The comparison is given in Figure 7-22.

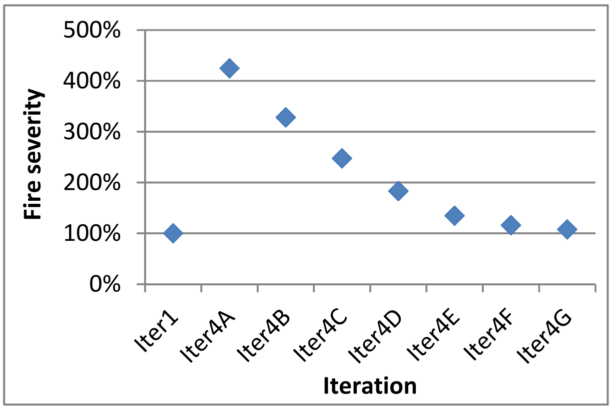


Figure 7-22: Fire severity

The adjacent graph shows the equivalent fire severity of the 4th iteration for the unprotect fire room with HFLD. The amount of relevant combustible structure plays a major role in determining the natural temperature-time curve.

In the horizontal axis the capital letters represent the amount of additional fire load taken into account, from A = 100% to G= 5%.

7.5.2 High and low initial fire loads

Two natural fire scenarios were evaluated with two different movable design fire loads, 932.8 MJ/m^2 and 328.7 MJ/m^2. This analysis has been carried out only for the unprotect configuration and its aim is to evaluate if there is any difference in the number of iterations needed to converge to a steady value for the relevant structural fire load.

In the HFLD configuration, due to the large amount of charred structural timber, the fire load density increases sharply and it affects the maximum number of iterations that could be done. In fact the analysis had been stopped at the 4th iteration because the obtained total fire load density produced a temperature-time curve with too high temperatures to be taken into account with the defined material properties.

The LFLD configuration has a percentage increase, related to the charred depth due to the initial fire load density, much higher than the HFLD configuration, this is shown in Figure 7-23. This means that also if the LFLD has a lower charred depth compared to the HFLD, it will converge to a steady value slower than the HFLD. This can be explained by the amount of additional fire load determined on each iteration. In the LFLD this value is soon comparable with the initial fire load density, this is shown in Figure 7-24 where are shown the total fire load densities for each iterations for both the configurations (in percentage value related to the initial fire load).

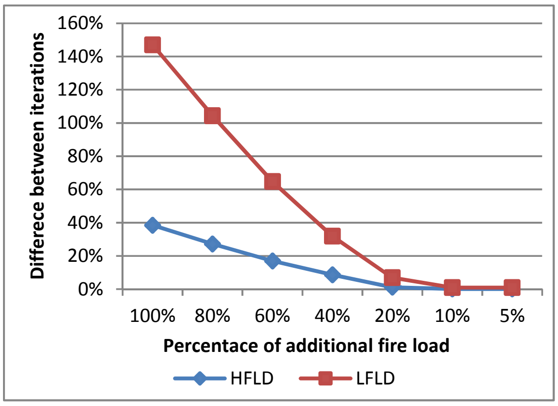


Figure 7-23: Scatter of charred depths between 2nd and 3rd iteration for LFLD and HFLD

As can be seen from Figure 7-23, the low fire load density configuration has a charred depth increment more than 140% between the 2nd and the 3rd iteration (the percentage value is related to the charred depth due to initial fire load density).

The high fire load density configuration, instead, has a charred depth increment less than 40% between the 2nd and the 3rd iteration (the percentage value is related to the charred depth due to initial fire load density).

If only the 20% of the charred depth is accounted in the next iterations, a convergence is obtained approximately at the 3rd iteration.

As shown in Figure 7-24, the HFLD converges faster to a stable value than a LFLD, this can be explained by relative increase in fire load density. An increase of a certain fire load on both, low and high fire densities, will produce a bigger effect on the outcomes from the low fire density since in percentage values is bigger.

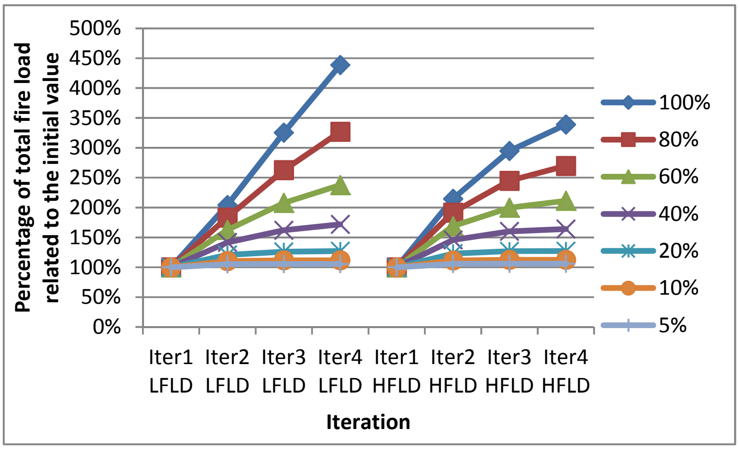


Figure 7-24: Total fire load per each iteration

In Figure 7-24 the total fire load is plotted for the High Fire Load Density and for the Low Fire Load Density, please mind that the percentage values are referred to the initial fire load for each configuration ($932.8\ MJ/m^2$ and $328.7\ MJ/m^2$).

As also shown in Figure 7-23, the HFLD converges faster to a steady fire load.

In (41), the total fire load taking into account the additional fire load due to the structure was 1.11%.

The percentage values on the right refers to the amount of additional fire load accounted in each iteration.

7.5.2.1 Differences with prescriptive codes

From a comparison of the charred depths (obtained with FEM analysis) and the char front proposed by the codes, it can be seen that the prescriptive proposed value is not always on the safe side. In Figure 7-25, the charred depths according to natural fire exposures above (performance based codes) for low and high fire load densities are shown together with the charring depth related to an ISO fire exposure (prescriptive codes) of 120 min and 210 min. The charred depths for natural fire exposures presented in Figure 7-25 are determined for a complete burnout of combustible material.

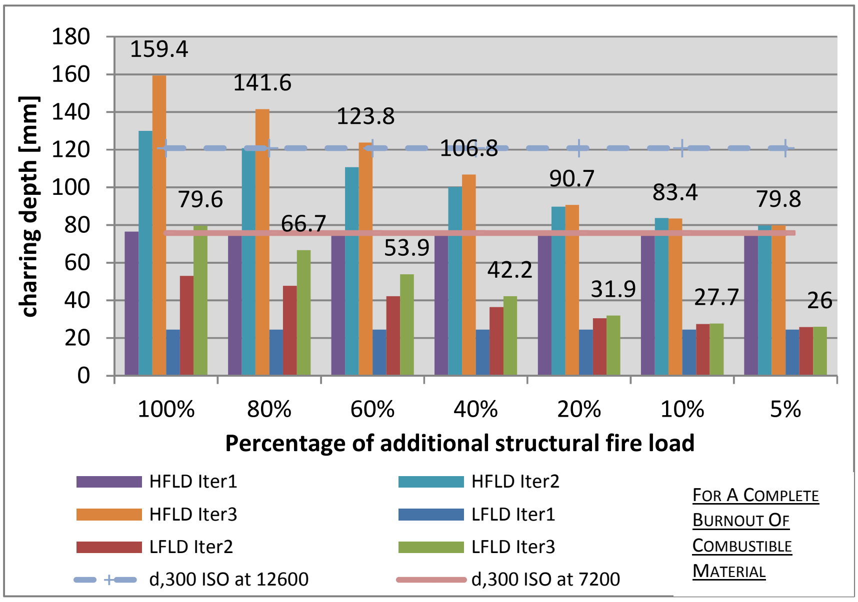
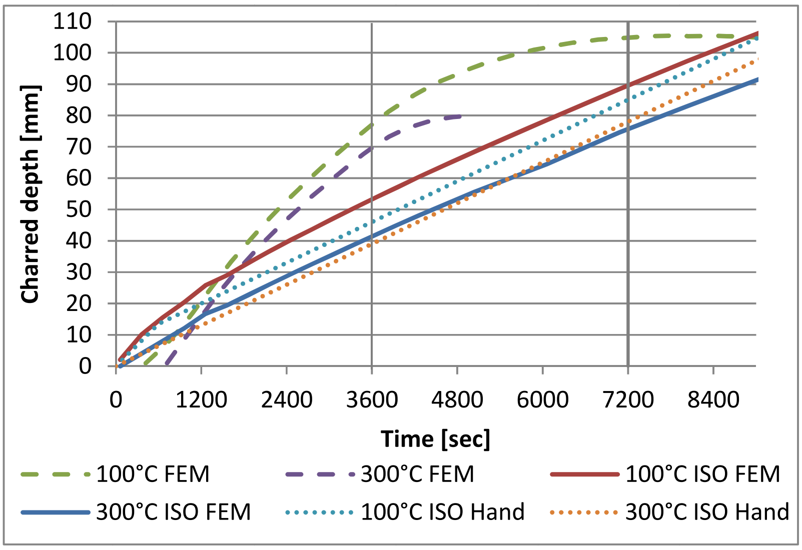


Figure 7-25: Charring depths compared for HFLD and LFLD - Unprotect

In the HFLD Iter3 with 100% of additional fire load taken into account, the maximum charred depth is 159 mm and it is reached at 12600 sec, while the maximum peak temperature in the fire room is reached at 8340 sec. This behaviour has a great effect when the char front is determined before a complete burnout of combustible material, leading to an underestimation of the charred depth.

If a building with structural timber in sight is designed according to a fire resistance time of 120 min (pink solid line), the charred front is underestimated for HFLD, on the other hand it is too conservative for LFLD. The charring depth relative to the ISO fire curve has been computed by finite element analysis with SAFIR, and its value is 75.8 mm, while for the zero strength layer is 89.5 mm.

For HFLD with large percentage of additional fire load accounted for each iteration, the charring depth from the ISO fire exposure is greatly exceeded. However there are no guidance on how much combustible material should be taken into account.

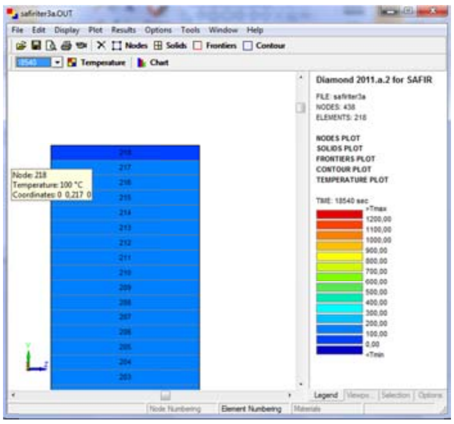


In the graph beside, the depth for the 300°C and the 100°C isotherms are given for different fire exposures.

Natural fire curve, HFLD-Iter3G (dashed lines), ISO fire exposure computed by finite element analysis (solid lines), and for an ISO fire exposure computed by formulas in EN1995-1-2 (dotted lines).

Figure 7-26: Charring rates for unprotect configuration

As can be seen from Figure 7-26, the values of charred depth and the zero strength layer determined by prescriptive codes are underestimating the values obtained by advanced calculation methods with a natural fire curve with a load fire density of $988.54\ MJ/m^2$ (Iter3G). However in the iteration Iter3A-HFLD (with a fire load density of $2750\ MJ/m^2$), for a complete burnout of combustible material, the 100°C isotherm was still propagating inward at 5 hours and its depth was deeper than $218\ mm$ (Figure 7-27), it was not possible to establish the actual depth due to the limitation in the finite element program.

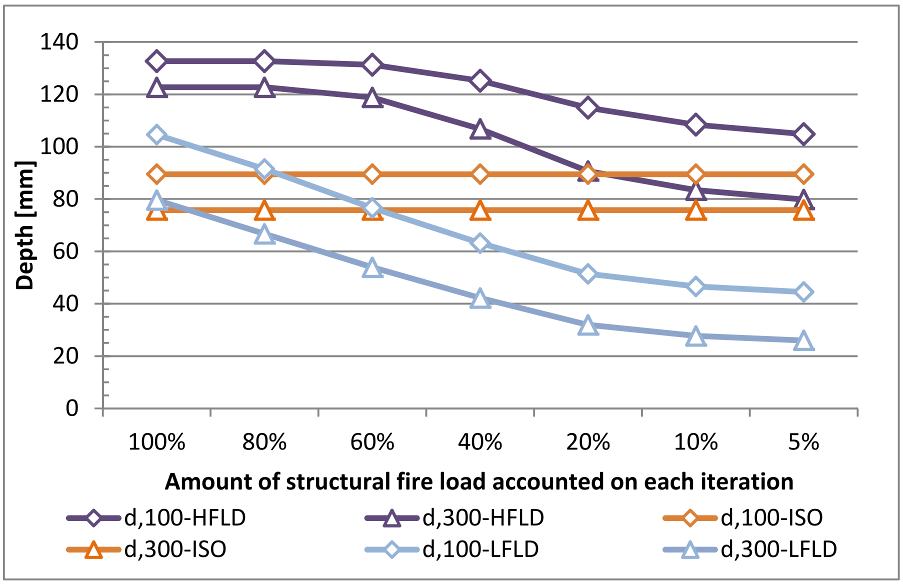


Depth of the 100°C isotherm according to finite thermal analysis for the high initial fire load density with an unprotected fire compartment.

It worth noting that the fire curve has its maximum temperature of approximately 1300°C at 8340 sec (2h19min).

Figure 7-27: Time step of Iter3A - HFLD unprotected

In Figure 7-28, the depth of the char front (300°C isotherm) and the depth of the zero strength layer (100°C isotherm) are shown. It is important to note that those depths are the maximum depths reached before a complete burnout. The values in Figure 7-28 are obtained for a fire exposure of $120\ min$ to natural fire exposures and to the ISO fire curve.



In the graph beside, the depth for the 300°C and the 100°C isotherms are given for three fire exposures, namely: the ISO fire curve (orange lines), the High Fire Load Density (violet lines), and the Low Fire Load Density (sky-blue lines). All these values are obtained at 120 min. As it is shown, for the highest fire load density, the distance between the violet lines is reducing. It is due to the fire period and it is explained in more details below.

Figure 7-28: Calculated depths for HFLD and LFLD, at 120 min - unprotect

From Figure 7-28 two main conclusions can be drawn:

1. The ISO fire exposure gives a best fit for two iterations: the HFLD with 5% of additional fire load, and the LFLD with 100% of additional fire load. Their value is around $1000\ MJ/m^2$. Please refer to Figure 7-29.
2. The distance between the char front and the zero strength value is reducing for the analysis with the highest fire load densities. If the fire resistance time, is before the beginning of the decay stage, the two line will be closer, otherwise, the zero strength layer will move inwards also after the 300°C isotherm has reach its maximum value.

As shown in Figure 7-28, the 300°C and the 100°C isotherms for the natural fire exposures are not well represented by the values obtained by the ISO fire curve. The ISO fire curve underestimate the charred depth for high fire loads, while for low fire loads it is too conservative. This behaviour is better represented in Figure 7-29, where is presented the ratio expressed in equation (68). The ISO charred depth gives the best fit for fire load densities around $1000\ MJ/m^2$.

$$\frac{Natural\ fire\ char - ISO\ char}{ISO\ char} \tag{68}$$

It is also important to note that in Figure 7-28 the distance between the 300°C and the 100°C isotherms is increasing with the fire load, as shown by the sky-blue lines, this is shown also in Figure 7-30 by the two dashed lines. However, when the fire load is increasing significantly the gap between the two violet lines in reducing, this can be explained as follows: at 120 min for greater fire loads the peak room temperature is not reached yet therefore the isotherms are still in the growing stage depicted in Figure 7-30 (before the maximum fire room temperature).

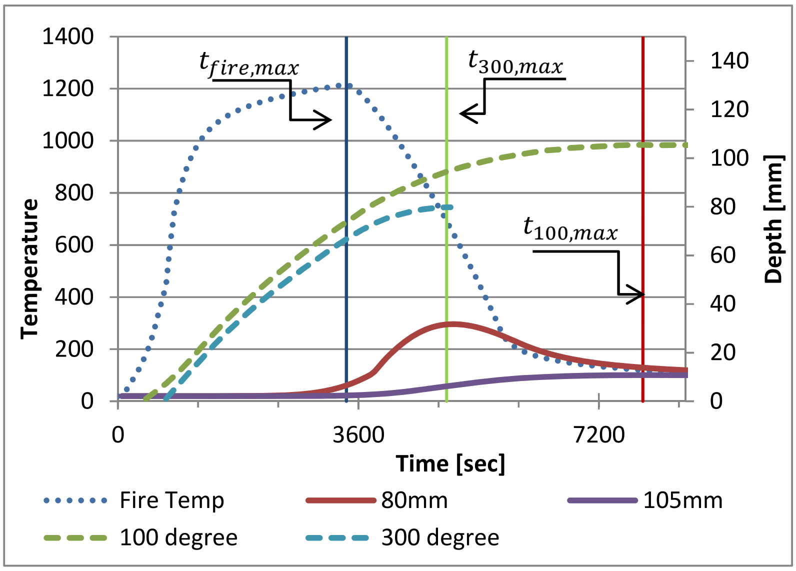
Figure 7-29: differences between charred depths in Figure 7-28

In Figure 7-29, it is shown the difference between the charred depths for a natural fire exposure limited at 120 min and the charred depth for the ISO fire exposure (120 min).

As can be seen, for values around 1000 MJ/m^2 the ISO fire curve has a good approximation.

For values smaller than 1000 MJ/m^2 the ISO value is overestimating the charred depth, while for higher values it is underestimating the charred depth.

In Figure 7-28 for the HFLD the distance between the 300 and the 100°C isotherms is reducing with the increasing of the additional fire load and in turn, the total fire load. This can be explained by looking at the time when the fire reaches the max temperature. According to natural fire curves, the maximum 300°C isotherm depth occurs with a certain time lag from the maximum temperature in the fire room, and the maximum 100°C isotherm depth occurs with a certain time lag from the maximum 300°C isotherm depth. For instance in HFLD Iter3G, the maximum peak temperature in the fire room is reached at 3420 sec, while the maximum charred depth is reached at 4980 sec and the maximum depth of the zero strength layer is reached at 7860 sec. This means that the char front and the unaffected timber section are still affected by the fire even after the decay stage. In Figure 7-30 this behaviour is shown for the HFLD fire exposure Iter3G.



In Figure 7-30 the results from Iter3G with the HFLD are shown.

From this graph it is possible to see the time lag between the maximum fire room temperature, the time when the maximum charred depth is reached, and the time when the maximum depth of the 100°C isotherm is met.

It is also important to note that the distance between the two depths is increasing in the decay stage of the fire.

Legend:

Solid line = temperatures at given depth;

Dotted lines = fire room temperature

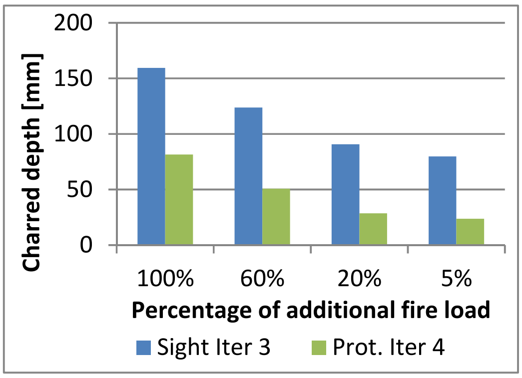
Dashed lines = depth of isotherms

Figure 7-30: Time lag between the maximum charred depth and zero strength layer (unprotected configuration)

The maximum peak temperature in the fire room occurs at: 3420 sec and its value is: 1215°C
The maximum 300°C isotherm depth occurs at: 4980 sec and its value is: 79.8 mm
The maximum 100°C isotherm depth occurs at: 7860 sec and its value is: 105.5 mm

7.5.3 Protected and unprotected configuration

In this section the effect of non-combustible lining materials is evaluated. The protected configuration has a design fire load density of $917.9\ MJ/m^2$, while the unprotected fire room had a fire load density of $932.8\ MJ/m^2$. The thermal FEM analysis carried out showed that in the protected configuration the amount of charred timber after a complete burnout of the combustible material was highly reduced. This is shown in Figure 7-31.

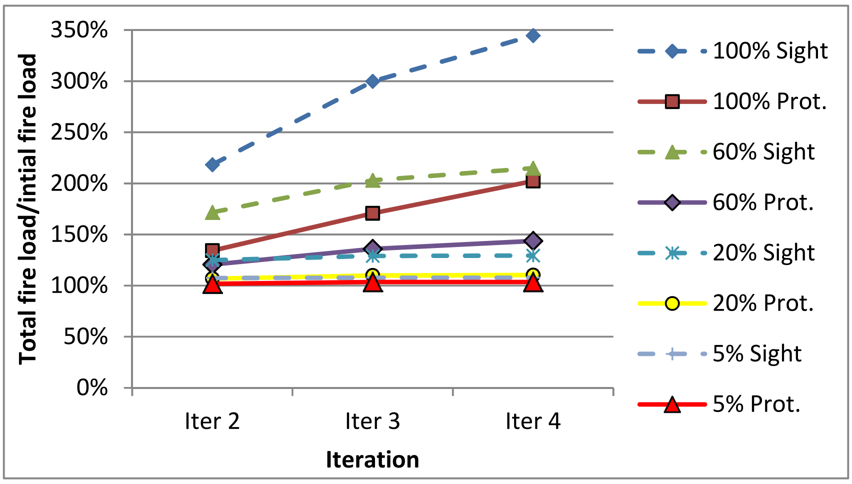


In Figure 7-31, it is shown the charred depths for the in-sight (blue) and protected (green) element.

As can be seen, the protected configuration has the lower charred depth.

Figure 7-31: Charred depth difference

Figure 7-32, below, shows the percentage increase for each iteration, where the in-sight analysis is depicted as dashed lines, while the protected analysis is described by solid lines. The slope of the different curves (for the same percentage of additional fire load taken into account) is almost identical, however the curves differ in the amount of charred depth.

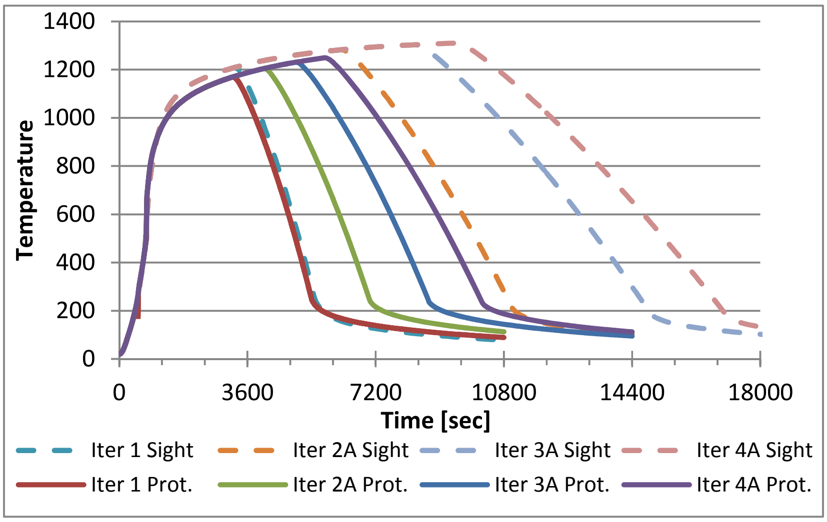


In Figure 7-32, it is shown the total fire load per iteration for different percentage of additional fire load.

As can be seen, the two configurations studied converge to a steady value with the same rate, however the protected configuration has a low amount of total fire load which makes it possible to carry out more iteration.

Figure 7-32: Total fire load of protected and unprotected configuration

The encapsulation layer affects the both, maximum fire temperature and the fire period, as depicted in Figure 7-33, however due to the reduction of the combustible fire load, the fire period is most affected.



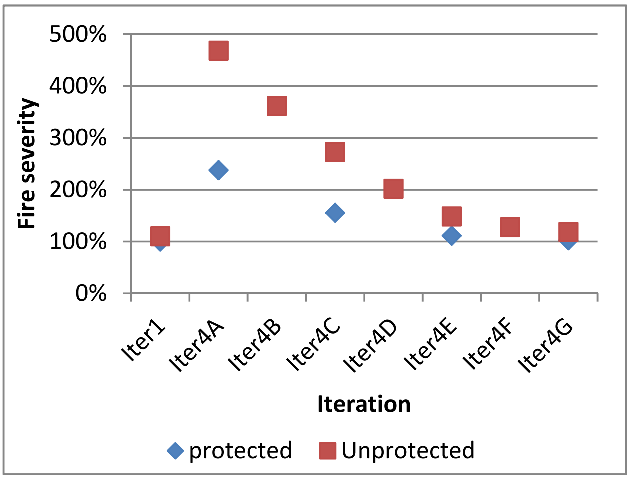
In the figure beside the different fire curves for the protected (solid lines) and unprotected (dashed lines) configuration are shown.

The curves represent different iterations for the additional fire load value of 100%.

Please mind that the different height in the growing stage of the fire is due to the different initial fire load.

Figure 7-33: Time temperature curves for 100% of additional fire load

In order to have a better estimation of the differences in the temperature-time curves, the fire severity for the room setup with and without the combustible wall and ceiling linings with a high initial fire load density is shown in the next graph. The fire severity of the unprotected configuration is almost twice as the protected fire room.



The adjacent graph shows the equivalent fire severity for protected and for unprotected fire room with a high fire load density.

For high fire loads, the encapsulating layer roughly reduces the fire severity by half.

In the horizontal axis the capital letters represent the amount of additional fire load taken into account, from A = 100% to G= 5%.

Figure 7-34: Summary of fire severities

The main advantage of encapsulating the combustible timber element is represented by the reduction of the charred depth and in turn, the reduction of the fire load which consecutively affects the fire room temperature. As already done for the unprotected configuration (Figure 7-30), in the following Figure 7-35 are described the maximum fire room temperature, the temperatures at 24 mm and 49 mm depth, and the charred depths for two isotherms (300°C and 100°C).

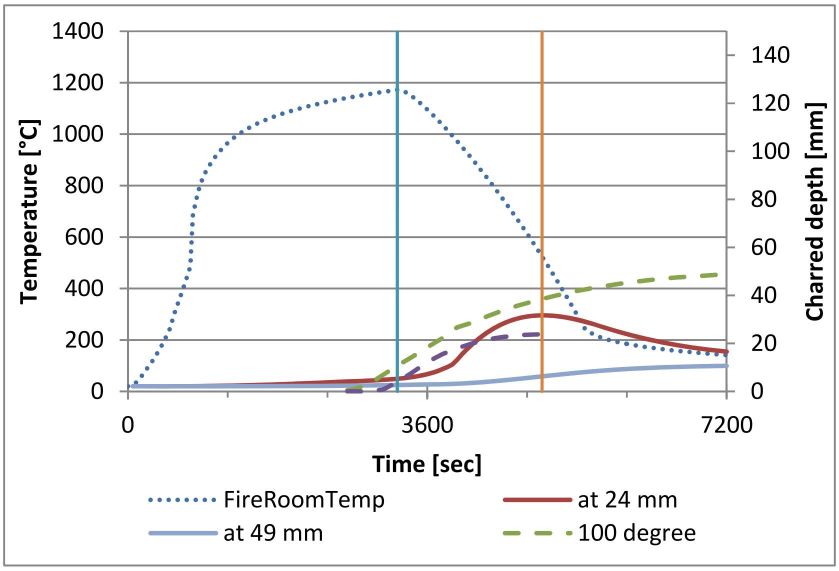


Figure 7-35 beside shows the values obtained from Iter4G (HFLD - protected configuration).

The zero strength layer at the end of the analysis (7200 sec) was still penetrating inward the element and therefore was not possible to determine the unaffected timber section.

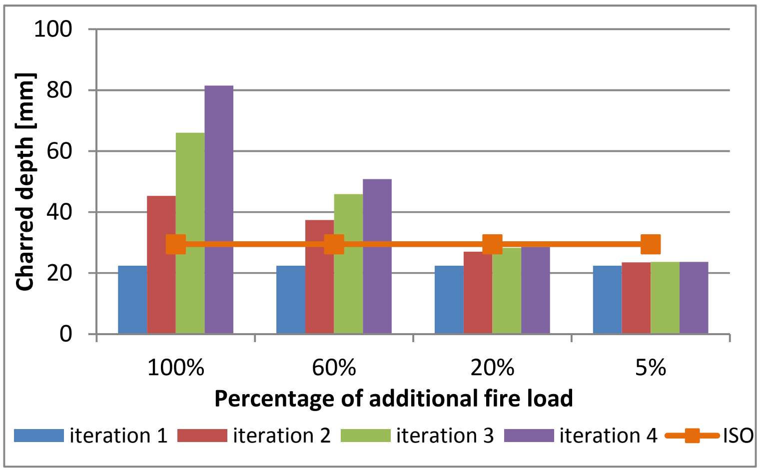
Mind that the vertical axes are in agreement with Figure 7-30.

Figure 7-35: Time lag between the maximum fire room temperature and the charred depth (protected configuration)

The maximum peak temperature in the fire room occurs at: 3240 sec and its value is: 1172°C
The maximum 300°C isotherm depth occurs at: 4980 sec and its value is: 23.8 mm
The maximum 100°C isotherm depth occurs later than: 7200 sec and its value is deeper than: 48.6 mm

7.5.3.1 Differences with prescriptive codes

The results obtained from the natural fire exposures to the HFLD in the protected configuration were compared with the results from prescriptive codes (ISO fire exposure for a fire period of 120 min). Both the values are derived by thermal FEM analysis.

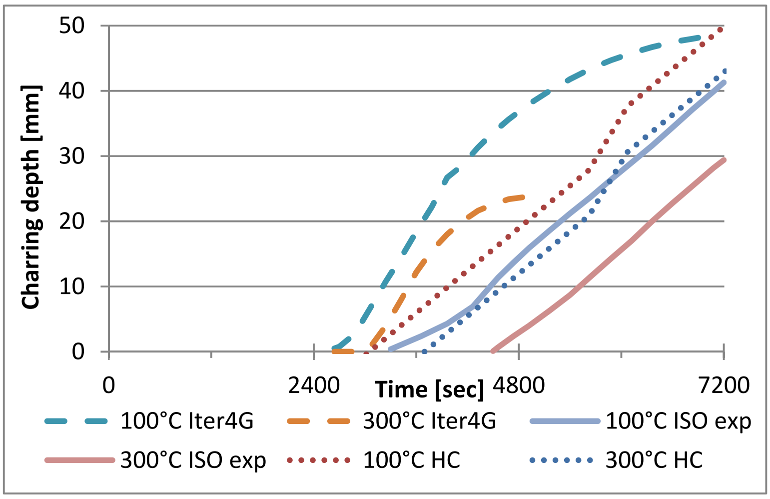


The figure beside shows the charred depths according to the ISO fire exposure (solid orange line) and to a HFLD natural fire exposures (with different amount of additional fire load) represented by columns.

All the valued are determined by FEM analysis.

Figure 7-36: Charred depths compared - protected

As described in Figure 7-37, below, the charred depth for a natural fire exposure differs from the value determined by the ISO fire exposure. The maximum depth for the 300°C isotherm, according to the fire curve Iter4G (fourth iteration, 5% of additional structural fire load), is about 23.8 mm, and it is reached at 4980 sec, while according to the standard fire exposure at 120 min, the charred depth is 29.5 mm. There is not too much scatter between those values, however if the unaffected timber depth are compared the scatter may be relevant. At 7200 sec, the zero strength layer for the standard fire curve is located at 41.3 mm, for the natural fire exposure, the $100°C$ isotherm is still moving inward and at 7080 sec its depth is $48.6\ mm$. This behaviour is show in Figure 7-37, where the charring depths for the ISO fire exposure and the Iter4G fire curve are shown.



In the figure beside the 100°C and 300°C isotherms are shown for a natural fire exposure, according to natural fire curve, Iter4G (dashed lines), for an ISO fire exposure computed by finite element analysis (solid lines), and for an ISO fire exposure computed by formulas in EN1995-1-2 (dotted lines).

Figure 7-37: Charring depths compared - protected

Figure 7-37 shows the depth for the $300°C$ and $100°C$ isotherms for different calculation methods. It is important to note that only the dotted lines (according to formulas for hand calculations) and the solid lines (finite thermal analysis with the ISO fire curve) are independent from the fire properties, which means that they are only affected by the amount of protective layers applied. While the dashed lines are derived by the natural fire curve (Iter4G-protected) and in turn they are dependent on fire load density, thermal properties of boundary elements and opening factor. The natural fire curve used for determining the depth values in Figure 7-37 has a load fire density of $934.48\ MJ/m^2$.

7.5.4 Review on the obtained charred depths

The next tables show the charred depths obtained in the sections above, in here all the results are shown together and compared. The charred depths are representative of the fire load. However for determining the residual strength of a timber element, the $100°C$ isotherm is needed. The distance between the two isotherms can vary considerably.

In Table 40 are shown the charred depths according to complete burnout of combustible material for natural fire exposures with low and high fire load density. It is also shown the effect of different percentages for the additional fire load used.

Amount of additional fire load	*Charred depths after a complete burnout of combustible material* HLFD Protected iteration 4 $[mm]$	*HLFD Unprotect iteration 3* $[mm]$	*LFLD Unprotect iteration 4* $[mm]$
Case A - 100%	81.5	159.4^{a}	79.6
Case B - 80%	–	141.6^{a}	66.7
Case C - 60%	50.8	123.8^{a}	53.9
Case D - 40%	–	106.8	42.2
Case E - 20%	28.6	90.7	31.9
Case F - 10%	–	84	27.7
Case G - 5%	23.7	79.8	26

^{a}: The maximum charred depth is reached later than $7200\ sec$

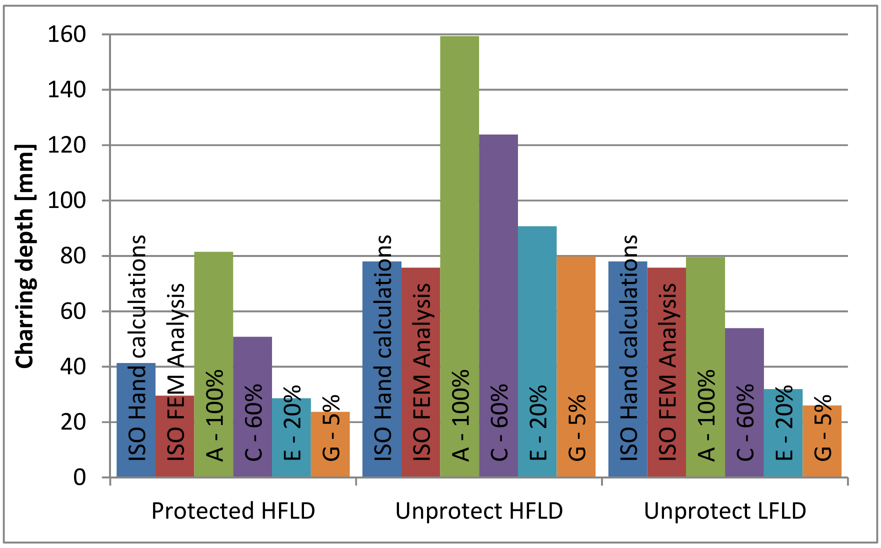
Table 40: Charred depths for $t \to \infty$

Table 41 shows the depths for the 300℃ and 100℃ isotherms according to prescriptive codes (ISO fire exposure), the depths are computed by both, thermal finite element analysis and by hand calculations.

	Depths according to an ISO fire exposure of $7200\ sec$			
	300℃ isotherm		100℃ isotherm	
	Protected $[mm]$	Unprotect $[mm]$	Protected $[mm]$	Unprotect $[mm]$
Hand calculations	41.3	78	48.3	85
FEM Analysis	29.5	75.8	41.3	89.5

Table 41: Depths for ISO exposure at $7200\ sec$

A graphical review is given in the next Figure 7-38, where are shown some of the values presented in the tables above. The fire load density is determined by the fire fighting measures used (§7.3.2), a high initial fire load density takes into account only the basic fire fighting measures, while a low fire load density takes into account more active fire fighting measures.



In Figure 7-38 the charring depths for three scenarios are given, the charred depth for each scenario has been computed according to prescriptive codes (blue and red columns) and performance based codes (remaining columns).

Each column of performance based codes uses a different amount of additional fire load due to the combustible structure.

Figure 7-38: Charred depths compared

As it is shown in the Figure above, the charring depth is deeply affected by the fire load and by the encapsulation; moreover hand calculations presented in EC5 give values which differ considerably from high fire load densities. It is worth noting that the reduction of fire load densities prescribed in EN1991-1-2 when fire active measures are used, according to thermal finite element analysis, has almost the same effect of encapsulating the structural element.

In the following tables the depths of the unaffected timber section are given for different calculation method, different fire load densities and different configuration factor. Although all the input data have the same values (apart the fire load density) the HFLD and the LFLD has a temperature time development which is slightly qualitatively different, as shown in the Figure 7-39 beside.

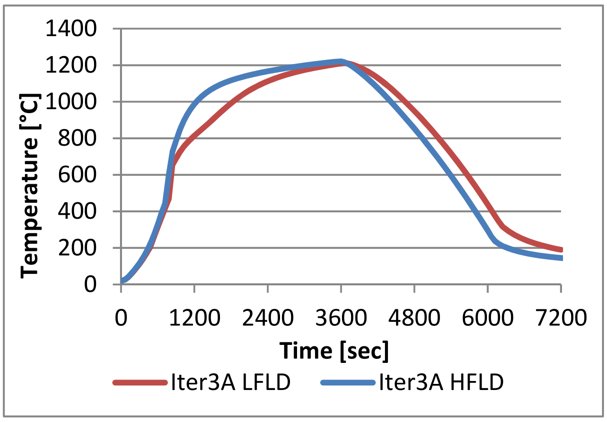


Figure 7-39: Differences in HFLD and LFLD

100°C isotherm depths for unprotected specimens

	Fire Model:	*Zone Model*	*Zone Model*	*ISO*	*ISO*
	Calculation method:	*FEM*	*FEM*	*Hand Calculation*	*FEM*
	Time:	*7200 sec*	*Complete burnout*	*7200 sec*	*7200 sec*
		$[mm]$	$[mm]$	$[mm]$	$[mm]$
Fire curve and fire load density $[MJ/m^2]$					
HFLD iter3a	*2750*	*132.6*	*>218*	*85*	*89.5*
HFLD iter3b	*2284*	*132.6*	*>184*	*85*	*89.5*
HFLD iter3c	*1862*	*131.3*	*>158*	*85*	*89.5*
HFLD iter3d	*1493*	*125.0*	*137.6*	*85*	*89.5*
HFLD iter3e	*1184*	*115.0*	*118.5*	*85*	*89.5*
HFLD iter3f	***1050***	***108.5***	***109.7***	*85*	*89.5*
HFLD iter3g	*986*	*104.8*	*105.5*	*85*	*89.5*
HFLD iter1	*933*	*101.3*	*101.5*	*85*	*89.5*
LFLD iter3a	***1070***	***104.6***	***107.3***	*85*	*89.5*
LFLD iter3b	*863*	*91.5*	*91.8*	*85*	*89.5*
LFLD iter3c	*683*	-	*76.6 at 6600 sec*	*85*	*89.5*
LFLD iter3d	*533*	-	*63.0 at 5580 sec*	*85*	*89.5*
LFLD iter3e	*414*	-	*51.4 at 4620 sec*	*85*	*89.5*
LFLD iter3f	*367*	-	*46.6 at 4320 sec*	*85*	*89.5*
LFLD iter3g	*346*	-	*44.5 at 4140 sec*	*85*	*89.5*
LFLD iter1	*329*	-	*42.7 at 4080 sec*	*85*	*89.5*

Table 42: 100°C isotherm depths for unprotected specimens

The bolted values show a discrepancy between the LFLD and the HFLD, however it is believed that a difference of few millimetres can be neglected.

100°C isotherm depths for protected specimens

	Fire Model:	*Zone Model*	*Zone Model*	*ISO*	*ISO*
	Calculation method:	*FEM*	*FEM*	*Hand Calculation*	*FEM*
	Time:	*7200 sec*	*Complete burnout*	*7200 sec*	*7200 sec*
		$[mm]$	$[mm]$	$[mm]$	$[mm]$
Fire curve and fire load density $[MJ/m^2]$					
HFLD iter3a	*1841*	*86.7*	*>119*	*48.3*	*41.3*
HFLD iter3c	*1303*	*71.8*	*80.4*	*48.3*	*41.3*
HFLD iter3e	*997*	*53.5*	*55.3*	*48.3*	*41.3*
HFLD iter3g	*934*	*48.7*	*49.9*	*48.3*	*41.3*
HFLD iter1	*917*	*37.4*	*48.2*	*48.3*	*41.3*

Table 43: 100°C isotherm depths for protected specimens

The values presented in the tables above are graphically compared in Figure 7-40 and Figure 7-41. In these graphs it is clear that the prescriptive values given in EN1995-1-2 are most of the time not representative of the values obtained by advance calculations. The depth of the $100°C$ isotherm determines the residual wood section with unaffected strength properties, but with HFLD it was not possible to determine the depth for a complete burnout of fuel. For low fire load densities (right part of the graphs) the prescriptive value is too conservative for unprotected fire room, while for protected specimens has a good approximation. When the fire load density is increased, moving from right to left, the difference increases as well, reaching up 53 mm for HFLD Iter3C in Figure 7-40, and 32 mm for HFLD Iter3B in Figure 7-41.

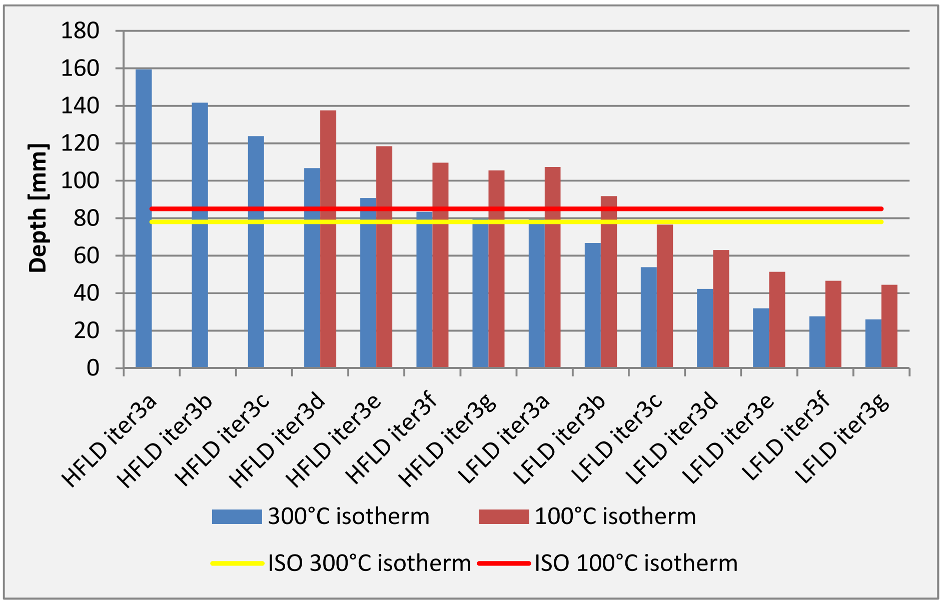


Figure 7-40: Charring depth and unaffected depth compared – unprotected

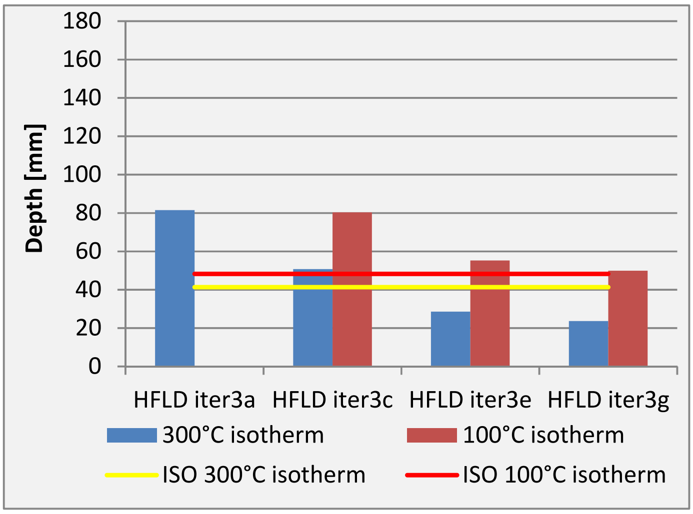


Figure 7-41: Charring depth and unaffected depth compared –protected

7.6 Comparison with real fire scale tests

The time-temperature curves obtained from the previous analysis are compared to real full scale fire test with wooden load bearing structure with and without combustible linings. The temperature-time curves of full scale experiments described in §3.3.2.2 are shown together in this section. In the table and graphs below all the main results are shown. The tests studied differ in the amount of fire load and in the opening factors therefore a straight comparison is hard to be done.

Fire curve name	*Temperature measure in*	*Configuration*	*Fire load density* $[MJ/m^2]$	*Opening factor* $[m^{1/2}]$
Test 1	Front	I sight	Movable: 720	0.042
Test 2	Front	Protected	Movable: 720	0.042
Ivalsa – ch5	Centre	Protected walls and in sight wooden floor	Total: 790 Movable: 711	0.007
Iter4G	N.a.	Protected	Total: 934 Movable: 918 M	0.063
Iter3G	N.a.	In sight	Total: 1004 Movable: 933	0.063
BÜ bb	Front & rear	In sight	Total: 855	0.033
BÜ nbb	Front & rear	Protected	Total: 363	0.033

Table 44: Summary of different temperature-time curves

In the graphs below the protected fire room temperatures are given first, followed by the temperatures related to the configuration with timber in sight.

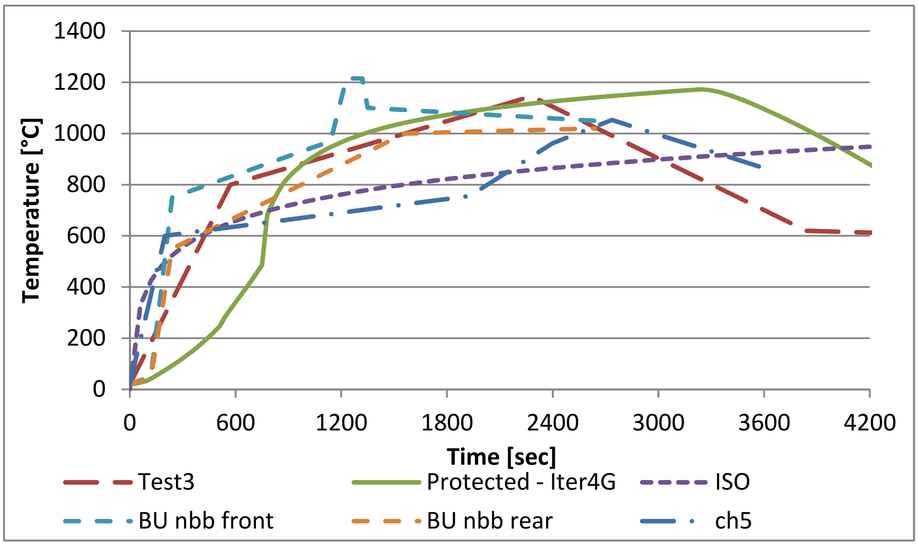


Figure 7-42: Temperature curves for protected fire room

Despite different fire load densities and opening factors in the fully developed stage the natural fire curves have a close fit, excluding the peak value for the BÜ nbb curve. All the real scale fire tests after 7 min from ignition reach higher temperatures than the ISO curve. As can be seen in Figure 7-42, the curve determined in this study (Iter4G), which has the highest fire load, has the longer fire period. Although the fire curve determined with OZone has also the highest opening factor, its growing stage is slower than the other curves.

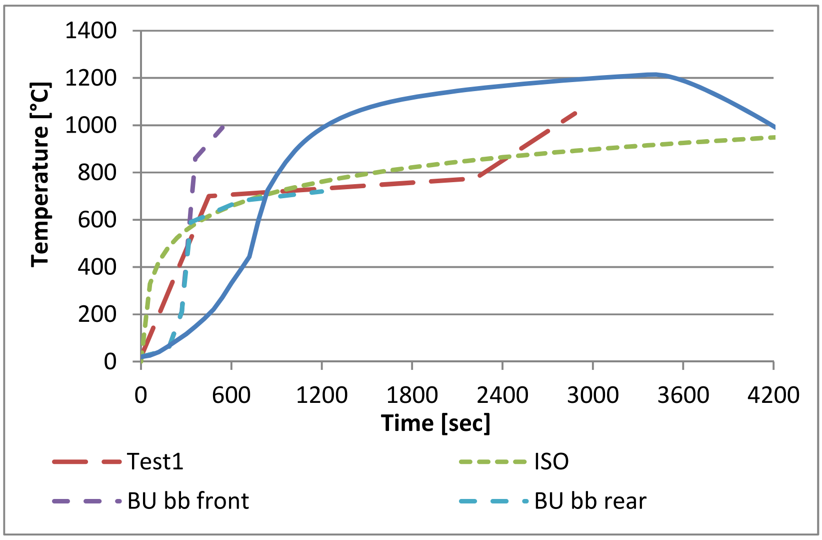


Figure 7-43: Temperature curves for unprotected fire room configuration

The fire room temperatures for an unprotect compartment are shown in Figure 7-43. The measured values have different trend which make difficult to determine the accuracy of the determined fire curve by OZone. Indeed, the fire curve obtained by OZone with combustible finishing material has much higher temperatures than the real scale fire tests (test 1). However, the test results, apart test 1, have a record period too short to be compared with show a good fit for values in the rear of the compartment. The temperatures in the front of the compartment for test BÜ bb were measured only for the first 10 min and they have a faster growing stage with higher temperatures than the computed fire curve. The test had been stopped after $20\ min$ due to excessive flaming.

The measured temperatures in the fire room with wooden structure in sight have a better fit of the ISO fire curve (test 1) until a complete burnout of the movable fire load occurs, then, when the pyrolysis gas production decrease and oxygen can enter the fire room, the temperature starts rising (approximately $40\ min$ from ignition).

7.7 Summary and discussions

7.7.1 Relevant part of combustible structure

The relevant part of combustible structure has a great effect on the fire severity. In the analyses different percentages of the additional fire load were evaluated. Higher percentages require more iterations in order to converge to a steady value of charring depth. When it was assumed a low percentage of the additional fire load, it was possible to determine a stable charring depth.

In Table 45 it is shown the variation of charring depth with the amount of additional fire load. The unaffected timber depth is also greatly affected by the additional fire load density and, in turn, by the temperature-time curve, as shown in Figure 7-27. As shown in Figure 7-40 and Figure 7-41, prescriptive codes fail for extremely high and extremely low fire load densities.

Configuration	*Initial fire load density* $[MJ/m^2]$	*Iteration*	*Charred depth* $[mm]$	*ISO charred depth at 120 min* $[mm]$
In sight	933	3rd	From 160 to 80	78
Protected	918	4th	From 82 to 24	41.3
In sight	329	4th	From 80 to 26	78

Table 45: Summary of charred depths

The initial fire load density does not affect only the fire severity and the charring depth, but it also affects the computational effort, it has been shown that initial low fire load densities need more iteration to converge to a steady value of charred depth.

7.7.2 High and low initial fire load densities

The presence of fire fighting measures was assumed by reducing the initial fire load density. It is important to note that in reality when fire fighting measures are present, the fire load of the compartment is not affected. The only effect they have is the reduction of the fire risk, which is taken into account by a statistical approach, then the presence of fire fighting measures is “translated” in engineering terms as a lower fire load.

From a comparison between the HFLD and the LFLD it can be seen that the low initial fire load needs more iterations to converge to a stable value of charring depth. This behaviour can be explained by the relative increase in fire load density. In fact, the LFLD has an additional fire load on each iteration which has almost the same magnitude of the initial fire load.

7.7.3 Protected fire compartment

According to the analyses carried out, when a timber compartment is encapsulated, the charring of wood is greatly reduced. When the charring of wood is reduced also the additional fire load is decreased. Reducing the total fire load affects the overall fire severity, which is extremely reduced. It is interesting to note that applying protective linings in a timber compartment has almost the same effect of reducing the fire load. It can be argued that fire passive measures should have the same reliability of fire active measures.

7.7.4 Unaffected timber depth

The maximum depth of the unaffected timber section is reached later than the maximum charred front depth and later than the maximum fire room temperature. It may be even reached after the decay stage. Due to this behaviour, the distance between the char front and the zero strength layer it is not always 7 mm wide, as determined in prescriptive codes. In a natural fire exposure as soon as the maximum char front depth is reached, the distance between the $100°C$ isotherm and the $300°C$ is widening.

7.7.5 Applicability of nominal fire curve

In reference to calculations carried out in §5.7, the concept building will withstand a fire which causes a reduction of the section of $91\ mm$. As it can be seen from Figure 6-16, the $100°C$ isotherm line lays on the perpendicular layer. Since the structural contribution of the perpendicular layers have been neglected a further section reduction of $29\ mm$ may be arranged. Therefore the maximum allowable depth for the unaffected timber section is $120\ mm$. If a greater reduction occurs the structural resistance is not granted anymore and the new residual section has to be checked again.

Based on fire advanced analysis of different natural fires, the concept building will withstand a natural fire exposure according to the analysis: HFLD Iter3E characterised by a fire load density (at the 3rd iteration) of $1184\ MJ/m^2$ and an unaffected depth of $d_{100} = 118.5\ mm$. In this analysis only 20% of the structural fire load has been added to the initial fire load. Therefore if the initial fire load is higher than $932.8\ MJ/m^2$, or a higher the percentage has to be used, the residual section will not able to handle the loads and the structure may collapse.

On the other hand, if it is reasonable to add 40% of the structural fire load to the initial movable fire load, as it is done in HFLD Iter3D, it is clear that the fire design time requested by prescriptive codes is not on the safe side. The analysis HFLD Iter3D is characterised by a fire load density (at the 3rd iteration) of $1493\ MJ/m^2$ (the initial fire load density was $932.8\ MJ/m^2$) and an unaffected depth of $d_{100} = 137.6\ mm$. In this analysis a full burnout of combustible material is reached approximately after 3 hours from ignition. In order to obtain the same unaffected depth with prescriptive codes a fire design time of 3.5 hours ($d_{100,ISO,3.5h} = 136.5\ mm$) should be requested instead of two hours ($d_{100,ISO,2h} = 85\ mm$).

8 Conclusions

8.1 General conclusions

The main object of this MSc thesis was to study the feasibility of a high rise timber building with regard to fire safety aspects. In particular, the fire safety of such building was extensively addressed. From the knowledge obtained from the literature review and the fire analyses carried out, it can be concluded that **it is possible to build a $100\ m$ high timber building made of CLT panels**.

According to calculations based on prescriptive rules, a CLT section of $360\ mm$ thick is able to resist the load combination due to the fundamental and the accidental action. The accidental action has been considered as a two hours exposure to the standard fire curve (according to building codes). During the accidental design situation, there is a great reduction of the applied loads, and at the same time, the strength of the material is increased due to the exceptional event.

Current building regulations rely on prescriptive rules but they also allow the use of performance based design. It is not always easy to determine the requirements in the building codes and they lead to misunderstandings. First of all building codes should be written much clearer and with the aids of graphs and drawings, as it is done in the English and in the New Zealand building codes. Italian and Dutch building decrees refer to some other documents that may be not known. Apart from those difficulties, no restrictions are present in the use of timber as a construction material.

Doubts arise due to the combustibility of the building material. In fact, for a structure $100\ m$ high, the prescriptive codes require two hours of fire design time, which is considered not on the safe side in the case of a high rise timber building. Indeed, concerns arise on the philosophy behind the applicability of prescriptive rules on timber structures because they have been determined for non-combustible structures. In fact they assume that after the fire design time all the combustible material is consumed and the fire is extinguished. This is not the case when a combustible load bearing structure is exposed to a fire, because of:

– The load bearing elements are reducing their cross-sectional area, it is an irreversible effect;
– The charred timber section contributes to the fire load, increasing the fire severity;

The contribution of the charred section to the initial fire load is recognized also in the Eurocodes, where it is prescribed that “the relevant combustible parts of the construction should be considered into the fire load” when using advance calculation methods. However, no source has been founded in literature addressing how to take into account the relevant combustible parts of the construction. Thus, in this work the effect of different percentages of additional fire load on the fire severity has been studied but no guidance has been obtained on which percentage should be used. However, it has been recognized that the fire severity depends greatly on how much additional fire load is taken into account. The effect on the fire severity has been studied by advanced fire analyses carried out with the zone model OZone, developed for steel structures.

Due to the structural contribution to the fire load, it is reasonable to think that the fire will not go out until the entire load bearing structure has been consumed. If this is the case then, the fire will inevitably lead to a structural collapse. The results show that depending on the percentage of additional fire load taken into account, the charred depth converges to a steady value after several iterations. Unfortunately, due to limitations on the finite element program and on the thermal timber properties, it was not possible to determine if the charred depth was converging to a steady value for large percentages of additional fire load.

The codes clearly address the height of buildings as the major problem during fires. In fact the requirements increase with the height of buildings, and with the function of buildings (office or residential use).

The height will affect the escaping time, the time needed by fire-fighters to reach the fire and the possibility to carry an external attack to fires.

Different fire protective measures are commonly used in steel-concrete high rise buildings in order to reduce the fire risk, for both inhabitants and the structure. The most common measures are:

- Fire-fighting elevators;
- Protected lobbies equipped with stand pipes;
- Self-closing and fire resistance entry doors;
- Automatic detection systems;
- Automatic fire-fighting systems (Sprinklers);
- Manual fire-fighting devices (fire hand extinguishers, hose reels)
- Fire refuge floors and areas;

All these protective measures shall be also implemented in high rise timber buildings. They increase greatly the fire safety of the building but they will not ensure the possibility to carry an external attack to the fire. It is strongly believed that the possibility of an external attack is the paramount protective measure for high rise timber buildings.

It is reasonable to believe that a timber building will not survive to a fire which is left unattended. This is the main difference with steel-concrete buildings. In fact, in steel-concrete high rise buildings when fighting the fire becomes impossible (either from inside or outside), the fire is let consuming all the combustible material in the compartment. In order to have more redundancy against a potential out-of-control fire, timber buildings must provide a safe external attack whichever the fire floor is. For this reason **the three towers building concept is believed being the main protective measure for timber buildings**. This concept allows fire-fighters to attack fires externally independently from the fire floor. The three towers concept has the following advantages:

- External attack always possible from the sky gardens and from adjacent towers;
- Inhabitants may flee crossing the sky gardens;
- Fire-fighting elevators may be not needed since fire fighters shall use the unaffected towers to reach the fire;

It is important to note that a fire safety education of inhabitants plays a major role in fire safety whatever fire protection measures are used. It is believed that if inhabitants are able to properly start fighting the fire, the fire risk could be greatly reduced. For instance, the Eurocodes allow a reduction of fire load when an onsite fire fighting team is used. It is believed that well trained inhabitants may also ensure a reduction of the fire risk.

In addition to that it is believed that a quantitative assessment with probabilistic analysis of all the fire protective measures applied to the fictional building has to be performed in order to guarantee an adequate fire safety level. However, it was believed that a probabilistic analysis was outside the scope of this study and it was neglected. Thus it was assumed that the three connected towers concept together with the common protective measures ware providing enough safety.

8.2 Proposal for a new fire model for timber compartments

When unprotected timber compartments are exposed to fires, the temperature time curve is greatly affected by the large production of pyrolysis gasses. It has been shown by real scale experiments which were performed on exposed wooden structure. The fire model used in the analyses was not able to taken into account this effect and therefore it was neglected.

At the time of writing this thesis, no fire model have been specifically derived for timber compartments with or without protective linings. This is a main disadvantage for the development of high rise buildings made of CLT panels.

It is believed that in order to ensure more accurate results, a fire model for timber compartment should be developed. If a finite element program, as SAFIR, could be implemented in a fire model program, as OZone, a more accurate fire growth may be evaluated. If these two models are merged together there is no need to perform iterations for determining the additional fire load.

In fact, if the mutual influence of the heat release rate, the temperature time curve, and the development of temperature inside the timber element can be determined determined step by step. In this way it will be possible to evaluate with only one analysis the final charred depth of the entire compartment. Furthermore, this model could also take into account the effect of pyrolysis gas production by reducing the maximum rate of heat release.

The decay stage of a fire is the main aspect that has to be considered in a fire model for timber compartments. It is the key factor which will ensure that a fire will not consume the entire load bearing structure. In fact, it is important to evaluate if the fire has sufficient energy to heat up more timber section beyond ignition temperature. If the energy will not be enough then a decay stage could start, otherwise the fire will keep burning.

8.3 Recommendations for further study

Performing this study on fire safety of high-rise timber buildings, several aspects have been encountered where lack of knowledge was observed. Those aspects are mainly related on how to model fires in timber compartments. This is mainly due to the fact that CLT panels have been used mostly for low and medium rise buildings, where a fire risk was low and there was no need for fire safety engineering. When higher structures have to be built, fire safety engineering is needed, therefore a deep knowledge of the behaviour in fire has to be obtained. In literature, many experiments on panels have been carried out, but only few real scale tests on CLT compartments have been founded. Further research is needed to address the following aspects:

- How do different thermal histories affect the thermal properties of wood? How much is the error encountered if those properties (which are determined for a standard fire exposure) are used for different fire curves?
- How is it possible to take into account the additional fire load due to the combustible structure? How much of the charred depth should be added to the initial fire load?
- How can be taken into account the effect of pyrolysis gasses production on the temperature-time curve?
- Is it possible that a fire be self-extinguishing? When all the movable fire load has been consumed, is the additional fire load sufficient to heat up the remaining timber section beyond ignition temperature and continuing burning?

The last question is considered vital for high rise timber buildings because it is a major concern in case of failure of active protection systems or when an external fire attack cannot be performed. All of the real scale experiments performed in literature have been manually extinguished before the fire could burn the entire combustible material.

Bibliography

1. **Franssen, J. -M.** *SAFIR. A Thermal/Structural Program Modelling Structures under Fire.* Engineering Journal, A.I.S.C., Vol 42, No. 3 (2005), 143-158, http://hdl.handle.net/2268/2928.

2. *Fire safety regulations for housing in europe compared.* **Visscher, H., Sheridan, L. and Meijer, F.** 4, 2008, Building Research Journal, Vol. 56, pp. 215-227.

3. **Lay, Simon F.** *Design, Fire Safety Challenges in UK High-Rise Residential and Mixed Use.* UK : s.n.

4. *Fire Safety Engineering, New possibilities for wood products.* **Mikkola, Esko.** Tallin : s.n., 2005.

5. **Ostman, Birgit and Rydholm, Daniel.** *National fire regulations in relation to the use of wood in Eurperan and some other countires.* Stockholm : Tratek, 2002.

6. **Östman, B., et al., et al.** *Fire safety in timber buildings, Technical guideline for Europe.* Sweeden : SP Technical Research Institute of Sweden, 2010. p. 210 pages.

7. **Karacabeyli, Erol and Popovski, Marjan.** *Possibilities with Wood System in Construction.* April 07. Minneapolis : The Wood Products Council, 2011.

8. **Ward, Roxane.** Going to New Heights, Building the World's Tallest Mixed-use Wood Structure. *Structure.* [Online] [Cited: 06 28, 2011.] http://www.structuremag.org/article.aspx?articleID=947.

9. **Warwick Banks , B.E.** MULTI-STOREY TIMBER CONSTRUCTION – A FEASIBILITY STUDY. *New Zealand Timber Design.* [Online] http://www.timberdesign.org.nz/journal.cfm/view/18/parent/2.

10. **Purkiss, J. A.** *Fire Safety Engineering, desing of structures.* Oxford : Butterworth-Heinemann, 1996.

11. **SP-Trätek.** *Fire safety in timber buildings.* Stockholm, Sweden : SP Technical Research Institute of Sweden, 2010.

12. **Eurofer.** *"Steel and Fire Safety - A Global Approach".* Brussels : Eurofer Steel Promotion Committee Brochure, 1990.

13. **Buchanam, Andrew H.** *Structural design of fire safety.* University of Canterbury, New Zealand : John Wiley & sons, LTD, 2001.

14. **Sheridan, L., Visscher, H.J. and Meijer, F.M.** *Building regulations in Europe: Part II, A comparison of the technical requirements in eight European countries.* Delft, Netherlands : DUP Science, 2003.

15. **Spearpoint, Micheal.** *Fire Engineering Design Guide.* Christchurch, New Zealand : New Zealand Centre for Advanced Engineering CAENZ, 2008.

16. **Nystedt, Fredrik.** *Verifying Fire Safety Design in Sprinklered Buildings.* Lund, Sweden : Department od Fire Safety Engineering and Systes Safety, 2011.

17. *DM n. 246 - Norme di Sicurezza per gli edifici di civile abitazione.* 16 Maggio 1986.

18. **Piazza, Maurizio, Tomasi, Roberto and Modena , Roberto.** *Strutture in legno. Materiale, calcolo e progetto secondo le nuove normative europee.* s.l. : Hoepli, 2005.

19. **Andreolli, Mauro.** *promo_legno.* [Online] [Cited: 06 27, 2011.] http://www.promolegno.com/risponde/domanda/data/quali-sono-le-limitazioni-previste-per-gli-edifici-in-legno.htm?L=0&cHash=49af9d15b5.

20. **Mahapatra, Krushna and Gustavsson, Leif .** *General conditions for construction of multi-storey wooden buildings in Western Europe.* Östersund, Sweden : School of Technology and Design Växjö University, Sweden , 2009.

21. *Bouwbesluit.* 2001.

22. *Building regulations on fire safety in Europe.* **Sheridan, L., Visscher, H.J. and Meijer, F.M.** Malaysia : s.n., 2003. CIB-CTBUH International Conference on Tall Buildings, 8-10 May .

23. *Fire Design Concepts for Tall Timber Buildings.* **Frangi, Andrea, Fontana, Mario and Knobloch, Markus.** Number 2, s.l. : International Association for Bridge and Structural Engineering, May 2008, Structural Engineering International, Vol. Volume 18, pp. pp. 148-155(8).

24. *International applications of elevators for fire service access and occupant egress in fires.* **Bukowski, Richard W.** Issue III, s.l. : CTBUH Jorunal, 2010, CTBUH Journal, pp. 28-33.

25. **Enjily, Vahik .** Roiadmap 2010 - Building with wood. [Online] [Cited: 06 24, 2011.] http://dfwm.ugent.be/woodlab/docs/dublin/Vahik.pdf.

26. **BRE.** *Barriers to the enhanced use of wood in Europe: particular attention to the regulatory barriers,.* UK : Building Research Establishment, 2004.

27. **Poon, Leong .** *Literature review on the contribution of fire resistant timber construction to heat release rate.* Victoria, Australia : WFRA, 2003.

28. **Bergman, Richard, et al., et al.** *Wood Handobook, wood as an engineerign matreial.* Madison, Wisconsin : Forest Products Laboratory. Wood handbook - Wood as an engineering material. General Technical Report FPL-GTR-190. Madison, WI: U.S. Department of Agriculture, Forest Service, Forest Products Laboratory, 2010.

29. **EUROFORTECH.** *STEP 2, Timber Engineering.* The Netherlands : Centrum Hout, 1995.

30. **SP-Trätek.** *InnoFireWood - Innovative eco-efficient, high-fire-performance wood products for demanding applications.* Stockholm : SP-Trätek, 2006.

31. *Durability of fire retardant treated wood products at humid and exterior conditions, review of literature.* **Ostman, Birgit, et al., et al.** 2001, Fire and materials, pp. 95-104.

32. *Charring rate and temperature profiles of wood sections.* **Fontana, Mario and Frangi, Andrea.** s.l. : Fire and Material, 2003, Fire and Materials, pp. 91-102.

33. **Friquin, Kathinka Leikanger.** *Charring rate of heavy timber structures for fire safety design.* Trondheim : NTNU Norwegian University of Science and Technology, 2010.

34. **Schaffer, E.L.** *Structural Fire Design: Wood.* Madison, Wis. : Forest Products Laboratory, 1984.

35. **EUROFORTECH.** *STEP 1, Timber Engineering.* The Netherlands : Centrum Hout, 1995.

36. **Mikkola, E.** *Charring of wood. VTT Research Report 689.* Espoo, Finland : VTT Technical Research Centre of Finland, 1990.

37. *Brandeversuche an modulhotels in holtbauweise.* **Maag, T. and Fontana, M.** ETH Zurich : s.n., August, 2000, Institute of Structural Engineering, p. IBK Report No. 250.

38. *EN 1995-1-2.*

39. *Fire performance of timber construction.* **Buchanan, Andrew H.** Christchiurch, New Zealand : university of Canterbury, 2000, Progress in Structural Engineering and Materials;2, pp. 278-289.

40. *Experimantal analysis of cross-laminated timber panels in fire.* **Frangi, Andrea, et al., et al.** 2009, Fire Safety Journal.

41. **Frangi , Andrea, et al., et al.** *Natural full-scale fire test on a 3-storey Xlma timber building.* 2006.

42. *Fire Behaviour Of Cross-laminated Solid Timber Panels.* **Frangi , Andrea, et al., et al.** 2009, Fire Safety Science, pp. 1279-1290.

43. *Post-Flashover Fires in Light and Heavy Timber Construction Compartments.* **Hakkarainen, Tuula .** Issue: 2, MAR 2002, JOURNAL OF FIRE SCIENCES, Vol. Volume: 20 , pp. Pages: 133-175.

44. **Maag, Trond and Fontana, Mario.** *Brandversuche an modulhotels in holzbauweise.* Zürich : Institut für Baustatik und Konstruktion Eidgenössische Technische Hochschule Zürich, August 2010.

45. **NFPA.** *The SFPE handbook of fire protection engineering.* 3rd Ed. Quincy, Massachusetts : Nationa Fire Protection Association, 2002.

46. **Cajot, L.G., Haller, M. and Pierre, M.** *PART1: Thermal & Mechanical Actions.* s.l. : DIFISEK-project, 2005.

47. **Barnett, Cliff, et al., et al.** *Fire safety in tall buildings, Council on Tall Buildings and Urban Habitat.* New York : McGraw-Hill, 1992.

48. **Canadian Wood Council.** *Fire safety in Residential Buildings.* Building performance bulletin : series 2, 2000.

49. **Twilt, L.** *Fires in biuldings: the facts.* The Netherlands : TNO.

50. **Richardson, K.** *Fire safety in high-rise apartment buildings.* Canada : Ontario Association of Architects.

51. **Eurofer.** *Steel and Fire Safety - A Global Approach.* Brussels : Eurofer Steel Promotion Committee Brochure, 1990.

52. **Best, Richard.** *Investigation report on the MGM grand hotel fire.* Las Vegas, Nevada : National Fire Protection Association, 1980.

53. **Nystedt, Fredrik.** *Verifying Fire Safety Design in Sprinklered Buildings.* Lund, Sweden : Department od Fire Safety Engineering and Systes Safety, 2011.

54. **Wikipedia.** Carbon monoxide poisoning. *Wikipedia.* [Online] [Cited: 07 07, 2011.] http://en.wikipedia.org/wiki/Carbon_monoxide_poisoning.

55. *Integrated design of safe skyscrapes: problems, challenges and prospects.* **Ali, M.** Malaysia : CIB Publication No:290, 2003. CIB-CTBUH International Conference on Tall Buildings. pp. 689-696.

56. **Department of the Enviroment.** *England and Wales Building Regulations, Approved Document B, Fire Safety.* s.l. : HMSO, 1992.

57. **The Chicago High-Rise Safety Commission.** *High-rise fire safety: problems & solutions.* Chicago : s.n., 1999.

58. **Routley, Gordon, Jennings, Charles and Chubb, Mark.** *High rise office building fire One Meridian Plaza.* s.l. : United States Fire Administration, 1991.

59. **Hall, John R.** *U.S. Experience With Sprinklers.* Quincy, MA : NFPA Fire Analysis and Research, 2011.

60. **Koffel, William E.** *Reliability of Automatic Sprinkler Systems.* s.l. : Alliance for Fire and Smoke Containment and Control, 2005.

61. **Hall, J.R.** *U.S. Experience with sprinklers and other Fire Extinguishing Equipment.* s.l. : Fire Analysis and Research Division National Frie Protection Association, 2010.

62. *Fire Engineering Super-Tall: A New Approach to Escape.* **Kealy, Martin J.** Dubai : CTBUH 8th World Congress, 2008. CTBUH 8th World Congress. pp. pp.723 - 731.

63. *Evacuation design strategy for high rise buildings.* **Sano, T., Hagiwara, I. and Tanaka, A.** Malaysia : CIB Publication, 2003. CIB-CTBUH International Conference on Tall Buildings. pp. 567-572.

64. *Elevators for occupant evaquation and fire department.* **Kuligowski, E.** Malaysia : CIB Publication No: 290, 2003. CIB-CTBUH International Conference on Tall Buildings. pp. 193-200.

65. *Alternative evacuation design solutions for high rise buildings.* **Lay, Simon.** October 19, 2007, The structural design of tall and special buildings, Vol. 16, pp. 487-500.

66. *High-rise evacuation: a questionable concept.* **Proulx, Guylene.** Boston, MA., U.S.A. : s.n., 2001. 2nd Internarional Symposium on Human Behaviour in Fire. pp. pp. 221-230.

67. *Fire engineering the tallest building in the historic city of Manchester, UK.* **McGrail, Dean Kevin, Lay, Simon Francis and Chow, W. K.** Seoul, Korea : CTBUH, 2004. CTBUH. pp. 735-742.

68. **U.S. Fire Administration.** *Interstate Bank Building Fire.* Technical Report Series. Los Angeles, California : Homeland Security, 1988. USFA-TR-022.

69. **Ling, Florence Yean Yng and Soh, Lay Hong.** *Improving the design of tall buildings after 9/11.* Singapore : Department of Building, National University of Singapore, 2005.

70. **Department of Building and Housing.** *Compliance Document for New Zealand Building Code, Clauses C1, C2, C3, C4, Fire Safety.* Wellington, New Zealand : Department of Building and Housing, 2010.

71. **Lay, Simon F.** *Fire Safety Challenges in UK High-Rise Residential and Mixed Use Design.* Manchester : WSP Buildings.

72. *Vibration control of a tower complex connected by sky gardens.* **Nishimura, Akira.** II, s.l. : CTBUH Journal , 2011.

73. *Review of evacuation procedures for the Petronas twin Towers.* **Ariff, Arlida.** Malaysia : CIB Publication No:290, 2003. CIB-CTBUH International Conference on Tall Buildings. pp. 35-42.

74. **Waugh Thistleton Architects.** *Tall Timber Buildings The Stadthaus.* London : Techniker, 2010.

75. **Serrano, Eric.** *Documentation of the Limnologen Project.* Växjö, Sweden : School of Technology and Design, 2009. Reports, No. 56.

76. **Dunn, Vincent.** *Strategy of Firefighting.* New York : s.n., 2007.

77. **van Egmond, Simon.** *Medium rise tiber buildings in the Netherlands.* s.l. : Faculty of Civil Engineering and Geosciences, TU Delft, 2011.

78. **van De Kuilen, J.W.G., et al., et al.** *Very tall wooden buildings with cross laminated timber.* s.l. : Elsevier, 2011.

79. **S.G.C.Timmer.** *Feasibility of Tall Timber Building.* s.l. : Faculty of Civil Engineering and Geosciences, TU Delft, 2011.

80. **Simone, Angelo.** *An Introduction to the Analysis of Slender Structures.* Delft : Delft University of Technology, Faculty of Civil Engineering and Geosciences, 2009.

81. *A parametric natural fire model for the structural fire design of multy-storey buildings.* **Zehfuss, J. and Hosser, D.** 2007, Fire Safety Journal, Vol. 42, pp. 115-126.

82. **Torero, J.L., Quintiere, J.G. and Steinhaus, T.** *Fire Safety in High-rise Buildings: Lessons Learned from the WTC.* Germany : Jahresfachtagung der vereingung zur Forderrung des Deutschen Brandschutzez, 2002.

83. *Post-flashover fires for structural design.* **Feasey, R. and Buchanam, A.H.** 2002, Fire Safety Journal, pp. 83-105.

84. Zone Models. *University of Manchester.* [Online] [Cited: 04 09, 2012.] http://www.mace.manchester.ac.uk/project/research/structures/strucfire/Design/performance/fireModelling/zoneModels/default.htm.

85. **Friquin, Kathinka Leikanger.** *Charring rate of heavy timber structures for fire safety design.* Trodheim : Norvegian University of Science and Technology, April 2010.

86. **Buchanan, Andrew H.** *Structural design for fire safety.* s.l. : John Wiley, 2001.

87. **Franssen, Jean-Marc and Cachim, Paulo B.** Comparison between charring rate model and conductive model of Eurocode 5. *Fire & Materials.* 2009, Vol. 33.

88. **Konig, Jurgen.** Structural fire design according to Eurocode 5 - design rules and their background. *Fire and Material.* 29, 2005.

89. **Franssen, Jean-Marc.** Assessment of Eurocode 5 charring rate calculation method . *Fire Technology .* Springer Science Business Media, 2009, Vol. 46, I.

90. **Frangi, Andrea, Schleifer, Vanessa and Fontana, Mario.** *Design model for the verificationof the separating function of light timber frame assemblies.* ETH Zuroch, Institute of Structural Engineering, 8093 Zurich, Switzerland : ELSEVIER, 2010.

91. **Trada.** *Guidance Document 10 - Cross laminated timber design guide for project feasibility.* s.l. : TRADA technology Ltd, 2009.

92. **A. Frangi, M. Fontana, M. Knobloch and G. Bochicchio.** *Fire Behaviour Of Cross-laminated Solid Timber Panels.* 2009.

93. *One-dimensional charring of timber exposed to standard and parametric fires in initially protected and non-protected fire situations.* **Konig, J. and Walleij, L.** Stockholm : Tratek - Swedish Institute for Wood Technology Research, 1999. I 9908029.

94. **Konig, Jurgen.** *Timber frame assemblies exposed to standard and parametric fires, Part 1 and 2.* Stockholm : Tratek, 2000.

95. **STEP 2.** *Timber Engineering, STEP 2.* 1995.