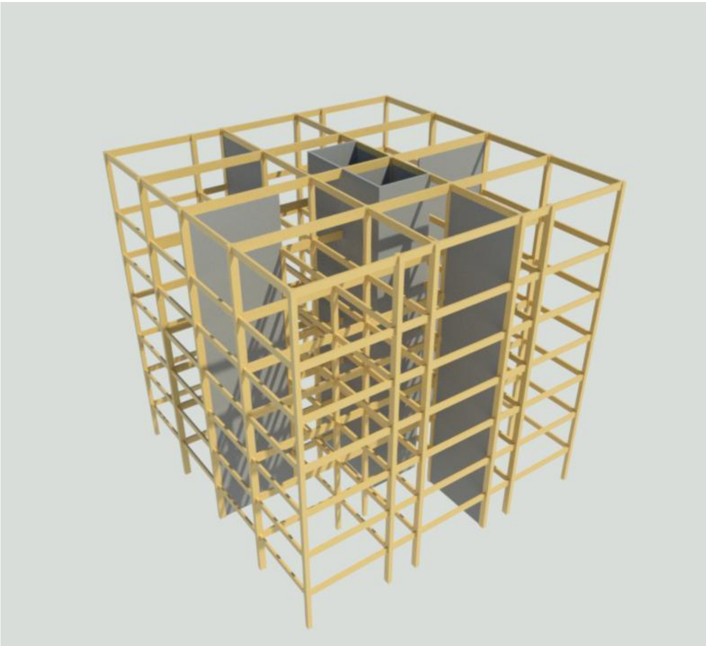

FIRE-RESISTANCE OF GLUE-LAMINATED TIMBER

ABSTRACT

Wood is becoming an increasingly popular material for construction of high-rise buildings. However, it has a bad reputation for not being a fire-safe alternative. But is this reputation rooted in truth or mostly due to negative public perception? How does glue-laminated timber compare to traditional materials like concrete and steel when considering fire-safety, economic and environmental factors?

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Introduction

The use of timber and glulam (glue laminated timber) has, in recent years, become increasingly popular in construction of tall buildings. It offers a sustainable and environmentally friendly alternative to steel and concrete in construction and is also renewable to a degree that the others is not. Due to the lower cost of materials and possibility for offsite prefabrication of elements, it is also a time-and cost-efficient alternative to more traditional construction methods for high-rise buildings.

Timber tends to have a bad reputation when it comes to fire safety in buildings, however, this is mostly due to public perception. Although traditional materials like steel and concrete are fireproof, meaning they do not catch fire, they are not fully fire resistant, meaning a lot of its structural strength is lost during a fire. This fact coupled with the knowledge that its usually the contents (furniture, walls, flooring) of the building that contribute the most to the danger of a fire rather than the structure of the building itself, means that the focus should be on designing a firesafe building instead of one that is fireproof. During a building-fire, the average temperature reached is between 680^0C and 900^0C. A concrete structure is subject to heavy spalling, even at lower temperatures, and shows a significant reduction in strength at around 650^0C [1], depending on aggregates used. Tests done for steel show that about 90% of the strength is lost if exposed to 750^0C for 30minutes, while timber has a strength loss of only 25% at the same temperature. One reason for the huge difference in strength-loss can be attributed to the Charing that occurs when wood burns, giving the structural element a natural insulation against the rising temperature [2].

It is easy to see why timber could be a good choice of material when designing a high-rise building. In this thesis, we will make a review of the techniques used for calculating and ensuring the fire safety of glue-laminated timber elements and design a building of 7 floors, with dimensioning for fire safety in R90 using a structure made from glue-laminated timber. For simplification the joints between structural elements is not considered in this thesis, but should normally get special attention as it is one of the main failure points of a timber structure during a fire [3]. A comparison of prize and CO_2-emmissions between the same structure made from glue-laminated wood, steel and concrete will also be done.

2 Literature review of current fire safety methods and design methods.

When determining the effects of a fire on a structural component made from timber, we first need to find the charring-depth at a given time during the fire-situation. This is done by using the charring-rate multiplied by the number of minutes the structure has been exposed to fire. The element should be divided into two separate categories that have different calculations. For slabs and panels without any cracks or fissures the method used is called one-dimensional. It is one-dimensional as it does not have any exposed corners and only one side is subjected to the fire. For beams and columns or panels with fissures, the two-dimensional method is used. In the two-dimensional method the structural element is exposed to fire on two or more sides, causing a rounding off the corners and a faster reduction in cross-section. The method of calculation is similar in both one- and two-dimensional cases, but the charring-rate changes.

2.1 one-dimensional calculation of fire resistance:

The charring-rate is calculated from the expression:

$d_{char,0} = \beta_0 \times t$

Where "t" is the time since exposure to fire in minutes, and β_0 is the charring-rate. The charring-rate is usually taken as being 0.9 mm/min for wood panels with a thickness less than 30 mm, and 0.65 mm/min for panels thicker than 30 mm. with a characteristic density of 450 kg/m^3. For wood panels with different density, and thickness less than 20mm, the charring-rate is calculated as:

$\beta_{0,p,t} = \beta_0 \times k_p \times k_h$

$K_p = (450/p_k)^{1/2}$ where p_k is the characteristic density of the wood.

$K_h = (20/h_p)^{1/2}$ where h_p is the thickness of the panel.

2.2 Two-dimensional charring:

In order to simplify the calculation for loss of cross-section with the two-dimensional method, the notional charring depth is used to include the effect of the rounding of corners.

$d_{char,n} = \beta_n \times t$

Where β_n is the notional charring rate according to EN 1995-1-2.

β_n = 0.7 mm/min is commonly used for glue-laminated timber beams and columns. Meaning that after 30 minutes of exposure, 21 mm of cross-section is lost on all exposed sides.

Different densities and species of wood have different charring rates, EN 1995-1-2:2004 use this table for determining the charring rate β_0 and β_n for glue-laminated timber.

Table 1. charring rates for wood [4]

Density	β_0 mm/min	β_n mm/min
Softwood: Density ≥ 290 kg/m^3	0.65	0.70
Hardwood: Density ≥ 290 kg/m^3	0.65	0.70
Hardwood: Density ≥ 450 kg/m^3	0.50	0.55
Wood Panels with thickness ≤ 30mm	0.90	One-dimensional only
Wood Panels with thickness ≥ 30mm	0.70	One-dimensional only

2.3 Reduced cross-section method

The general method for calculating fire resistance in timber structures is known as the “reduced cross-section method”. This is the simplified method for calculating the fire resistance of timber. The method uses the reduction of cross-section of the structural element due to the charring and the thermal effect on the remaining wood to calculate its strength, in ALS (accidental limit state), during a fire.

timber beyond the charring depth is also affected by the heat of the fire, reducing its strength due to decomposing of the timber. It is therefore necessary to calculate an effective cross-section by determining the effective charring depth d_{ef}. This method of calculation is for timber without any fire-protection.

$D_{ef} = d_{char,n}+K_0 \times d_0$

$d_0 = 7.0mm$ (constant)

K_0 is determined by the table:

Table 2 K_0 for effective depth

Time in minutes	K_0
t≤ 20min	t/20
t≥ 20min	1

As seen in table 2, with exposure to a fire for 20 minutes or more k_0 should be taken as 1. meaning that in most cases of fire resistance calculation, the timber structure is affected by a thickness of the charring depth +7mm on all exposed sides. [4]

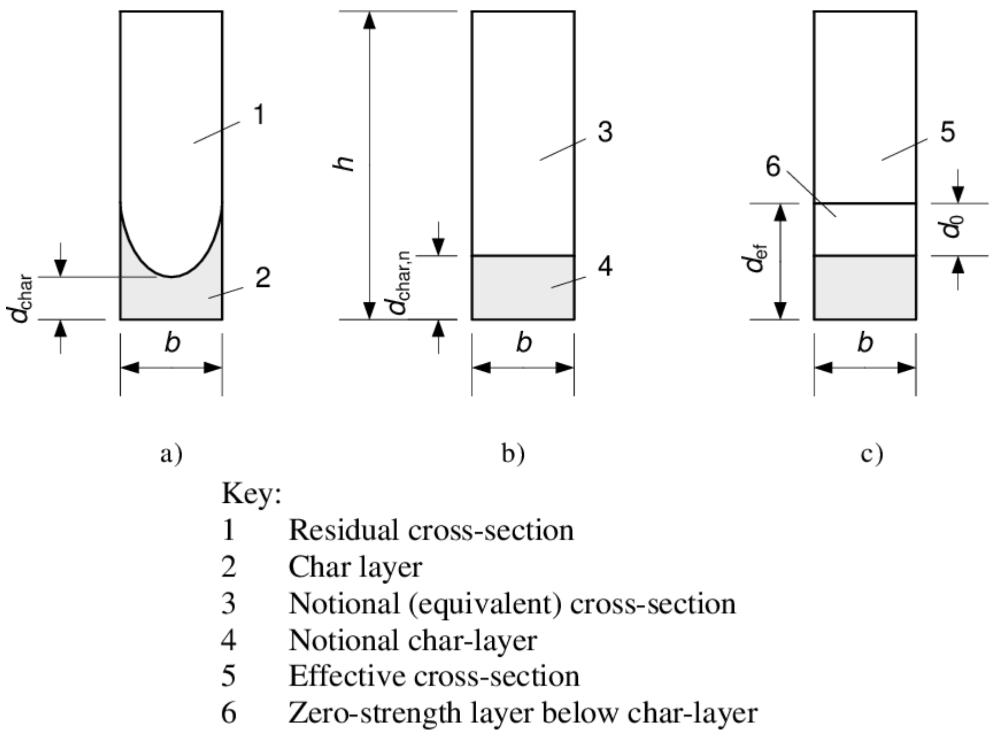

Figure 1 Reduced cross-section method [5]

2.4 The reduced properties method.

The reduced properties method uses the reduced cross-section and the strength loss of the material in a fire-situation to calculate the design resistance of a member in ALS. It can be used on rectangular cross-sections exposed on three or more sides and round columns. It is used by calculating a strength modification factor $k_{mod,fi}$ instead of effective charring-depth to find the strength of a member. To calculate the strength modification factor for the material these equations are used:

$k_{mod,fi}=1.0-(1/200)\times Þ/A_t$ for members in bending.

$k_{mod,fi}=1.0-(1/125)\times Þ/A_t$ for members in compression.

$k_{mod,fi}=1.0-(1/300)\times Þ/A_t$ for members in tension or to calculate modulus of elasticity.

Where Þ=perimeter of the residual cross-section and A_t=area of the residual cross-section.

3 Protection and Cladding

In members protected by insulation or gypsum boards, the charring rate, immediately after failure of the protection, is exceedingly high until a charring depth of 25 mm is reached. After this initial charring it slows down to the notional charring rates for timber. This is especially important when calculating the fire resistance of walls and floors since they are usually clad with gypsum or another type of fire protection. The cross section of floor joists and wall studs is also smaller than most beams and columns and will usually fail before reaching a protective charring depth of 25 mm. [6]

A timber element can start charring due to an increase in heat before complete failure of the protection. There is no good way of calculating the real failure time of the protection, but it has been determined through experimentation with specific types of protection and methods of fastening. the start of charring behind the protection, however, is possible and important to calculate and is typically used as a failure time for protective cladding as well. Failure of protection made from gypsum is often related to dehydration of the panel, glass-fiber reinforcement is sometimes used to increase the failure time.

If the protective layer consists of a wood-based panel, the time of start of charring for the undelaying element is found by the expression:

$t_{ch} = h_p/\beta_0$

t_{ch} = time before start of charring.

h_p = thickness of the panel.

β_0= charring rate of the panel.

With a protective cladding consisting of one layer of type A, F or H gypsum, the given expressions can be used for calculation of delayed charring time. Even tough gypsum of type E, D, R and I have better thermal properties, the expressions can be used as a conservative calculation.

$t_{ch} = 2.8 \times h_p - 14$ (with filled joints or gaps with a width less than 2 mm)

$t_{ch} = 2.8 \times h_p - 23$ (with unfilled joint or gaps larger than 2 mm)

if two layers of type A or H protective cladding is used, the inner layer should be assumed as having only 50% fire resistance since it will already be affected by the increase in temperature.

If using different layers, the strongest type should form the outer layer of the protection. In this case the inner layer retains 80% of its fire resistance [4].

A conservative way of determining the failure time of the protective layer is to calculate it as $t_f = t_{ch}$. Meaning that a protective layer has failed immediately after the timber behind it starts charring. This method of calculation is relevant for gypsum boards of type A, H and wood panels. the failure time of the protective layer is influenced by factors such as vertical or horizontal orientation, size of the panels, and spacing and penetration of screws used for fastening. It has been determined that the minimum penetration depth of screws into the uncharred timber is 10mm. calculated by the expression:

$l_{f,req} = h_p + d_{char,0} + l_a$

h_p = thickness of the panel.

$d_{char,0}$ = charring depth of the timber member.

l_a = 10mm (minimum depth of screw)

the failure time for a specific protection may be specified by the producer. In the case of $t_{ch} \leq t \leq t_f$ meaning that the protective layer fails some-time after the start of charring of the layer behind it. The charring rate can be calculated as β_0 or β_n multiplied by factor k_2.

$K_2 = 1-0.018 \times h_p$ (where h_p is the thickness of the gypsum layer. In the case of several layers, it is the thickness of the inner layer of gypsum)

for timber protected with rock fiber with a thickness between 20-45 mm, with a density of 26 kg/m^3 and being able to withstand 1000 C°, the value for k_2 can be taken from the table.

Table 3 K_2 for protected members.

Thickness in mm	K_2
20	1
≥45	0.6

K_2 with thicknesses between 20 and 45 mm can be determined by linear interpolation.
The expression $t_{ch} = 0.07 \times (h_{ins} - 20) \times (p_{ins})^{1/2}$ can be used to calculate the charring time of timber protected by rock-fiber [4]. Where h_{ins} is the thickness of the insulation, and p_{ins} is the density.

After the failure of the protection, $t_f \leq t \leq t_a$ where t_a is the time limit, the charring-rate can be calculated by multiplying β_0 or β_n with a factor $k_3 = 2$. When $t \geq t_a$ the standard charring-rate, β_0 or β_n, is used.

In the case of $t_{ch} = t_f$, as used with wood-panels and gypsum protection, t_a can be calculated by the expressions:

t_a = minimum value of $(2 \times t_f)$ or $(25/k_3 \times \beta_n + t_f)$

for $t_{ch} < t_f$:

$t_a = (25-(t_f-t_{ch}) \times k_2\beta_n)/(k_3\beta_n) + t_f$

t_a being the time it takes for a protective charr-layer of 25 mm to build up, thus slowing down the charring-rate.

As walls and floors usually consist of several layers of wood panels, gypsum and insulation, it is necessary to calculate a sum fire-resistance for the element. This can be done by calculating the start of charring for the load-bearing structure, usually the studs in a wall or joists in a floor. the charring rate of the structure must also be determined, both before and after the failure of the protection.

example:

a wall structure made with 20 mm timber CLT panel with density of 450kg/m^3 and one layer of 15 mm gypsum panel of type F. the voids in the wall is filled with rock-wool insulation with a density ≥ 26kg/m^3. The studs are made from hardwood with a cross section of 48×148 mm and a charring rate of $\beta_n = 0.7$ mm/min, $\beta_0 = 0.65$ mm/min.

3.1 Method 1: Calculation of protection time

$t_{f,panel} = t_{ch,panel} = h_p/\beta_0 = 20/0.9 = 22$ min

$t_{f,\ gypsum} = t_{ch,gypsum} = 2.8\times h_p - 14 = 28$ min

$t_{ch,\ stud} = 22+28 = 50$ minutes.

immediately after failure of the gypsum, t=50 minutes, the charring rate is:

$\beta_n \times k_3 = 0.7 \times 2.0 = 1.4$ mm/min

t_a = min (2×50) or (25/2×0.7 + 50)

t_a = 17.8+50 = 67.8 minutes from the start of the fire. Meaning that the full insulation effect from the char at 25 mm thickness will occur 67.8 minutes after the start of the fire. Since this is a calculation of fire-resistance on a wall-stud, it is likely to fail before reaching a char level of 25 mm. it would be a good idea to add a layer of gypsum on this wall if it is for a high-rise building.

3.2 Method 2: components additive method

The components additive method uses the sum protection time of all the layers, $t_{ch} = \sum t_{prot}$, to calculate the charring time for a member. [6]

layer 1. 20 mm wood panel.

$t_{prot,0,1}$ = min of (23 × $(h_1/20)^{1.1}$ or h_1/β_0 = 23

$k_{pos,\ exp,1}$ = 1.0 (no preceding layer)

$k_{pos,\ unexp,1}$ = 1.0 (backed by gypsum)

$k_{j,\ 1}$ = 1.0 (joint coefficient, backed by gypsum)

Δt_1 = 0 (no preceding layer)

$t_{prot,\ 1} = (t_{prot,\ 0,\ 1} \times k_{pos,\ exp,\ 1} \times k_{pos,\ unep,\ 1} + \Delta t) \times k_{j,\ 1}$ = 23 minutes

Layer 2. 15 mm gypsum type F.

$t_{prot,\ 0,\ 2}$ = min of $(24 \times (h_2/15)^{1.4})$ or $(30 \times (h_2/15)^{1.2})$ = 24

$k_{pos,\ exp,\ 2} = 0.5 \times (t_{ins,\ 0,\ 2}/\sum t_{prot,\ n\text{-}1})^{1/2} = 0.51$

$k_{pos,\ unexp,\ 2}$ = 1.0

$k_{j,\ 2}$ = 1.0

$\Delta t_2 = 0.22 \times t_{prot,\ n\text{-}1} - 0.1 \times t_{ins,\ 2} + 4.7 = 7.36$

$t_{prot,\ 2} = (t_{prot,\ 0,\ 2} \times k_{pos,\ exp,\ 2} \times k_{pos,\ unep,\ 2} + \Delta t) \times k_{j,\ 2}$ = 19.6 min

t_{ch} = 42.6 min

the stud will start charring after 42.6 minutes.

the charring-rate is then calculated the same way as with method 1.

3.3 Charring rates for studs and joists

There is a simplification done in the above calculation, as the voids in the wall is filled with insulation the charring would mostly be concentrated to the narrow side of the studs. due to the thermal heat-flux around the sides of the insulation there will be extensive rounding of the corners, and no consolidation of the charring rate due to an insulating charr-layer should be considered. For this to be true, it would have to be insured that the insulating batts are tightly fitted to the voids and fastened properly, this is commonly done by oversizing the insulation and mechanically fixing it to the studs with wire. Mineral wool insulation has the best properties for reducing charring on the sides of the studs after the failure of the protective cladding because it retains its shape, even during exposure to high temperatures. the charring rate can be calculated by the expression: $\beta_n = \beta_0 \times k_s \times k_n \times k_p$

k_s is related to the width of the stud, for insulation with stone-wool with a density of 26 kg/m^3 it can be found by:

Table 4. K_s for voids with insulation.

Width of stud or joist in mm	k_s
$38 \leq b \leq 90$	$0{,}00023 \times b^2 - 0{,}045 \times b + 3{,}19$
$b \geq 90$	1

K_n is a conversion of charring depth from the actual rounding of corners to an equivalent rectangular shape used to simplify the expression:

Table 5. K_n for protected members

Width of stud or joist in mm	k_n
$b \leq 60$	1.5
$b \geq 60$	1.25

K_p is dependent on protection given by the cladding at different stages:

Table 6 K_p for protected members

Protection stage	k_p
Un-protected structure	Equal to k_1
Protected but charring $t_{ch} \leq t_f$	Equal to k_2
After protection fails $t \geq t_f$	Equal to k_3

Using this information, assuming a stud width of 48mm and the protection to fail as soon as charring of the wood behind it we get the values:

$\beta_0 = 0.65$ mm

$k_s = 1.63$

$k_n = 1.5$

$k_p = k_3 = 2.0$

$\beta_n = 3.18$ mm/min

previous charring-rate calculated without considering the insulation was $\beta_n = 1.4$ mm/min.

The new charring-rate is considerably higher than the value without rock-wool, however only for the narrow face of the stud instead of on all sides.

the effective charring-depth can then be calculated as $d_{eff} = d_{char,n} + d_0$ where d_0 is found using:

Table 7 d_0 for joists

Exposed side	d_0
In tension	13.5+0.1×h
In compression	21.5+0.1×h

For floor joists with a height ≥ 95 mm and width ≥ 38 mm.

Table 8 d_0 for studs

Exposure	Limitations mm	d_0 mm
Fire on one side. stiff y-y axis.	b≥38 h≥95	13.5+0.1×*h*
Fire on one side. weak x-x axis.	b≥38 h≥95	17+0.25×*h*
Fire on both sides. stiff y-y axis.	b≥38 h=140	25
Fire on both sides. weak x-x axis.	b≥38 h=140	44

For wall studs. [6]

3.4 Origin and use of the components-additive method

There is a difference in the charring time of 7.4 minutes between the two methods. The components additive method being the more conservative. This basic method for calculating the charring time of walls and floors can be found in annex E in Eurocode EN 1995-1-2 and is called the separating function. It relies on the use of tabulated values to determine the coefficient values needed to calculate the fire resistance. The components additive method is meant to offer a more precise way of calculating the fire resistance of a complex structure, built up with different layers of protection and aesthetic panels. It is based on the separating function, which uses tabulated values found by testing, but instead offers general equations to calculate each coefficient. These equations have been formed by a finite number of tests done on both small- and full-scale fire-simulations. The tests were done on unloaded specimen, but the calculation should be accurate for loadbearing structures as well. By considering the fire resistance of each layer, t_{ins} and t_{prot} as the basic insulation and protection value, together with the interaction of the preceding and backing layer by using the position coefficients $k_{\mathrm{pos,\,exp}}$ and $k_{\mathrm{pos,\,unexp}}$. It also considers the effect from different types of joints and their backing with the coefficient k_{j}, and the correction time for the insulation or protection as Δt for layers protected by type F gypsum.

This method has been extensively tested using the Eurocode testing standards and found to be a precise, versatile and safe way of calculating the fire resistance of complex structures consisting of an infinite number of layers.

4 design strength during a fire-situation

The calculation for design strength is done the same way in a fire-situation as in ultimate-limit state but with different safety-coefficients [4].

$f_{d,fi} = k_{mod,\,fi} \times \mathrm{f}_{20}/\Upsilon_{\mathrm{m,\,fi}}$

$S_{d,\,fi} = k_{mod,\,fi} \times s_{20}/\Upsilon_{\mathrm{m,\,fi}}$

$f_{d,\,fi}$ is the design strength during a fire.
f_{20} is the strength fractile 20% of a strength property at normal temperature, calculated as $f_{20}= f_k \times k_{fi}$.

$S_{d,\,fi}$ is design stiffness properties, modulus of elasticity or shear modulus.
s_{20} is the strength fractile 20%, calculated as $s_{20}=s_{0.5}\times k_{fi.}$

$K_{mod,\,fi}$ = 1.0 for glue-laminated wood in a fire-situation.
$Y_{m,\,fi}$ = 1.0 in a fire situation.
k_{fi} = 1.15 for glue-laminated timber.

For a structural element to be suitable in a fire-situation, the following expression must be satisfied: $E_{d,fi} \leq R_{d,t,fi}$. Meaning that the design effect on the structure during a fire is smaller than the resistance of the structural element at the required time of fire resistance.

$E_{d,fi}$ can be simplified to, $E_{d,fi} = E_d \times \eta_{fi}$

η_{fi} = the smaller value of $(G_k+\psi_{fi} \times Q_{k,1})/(\gamma_G \times G_k + \gamma_{Q,1} \times Q_{k,1})$ or

$(G_k+\psi_{fi} \times Q_{k,1})/(\xi \times \gamma_G \times G_k + \gamma_{Q,1} \times Q_{k,1})$

referring to load combinations from (6.10 a) and (6.10 b) in EN 1990:2002.

Equations (6.10 a) and (6.10 b) can also be used directly to calculate design-loads by using reduction-factors for ALS design.

Strength calculations and dimensioning of construction elements will be discussed in greater detail in chapter 9- members in accidental limit state.

5 Factors and design considerations that may influence fire resistance.

There are several factors that influence the charring rate and fire resistance of timber. These include the type of lamination used, if its glue-laminated, what type of adhesive is used and how fire resilient is it? as well as the direction of the grain and density of the wood. The design detailing of elements can also have a large impact on fire resistance. Especially with regards to floors, ceilings and walls, where there can be huge variation in build-up of the protective cladding and structure. Even if the walls and floors are not part of the load-bearing structure, they help dividing a building into fireproof cells to prevent the spread of a fire. In order to design a building that maximizes fire-safety in the most economical and practical way possible it is important to have a good understanding of the factors that affects fire-resistance. A good way to get an understanding of these influencing factors would be Studying tests done on different encapsulation methods, adhesives and external factors. Even though there can be a lot of variation on how the tests are performed with regards to information about wood density, moisture content and the type of fire-test used, they can still give useful insight and guidelines into how we should- or should-not design a building for fire safety.

5.1 Considering different encapsulation methods.

Tests to determine the effect of encapsulation to increase the fire resistance has showed the effects of including an airgap between the protective gypsum-boards and the structural element. Initially the airgap has a positive impact on the rise of temperature of the wood due to the insulating effect of air. However, as the protective layer fails, the charring of the timber is much greater than if the gypsum had been mounted directly on the wood. This might be due to a greater surface of timber being exposed to hot gasses when a crack eventually forms in the gypsum [7]. Due to the high standard for fire-safety, a fire resistance level of R90 being enforced on buildings up to 30 meters high, and the probability of the protection to fail before 90 minutes of exposure. It can be concluded that an unfilled airgap between the protective layer and the timber should be avoided.

A gap between the protective layer and the timber, filled with rock-wool or glass-wool, however proved to have a positive effect on the charring rate even after the failure of the gypsum protection. The increased thermal properties from the layer of insulation, and the elimination of hot gasses affecting the timber after failure of the protection resulted in a delayed charring time, t_{ch}. This method of encapsulation has also shown to greatly increased the failure time of the protection. From t_f = 27.7minutes with gypsum mounted directly on the wood, t_f = 26.7min with a 100 mm airgap (with an increased Charing-rate compared to directly mounted gypsum), to t_f = 78.6min for gypsum with 100 mm of rock-wool insulation. With two layers of gypsum the failure time was found to be 70.8min, which is more than double the failure time found for one layer of gypsum in the same test. these tests were done with 16mm type X gypsum, which is equivalent to type F in terms of fire resistance. Failure of the protective cladding was considered as the time of charring of the wood behind it which is the criteria used according to EN 1995-1-2, $t_f = t_{ch}$ [7]. although this was a study done on small-scale specimen, the results correspond well with those from larger scale testing of fire protection [8], and calculations made according to the standard.

5.2 Adhesives and finger-joints

The methods used for calculating fire resistance is the same for solid timber as for glue-laminated timber. However, with a load bearing structure made from glue-laminated wood, one should consider the effects that the degradation of the adhesive both due to sunlight and heat from a fire-situation may have on strength. As the effect of adhesive strength and the mechanical strength of finger joints are closely related, they should be considered together. The current European standard for testing adhesives is a loaded specimen being heated for two weeks at 70C^{o}, which might be okay to check for degradation due to heating from sunlight, but not applicable in a fire situation. Even though Timber has naturally good thermal properties tests show that the temperature of timber reaches 360 ^{o}C at the effective charr-depth (d_{eff}), and 100^{o}C at 15 mm from d_{eff} [9], during a fire. Meaning the standard test parameters are not satisfactory to check suitability of an adhesive during a fire [10]. There are two main aspects to take into consideration when deciding if an adhesive is suitable.

1. what is the limiting factor, the strength loss of the adhesive, or the timber during a fire situation?
2. Does the charr-layer fall of when it reaches the glue-line (de-lamination)?

The ability of an adhesive to resist against heat is most important for beams in bending. Since the stress distribution for shear force is mostly towards the center of the beam, the fire resistance of the adhesive is less important for the timber element to resist shear force. bending moment, however, has a stress distribution towards the over-and under-side of the beam, meaning that a worse performing adhesive can have a large impact on its ability to resist bending. Research also show that adhesive strength has the largest impact on strength for members in tension, which is usually found on the underside of beams in bending [10].

Since the fire resistance of timber is highly dependent on it forming an insulating char-layer of 25 mm in order to reduce the rising temperature in a member and reduce the charring rate. It is important that the charr-layer stay in place even after reaching the glue-line between lamination layers, a failure to achieve this would result in de-lamination of the cross-section. An adhesive that allows the char to simply fall of will not have a chance to reduce the charring-rate and is therefore not satisfactory when designing a building with a high focus on fire resistance. A lamination thickness of 45mm or more per layer will ensure that the insulating char layer of minimum 26mm is maintained without excessive spalling when the char-depth reaches the glue-line. [11]

The National Standard specifies phenol-formaldehyde and amino-plastic type 1, as suitable adhesives for laminated structural elements. According to the standard no testing at elevated temperatures are required for these types of adhesives [6]. In order to cut cost and lower production time on structural elements, adhesives based on melamine-urea–formaldehyde and one-component polyurethane have been used in recent years. these are classified as type 1 adhesives, and therefore approved according to EN-301 and EN-15425. Polyurethane based adhesives have the added benefit of being more environmentally friendly than previously used adhesives as well. Research has been done at ETH Zurich [10] involving one melamine-urea– formaldehyde and four different one-component polyurethane adhesives to determine their strength in a fire-situation. In their research they tested the tensile strength of a finger-joint with different types of adhesives at temperatures up to 140°C. They also included a control experiment with a solid, unjointed member at elevated temperature as a comparison to see how much the adhesive was compromised relative to the wood itself. The test results can be found in table 9.

Table 9 performance of adhesives

temperature	Control	p-1	p-2	p-3	p-4	m-1
20	42.9	35.6	32.0	31.7	40.0	35.4
60	35.4	25.1	26.3	25.5	33.8	35.1
100	31.2	17.5	16.2	18.1	25.1	30.2
140	25.4	14.1	20.4	16.9	23.4	21.7
$\Delta F_{140^{o}C}$	-	11.3	5.0	8.5	2.0	3.7

Table 9 lists the temperature in Celsius. the control-specimen is timber without any joint. p-1 to p-4 being the different one-component polyurethane based adhesives tested, and m-1 is based on melamine-urea– formaldehyde. All the tests are for tensile forces in N/mm^2. $\Delta F_{140^{o}C}$ lists the difference in tensile strength between the control specimen and the different adhesives at 140°C in N/mm^2. As can be seen from the table, the adhesive listed as p-4 had the least strength loss at 140°C, losing only 2.0 N/mm^2 of tensile strength when compared with the control-specimen. This makes it suitable for use in glue-laminated timber structures with high requirements for fire resistance. The chemical structure of the adhesives was not specified in the test. large variations in temperature-resistance were observed between the different adhesives of the same type. Further testing is therefore necessary to decide the best chemical structure for fire safety, but an adhesive based on polyurethan can be satisfactory. Even though the adhesive did not decompose significantly compared to the wood, it is advised to avoid placing finger-joints near the point of maximum bending moment on a beam due to possible deficiency or variation in quality of the joint.

5.3 Density, moisture content and direction of the grain

The density of the wood could influence the charring rate. Even though it is widely accepted, and the table on charring-rates from EN 1995-1-2 states that a higher density timber has a lower charring-rate, there is contradicting evidence that support the case for using both low- and high-density timber for structural elements. Some tests done on correlation between charring-rate and density found that a higher density has a huge impact on charring rates. Logically a structural element with a higher density has more mass to decompose during a fire. However, many tests have found there to be little correlation between density and charring-rates, or a correlation only in specific circumstances like moisture-content or specific ranges of density in certain types of wood. The argument for low density timber being that the lower thermal conductivity works to keep the internal temperature of the element lower [9]. All tests done concluded that the charring-rates from the standard is conservative.

A higher moisture level will lead to lower charring-rates. In structural elements the moisture content is typically around 12-16%. Most of the tests done on timber during a fire is within this moisture range. Since a higher moisture content can lead to lower structural strength and a faster degradation of the timber it should not be increased for the purpose of fire resistance.

Test done on the effect of grain-direction also show contradictory evidence, with some tests concluding that the charring rate is highest along the grain direction and others finding that the grain direction is of little importance [9]. Timber is the strongest in both tension and compression parallel to the grain, with a loss of strength of more than 87% with forces perpendicular to the grain, calculated from table 1 in NS-EN 338:2009. Due to the huge strength-loss as a result of grain direction, Any structural element should be constructed to ensure that the timber is loaded in its strongest direction.

6 Load calculation

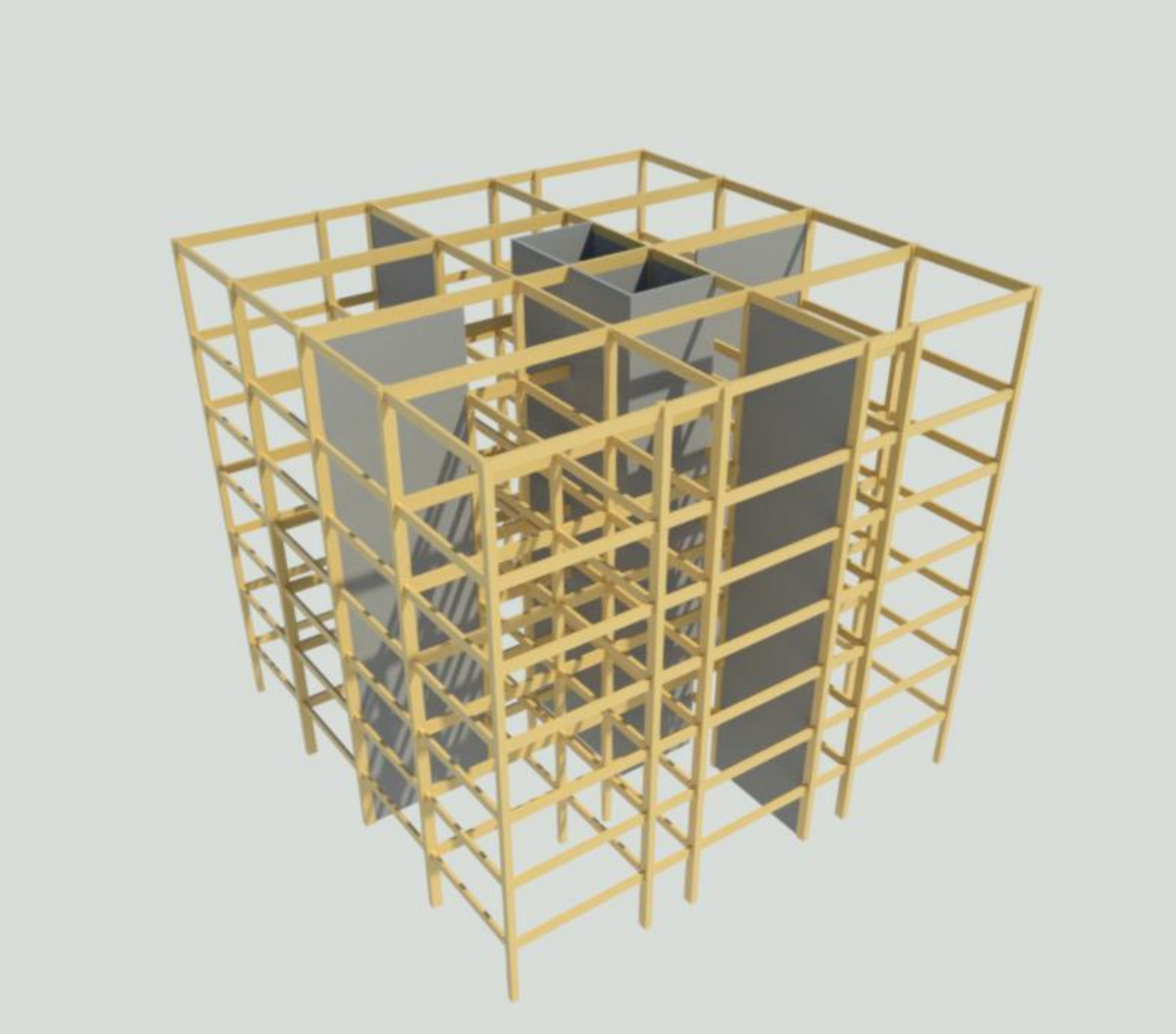

Figure 2 overview of the load-bearing structure

In this thesis I will design and dimension the loadbearing structure of a building according to calculations from the standard. The external forces taken into consideration will be snow load and wind load, the effect of seismic activity will not be calculated. Considering a building with a height of 29meters, the load bearing structure needs to be fire-resistant in R90, meaning that it can burn for 90 minutes without collapsing. The building will have a perfectly square cross-section with sides of 30 meters and an exterior covered with a glass facade. It will have 7 floors, each being 4 meters high except for the top floor which has a height of 5 meters, giving the building a total height of 29 meters. The walls dividing the rooms at each floor will not be loadbearing but will function as a fire-barrier to divide the building into fire-cells, therefore they also need to be fire-resistant. The loadbearing structure will be made from glue-laminated beams and columns without any protective cladding. It also has loadbearing shear-walls on each side as well as the elevator-shaft/stairwell to resist against wind-load and seismic activity. All beams are supported against torsional buckling by the floor. All elements made from glue-laminated timber will be constructed by using one-component polyurethane based adhesives. The building components would be prefabricated in a factory before assembly at the construction-site. The first floor will function as a reception and cafeteria. 2^{nd} - 6^{th} floor will have offices and 7^{th} Floor will be reserved for conference-rooms.

the building will be located next to scandic forum hotel in Tjensvoll, Stavanger with a height of 50 meters above sea level. Located 8 km from the coast with a terrain category III and being 2.5 km from terrain category II. The terrain surrounding the building is mostly flat.

6.1 Snow load

Using formulas and statistics found in NS EN 1991-1-3 2003+NA 2008. [12]

$S = \mu_i \times c_e \times c_t \times s_k$

μ_i is a shape coefficient regarding the angle of the roof, in this case the angle is 0^{o} which means $\mu_i = 0.8$.

c_e is an exposure coefficient and should be considered as 1.0 unless otherwise specified.

c_t is a thermal coefficient which should be considered as 1.0.

s_k is the characteristic snow load on the ground found in table NA.4.1 (901) for Stavanger it is specified as 1.5 kN/m^2 under a height of 150 meters above sea level.

for our building the snow load on the roof is:

$s = 0.8 \times 1.0 \times 1.0 \times 1.5 \underline{= 1.2 \text{ kN/m}^2}$

with a surface area of 900 m^2 the total snow-load on the roof is 1080 kN.

6.2 Wind load

using the simplified method of calculation from NS EN 1991-1-4 2005+NA 2009. [13]

$q_{(z),\, p} = k_1 \times k_2 \times k_3 \times c^2_{dir} \times c^2_{alt} \times c^2_{season} \times c^2_{prob.} \times q_{p,\, 0,\, z}$

k_1 is an orography factor considering the terrain. Set to 1.0

k_2 is a factor that takes steep terrain into account. Set to 1.0

k_3 is a factor considering the change of terrain, using values found in table v.1 of the standard and Δn_{BA}=1.

K_3 = 1.05.

c^2_{dir} / c^2_{alt} / c^2_{season} / c^2_{prob} are factors considering direction, altitude, season and the probability of wind strength in the next 50 years, they will all be considered as 1.0.

$V_{b,\,0}$ = 26 m/s (value from table NA 4. (901.1)) is the basic wind velocity in Stavanger.

$q_{p,\,0,\,z}$ with a building height of 29 meters and $v_{b,\,0}$ = 26 m/s is 1.1 kN/m^2 using fig. v.1 c to calculate.

$q_{p,\,0,\,z}$ = 1.1 kN/m^2

$q_{(z),\,p}$ =1.05 × 1.1 = 1.16 kN/m^2

To find the wind pressure working on the building we will simplify the calculations by assuming the wind will hit with 90^o on the sides being 30 meters wide.

Table 10 wind load on each side of the building

side	$C_{pe,10}$	$C_{pe,\,1}$	Area m^2	$q_{(z),\,p}$	Total wind force kN
A	-1.2	-1.4	180	1.16	-250.6
B	-0.8	-1.1	720	1.16	-668.2
D	0.8	1.0	900	1.16	835.2
E	-0.5		900	1.16	-522

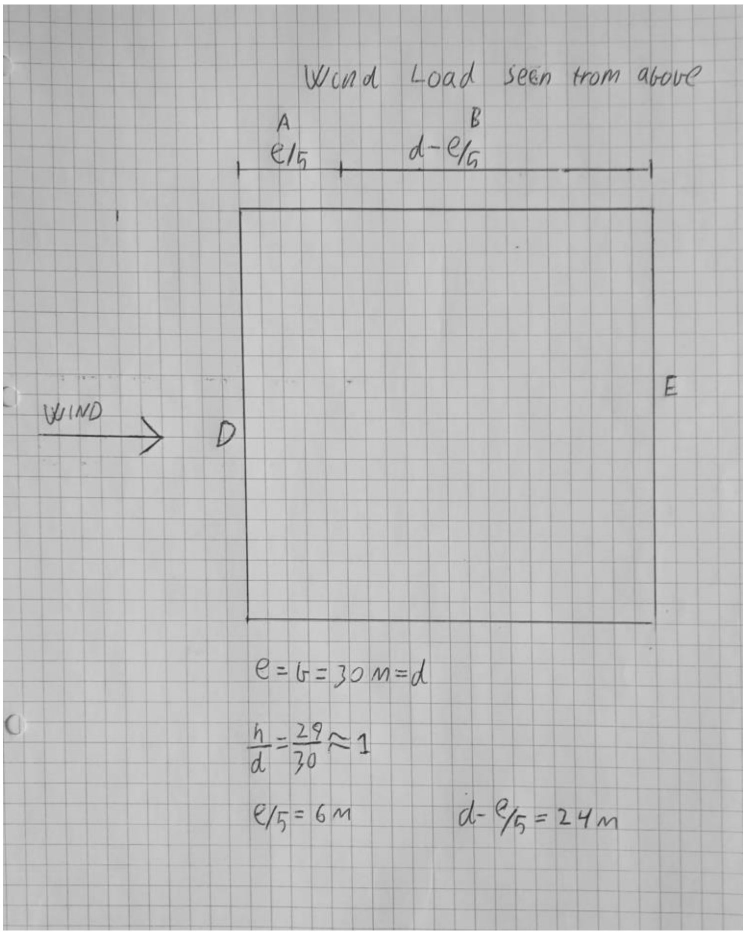

Figure 3 wind load on the building as seen from above.

using the self-weight of the building calculated in chapter 7, the building's ability to resist overturning was calculated: M_w=(835.2+522)kN×15m=20 358kNm is the moment on the building from wind-load.

M_b=7×(1.47×30×30)kN×15m=138 915kNm is the moment just from the self-weight of the floor.

M_b=138 915kNm>M_w=20 358kNm -> the building is safe from overturning.

6.3 imposed load

This is a building with offices and conference rooms. The values for imposed load are taken from table NA. 6.1 and NA. 6.2 in NS-EN 1991.1.1:2002/NA:2008.

floors 2 - 6 contain offices in category B, they have a characteristic imposed load of 3.0 kN/m^2.

7^{th} floor is reserved for conference rooms, in category C2 with a characteristic imposed load of 4.0 kN/m^2.

A reduction factor for the floors assigned to offices can be calculated as: $\alpha_n = 2+(n-2)\times\psi_0/n$ with n being the number of consecutive floors of the same category and ψ_0 being the combination factor for offices.

$n = 4$

$\psi_0 = 0.7$

this reduction-factor is meant to be used when dimensioning the columns on the first floor.

7 Design and charring-times for walls and floors

To calculate the self-weight of each floor we first need to design the inner walls and floors. Even though the walls are not loadbearing, they will need to be fireproof in EI90 to make each room and floor function a fire-cell. EI90 means that the wall can be exposed to a fire for 90 minutes without the fire spreading to the unexposed side. Calculations will be done according to the components additive method.

7.1 Walls

Layer 1: type F gypsum, 15 mm.

Layer 2: type F gypsum, 15 mm.

Layer 3: CLT plywood, 12 mm.

studs: 148x48 mm glue-laminated timber (hardwood) with a density ≥ 290 kg/m^3. CC 600 mm.

the voids between studs are filled with mineral wool which we assume is properly fitted.

both sides of the walls are built up in the same way to create a fire-cell.

Fire- resistance, Layer 1:

$t_{prot,\,0,1} = 30 \times (h_1/15)^{1.2} = 30.0$ min

$k_{pos,exp,1}$ = 1.0 (first layer)

$k_{pos,unexp,1}$ = 1.0 (gypsum backing)

$k_{j,1}$ = 1.0 (gypsum backing)

Δt = 0 (first layer)

$t_{prot,1} = (t_{prot,0,1} \times k_{pos,exp,1} \times k_{pos,unexp,1} + \Delta t) \times k_{j,1}$ = 30.0 min

layer 2:

$t_{prot,\,0,\,2} = 30 \times (\,h_1/15)^{1.2}$ = 30.0 min

for $\sum t_{prot,1} > t_{prot,0,2} / 2$ -> $k_{pos,exp,2} = 0.5 \times (t_{prot,0,2}/\sum t_{prot,1})^{1/2} = 0.5 \times (30.0/30.0)^{1/2} = 0.5$

$k_{pos,unexp,2}$ = 1.0 (backed by timber)

for $t_{prot,0,2} \geq 12$ -> $\Delta t = 0.22 \times t_{prot,1} - 0.1 \times t_{prot,0,2} + 4.7$ = 8.3 min

$k_{j,2}$ = 1.0 (backed by timber)

$t_{prot,2}$ = (30.0 × 0.5 × 1.0 + 8.3) × 1.0 = 23.3 min

layer 3:

$t_{ins,0,3} = 19 \times (h_3/20)^{1.4} < h_4/\beta_0 = 12/0.9$ = 13.3 min

for $\sum t_{prot,1,2} > t_{ins,0,3} / 2$ -> $k_{pos,exp,3} = 0.5 \times (t_{ins,0,3}/\sum t_{prot,1,2})^{1/2} = 0.5 \times (13.3/53.3)^{1/2} = 0.25$

$k_{pos,unexp,3} = 0.5 \times h_3^{0.15} = 0.72$ (backed by insulation)

for $t_{ins,0,3} > 12$ -> $\Delta t = 0.22 \times t_{prot,2} - 0.1 \times t_{ins,0,3} + 4.7$ = 8.5 min

$k_{j,3}$ = 1.0 (backed by insulation)

$t_{prot,3}$ = (13.3 × 0.25 × 0.72 + 8.5) × 1.0 = 10.89 min

total time until charring of the studs occur = $\sum t_{prot}$ = 64.19 minutes.

The voids in the wall are filled with stone-wool, it is assumed that this is properly fitted to the studs. since the wall is non-structural, the stud can burn 2/3 of the way through before we consider the wall as failed. Maximum charring depth is therefore 98.6 mm.

the charring rate can be calculated as $\beta_n = \beta_0 \times k_s \times k_n \times k_p$.

β_0 = 0.65 mm/min

$k_s = 0{,}00023 \times b^2 - 0{,}045 \times b + 3{,}19 = 1.56$

k_n = 1.5

$k_p = k_3$ = 2.0

β_n = 0.65 × 1.56 × 1.5 × 2.0 = 3.04 mm/min

The charring time will therefore be $t_{ch,stud}$= 98.6/3.04 = 32.4 min

the time from the start of fire until failure of the wall is then t_f = 64.19+32.4 = 96.59 minutes. This satisfies our requirement of EI90.

7.2 Self-weight of the wall

$g_{q,k,panels}$= (760kg/m^3 × 0.015m × 4 + 290kg/m^3 × 0.012 × 2) × 9.81/1000 = 0.51 kN/m^2.

$g_{q,k,studs}$ = 290kg/m^3 × 0.048×0.148m × 0.6/1 × 9.81/1000 = 0.012 kN/m^2

$g_{q,k,\ ins}$ = 26kg/m^3 × 0.148m × 9.81/1000 = 0.04 kN/m^2

self-weight of electrical equipment and piping is estimated at $g_{q,k,ext}$= 0.05 kN/m^2

total self-weight of the wall is then $g_{q,k,wall}$ = 0.62 kN/m^2

7.3 Floors

The construction of this element is different on each side. The floor construction needs to be fire-resistant in REI90, meaning it will both retain its structural integrity and limit the spread of fire for 90 minutes. One side acting as a ceiling for the floor below, and one side acting as a floor. The first calculation for fire resistance will be done by considering the floor as the side exposed to fire.

Layer 1: 30 mm hardwood with density > 290 Kg/m^3.

layer 2: 15 mm, type F gypsum.

layer 3: 15 mm, type F gypsum.

the voids in the floor is filled with stone-wool insulation.

Layer 1:

$t_{prot,0,1}$ = 30 × (h$_1$/20)$^{1.1}$ < h$_1$/β_0 = 30/0.70 = 42.8 min

$k_{pos,exp,1}$ = 1.0 (first layer)

$k_{pos,unexp,1}$ = 1.0 (gypsum backing)

$k_{j,1}$ = 1.0 (gypsum backing)

$\Delta t = 0$ (first layer)

$t_{prot,1} = (t_{prot,0,1} \times k_{pos,exp,1} \times k_{pos,unexp,1} + \Delta t) \times k_{j,1} = \underline{42.8 \text{ min}}$

Layer 2:

$t_{prot,\,0,\,2} = 30 \times (\,h_1/15)^{1.2} = 30.0$ min

for $\sum t_{prot,1} > t_{prot,0,2} / 2$ -> $k_{pos,exp,2} = 0.5 \times (t_{prot,0,2}/\sum t_{prot,1})^{1/2} = 0.5 \times (30.0/42.8)^{1/2} = 0.41$

$k_{pos,unexp,2} = 1.0$ (backed by gypsum)

for $t_{prot,0,2} > 8$ min -> $\Delta t = 0.1 \times t_{prot,1} – 0.035 \times t_{prot,0,2} + 1.2 = 4.43$ min

$k_{j,2} = 1.0$ (backed by gypsum)

$t_{prot,2} = (30.0 \times 0.41 \times 1.0 + 4.43) \times 1.0 = \underline{16.73 \text{ min}}$

Layer 3:

$t_{ins,0,3} = 24 \times (\mathrm{h}_3/15)^{1.4} = 24.0$ min

for $\sum t_{prot,1,2} > t_{ins,0,3} / 2$ -> $k_{pos,exp,3} = 0.5 \times (24.0/59.5)^{1/2} = 0.31$

$k_{pos,unexp,3} = 0.5 \times \mathrm{h}_3^{0.15} = 0.75$

for $t_{ins,0,3} > 8$ min -> $\Delta t = 0.1 \times t_{prot,1,2} – 0.035 \times t_{ins,0,3} + 1.2 = 6.3$ min

$k_{j,3} = 1.0$ (voids filled with stone wool)

$t_{ins,3} = (\,24.0 \times 0.31 \times 0.75 + 6.3) \times 1.0 = \underline{11.88 \text{ min}}$

t_{ch} for the joists when exposed to fire from the floor side is 71.4 minutes.

From the underside (ceiling) exposed to fire, the construction will consist of three layers of 15 mm, type F gypsum.

layer 1:
$t_{prot,\,0,1} = 30 \times (h_1/15)^{1.2} = 30.0$ min
$k_{pos,exp,1} = 1.0$ (first layer)
$k_{pos,unexp,1} = 1.0$ (gypsum backing)
$k_{j,1} = 1.0$ (gypsum backing)
$\Delta t = 0$ (first layer)
$t_{prot,1} = (t_{prot,0,1} \times k_{pos,exp,1} \times k_{pos,unexp,1} + \Delta t) \times k_{j,1} = \underline{30.0 \text{ min}}$

Layer 2:
$t_{prot,\,0,\,2} = 30 \times (\,h_1/15)^{1.2} = 30.0$ min
for $\sum t_{prot,1} > t_{prot,0,2} / 2$ -> $k_{pos,exp,2} = 0.5 \times (t_{prot,0,2}/\sum t_{prot,1})^{1/2} = 0.5 \times (30.0/30)^{1/2} = 0.5$
$k_{pos,unexp,2} = 1.0$ (backed by gypsum)
for $t_{prot,0,2} > 8$ min -> $\Delta t = 0.1 \times t_{prot,1} – 0.035 \times t_{prot,0,2} + 1.2 = 3.15$ min

$k_{j,2}$ = 1.0 (backed by gypsum)
$t_{prot,2}$ = (30.0 × 0.5 × 1.0 + 3.15) × 1.0 = <u>18.15 min</u>

Layer 3:

$t_{ins,0,3} = 24 \times (h_3/15)^{1.4} = 24.0$ min

for $\sum t_{prot,1,2} > t_{ins,0,3}/2$ -> $k_{pos,exp,3} = 0.5\times(24.0/48.15)^{1/2} = 0.35$

$k_{pos,unexp,3} = 0.5\times h_3^{0.15} = 0.75$

for $t_{ins,0,3}$ > 8 min -> $\Delta t = 0.1\times t_{prot,1,2} - 0.035\times t_{ins,0,3} + 1.2 = 5.17$ min

$k_{j,3}$ = 1.0 (voids filled with stone wool)

$t_{ins,3}$ = (24.0 × 0.35 × 0.75 +5.17) × 1.0 = <u>11.47 min</u>

The charring-time for the joists exposed to fire from the underside is t_{ch} = 59.62 minutes.

Using the information above it is possible to calculate an approximate self-weight of the floor to use when further dimensioning the floor-joists.

7.4 Self-weight of the floor

$g_{q,k,timber-panel}$ = 290kg/m^3 × 0.03m × 9.81/1000 = 0.085 kN/m^2.
$g_{q,k,gypsum}$ =760kg/m^3 × 0.015m × 5 × 9.81/1000 = 0.56 kN/m^2. [14]
the self-weight of ventilation, lighting and electrical equipment is estimated at
$g_{q,k,tech}$ = 0.55 kN/m^2.
the estimated thickness and width of the floor-joists is 270X73mm, thus putting the self-weight of the joist at $g_{q,k,joist}$ = 450 kg/m^3 × 0.27 × 0.073 × 9.81/1000 = 0.09 kN/m, and for the insulation at $g_{q,k,ins}$= 26 kg/m^3 × 0.270m × 9.81/1000 = 0.07 kN/m^2.
the longest span of the joists is 7.5 meters, with a spacing of 400 mm. we can therefore calculate the self-weight of the joists to be $g_{q,k,joist}$ = 0.09 × 1/0.4 = 0.225 kN/m
the total self-weight of the floor is therefore <u>$g_{q,k,floor}$= 1.47 kN/m^2.</u>

8 Dimensioning of structural elements

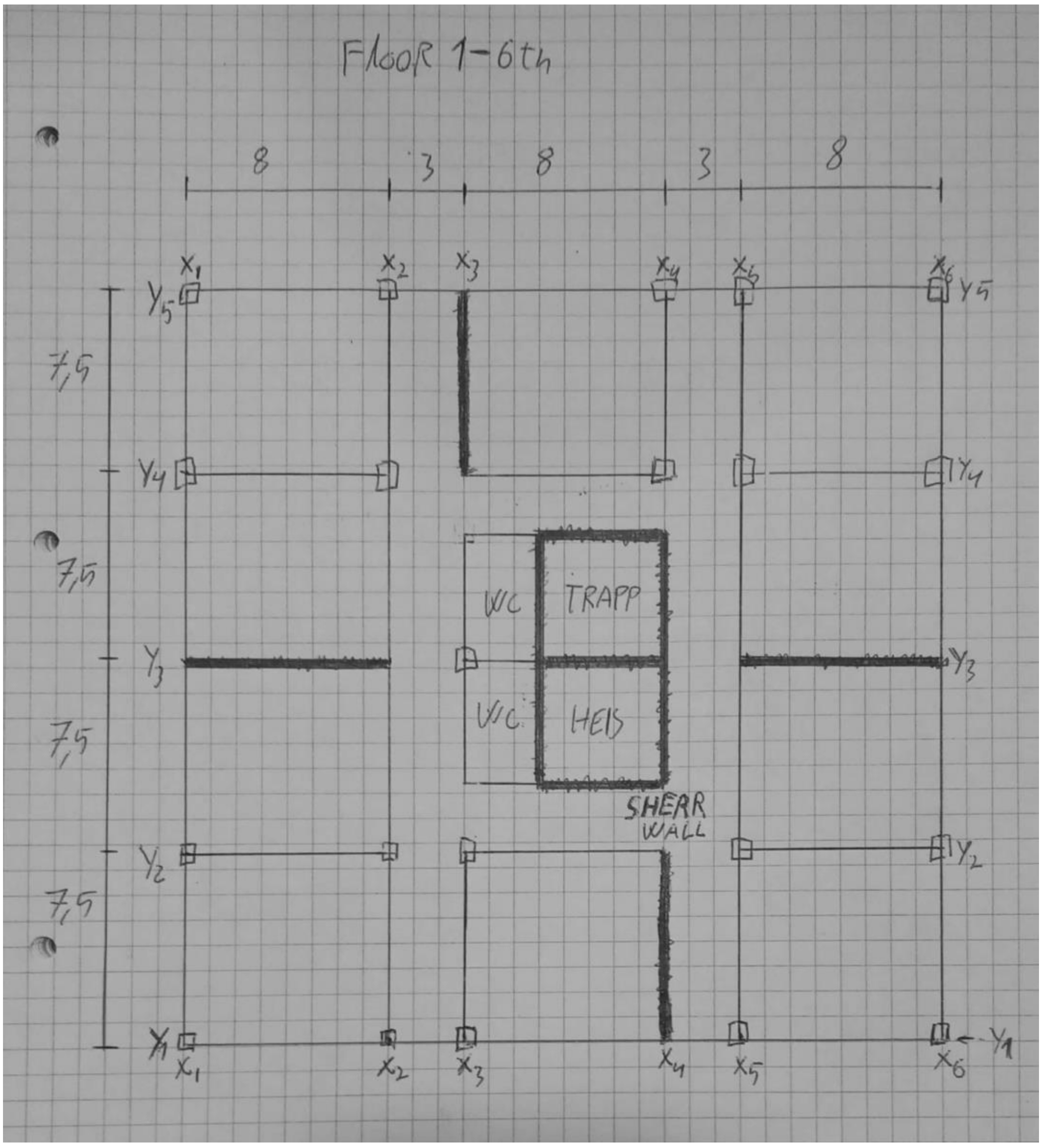

Figure 4 simple plan view of the building, floors 1-6th.

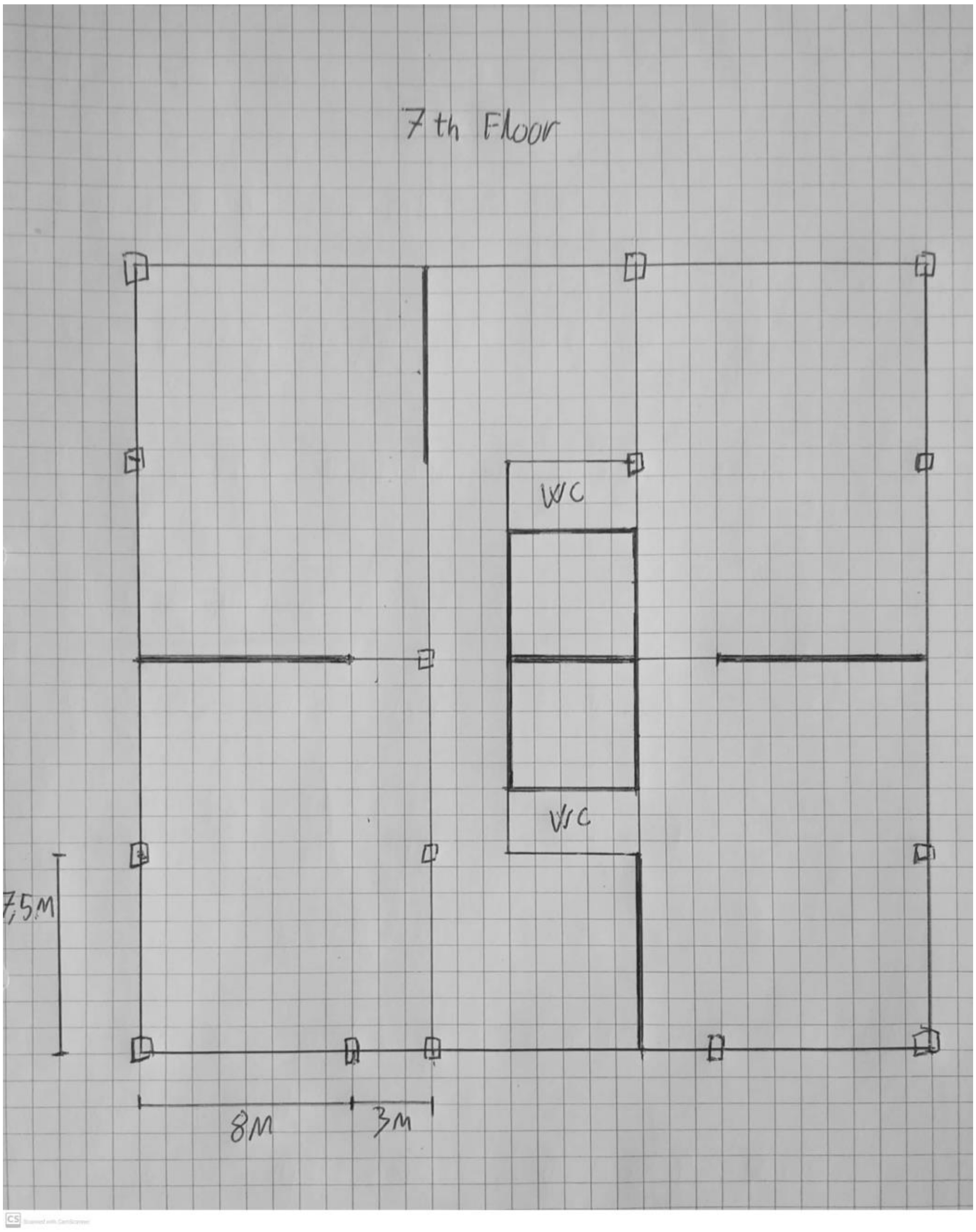

Figure 5 plan view of 7th floor

From figure 4 we can see the plan view of the building from 1st-6th floor. The beams and columns are named X_{1-6} and Y_{1-5}. The beams are continuous from one side of the building to the other. The shear-walls are marked with thick lines.

The internal columns on line X_2 and X_5 stop after the 6th floor with a total height of 24meters. they do not help support the roof. This is done to give more space for the conference-rooms on the 7th floor as seen in figure 5. The beams holding up the roof in the Y-direction therefore span 11meters.

All the internal walls are supported by glue-laminated beams, they also support the floors. The longest span for a beam supporting an internal wall is 8 meters. All the calculations and dimensioning is done according to NS-EN 1995-1-1: 2004+A1: 2008+NA:2010. [15]

for the 7th floor we have an imposed load of 4.0 kN/m^2. From table NA 6.1 in NS-EN 1991-1-1:2002+NA: 2008. [16]

the beam has a self-weight of 490 kg/m^3.

the wall assembly has a load of 0.62 kN/m^2.

the floor and ceiling has a load of 1.47 kN/m^2.

due to the floor joists the beams are supported against lateral torsional buckling. A simplification has been made by assuming that the loads from the floor distribute evenly to all beams in its influence area. Using 6.10 a and b with an approximated influence area of 30m^2 to find design values for loads:

$$6.10\text{ a.} = 1.35 \times (4m \times 8m \times 0.62kN/m^2 + 1.47kN/m^2 \times 30m^2) + 1.5 \times 0.7 \times 4.0kN/m^2 \times 30m^2 = \underline{212.3\text{ kN}}$$

$$6.10\text{ b} = 1.2 \times (4 \times 8 \times 0.62 + 1.47 \times 30) + 1.5 \times 4.0 \times 30 = \underline{256.7\text{ kN}}$$

Using the partial factors from expression 6.10 b for dimensioning:

distributed design-loads from:

walls = 3.0kN/m

floor = 1.76kN/m^2

imposed load on floor = 6kN/m^2

imposed load from snow = 1.8kN/m^2

as the forces on the 7th floor is slightly higher than the others due to being in class C2, we will use these values for dimensioning of all beams of the same position in all floors.

for a beam in the class GL 36h, values for characteristic strength are taken from NS-EN 1995-1-1: 2004+A1: 2008+NA :2010 table 1. [15]

Bending: $f_{m,g,k}$= 36 N/mm^2

Shear: $f_{v,g,k}$= 4.3 N/mm^2

Compression: $f_{c,0,g,k}$=31N/mm^2

and the design strength of the beam: $X_d = K_{mod} \times X_k/Y_m$

where:

X_k is the characteristic value of strength is ether 36N/mm^2 for bending, 4.3 N/mm^2 for shear or 31N/mm^2 for compression.

Y_m is the partial factor of a material property = 1.15 for glue-laminated elements.

K_{mod} is a modification factor = 0.8 for service class 1 with medium term load such as imposed load being dominant.

$f_{m,d}$ = 0.8 × 36/1.15 = 25 N/mm^2

$f_{v,d}$ = 0.8 × 4.3/1.15 = 2.99 N/mm^2

$f_{c,d}$=21.56N/mm^2

8.1 Beams

As the thickness of the floor joists is set at h=270mm and the thickness of the walls is b=202mm, we will use a cross-section of 700x202 mm for the beam. Doing this we can ensure the beam is not protruding outside the wall. maximum bending moment for beams has been calculated by using an online calculator with a triangular load distribution from the floor and imposed load, and a uniformly distributed load from the wall.

for the beam with an 8m span:

bending moment:

$M_{y,ed}$= 334.56 kNm

Shear forces:

$V_{z,ed}$= 128 kN

The beams are named $Y_{2,4}$,as marked in figure 4, and span 8m. the load from the floor and imposed load is distributed triangularly while self-weight of the beam and walls are uniformly distributed.

Dimensioning against bending:

$M_{y,rd} = W_y \times f_{m,d}$

$W_y = b \times h^2/6 = 16496666.7\ mm^3$

$M_{y,rd} = 412.4$ kNm (design resistance of the beam against bending)

self-weight of the beam being: 490×0.7×0.202×9.8/1000 = 0.624kN/m × 1.2 =0.75kN/m it adds to the maximum moment with 6kNm and the shear force with 3kN.

$M_{y,rd}= 412.4\ kNm \geq M_{y,ed}=340.56kNm$

Dimensioning against shear stress:

$\tau_d \leq f_{v,d}$ needs to be satisfied.

$\tau_d= 3v_d /2b_{eff} \times h$

$b_{eff} = k_{cr} \times b = 0.8 \times 202 = 161.6mm$

$V_{z,r,d} = V_{z,d}= 2/3 \times (2.99 \times 161.6 \times 700) = 225485.8N = 225.48kN$

$V_{z,ed} = 128+3\ kN \leq V_{z,r,d} = 225.48\ kN$ cross-section is good in shear.

the beams designated for the outer perimeter supports a glass-front with a weight of 30 kg/m^2.

the force on the beam from the glass is therefore $q_{g,k}$=0.29 kN/m^2 and design force $q_{g,d}$=0.35 kN/m^2. The beam has a length of 8m on the X-axis and 7.5m on the Y-axis. the imposed load and load from the floor has a triangular shape. The beams are named $X_{1,5}$ and $Y_{1,6}$.

The design load is therefore:

glass-front = 1.4 kN/m

floor = 1.76kN/m^2

imposed load floor = 6kN/m^2

the forces are greatest on the beam in the X-axis and is therefore the dimensioning side.

maximum bending moment is then: $M_{y,ed}$= 170.0 kNm

with a shear force of: $V_{y,ed}$=65.0 kN

Dimensioning against bending:

thickness of the beam is 148mm. height is 550mm.

W_y= 7461666.6mm^3

$M_{y,rd}$= 186.5 kNm

Shear:

τ_d= 3v_d /2b_{eff}×h

b_{eff} = k_{cr}×b = 0.8×148 = 118.4mm

$V_{z,r,d}$ = $V_{z,d}$= 2/3 × (2.99×118.4×550) = 129805.8N = 129.8kN

forces from the self-weight amounts to 0.47 kN/m. added bending moment is then 3.3 kNm and shear 1.76 kN.

$M_{y,rd}$= 186.5 kNm ≥ $M_{y,ed}$=173.3kNm

$V_{z,r,d}$ = 129.8 kN > $V_{z,ed}$=65.5 kN

cross-section is good in both bending and shear.

For beam $X_{2,3,4,5}$. The length of the beam is 7.5 meters. loads from the wall, floor and imposed loads are the same as earlier and also in a triangular shape on each side of the beam.

Design bending stress:

$M_{y,ed}$ = 227.3 kNm

Design shear stress:

$V_{y,ed}$ = 94.3 kN

With a cross-section of 202X550mm:

$M_{y,rd}$ = 254.6 kNm>$M_{y,ed}$=227.3kNm

$V_{y,rd}$ = 177.1 kN>$V_{y,ed}$=94.3kN

The floor joists, with dimensions of 270x73mm are experiencing a uniformly distributed load of 1.76kN/m^2×0.4m+6×0.4=3.10kN/m per meter of joist. They have a span of 7.5 meter.

$M_{y,ed}$ =21.8kNm.

w=b×h^2/6= 886950mm^3

$M_{y,rd}$ = 22.17kNm > $M_{y,ed}$ = 21.8kNm

$V_{z,ed}$ = 11.63kN

$v_{z,rd}$ = 2/3×0.8×b×h×2.99 = 31.4kN > $v_{z,ed}$ = 11.63kN

cross-section is suitable in bending and shear.

The beams holding up the roof for the conference rooms:

The floor is divided into a hallway and four conference rooms of equal size as seen in figure 5. The roof is being held up by columns giving the beams a span of 11m in the x-direction with a spacing of 7.5m in the y-direction. The snow-load is 1.2 kN/m^2 and the self-weight is assumed to be 2 kN/m^2. Using Eq 6.10 b the forces are 4.2 kN/m^2. The beam around the edge is the same as for floors below. Using an influence area with 7.5m of maximum width per meter of beam for calculations.

Bending moment: $M_{y,ed}$ = 317.63 kNm
shear force: $V_{y,ed}$ = 86.63 kN

Seeing that a beam of 202x700mm as used in the lower floors is sufficient it will be used for holding up the roof as well.

8.2 Columns

each of the columns will be built as a single unit with the same cross section for each floor. GL36h has a compressive strength of $f_{c,0,g,k}$= 31N/mm^2 giving the timber a design compressive strength of $f_{c,0,d}$=21.56 N/mm^2.

For the corner column the loads are:

from beams = 1.2×490×(7.91)×9.81/1000 = 45.62kN.
from floors = 1.764(20.6+15×6) = 195.1 kN.
imposed loads = 1.5×(4+1.2)×20.6+1.5×(3×5)×15 = 498.18kN.
from glass= 1.2×0.29×240=83.5
combined load is N_{ed}=822.4kN.
Dimension will then be 300x300mm. $N_{r,d}$=A×$f_{c,0,d}$=300×300×21.56. giving the cross-section a design compressive strength of $N_{r,d}$=1940.4kN.

For the columns of the outer perimeter y-axis the loads are:

from beams = 1.2×490×11.59×9.81/1000 = 66.85kN.

from floors = 1.764×(30×5+41.25) = 317.52 kN.

from walls= 1.2×0.62×4×3.3×5=49.1kN

imposed loads = 1.5×(4×41.25+3×5×30) = 909kN.

from roof=(1.2×2+1.5×1.2) ×41.25=173.25kN

combined load is N_{ed}=1515.7kN.

we will utilize the same column as for the corners: 300x300mm cross-section.

For the columns of the outer perimeter x-axis loads are:

from beams = 1.2×490×14.7×9.81/1000 = 84.8kN.

from floors = 1.764×(20.6×5+35.7) = 244.7kN.

imposed loads = 1.5×(4×35.7+3×5×20.6) = 677.7kN.

from walls= 1.2×(0.62×74.25+0.29×185) = 119.6

from roof=(1.2×2+1.5×1.2)×35.7=149.9kN

combined load is N_{ed}=1276.7kN.

we will utilize the 300x300mm cross-section.

The internal columns of $x_{3,4}$ have loads as:

from beams = 1.2×490×(2.4+1.84×6)×9.81/1000 = 78.21kN.

from floors = 1.764×(41.25×6) = 436.6kN.

from roof= 1.2×2×71.25=171.0kN

from walls= 1.2×42.9×0.62×6=191.5kN

imposed loads = 1.5×(3×5×41.25+4×41.25+1.2×71.25)= 1303.9kN.

combined load is N_{ed}=2181.2kN.

a cross section of 350x350 mm will be utilized, giving the cross-section a design strength of $N_{r,d}$=2641.1kN.

The internal columns of $x_{2,5}$ have the same loads as $x_{3,4}$ but without the roof.

combined load is therefore N_{ed}=2010.2kN and cross-section 350x350 will be used.

8.3 Shear-walls

with a building of this size it is necessary to utilize shear-walls in order to stabilize the construction against horizontal forces such as wind load and seismic activity. The shear walls for this building is made up of massive cross-laminated timber that cover the full height of the building with a thickness of 202mm. They are in position Y_3 on each side of the building, extending 8 meters towards the center to give support in the X-direction. As well as X_3 and X_4 on either side to support in the Y-direction. The elevator-shaft and staircase walls are made in the same way and will also support against horizontal force while also supporting weight of the elevator and staircase vertically. The total wind load that will affect the building in each direction was calculated to be 1357.2kN. as calculated before, GL36H has a design shear strength of $f_{v,d}$=2.99N/mm^2.

The shear-walls in the Y-direction has a surface-area of A=200×7500mm on each side of the building, giving it a design strength of

$V_{z,rd}$=2/3×0.8×200×7500×2.99=2392.0kN>$V_{z,ed}$=1357.2kN.

While in the X-direction they have a design strength of $V_{z,rd}$=2551.4kN>$V_{z,ed}$=1357.2kN.

they also support the surrounding floor-area with a load of N_{ed}=3697.0kN each.

The walls supporting the elevator and staircase have a design compressive strength of $N_{r,ed}$=A×$f_{c,0,ed}$=(10 000+5000+5000)×200mm×21.56N/mm^2=862 400kN when considering one wall as compromised due to doorways. These walls also support the surrounding floor-area with a load of 4362.4kN.

9 Members in the accidental limit state

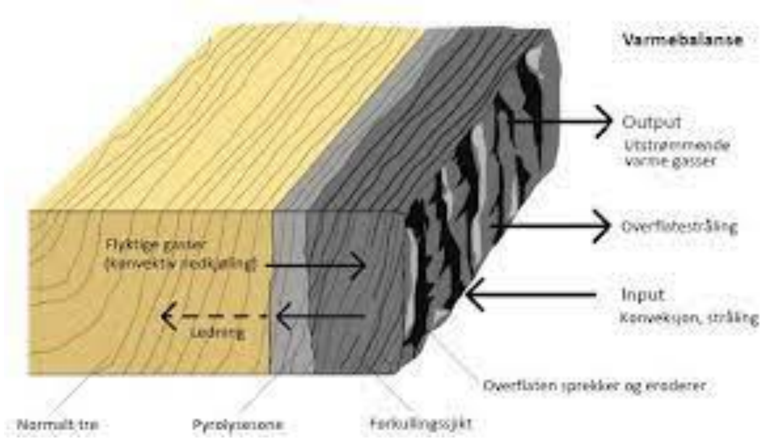

Figure 6 pyrolysis of a joist [17]

Figure 6 is taken from Limtreboka, [17] it shows charring on a wood element.

Design strength of timber in a fire situation is calculated as before with equations from the strandard [15] and limtreboka [17]. Using different values for k_{mod} and $Y_{m.}$

$X_d = K_{mod,fi} \times X_k / Y_{m,fi}$

$K_{mod,fi}=1.0$

$Y_{m,fi}=1.0$

using this equation we find design strength in:

bending: $M_{d,fi}=36.0N/mm^2$

shear: $V_{d,fi}=4.3N/mm^2$

compression: $f_{c,d,fi}=31.0N/mm^2$

In the situation of a fire, the design loads are calculated as:

$1.0 \times g_k + 1.0 \times \psi_1 \times q_k + 1.0 \times \psi_2 \times s_k$

where $\psi_1=0.5$ for category B and 0.7 for category C.

$\psi_2= 0.3$ for category B and 0.6 for category C.

we will first calculate the limit state for 7th floor, category C.

values taken from table NA.A1.1. [16]

load due to self-weight of the beams=0.624kN/m

from walls = 1.0 ×4m × 0.62 =3.104kN/m

from floors = 1.0×1.47kN/m^2 = 1.47kN/m^2

imposed load = 0.7 × 4.0kN/m^2 = 2.8kN/m^2

9.1 Beams

for the span of 8.0 meters with 202x700mm cross-section this means:

$M_{ed,fi}$=195.5kNm

$v_{ed,fi}$= 76.42kN

parts of the cross-section is protected by the wall and floor, however for simplicity and safety the effects of charring will be calculated with regards to a beam with two exposed sides.

Using two dimensional charring with the reduced cross-section method in R90:

β_n=0.55 mm/min

t=90 min

$d_{char,n}=\beta_n \times t$ = 90×0.55= 49.5

$d_{eff} = d_{char,n} + k_0 \times d_0$

k_0 = 1.0

d_0 = 7.0mm

d_{eff} = 49.5+7.0 = 56.5 mm

the residual cross-section is then 643.5x145.5mm.

with a design strength of $M_{y,rd}$ = 361.5kNm> $M_{ed,fi}$=195.5kNm and

$V_{z,rd}$ = 214.7kN> $v_{ed,fi}$= 76.42kN.

this beam is sufficient in a fire situation.

the internal beams with a length of 7.5 meters and cross-section of 550x202 mm experience the same loads and a $M_{ed,fi}$ = 131.9kNm and $V_{ed,fi}$ = 55.67kN.

the reduction in cross-section is the same as before, a reduction of 56.5 mm after 90 minutes of exposure leaves a cross-section of 493.5x145.5mm with a $M_{y,rd}$ = 212.1kNm> $M_{ed,fi}$ = 131.9kNm and $V_{z,rd}$= 164.6kN> $V_{ed,fi}$ = 55.67kN.

for the beams holding up the roof the span is 11m the cross-section is 202x700 and the loads are:

from snow = 1.2×0.6= 0.72kN/m^2

self- weight of roof= 2 kN/m^2

self-weight of beam= 0.624kN/m

$M_{ed,fi}$ = 181.86kNm and $V_{ed,fi}$ = 51.39kN

the beam is exposed on three sides, the effective cross-section in R90 is therefore 643.5x89mm and $M_{y,rd}$=221.1kNm>$M_{ed,fi}$=181.86kNm and $V_{z,rd}$=131.3kN> $V_{ed,fi}$ = 51.39kN.

For the beams at the perimeter of the building the loads are:

from glass=1.45kN/m

from floor=1.47kN/m^2

imposed load=2.8kN/m^2

self-weight=0.4kN/m

$M_{ed,fi}$=107.8kNm

$V_{ed,fi}$=42.9kN

the beam is exposed on 2 sides. After 90 minutes of exposure the cross-section is 493.5x91.5mm with a strength of $M_{y,rd}$= 133.7kNm> $M_{ed,fi}$=107.8kNm and $V_{z,rd}$=103.5kN> $V_{ed,fi}$=42.9kN.

From the ceiling side, the protection of the joists in the floor is 59.6minutes. 90-59.6= 30.4 minutes that they will be charring. They will charr at an increased pace due to the protection, since they are protected on the sides by rock-wool the charring is limited to the front of the member.

loads in accidental limit state is:

from self-weight=1.47kN/m^2×0.4m= 0.6 kN/m

due to imposed load= 0.6×4×0.4= 0.96kN/m

$M_{ed,fi}$= 10.97kNm and $V_{ed,fi}$=5.85kN.

charring rate: $\beta_n = \beta_0 \times k_s \times k_n \times k_p$.

β_0= 0.5mm/min

k_s = 0,000167×b^2 - 0,029×b + 2,27=1.04

k_n=1.25

k_p= k_3=2.0

β_n=1.3mm/min

d_o=13.5+0.1×h=40.5mm

the effective charring after 30.4 min of burning is therefore d_{eff}=80.02mm. with a new cross-section of 190.0x73mm the strength is then $M_{y,rd}$=18.1kNm> $M_{ed,fi}$= 10.97kNm and $V_{z,rd}$=31.8kN> $V_{ed,fi}$=5.85kN.

With the floor-side exposed to fire the protection time is t_{ch}=71.4minutes. the joists will therefore char for 18.6 minutes. Charring rate is the same as before, β_n=1.3mm/min.

d_o=21.5+0.1×h=48.5mm

d_{eff}=72.68mm

The remaining cross-section is larger than for a fire-situation on the ceiling-side, the joists are strong enough to withstand a fire in R90.

9.2 columns

The columns experiencing the highest pressure are the internal columns on the first floor with a force of 2181.2kN. It's also exposed on four sides. After 90 minutes of exposure the effective cross-section is then 237x237mm with a strength of N_{rd}=1211.0kN.

from beams = 490×(2.4+1.84×6)×9.81/1000 = 64.6kN.

from floors = 1.47×(71.25+41.25×6) = 468.6kN.

from roof= 2×71.25=142.5kN

from walls= 0.62×42.9×6=159.6kN

imposed loads = 0.5×(3×5×41.25) +0.7×(4×71.5)+0.6×(1.2×71.25)= 560.87kN.

combined load is $N_{ed,fi}$=1396.1kN

the strength after exposure to fire is smaller than the design load in ALS. We need to use a bigger cross-section. 400x400mm has a strength of N_{rd}= 1775.8kN> $N_{ed,fi}$=1396.1kN which is sufficient.

For the outer columns, the highest compressive force in ALS is:

from beams = 490×11.59×9.81/1000 = 55.7kN.

from floors = 1.47×(30×5+41.25) = 281.1kN.

from walls= 0.62×3.75×3.3=7.67kN

imposed loads = 0.7×(4×41.25)+0.5×(3×5×30) = 340.5kN.

from roof=(0.7×2+0.6×1.2) ×41.25=87.45kN

combined load is then $N_{ed,fi}$= 772.42kN.

The column is exposed on three sides and the cross-section after 90 minutes is 243.5x187mm with a design strength of N_{rd}= 981.7kN> $N_{ed,fi}$= 772.42kN. The cross-section is good in R90.

The columns on the corners has less load and is more protected in a fire-situation by only being exposed on two sides, it is not necessary to calculate the fire-resistance for the columns on the corners as we know it will be strong enough.

9.3 Shear walls

They are exposed to fire on one side and have only one-dimensional charring. For hardwood with a density>450kg/m^3 β_0=0.5 and $d_{char,eff} = \beta_0 \times t$+7mm after 90minutes. Giving them a thickness of 148mm and a strength of $V_{z,rd}$=2545.6kN and N_{rd}=23931.6kN each. Shear walls are good in R90.

10 Check for buckling of columns.

The columns on the first floor are not supported against buckling by the walls, it is therefore necessary to check for buckling.

the internal columns are not subject to bending, while the columns on the outside of the structure are subjected to bending due to uneven loading. Calculations are done according to:

NS-EN 1995-1-1:2004+A1:2008+NA:2010. [15]

L_{cr}=3.5m

$E_d = E_{0,g,05}/Y_m$ = 11900/1.15 = 10347.8 N/mm^2

$G_d = G_{g,mean}/Y_m$ = 910/1.15 = 791.3 N/mm^2

Radius of gyration:

$i_y=i_z = h/12^{0.5} = 300/12^{0.5}$= 86.6mm for the 300mm column and 115.4mm for the 400mm column.

Relative Slenderness:

$\lambda=L_{cr}/i$= 3500/86.6mm=40.41 for the 300mm column and 30.3 for the 400mm column.

$\lambda_{rel}=\lambda/3.14\times(f_{c,0,gk}/E_{0,g,05})^{0.5}$=0.70 for the 300mm column and 0.49 for the 400mm column.

K_{300}= 0.5×(1+β_c×(λ_{rel}-0.3)+λ_{rel}^2=0.765

K_{400}= 0.63

β_c=0.1 for glulam.

Buckling reduction factor:

$K_c=1/K+(K^2-\lambda_{rel}^2)^{0.5}$ =0.93 for 300mm and 0.97 for 400mm.

design compressive stress:

$\sigma_{c,0,g,d}=N_{ed}/A$ = 16.84N/mm^2 for the 300mm and 13.63N/mm^2 for the 400mm.

$\sigma_{m,d}=M_{e,d}/w_y=383\times10^6/4.5\times10^6=85.11$ only for the 300x300mm cross-section.

stability check of the column:

$(\sigma_{m,d}/f_{m,d}) + \sigma_{c,0,g,d}/K_c \times f_{c,0,gd} = 4.24 < 1.0$ = for the 300x300mm cross-section on the perimeter.

Failure of cross-section.

$\sigma_{c,0,g,d}/K_c \times f_{c,0,gd} = 0.65 < 1.0$ = for the 400x400mm cross-section.

The cross-section is suitable.

the columns on the perimeter will buckle.

new cross section for testing is 600x400mm:

$\sigma_{m,d}$=15.95N/mm^2

$\sigma_{c,0,g,d}$=7.04N/mm^2

$(\sigma_{m,d}/f_{m,d}) + \sigma_{c,0,g,d}/K_c \times f_{c,0,gd} = 0.96 < 1.0$ = for the 400x600mm cross-section.

The new cross-section is suitable.

The corner columns experience moment in two directions and must satisfy the expression:

$(\sigma_{m,y,d}/f_{m,d}) + k_m \times (\sigma_{m,z,d}/f_{m,d}) + \sigma_{c,0,g,d}/K_c \times f_{c,0,gd} < 1.0$

with a cross-section of 400x400mm N_{ed}=822.4kN and $M_{y,ed}=M_{z,ed}$= 82.24kNm.

$\sigma_{m,d}$=7.71

$\sigma_{c,0,g,d}$=5.14

k_m=0.7 for rectangular cross-sections

$(\sigma_{m,y,d}/f_{m,d}) + k_m \times (\sigma_{m,z,d}/f_{m,d}) + \sigma_{c,0,g,d}/K_c \times f_{c,0,gd} = 0.77 < 1.0$

The cross-section is suitable.

11 checking beams for failure

11.1 failure due to combined action

The beams will have a slight compressive force due to the strength of the wind acting on the building. It is necessary to check the beams for failure due to combined bending and compression. In order to calculate it a simplification was made where the wind force spreads evenly across all the beams on one side of the building. Since the combined wind load for oppsing sides is 1357.2kN, divided by 30 beams on each side the compression is then 45.2kN.

the expression that needs to be satisfied is: $(\sigma_{m,d}/k_{crit}\times f_{m,d})^2 + (\sigma_{c,0,d}/k_{c,z}\times f_{c,0,d})<1.0$

$\sigma_{m,d}$=design bending stress

k_{crit}=is a factor to compensate for torsional buckling,

the beams are well supported in the z-axis $k_{crit}=1.0$

$\sigma_{c,0,d}$=design compressive stress

$k_{c,y}= 1/k_y+(k_y^2-\lambda_{rel,y}^2)^{0.5}$ calculated in the same way as for columns.

<u>8m beam with 700x202mm cross-section:</u>

$\sigma_{m,d}=M_{ed,y}/w_y=17.4$

$f_{m,d}=25N/mm^2$

$\sigma_{c,0,d}=N_{ed}/A=0.29$

$f_{c,0,d}=21.56$

$K_{c,y}=0.72$

$(\sigma_{m,d}/k_{crit}\times f_{m,d})^2 + (\sigma_{c,0,d}/k_{c,z}\times f_{c,0,d})=0.7 <1.0$

good.

<u>7.5m beam with a 550x202 cross-section:</u>

$\sigma_{m,d}=M_{ed,y}/w_y=22.3N/mm^2$

$f_{m,d}=25N/mm^2$

$\sigma_{c,0,d}=N_{ed}/A=0.406N/mm^2$

$f_{c,0,d}=21.56N/mm^2$

$K_{c,y}=0.8$

$(\sigma_{m,d}/k_{crit}\times f_{m,d})^2 + (\sigma_{c,0,d}/k_{c,z}\times f_{c,0,d})=0.82 <1.0$

good.

<u>Beam at the perimeter of the building 550x148mm</u>

$\sigma_{m,d}=M_{ed,y}/w_y=22.3N/mm^2$

$f_{m,d}=25N/mm^2$

$\sigma_{c,0,d}=N_{ed}/A=0.407N/mm^2$

$f_{c,0,d}=21.56N/mm^2$

$K_{c,y}=0.8$

$(\sigma_{m,d}/k_{crit}\times f_{m,d})^2 + (\sigma_{c,0,d}/k_{c,z}\times f_{c,0,d})=0.82 <1.0$

good.

<u>11m beam supporting the roof with 700x202 cross-section:</u>

$\sigma_{m,d}=M_{ed,y}/w_y =19.25$

$f_{m,d}=25N/mm^2$

$\sigma_{c,0,d}=N_{ed}/A=0.29$

$f_{c,0,d}=21.56$

$K_{c,y}=0.92$

$(\sigma_{m,d}/k_{crit}\times f_{m,d})^2 + (\sigma_{c,0,d}/k_{c,z}\times f_{c,0,d})=0.61 <1.0$

All beams pass the test for combined action.

11.2 serviceability limit state

The beams need to be checked for failure in serviceability limit state due to deflection. If a beam bends more than the limiting value it needs to be re-dimensioned. Calculations are done according to NS-EN 1995-1-1:2004+A1:2008+NA:2010. [15]

$w_{inst,G}$= deflection due to permanent loads= $5/384\times(1.0\times g_k/E_{0,mean}\times I)\times L^4$
$w_{fin,G}=w_{inst,g}\times(1-k_{def})$ where k_{def} = 0.6 for service class 1.
$w_{inst,Q}=5/384\times(1.0\times q_k/E_{0,mean}\times I)\times L^4$
$w_{fin,G}=w_{inst,Q}\times(1- \psi_2\times k_{def})$
$\psi_2=0.6$
$W_{fin}= w_{fin,G}+w_{fin,Q} > L/300$ mm to pass the check. From table 7.2 in the NS.

<u>11m span:</u>

$g_k= 2.624kN$

$I=bh^3/12=5.773\times10^9 mm^4$

$E_{0,mean}=14700$

$w_{inst,G}=5.89mm$

$w_{fin,G}=w_{inst,G}\times(1-0.6)=2.36mm$

q_k=1.2kN

$w_{inst,Q}$=10.8mm

$w_{fin,Q}$= 10.8×(1-0.6×0.6)=6.91mm

w_{fin}=9.27<w_{lim}=11000/300=36.6mm

Good.

8m span:

g_k= 8.3kN

q_k= 15kN

$w_{inst,G}$=5.21mm

$w_{fin,G}$=2.08

$w_{inst,Q}$=18.9mm

$w_{fin,Q}$=12.66mm

w_{fin}=14.74mm<w_{lim}=26.66mm

Good.

7.5m span:

I=2.8×10^9mm^4
g_k= 8.3kN
q_k= 15kN
$w_{inst,G}$=8.31mm
$w_{fin,G}$=3.32
$w_{inst,Q}$=15.0mm
$w_{fin,Q}$=9.6mm
w_{fin}=12.92<w_{lim}=25mm
Good.

Floor joists:

I=1.2×10^8mm^4
g_k=0.6 kN/m
q_k=0.96kN/m
$w_{inst,g}$=14.0mm
$w_{fin,G}$=5.61mm
$w_{inst,Q}$=22.4mm
$w_{fin,Q}$=14.3mm

w_{fin}=19.9mm<w_{lim}=25mm
Good.

all beams and joists pass the check for failure in the serviceability limit state.

After checking for failure, we end up with these beams:

Table 11 final beam dimensions

position	Thickness mm	Height mm	$M_{y,rd}$ kNm	$V_{y,rd}$ kN
$X_{1,5}$	202	550	254.6	177.1
$Y_{2,4}$	202	700	412.4	225.48
$Y_{1,6}$	202	550	254.6	177.1
$X_{2,3,4,5}$	202	550	254.6	177.1
For roof	202	700	412.4	225.48

and these columns:

Table 12 final column dimensions

Position	Cross-section	N_{rd}
Corners	400x400	3449.6kN
Edges	600x400	5174.4kN
Internal	400x400	3449.6kN

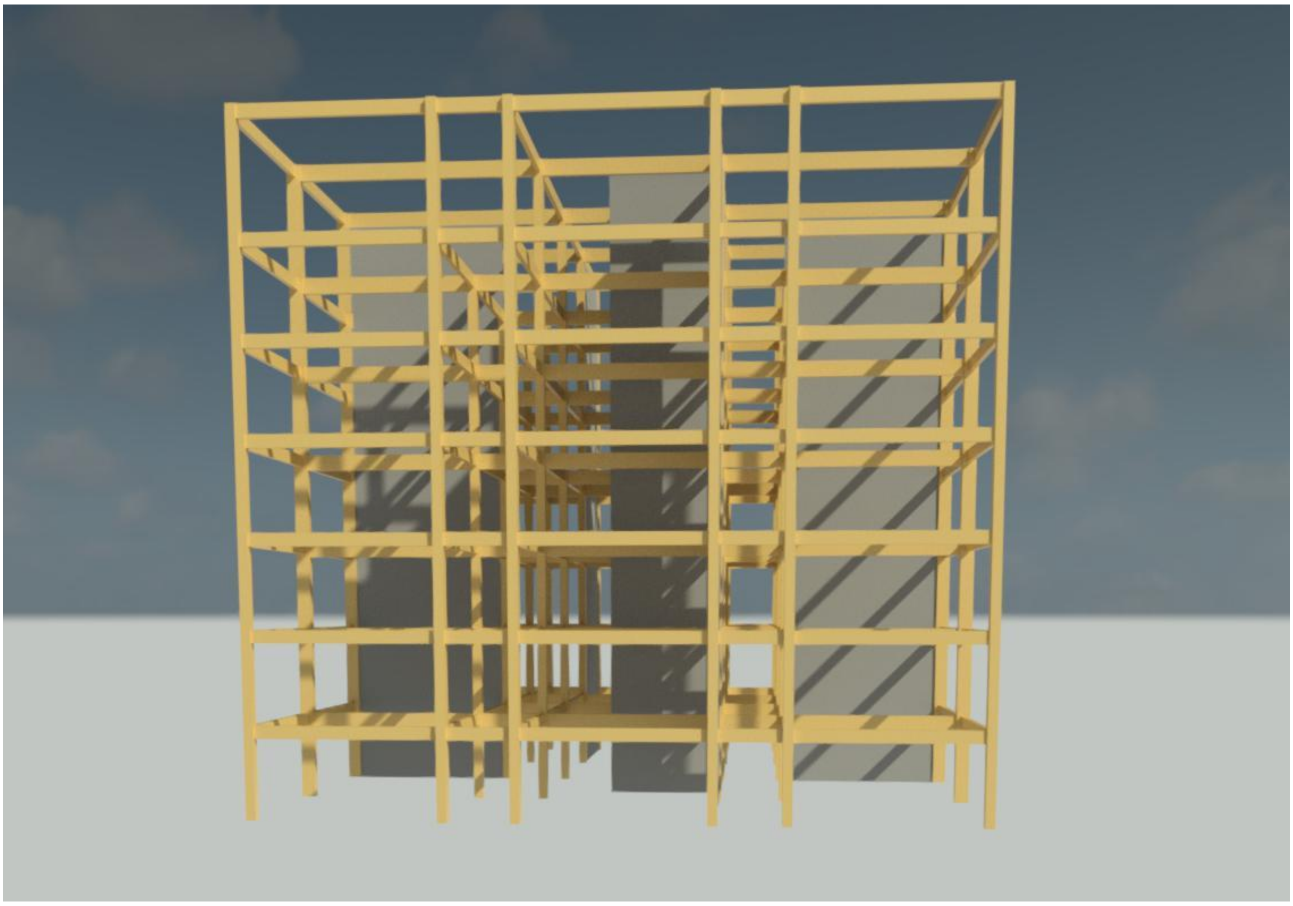

Figure 7 the building seen from X-axis

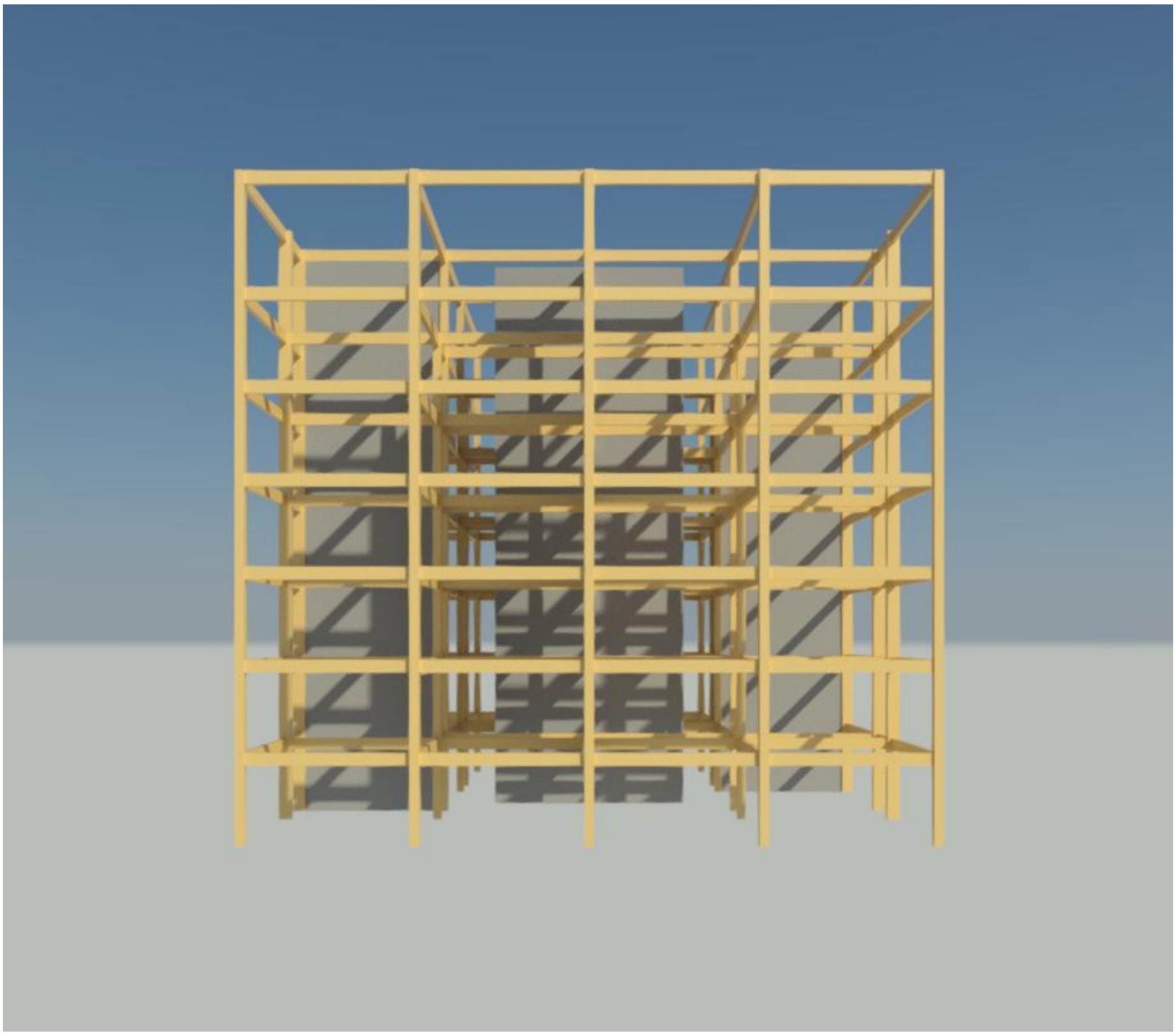

Figure 8 the building seen from Y-axis

12 structure made from concrete

The walls and floors will remain the same as before, the dimensioning being done with columns and beams made from B30 concrete with c/s 20mm double rebar and 50mm c_{nom}. Using the same bending moments due to the weight of floors, walls and imposed loads as before. Since the biggest strength loss of concrete in a fire-situation is the loss of tensile-strength, shear forces for the beams will not be calculated. The calculations are done in accordance with NS-EN 1992-1-1:2004+NA:2008. [18]

12.1 Beams in bending

<u>8meter beam:</u>

M_{ed}=340.56kNm

V_{ed}=128kN
f_{yk}=500N/mm^2
f_{yd}=434.8N/mm^2
f_{ck}=30N/mm^2
f_{cd}=17N/mm^2
c_{nom}= 50mm
h=700mm
t=202mm
d=632mm
d'=68mm

Calculating the steel area in tension:

$A_{s,tension}=M_{bal}/0.82\times d\times f_{yd} + (M_{ed}-M_{bal})/(d-d')\times f_{yd}=1640mm^2$->with 5 bars $A_s=1570mm^2$
$A_{s,compression}$= using 2 bars= 628mm^2
$M_{bal}=0.167\times b\times d^2\times f_{ck}=4.15\times10^8$
$x=f_{yd}\times(A_{s,tension}-A_{s,compression})/0.8\times b\times f_{cd}=149.1<0.617\times d=394.88$ to ensure failure of tensile r/f.
$M_{rd}=0.8\times X\times b\times f_{cd}\times(d-0.4X)+f_{yd}\times A_{s,comp}\times(d-d')=396kNm$
bar spacing, A_h, is then 17.2 mm. minimum spacing is 40mm. we need to try a wider cross-section of 400mm.

$A_{s,tension}=M_{bal}/0.82\times d\times f_{yd} + (M_{ed}-M_{bal})/(d-d')\times f_{yd}=1676.8mm^2$->with 6 bars $A_s=1884mm^2$
$A_{s,compression}$= using 2 bars= 628mm^2
$M_{bal}=0.167\times b\times d^2\times f_{ck}=8.0\times10^8$
$x=f_{yd}\times(A_{s,tension}-A_{s,compression})/0.8\times b\times f_{cd}=100.4<0.617\times d=390$ to ensure failure of tensile r/f.
$M_{rd}=0.8\times X\times b\times f_{cd}\times(d-0.4X)+f_{yd}\times A_{s,comp}\times(d-d')=477.2kNm$
A_h= 44mm. the self-weight of the beam adds to the moment with 56kNm.
$M_{rd}=477.2kNm > M_{ed}=396.56kNm$.

For the 7.5m span:

M_{ed}=227.3kNm
f_{yk}=500N/mm^2
f_{yd}=434.8N/mm^2
f_{ck}=30N/mm^2
f_{cd}=17N/mm^2
c_{nom}= 50mm
h=550mm
t=400mm
d=482mm
d'=68mm

Calculating the steel area in tension:

$A_{s,tension}=M_{bal}/0.82\times d\times f_{yd} + (M_{ed}-M_{bal})/(d-d')\times f_{yd}$=1385mm^2->with 5 bars A_s=1570mm^2
$A_{s,compression}$= using 2 bars= 628mm^2
$M_{bal}=0.167\times b\times d^2\times f_{ck}=4.66\times10^8$
$x=f_{yd}\times(A_{s,tension}-A_{s,compression})/0.8\times b\times f_{cd}$ =75<0.617×d=297.4 to ensure failure of tensile r/f.
$M_{rd}=0.8\times X\times b\times f_{cd}\times(d-0.4X)+f_{yd}\times A_{s,comp}\times(d-d')$=297.4kNm
with two layers of rebar we can make the beam 300mm wide with A_v=32mm, new d=465mm and A_h=55mm. M_{rd}=282.4kNm> M_{ed}=256.7.

For the span of 11 meters

M_{ed}=227.3kNm
f_{yk}=500N/mm^2
f_{yd}=434.8N/mm^2
f_{ck}=30N/mm^2
f_{cd}=17N/mm^2
c_{nom}= 50mm
h=700mm
b=300mm
d=632mm
d'=68mm

Calculating the steel area in tension:

$A_{s,tension}=M_{bal}/0.82\times d\times f_{yd} + (M_{ed}-M_{bal})/(d-d')\times f_{yd}$=1676.8mm^2->with 6 bars A_s=1884mm^2
$A_{s,compression}$= using 2 bars= 628mm^2
$M_{bal}=0.167\times b\times d^2\times f_{ck}=8.0\times10^8$
$x=f_{yd}\times(A_{s,tension}-A_{s,compression})/0.8\times b\times f_{cd}$ =100.4<0.617×d=390 to ensure failure of tensile r/f.
$M_{rd}=0.8\times X\times b\times f_{cd}\times(d-0.4X)+f_{yd}\times A_{s,comp}\times(d-d')$=477.2kNm
A_h= 44mm. using two layers of rebar with A_v=32mm new d=618mm we can make the cross-section 300mmx700mm. A_h=41mm with a M_{rd}=456.0kNm. seeing that this strength is sufficient for beams $Y_{2,4}$ it will be used there as well.

the self-weight adds to the moment with M_{ed}=79.4kNm, new M_{ed}=306.7kNm.

<u>Beams around the perimeter of the building</u>

M_{ed}=157.9kNm
f_{yk}=500N/mm^2
f_{yd}=434.8N/mm^2
f_{ck}=30N/mm^2
f_{cd}=17N/mm^2
c_{nom}= 50mm
h=550mm
b=300mm
d=482mm
d'=68mm

Calculating the steel area in tension:

$A_{s,tension}=M_{bal}/0.82\times d\times f_{yd} + (M_{ed}-M_{bal})/(d-d')\times f_{yd}=1354mm^2$->with 5 bars $A_s=1570mm^2$
$A_{s,compression}$= using 2 bars= 628mm^2
$M_{bal}=0.167\times b\times d^2\times f_{ck}=3.49\times10^8$
$x=f_{yd}\times(A_{s,tension}-A_{s,compression})/0.8\times b\times f_{cd}=100<0.617\times d=284.6$ to ensure failure of tensile r/f.
$M_{rd}=0.8\times X\times b\times f_{cd}\times(d-0.4X)+f_{yd}\times A_{s,comp}\times(d-d')=279.2$kNm
the bars will be spaced to close, bars in tension needs to be put in two layers with A_v=32mm

d=461.2mm. M_{rd}=279.2>new M_{ed}=191.3kNm.

Table 13 beams made from concrete

position	Cross-section	M_{rd} kNm	M_{ed} kNm
$Y_{2,4}$	700x300	456.0	396.56
$Y_{1,2}$ and $X_{1,2}$	550x300	279.2	191.3
$X_{2,3,4,5}$	550x300	282.4	256.7
For the roof	700x300	456.0	306.7

12.2 concrete columns

The columns will be designed in the same manner as the beams, with c_{nom} =50mm and B30 concrete with 8mm stirrups and 20mm bars. The compressive force will be higher than for the wooden construction due to the self-weight of the concrete columns and beams.

<u>The corner columns:</u>

The columns on the corners are subject to bending in two directions, the reinforcement should therefore be placed on all sides of the column.

from beams = 1.2×25×(0.55×0.3×7.75)×7 = 268.54kN.
from floors = 1.764(20.6+15×6) = 195.1 kN.
imposed loads = 1.5×(4+1.2)×20.6+1.5×(3×5)×15 = 498.18kN.
from glass= 1.2×0.29×240=83.5
combined load is 1045.32kN.
if we consider a cross-section of 350x350mm self-weight will be 88.8kN M_{ed}=182.9kNm, N_{ed}=1134.13kN and h'=214mm.
h'/h=0.61
A_c=b×h=122500
n=N_{ed}/f_{cd}×A_c=0.54
m=M_{ed}/f_{cd}×A_c×h=0.25
w=0.2 from diagram 1.3 found in EC clause 9.5
$A_s'=A_s=w\times A_c\times f_{cd}/f_{yd}$=957.9$mm^2$->4 bars on each side=1256mm^2 <$A_{s,max}/2$=4900mm^2
spacing of the of stirrups is 250mm. spacing of the longitudinal bars are 50mm.
$N_{rd}=A_c\times f_{cd}+A_s\times f_{yd}$=3174.7kN>$N_{ed}$=1134.13kN

<u>Columns on the perimeter</u>

from beams = 25×0.7×0.4×(7.5+4)×7 = 563.5kN.
from floors = 1.764×(30×5+41.25) = 317.52 kN.
from walls= 1.2×0.62×4×3.3×5=49.1kN
imposed loads = 1.5×(4×41.25+3×5×30) = 909kN.
from roof=(1.2×2+1.5×1.2) ×41.25=173.25kN
combined load is 2012.4kN. with a 400x600 cross-section M_{ed}=603.7kNm and N_{ed}=2186.4kN.
h'=464mm
h'/h=0.77
A_c=b×h=240000
n=N_{ed}/f_{cd}×A_c=0.53
m=M_{ed}/f_{cd}×A_c×h=0.25
w=0.2 from diagram 1.3 found in EC clause 9.5
$A_s'=A_s=w\times A_c\times f_{cd}/f_{yd}$=1876.7$mm^2$->6 bars on each side=1884mm^2 <$A_{s,max}/2$=9600mm^2
spacing of the of stirrups is 250mm. spacing of the longitudinal bars are 44mm.
$N_{rd}=A_c\times f_{cd}+A_s\times f_{yd}$=5718.3kN>$N_{ed}$=2186.4kN.

Central columns:

the central columns are only subjected to compression.

from beams = 25×0.7×0.4×(4+3.75+7.5)×7= 747.25kN.

from floors = 1.764×(41.25×6) = 436.6kN.

from roof= 1.2×2×71.25=171.0kN

from walls= 1.2×42.9×0.62×6=191.5kN

imposed loads = 1.5×(3×5×41.25+4×41.25+1.2×71.25)= 1303.9kN.

combined load is N_{ed}=2850.3kN. seeing that a cross-section of 350x350mm as used for the corners is strong enough it will be utilized.

Table 14 columns made from concrete

Position	Cross-section	N_{rd}	N_{ed}
corners	350x350	3174.7	1134.13
perimeter	400x600	5718.3	2186.4
internal	350x350	3174.7	2966.3

13 Concrete structure in a fire-situation

Calculations are done according to NS-EN 1992-1-2: 2004+NA: 2010- Design of Concrete Structures- Part 1-2: Structural Fire Design. [19]

13.1 Beams

<u>8 meter beam</u>

load due to self-weight of the beams=0.7×0.4×25=7.0kN/m

from walls = 1.0 ×4m × 0.62 =3.104kN/m

from floors = 1.0×1.47kN/m^2 = 1.47kN/m^2

imposed load = 0.7 × 4.0kN/m^2 = 2.8kN/m^2

this means $M_{d,fi}$=251.66kNm

the calculation of fire resistance is based on tabulated date regarding critical temperature of the reinforcement steel.

$A_{s,required}$=1676.8mm^2

$A_{s,provided}$=1884mm^2

M_d=396.56kNm

Y_s=1.15

$\sigma_{s,fi}=M_{d,fi}\times f_{yk}\times A_{s,required}/M_d\times Y_s\times A_{s,provided}$=245.57

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}$=0.49

θ_{cr}=530^{0}C>500^{0}C no adjustments needed (from fig.5.1 of the standard)

as it is exposed on 2 sides minimum width is 300mm and cover to reinforcement is 40mm.

From table 5.5. the cross section is good for R90.

<u>For the 7.5meter beam:</u>

$M_{d,fi}$=211.17

$A_{s,required}$=1385.0mm^2

$A_{s,provided}$=1570.0mm^2

M_d=256.7kNm

Y_s=1.15

$\sigma_{s,fi}=M_{d,fi}\times f_{yk}\times A_{s,required}/M_d\times Y_s\times A_{s,provided}$=315.5

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.63$

$\theta_{cr}=460^0C>500^0C$ adjustments are needed (from fig.5.1)

Need to change the minimum distance to reinforcement $\Delta a=0.1\times(500-\theta_{cr})=4mm$

as it is exposed on 2 sides minimum width is 300mm and cover to reinforcement is

$40+\Delta a=44mm$. From table 5.5. the cross section is good for R90.

<u>11meter beams holding up the roof:</u>

$M_{d,fi}=283.7$

$A_{s,required}=1676.8mm^2$

$A_{s,provided}=1884.0mm^2$

$M_d=456.0kNm$

$Y_s=1.15$

$\sigma_{s,fi}=M_{d,fi}\times f_{yk}\times A_{s,required}/M_d\times Y_s\times A_{s,provided}=240.8$

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.48$

$\theta_{cr}=540^0C>500^0C$ no adjustments are needed (from fig.5.1)

as it is exposed on 3 sides minimum width is 300mm and cover to reinforcement is 40mm.

From table 5.5. the cross section is good for R90.

<u>perimeter beams:</u>

$M_{d,fi}=130.4$

$A_{s,required}=1354mm^2$

$A_{s,provided}=1570.0mm^2$

$M_d=191.3kNm$

$Y_s=1.15$

$\sigma_{s,fi}=M_{d,fi}\times f_{yk}\times A_{s,required}/M_d\times Y_s\times A_{s,provided}=255.6$

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.51$

$\theta_{cr}=520^0C>500^0C$ no adjustments are needed (from fig.5.1)

as it is exposed on 2 sides minimum width is 300mm and cover to reinforcement is 40mm.

From table 5.5. the cross section is good for R90.

13.2 columns

Corner columns:

$N_{d,fi}$=959.9kN

N_d=1134.14kN

$A_{s,required}$=957.9mm^2

$A_{s,provided}$=1256mm^2

$\sigma_{s,fi}=N_{d,fi}\times f_{yk}\times A_{s,required}/N_d\times \Upsilon_s\times A_{s,provided}=280.64$

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.56$

$\theta_{cr}=510^0C>500^0C$ no adjustments are needed (from fig.5.1)

minimum required width and cover to reinforcement is then b=300mm and C_{min}=25mm

from table 5.2(a). of the standard.

Internal columns:

$N_{d,fi}$=1529.9kN

N_d=2186.4kN

$A_{s,required}$=300mm^2

$A_{s,provided}$=628mm^2

$\sigma_{s,fi}=N_{d,fi}\times f_{yk}\times A_{s,required}/N_d\times \Upsilon_s\times A_{s,provided}=145.33$

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.3$

$\theta_{cr}=610^0C>500^0C$ no adjustments are needed (from fig.5.1)

minimum required width and cover to reinforcement is then b=300mm and C_{min}=25mm

from table 5.2(a).

Columns in the perimeter:

$N_{d,fi}$=1500.2kN

N_d=2186.4kN

$A_{s,required}$=1876.7mm^2

$A_{s,provided}$=1884mm^2

$\sigma_{s,fi}=N_{d,fi}\times f_{yk}\times A_{s,required}/N_d\times \Upsilon_s\times A_{s,provided}=297.17$

$K_{s(\theta cr)}=\sigma_{s,fi}/f_{yk}=0.6$

$\theta_{cr}=500^0C>500^0C$ no adjustments are needed (from fig.5.1) minimum required width and cover to reinforcement is then b=300mm and C_{min}=25mm from table 5.2(a). well bellow our values of b=400mm and C_{nom}=50mm.

14 Structure made from steel

The building will be designed with a structure made from steel beams and columns. The calculations are done according to Eurocode3 –NS-EN 1993-1-1:2005+A1:2014+NA:2015 [20]

14.1 Beams

$\underline{Y_{2,4}\text{ with a span of 8meters}}$.

$M_{y,ed}$=340.56kNm

f_y=355N/mm^2

$W_{pl,y}>M_{y,ed}/f_y=959.3\times10^3$

$W_{pl,y}=2\times s_y=2\times527\times10^3mm^3$ for beam HE240-B

checking class-classification of web and flange.

$c_f/t_{fe}=((b-2\times r-t_w)/2)/t_{fe}=5.53<9$->flange is in class 1.

$c_w/t_{we}=(h-2\times t_f-2\times r)/t_{we}=16.4<72$->web is class 1.

$M_{c,rd}=f_y\times2\times s_y/\ Y_{m,0}=355\times2\times527\times10^3/1.05\times10^6=356.35kNm$

$\underline{X_{2,3,4,5}\text{ with a span of 7.5meters.}}$

$M_{y,ed}$=227.3kNm

f_y=355N/mm^2

$W_{pl,y}>M_{y,ed}/f_y=640.2\times10^3$

$W_{pl,y}=2\times s_y=2\times414\times10^3mm^3$ for beam HE220-B

checking class-classification of web and flange.

$c_f/t_{fe}=((b-2\times r-t_w)/2)/t_{fe}=5.47<9$->flange is in class 1.

$c_w/t_{we}=(h-2\times t_f-2\times r)/t_{we}=16.0<72$->web is class 1.

$M_{c,rd}=f_y\times2\times s_y/\ Y_{m,0}=355\times2\times414\times10^3/1.05\times10^6=279.9kNm$

$\underline{X_{1,6}\text{ and }Y_{1,5}}$

$M_{y,ed}$=157.9kNm

f_y=355N/mm^2

$W_{pl,y}>M_{y,ed}/f_y=444.8\times10^3$

$W_{pl,y}=2\times s_y=2\times241\times10^3mm^3$ for beam HE180-B

checking class-classification of web and flange.

c_f/t_{fe}=((b-2×r-t_w)/2)/t_{fe}=5.05<9->flange is in class 1.

c_w/t_{we}=(h-2×t_f-2×r)/t_{we}=14.4<72->web is class 1.

$M_{c,rd}=f_y\times2\times s_y/\ Y_{m,0}=355\times2\times241\times10^3/1.05\times10^6=162.96kNm$

<u>For the beams holding up the roof:</u>

$M_{y,ed}$=317.63kNm

we will utilize the same beam HE240-B as for $Y_{2,3,4}$ with a $M_{c,rd}$=356.35kNm.

14.2 Columns

<u>Corner column with a height of 4meters.</u>

Trying a cross-section of SHS200x200x8mm N_{ed}=1940.4kN and $M_{ed,y}=M_{ed,z}$=97.0kNm.

$W_{pl,y}=W_{pl,z}>M_{ed}\times Y_{m,0}/\chi_{LT}\times f_y$

$\chi_{LT}=1.0$

W_{pl}=432>358.8

classification of cross-section:

$c_1/s\times\varepsilon$=(h-3×s)/s×0.92=23.9<33 -> class 1.

$N_{pl,rd}=A\times f_y/Y_{m,0}$=2045.5kN

$M_{n,rd}=(W_{pl}\times f_y/Y_{m,0})\times(1-n/1-0.5a_w)$

$n=N_{ed}/N_{pl,rd}$=0.96

a_w=(A-2×b×t)/A=0.47 use 0.5

$M_{n,rd}$=7.8kNm<97.0 ->not ok

trying cross-section SHS300x300x10

W_{pl}=1238

classification of cross-section:

$c_1/s\times\varepsilon$=(h-3×s)/s×0.92=29.3<33 -> class 1.

$N_{pl,rd}=A\times f_y/Y_{m,0}$=3854.3kN

$M_{n,rd}=(W_{pl}\times f_y/Y_{m,0})\times(1-n/1-0.5a_w)$

$n=N_{ed}/N_{pl,rd}$=0.5

a_w=(A-2×b×t)/A=0.47

$M_{n,rd}$=273.5kNm>97.0 -> check is ok.

$$(M_{y,ed}/M_{y,rd})^{\alpha}+(M_{z,ed}/M_{z,rd})^{\beta}<1.0$$

$$\alpha=\beta=1.66/1-1.13\times n^2=2.31$$

$(M_{y,ed}/M_{y,rd})^{\alpha}+(M_{z,ed}/M_{z,rd})^{\beta}=0.3<1.0$ cross-section is good.

check for buckling:

expression that needs to be satisfied:

$$N_{ed}/(\chi_{LT}\times N_{rk}/Y_{m1})+k_{yy}\times(M_{y,ed}+\Delta M_{y,ed})/(\chi_{LT}\times M_{y,rk}/Y_{m1})+k_{yz}\times M_{z,ed}/M_{z,rk}<1.0$$

$$N_{rk}=A\times f_y=4047.0kN$$

$$M_{y,rk}=M_{z,rk}=w_p\times f_y=439.5kNm$$

$\Delta M_{ed}=0$ for class 1 cross-section

$k_{yy}=k_{yz}=1.0$ estimated as a simplification.

$$N_{ed}/(\chi_{LT}\times N_{rk}/Y_{m1})+k_{yy}\times(M_{y,ed}+\Delta M_{y,ed})/(\chi_{LT}\times M_{y,rk}/Y_{m1})+k_{yz}\times M_{z,ed}/M_{z,rk}=0.95<1.0$$

no buckling occurs.

<u>For the columns forming the perimeter of the building:</u>

$N_{ed}=1515.7kN$, if using a cross-section of RHS250x250x10mm $M_{ed}=189.46kNm$ bending in one direction.

$$W_{pl,y}=W_{pl,z}>M_{ed}\times Y_{m,0}/\chi_{LT}\times f_y$$

$$\chi_{LT}=1.0$$

$$W_{pl}=845>560.4$$

classification of cross-section:

$c_1/s\times\varepsilon=(h-3\times s)/s\times0.92=23.9<33$ -> class 1.

$$N_{pl,rd}=A\times f_y/Y_{m,0}=3195kN$$

$$M_{n,rd}=(W_{pl}\times f_y/Y_{m,0})\times(1-n/1-0.5a_w)$$

$$n=N_{ed}/N_{pl,rd}=0.47$$

$a_w=(A-2\times b\times t)/A=0.47$ use 0.5

$N_{ed}=1515.7kN>0.25\times N_{pl,rd}=798.75$ ->N_{ed} has an effect on bending resistance.

$M_{n,rd}=201.88kNm<189.46$ ->check ok.

check for buckling:

expression that needs to be satisfied:

$$N_{ed}/(\chi_{LT}\times N_{rk}/Y_{m1})+k_{yy}\times(M_{y,ed}+\Delta M_{y,ed})/(\chi_{LT}\times M_{y,rk}/Y_{m1})<1.0$$

$$N_{rk}=A\times f_y=3354.75kN$$

$$M_{y,rk}=M_{z,rk}=w_p\times f_y=299.98kNm$$

$\Delta M_{ed}=0$ for class 1 cross-section

$k_{yy}=k_{yz}=1.0$ estimated as a simplification.

$N_{ed}/(\chi_{LT}\times N_{rk}/Y_{m1})+k_{yy}\times(M_{y,ed}+\Delta M_{y,ed})/(\chi_{LT}\times M_{y,rk}/Y_{m1})=1.14<1.0$

buckling can occur. Try a bigger cross-section, SHS300x300x10mm.

$N_{rk}=A\times f_y=4047.0kN$

$M_{y,rk}=M_{z,rk}=w_p\times f_y=439.5kNm$

$\Delta M_{ed}=0$ for class 1 cross-section

$k_{yy}=k_{yz}=1.0$ estimated as a simplification.

$N_{ed}/(\chi_{LT}\times N_{rk}/Y_{m1})+k_{yy}\times(M_{y,ed}+\Delta M_{y,ed})/(\chi_{LT}\times M_{y,rk}/Y_{m1})=0.85<1.0$

no buckling occurs.

<u>For the columns in the center</u>

$N_{ed}=2181.2kN$.

choosing SHS300x300x10mm

$N_{c,rd}=A\times f_y/Y_{m,0}=3854.2kN>N_{ed}=2181.2$ ->no cross-sectional yielding.

$N_{b,rd}=\chi\times A\times f_y/Y_{m,1}$

$\chi=1/\Theta+(\Theta^2+\lambda^2)^{0.5}$

$\lambda=L_{cr}\times 1/i\times\lambda_1=0.27$

$i=118mm$

$\lambda_1=86.4$ for s355 steel

$L_{cr}=0.7\times L$ for a fixed column with pinned top.

$\Theta=0.5\times(1+\alpha\times(\lambda-0.2)\times\lambda^2=0.5$

$\alpha=0.21$ from table 6.2 of the standard.

$\chi=0.93$

$N_{b,rd}=3584.4kN>N_{ed}=2181.2kN$

cross-section will not buckle

Beams:

Table 15 beams made from steel

position	Cross-section	$M_{c,rd}$	M_{ed}
$X_{1,6}$ $Y_{1,5}$	HE180-B	162.96	157.9

$X_{2,3,4,5}$	HE240-B	356.35	340.56
$Y_{2,4}$	HE220-B	279.9	227.3
Roof	HE240-B	356.35	317.63

Columns:

Table 16 columns made from steel

position	Cross-section	$N_{b,rd}$	N_{ed}
corner	SHS300x300x10	3584.4	1940.4
perimeter	SHS300x300x10	3584.4	1515.7
internal	SHS300x300x10	3584.4	2181.2

15 Steel structure in a fire-situation

Calculations done according to NS-EN 1993-1-2:2005+NA:2009-Design of Steel Structures-Part 1-2: Structural Fire Design. [21]

15.1 Beams

for beams holding the roof $M_{ed,fi}$=181.86kNm

to be suitable in R90 the expression $\Theta_{a,cr}>\Theta_{a,t}$ must be satisfied.

$\Theta_{a,cr}=39.19\times\ln(1/0.9674\times\mu_0^{3.833}-1)+482^{o}C$ for a class 1 beam.

$\mu_0=E_{fi,d}/R_{fi,d,0}=181.86/356.35=0.51$

$\Theta_{a,cr}=581.0^{o}C$

$\Theta_{a,t}$ is found from table 13.7.

$A_m/V=68$

Table 13.7 in the standard show that after 30 minutes the temperature of the beam is around 750°C, this beam therefore needs to be protected. By using the calculator for protection with conlit150/150p from rockwool.no we see that with a 20mm layer of insulation the temperature of the beam will be 453°C after 90 minutes of exposure. This is satisfactory.

As the thickness of the wall is 202mm and the floor-joists is 270mm thick the other steel beams are already protected from the fire by the walls and floors.

15.2 Columns

<u>corner column:</u>

$\Theta_{a,cr}$=39.19×ln(1/0.9674×$\mu_0^{3.833}$ -1)+482°C for a class 1 beam.

μ_0=$E_{fi,d}$/$R_{fi,d,0}$=772.42/3584=0.22

$\Theta_{a,cr}$=710.6°C

$\Theta_{a,t}$ is found from table 13.7.

column is exposed on two sides.

A_m/V=52.6

After 30 minutes of exposure the temperature is around 700°C. The column needs protection. The calculator from rockwool shows that 20mm of conlit150/150p will keep the temperature at 383°C in R90, which is sufficient.

$\Theta_{a,t}$=383°C< $\Theta_{a,cr}$=710.6°C

<u>Perimeter columns:</u>

$\Theta_{a,cr}$=39.19×ln(1/0.9674×$\mu_0^{3.833}$ -1)+482°C for a class 1 beam.

μ_0=$E_{fi,d}$/$R_{fi,d,0}$=772.42/3584=0.22

$\Theta_{a,cr}$=710.6°C

$\Theta_{a,t}$ is found from table 13.7.

The column is exposed on three sides.

A_m/V=79.0

After 30 minutes of exposure the temperature is around 750°C. The column needs protection. The calculator from rockwool shows that 20mm of conlit150/150p will keep the temperature at 498°C in R90, which is sufficient.

$\Theta_{a,t}=498^oC< \Theta_{a,cr}=710.6^oC$

<u>Inner columns:</u>

$\Theta_{a,cr}=39.19\times\ln(1/0.9674\times\mu_0^{3.833} -1)+482^oC$ for a class 1 beam.

$\mu_0=E_{fi,d}/R_{fi,d,0}=1396.1/3584=0.4$

$\Theta_{a,cr}=619.8^oC$

$\Theta_{a,t}$ is found from table 13.7.

The column is exposed on four sides.

$A_m/V=105.3$

After 30 minutes of exposure the temperature is around 770°C. The column needs protection. The calculator from rockwool shows that 20mm of conlit150/150p will keep the temperature at 595°C in R90, which is sufficient.

$\Theta_{a,t} =595^oC<\Theta_{a,cr}=619.8^oC$

16 Comparison regarding economy and CO_2-emmisions.

After dimensioning the building for glue-laminated timber, steel and concrete the dimensions was used in IZY Calcus to get an estimate for the price and CO_2 emissions for the structural components with each of the solutions. The buildings made from concrete and steel both had shear-walls made from concrete with a thickness of 200mm. The results can be found in table 17:

Table 17 comparison for economy and CO_2-emmisions

material	Price in NOK	CO_2-emmisions in Kg
Glu-laminated timber	7 536 544+3 972 269 for shear-walls	34 537+32 137 for shear-wall
steel	9 086 453+3 878 128	371 985+152 982
concrete	9 011 083+3 878 128	347 693+152 982

The values listed is the price and emissions from the materials with a 60-year life cycle. The construction of the building its-self is not included. The difference in price is around 1.5 million kroners, with glue-laminated timber being the cheapest. This is not a huge difference, and some of the difference may be due to inaccuracies in the computer program or over dimensioning of elements with concrete and steel. The cost of construction is not taken into account, and might make a difference in the final cost-comparison of the building. The difference in emissions of CO_2 however is substantial. With Steel and concrete being almost 10 times more polluting, even when considering a possible flaw in the programing it is obvious that glue-laminated wood is a much more environmentally friendly option.

17 Conclusion

The background for this thesis was to do a literature review of the techniques and design considerations when using glue-laminated timber in construction of high-rise buildings. A special emphasis was put on the fire resistance of timber due to the public perception of it being an inferior building material to concrete and steel regarding fire-resistance. when reviving the literature and tests done on the fire-resistance of glue-lam I found there to be a lot of variation in results and in how different tests was performed. The techniques used for calculation and dimensioning of glue-laminated timber elements are also diverse, However, there are standardized and recommended methods for calculating fire resistance. These standard methods have been extensively tested and proves to be both reliable and accurate. The components additive method proved to be effective and practical for calculating protection time of cladding and the reduced cross-section is the advised and proven method for calculation of strength during a fire-situation.

For glue-lam to be a viable building material to replace concrete and steel in the future it must satisfy certain criteria. In best case scenario it would have better structural properties, economic benefits and lower emissions of CO_2 associated with its use in construction. In this thesis a load-bearing structure was designed in R90 and comparisons was made between glue-laminated timber, concrete and steel. Timber proved to perform just as well as or better then concrete in bending, we were able to design beams with a smaller cross-section to bear the loads. Timber also proved to be as good as concrete, and better than steel, for fire-resistance. When considering structural elements in compression, concrete performed better. And steel allowed us to design both beams and columns with a smaller cross-section than with timber, however needing protection against fire. After the dimensioning of the structure was finished, IZY calcus was used to calculate the price of the materials and emissions of CO_2 in their production. Material amounts for glue-lam and concrete had to be adjusted by volume since we did not utilize standardized dimensions. The difference in price between glue-laminated elements and those made from concrete and steel was found to be around 1.5million kroners, with glue-lam being the cheapest alternative. The CO_2 emissions was found to be almost 10 times higher for both concrete and steel than for a structure made of glue-laminated elements. Considering these results, I would say glue-laminated timber should become increasingly popular as a building material in the future.

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