VŠB - Technical University of Ostrava

Faculty of Civil Engineering

Department of Structures

Design of wooden house

Návrh dřevostavby

Diploma Thesis

Student: Bc. Ali Almansour

Supervisor: Ing. Kristýna Vavrušová. Ph.D

Ostrava 2019

VŠB - Technical University of Ostrava
Faculty of Civil Engineering
Department of Structures

Diploma Thesis Assignment

Student:	**Ali Almansour**
Study Programme:	N3607 Civil Engineering
Study Branch:	3607T037 Building Constructions
Title:	Desing of wooden house Návrh dřevostavby
The thesis language:	English

Description:

1) Research and description of historical and contemporary timber structures.
2) Layout of the proposed building.
3) Design and assessment of selected critical load-bearing elements and connections for ultimate and serviceability limit state.
4) Design and assessment of selected elements for fire resistance.
5) Corresponding drawing plans.

References:

1) Eurocode 5: Design of timber structures. Part 1-1: General – Common rules and rules for buildings.
2) Eurocode 5: Design of timber structures. Part 1-2:General – Structural fire design.
3) Eurocode 1: Actions on building structures.
4) Eurocode – Basis of structural design.
5) Porteous, J., Kermani, A., Structural Timber Design to Eurocode, 2013, John Wiley and Sons Ltd, ISBN 978-0-470-67500-7.
6) Larsen, H., Enjily, V., Practical Design of Timber Structures to Eurocode 5, ICE Publishing, 2009, ISBN-10: 0727736094, ISBN-13: 978-0727736093.

Extent and terms of a thesis are specified in directions for its elaboration that are opened to the public on the web sites of the faculty.

Supervisor: **Ing. Kristýna Vavrušová, Ph.D.**

Date of issue: 28.02.2019

Date of submission: 29.11.2019

doc. Ing. Antonín Lokaj, Ph.D.
Head of Department

VYSOKÁ ŠKOLA BÁŇSKÁ - TECHNICKÁ UNIVERZITA
Fakulta stavební
4
OSTRAVA

prof. Ing. Radim Čajka, CSc.
Dean

Declaration of the student

“I hereby declare that this Master thesis was written by myself. I have quoted all the references I have drawn upon.

In Ostrava………………. ………………………………

Student`s signature

Declaration of the Master thesis results utilisation

Herewith I declare that

- I am informed that Act No. 121 / 2000 Coll. – the Copyright Act, in particular, § 35
    - Utilisation of the Work as a Part of Civil and Religious Ceremonies, as a Part of School Performances and the Utilisation of a School Work – and § 60 – School Work, fully applies to my Master thesis;
- I take account of the VŠB – Technical University of Ostrava (hereinafter as VŠB TUO) having the right to utilize the Mater thesis (under § 35(3)) unprofitable and for own use; I agree that the Master thesis shall be archived in the VŠB-TUO's information system;
- It was agreed that, in case of VŠB-TUO interest, I shall enter into a license agreement with VŠB-TUO, granting the authorization to utilize the work in the scope of § 12(4) of Copyright Act;
- It was agreed that I may utilize my work, the Master thesis or provide a license to utilize it only with consent of VŠB-TUO, which is entitled, in such a case, to claim an adequate contribution from me to cover the cost expended by VŠB-TUO for producing the work (up to its real amount);
- Upon final submission of the Master thesis I agree with its publishing in accordance with Act no. 111 / 1992 Coll. on Higher Education Institutions and on Amendments and Supplements to some other Acts (the Higher Education Act)

Acknowledgements

I would like to thank Ing. Kristýna Vavrušová, Ph.D. for professional and guidance through my thesis.

Many thanks for my family for their support during my studies.

Abstract

The aim of this thesis is to design and static assessment of the construction of a house for living made of wood-based materials. A part of the thesis is also research of historical and contemporary structural systems of wood-based materials. For calculation of internal forces, a computational model was created using SCIA software, which was loaded with all permanent and imposed loads. The main load-bearing components are analysed statically and for fire resistance. At the end of the thesis selected connecting components are also analysed. In this thesis the material used for design is C24, and C30 for the wooden elements.

All the equations and factors used in the design are according to the Eurocode 5 Part 1 & 2.

Keywords:

Wooden House, research, historical, contemporary, fire resistance, column, girder, joist.

Contents of the Master thesis

1.Introduction……..…………………………………………………….10

2. Research and description of historical and contemporary timber structures………...……………………………………………………….11

2.2 History of Timber Houses…………………………………..…11

2.3 Construction Systems of wooden houses…………..….….………12

2.3.1 Log houses……………………………………………………12

2.3.2 Strut Frames…………………………………………….……14

2.4 Contemporary timber buildings………………………………15

3. Proposed House Plan and structural Material…………………...17

4. Design of critical Structural elements ULS and SLS, fire resistance design of critical elements………………………………...20

4.1. Roof Design…………………………………………………22

4.1.1 Actions on the roof………………………...………………23

4.1.2 Design of Tile batten……………………………………….36

4.1.3 Design of the Rafters……………………………………….41

4.1.4 Design of Purlins…………………………………………...48

4.1.5 Design of Post……………………………………………57

5. Floor Joist design…………………………………………..…….60

6. Beam Design……………………………………………………...63

7. Column Design…………………………………………………...69

8. Critical Beam Design..…………………………………………...71

9. Critical Column Design…..……………………………………...75

10. Connections……………………………………………………..80

11. Conclusion ..…………………………………………………….85

12. Tables and resources…………………………………….....…86

1.Introduction

Wood is a natural building material that has accompanied man throughout history. The main role in dwelling construction played mostly in prehistoric times and antiquity. With the advent of new materials such as masonry, later also concrete, limited the construction material to simple and small constructions.

With increasing demands on the quality of the environment at present, however at the same time, the requirements for ecological construction are increasing. Wood therefore seems appropriate material, it is a renewable material, has a positive effect on landscape protection and decreases demands on the extraction of non-renewable raw materials. Wood and wood products help reduce emissions

The greenhouse effect CO2, therefore, contributes to the stability of the Earth's climate. The advantage is also a clean construction site and minimal burden on the surroundings by heavy traffic.

Another advantage of timber structures; due to the thermal insulation properties of wood, heating costs are also reduced. For this reason, wood has been gaining importance in recent years, its share in the construction of houses, especially in Europe, North America, East Asia and other parts of the world.

When designing a timber structure, it is also necessary to deal with fire resistance construction. Wood is one of the flammable materials, but its fire behaviour can be avoided largely predict and fire resistance of wood is among the highest. Fire resistance can be increase by the use of tiles of flammable or non - flammable materials, impregnation or foaming or gasifying injections.

It is the fire resistance that is limiting in the design of multi-storey wooden buildings. In Czech in the Republic, the height is limited to 12 meters (calculated from the first floor to the last floor)

2. Research and description of historical and contemporary timber structures

2.2 History of timber houses

First timber-framed houses:

The first timber-framed houses were built 4500-3000 BC. The durability of these houses didn't exceed 20 years. Since the first farmers did not know structural detailing well, they had many problems, particularly with trusses and bracing.

The structure of long houses was the same. The width ranging between 5.5 to 7 m, and the length varied from 20 m to 45 m.

Rural Houses:

Between the 13th and 15th century, rural architecture came into existence and in this form existed until the 19th century.

In the middle ages 13th-15th century, rural architecture took on different regional forms. Materials traditionally used at that time were timber, stone and clay.

Urban House:

During the 12th and 13th century, the log cabin was widely built in towns of Central Europe. In contrast to the house in the countryside, there was only a passage leading to the backyard.

From the 14th century, stone and brick were used as a structural material for the construction of houses in towns.

The main reason of their spread was fire resistance of these materials. Floor structures of urban houses were made of timber until the 16th century.

2.3 Construction System of Wooden Houses:

1- Log houses.
2- Strut Frame.
3- Light Post and beam.
4- Frame (panel) houses.
5- Post and beam houses.
6- Massive panels timber houses.

2.3.1-Log Houses:

The tradition of log buildings dates to the past and has significantly influenced the European architecture of wooden buildings. Especially in Russia and Scandinavia, not only residential houses but also palaces, towers or churches were built as log buildings. They were also widespread in the mountains of Central Europe, especially in the Alps

Log buildings are still being built in mountain areas to this day, most of them can be found in Switzerland, the Bavarian Alps and Austria. Carpenters' knowledge is passed down from generation to generation, thereby retaining its characteristics, but at the same time the houses are still adapted to the latest standards of living.

Characteristics:

a) High handcrafts demands

b) Special quality timber

c) Difficult joints

d) Low variability

e) High consumption of timber

f) High settlement

Modern Log House:

However, this type of construction is built without the necessary knowledge and design principles resulting from long-term experience (see pic. 2.1).

Picture 2.1: Example of modern Log House [1]

2.3.2-Strut Frame:

At first, buildings dominated where the load-bearing structure (carrying skeleton) remained visible. Especially in the second half of the 19th century, half-timbered buildings began to be plastered in towns to imitate other massive buildings of stone or masonry, and also to suppress their rural appearance (see pic. 2.2, 2.3 and 2.4).

Widespread in Western, Central and Eastern Europe (in areas with mainly deciduous trees).

Characteristics:

a) Carrying skeleton is visible

b) Multi-Storey buildings

c) Carpenter joints

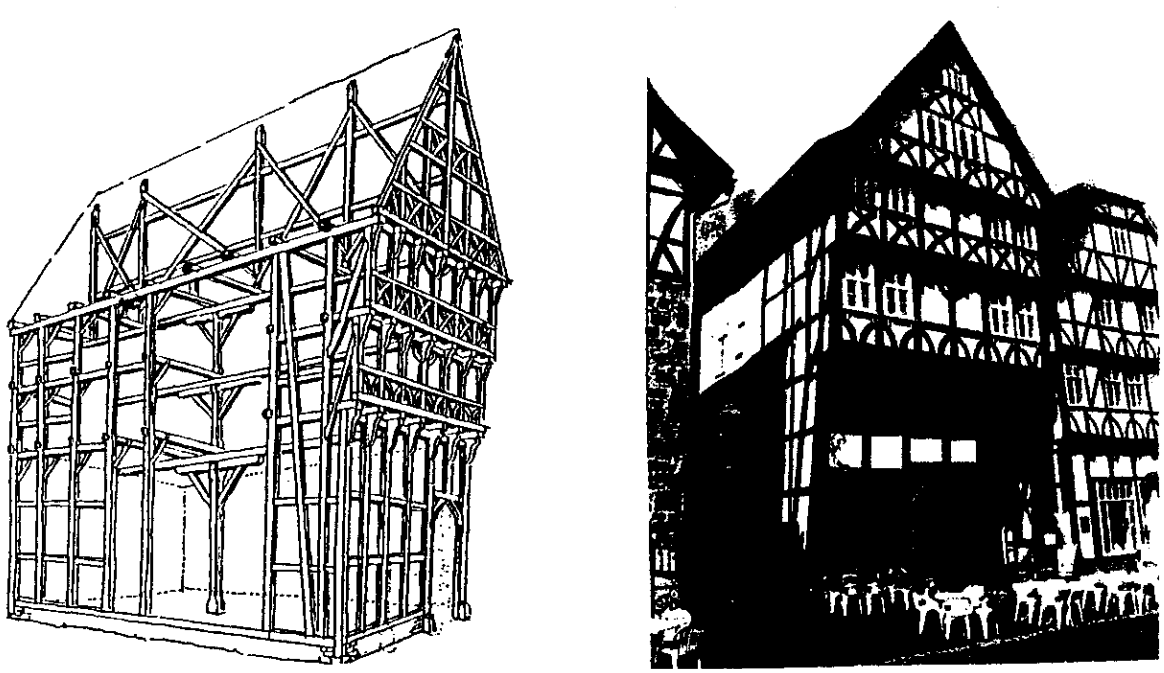
d) Massive cross-sections

e) High cost servicing

f) Settlement

Picture 2.2: Modern Sturt Frame houses: Mosbach (Germany) [2]

Picture 2.3: Zgorzelec (Germany) [3]



Picture 2.4: Traditional Strut Frame Houses, Fritzlar (Germany 1420) [4]

2.4. Contemporary timber houses

Multi-storey Buildings:

Muhlweg (Austria)

5 floors.

Picture 2.5: Contemporary timber house (Austria) [5]

Steinhausen (Switzerland)

6 floors

Picture 2.6: Contemporary timber house (Switzerland) [6]

Berlin e3 (Germany)

7 floors.

Picture 2.7: Contemporary timber house Berlin e3 (Germany) [7]

HOHO Wien (Austria)

Height 84 m (to be built)

THE WORLD'S TALLEST WOODEN TOWER

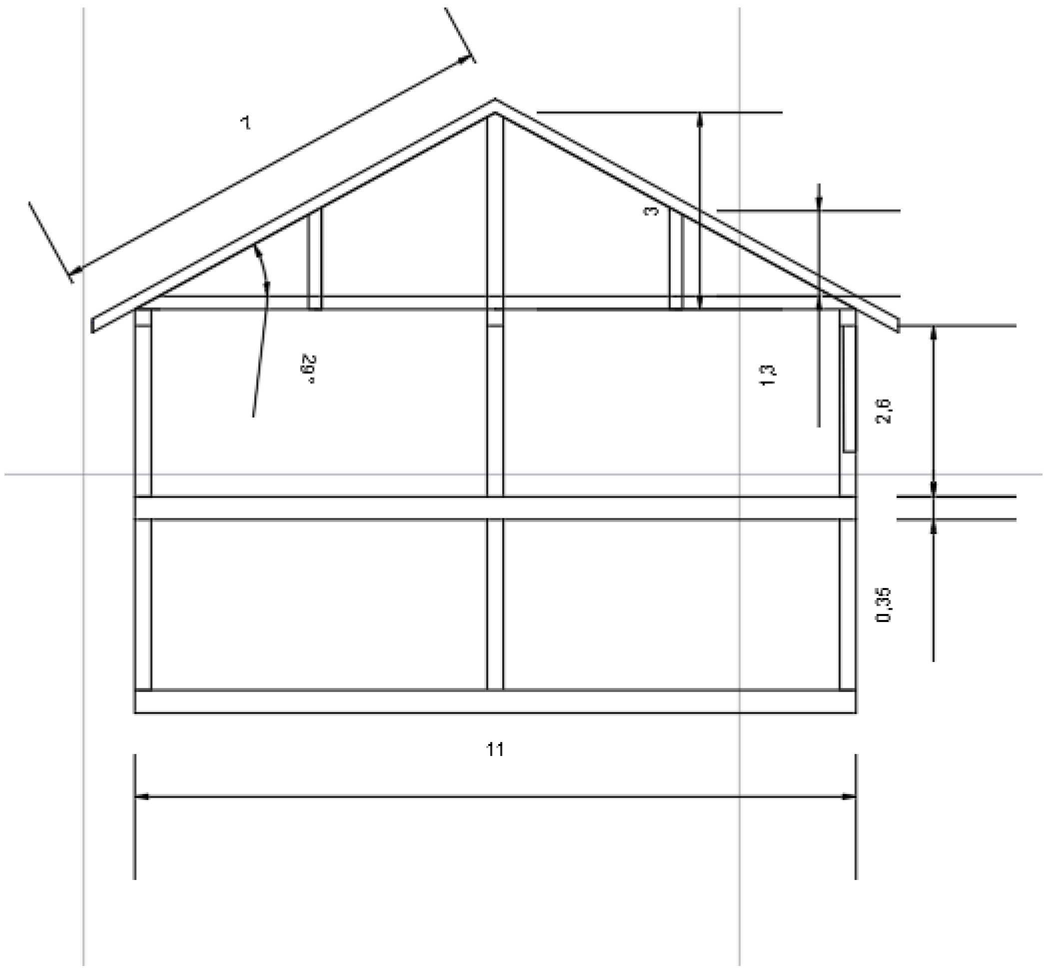
Looking at HoHo Vienna from the outside; the naturalness and, above all, the visibility of the wooden surfaces in the interior, are part of the core idea for the additional noticeable improvements and new tangible experience of the element of wood, in the world´s tallest wood structure high-rise.

Picture 2.8: Hoho Wien (Austria) [8]

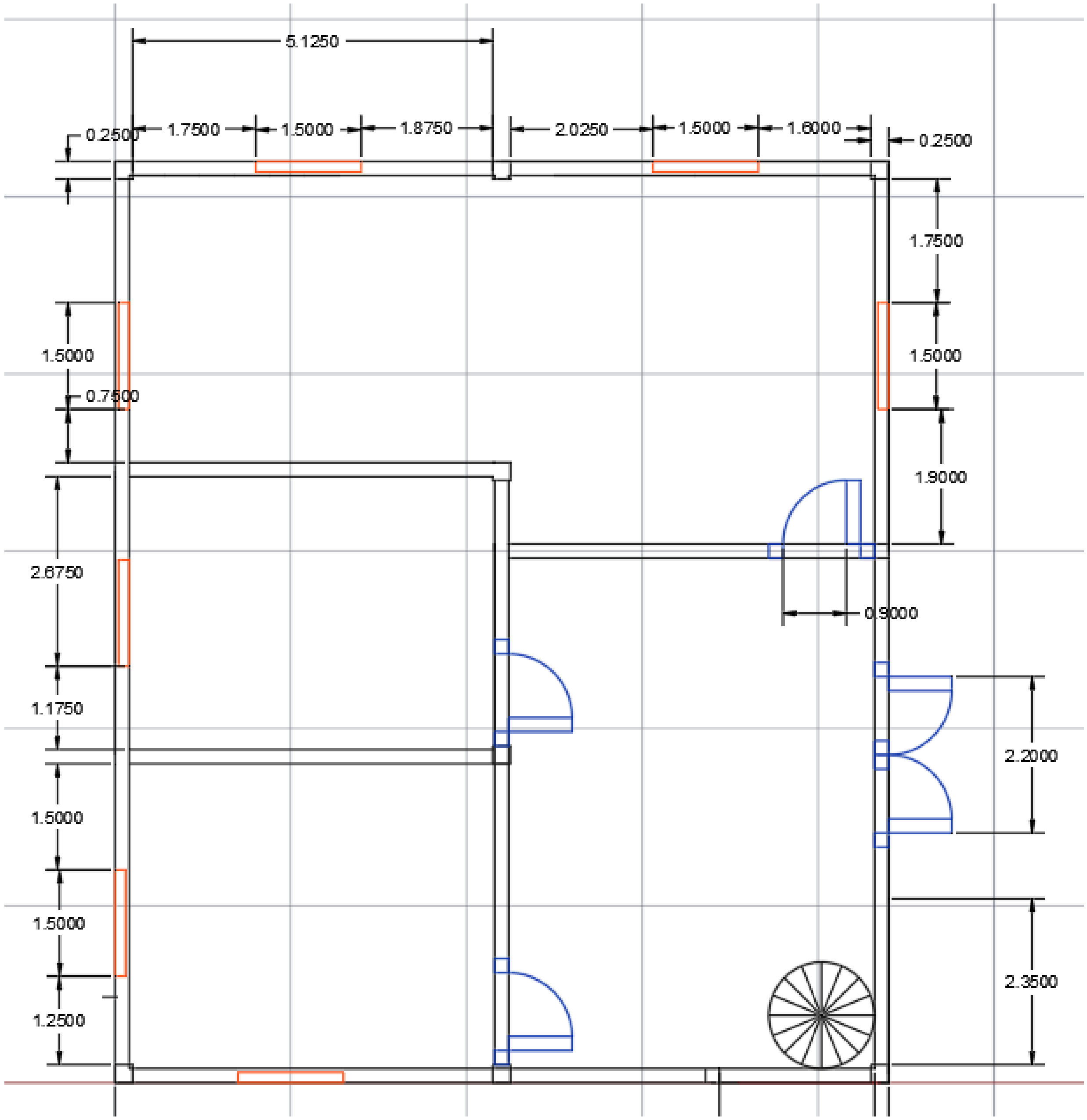
The HoHo Vienna is not only visually appealing, it also proves creativity when it comes to the use of space. The modular office structure allows individuality, and it can be modified at any time, transforming it without a lot of effort.

3. Proposed House Plan and structural Material:

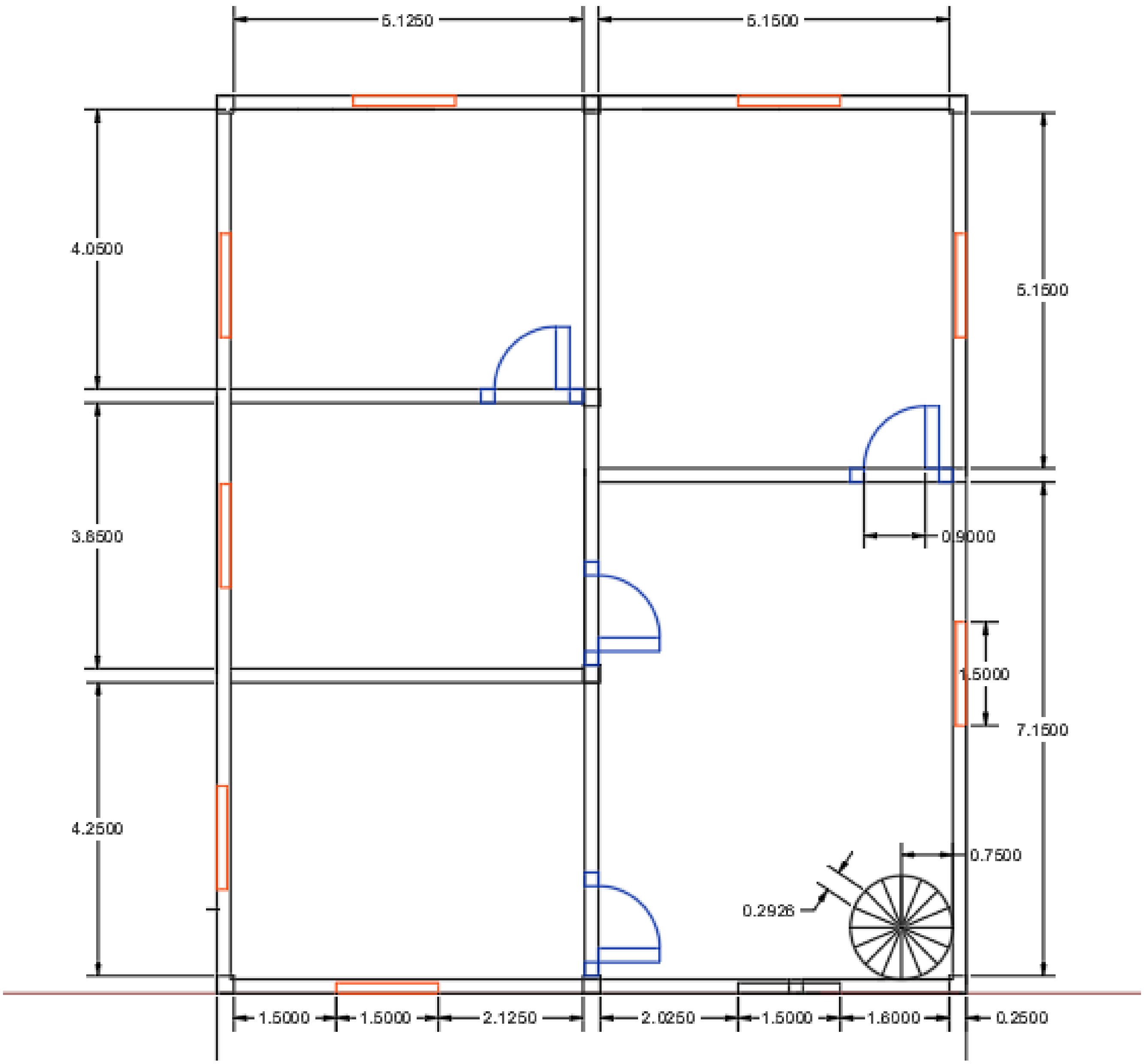
House Plans



Picture 3.1: Front View



Picture 3.2: First Floor Plan

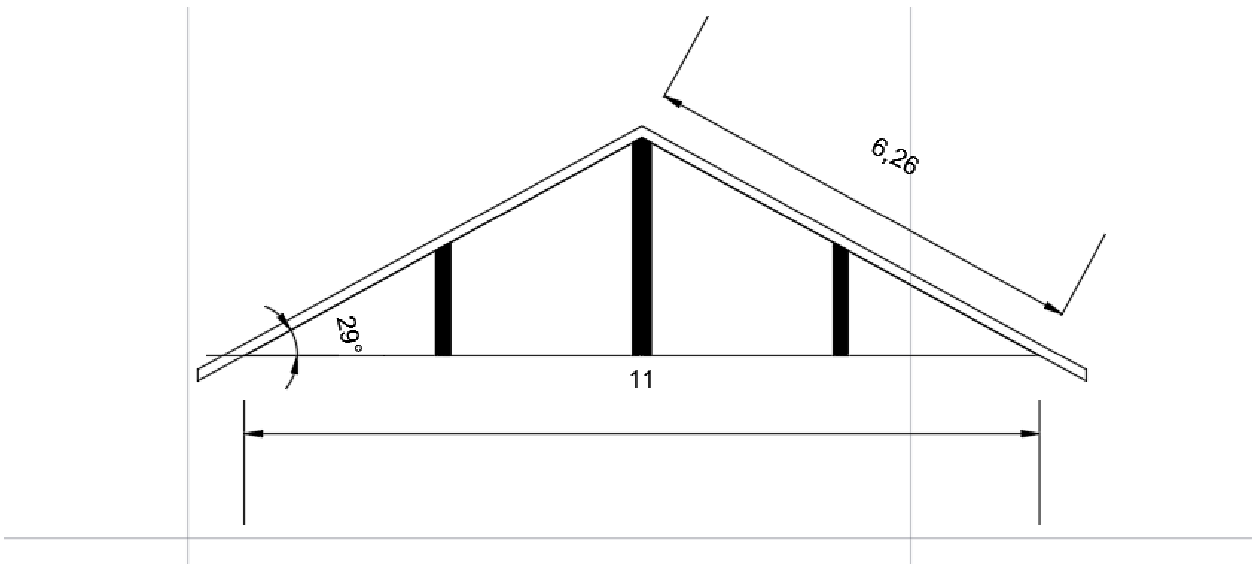


Picture 3.3: Second Floor Plan

4. Design of critical Structural elements ULS and SLS, fire resistance design of critical elements.

4.1 Roof Design

Roof type: Post-Beam



Picture 4.1: Roof dimensions

The roof is made of rafters at spacing 1 m.

The material used is Solid timber c24, the width of building is 11 m, and the length is 13 m.

The ridge height is 9m. Roof slope is (29°) and the overhang is 0.75 m on each side. The house is located in the eastern region of Czech Republic :

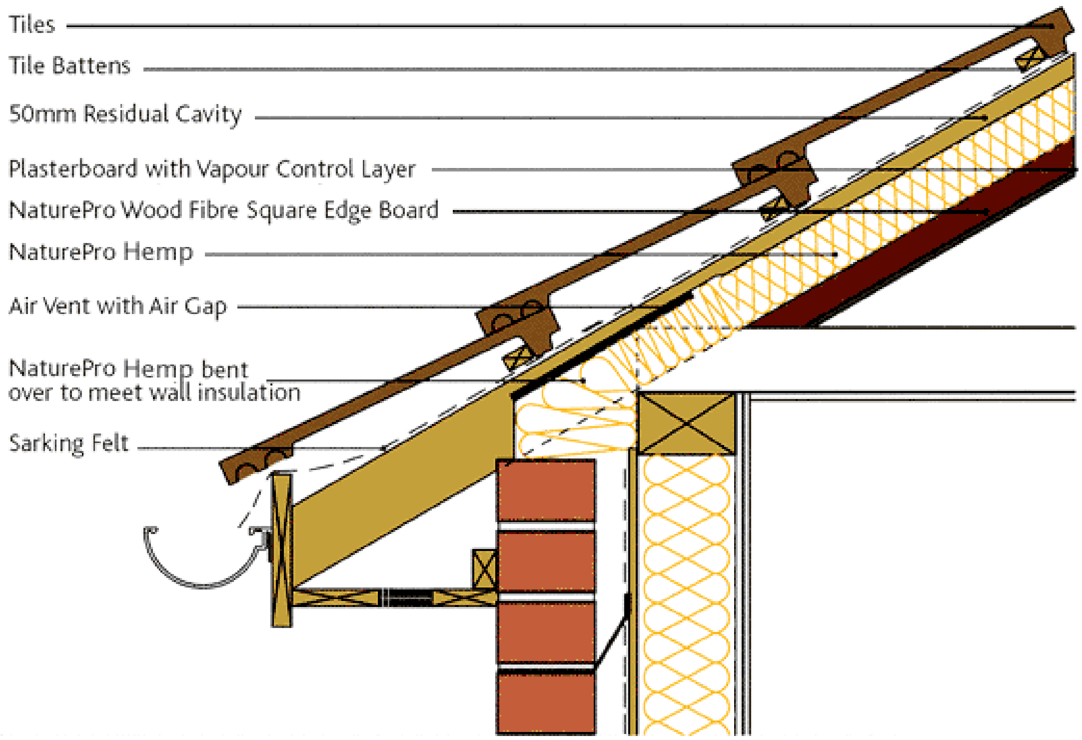
Snow Region III

Wind Region II

Terrain category III

Preliminary profile estimation:

-Rafter: 120/180



Picture 4.2: Roof details

4.1 Action calculation

Action on Rafter.

Description of Load	Calculation	Gk (kN/m)
Roof tiles γ=50 kg.m^{-2}	0.5 * 1 m	0.5
Tile batten γ=5 kg.m^{-2}	0.05 * 1 m	0.05
Roof Boards t. 20 mm. ρ = 500 kg.m^{-3}	5 * 0.02 * 1	0.10
Rafter Self Weight 120/180. ρ=420 kg.m^{-3}	4.2* 0.12 * 0.18	0.09
Other Insulations		0.26
Total	1 kN/m	

1.2) Imposed loads

1.2.1 Roof shell –

Category of roof: H => Roofs not accessible except for normal maintenance and repair

- $qk = 1\ kN/m^2 \cdot a = 1\ kN/m^2 \cdot 1m = 1\ kN/m$
- $Qk = 1\ kN$.

1.3) Snow load

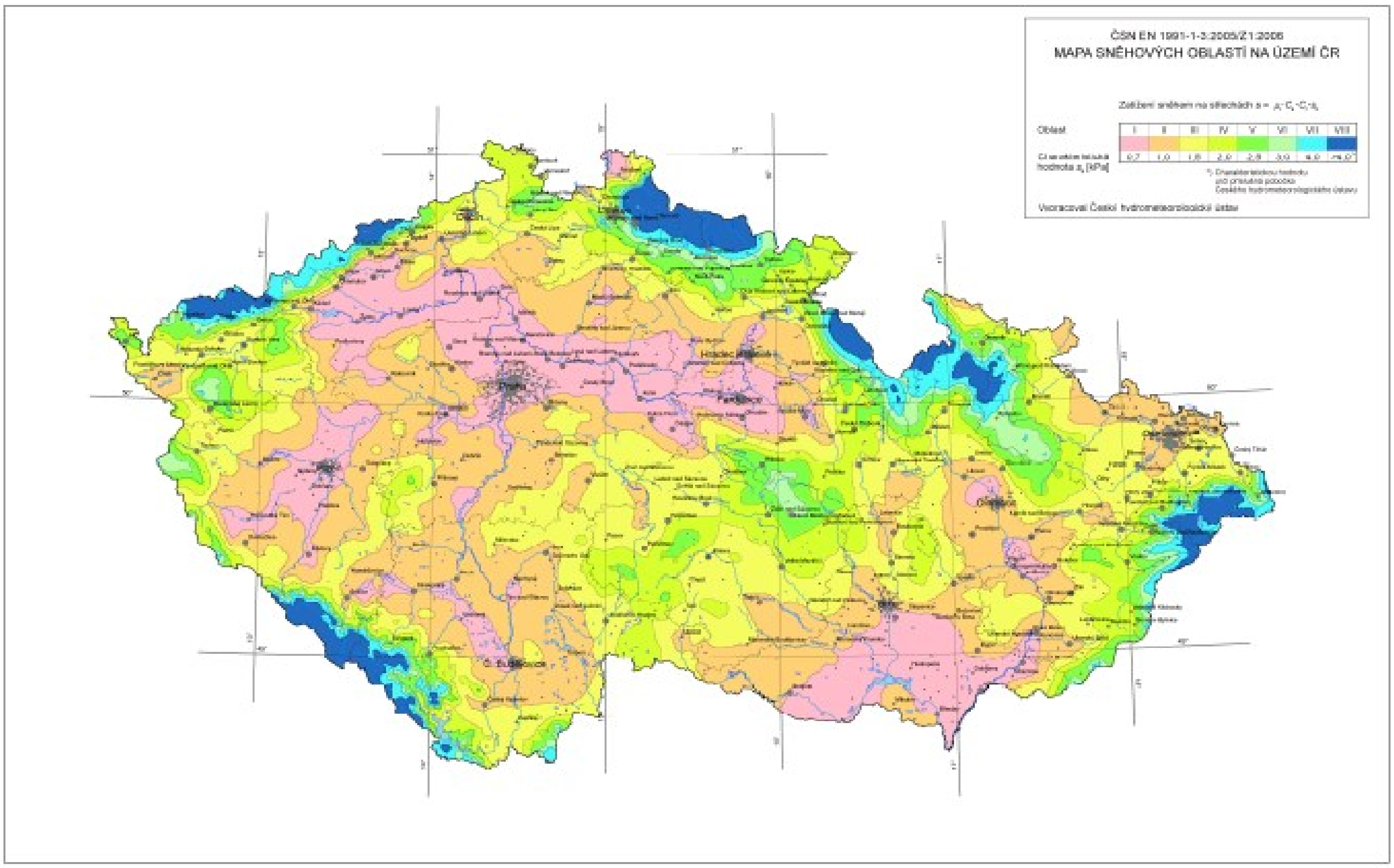
Many factors influence the snow load on roof:

a) Shape of the roof.

b) Thermal properties of the roof.

c) Roughness of the roof surface.
d) Heat quantity below the roof.
e) The distance of contiguous buildings.
f) Surrounding terrain.
g) Local climatic conditions.

Snow load map: Czech Republic



Picture 4.3: Snow map in Czech Republic [9]

TAB. 4 Coefficients of the shape

dip of the roof	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	0,8 (60 - α □) / 30	0,0
μ_2	0,8 + 0,8α / 30	1,6	---

Snow region III: sk = 1,5 kN/m2

Exposure coefficient (normal topography): *Ce* = 1,0

Thermal coefficient (λ < 1,0 $Wm^{-2}K^{-1}$) *Ct* = 1,0

Slope of the roof **α** = 29°

μ = (0.8)

Snow load on the frame:

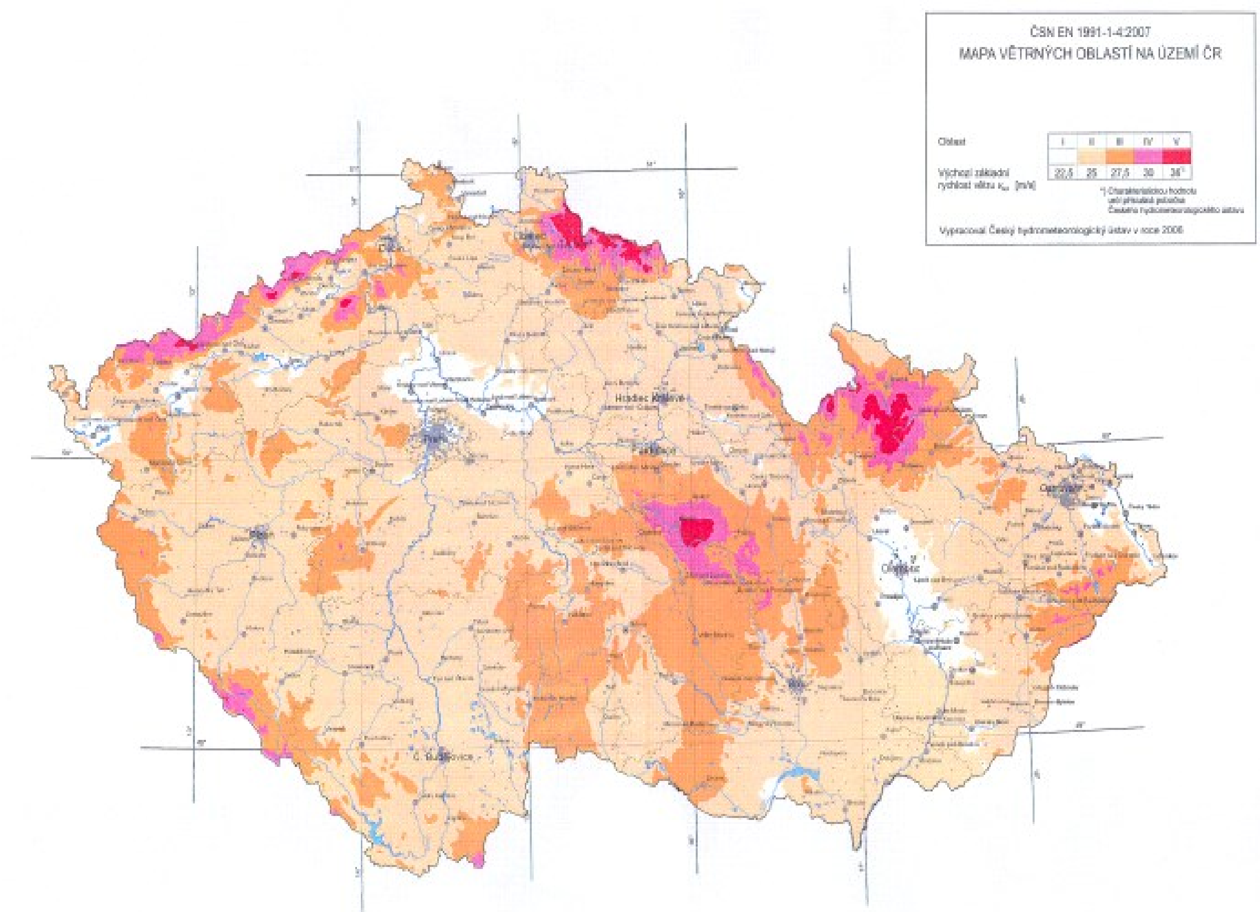
sk,1 = υ (29°) · Ce · Ct · sk · a = 0.8 · 1,0 · 1,0 · 1,5· 1m = **1.2 kN/m**

sk,2 = 0,5· sk,1 = **0,6 kN/m1**

1.4) Wind load

Wind velocity and pressure

Wind load map:



Picture 4.4: Wind map in Czech Republic [10]

Wind actions on structures and structural elements shall be determined taking account of both external and internal wind pressures

<u>The basic wind velocity:</u>

$v_b = C_{,dir} * C_{,season} * v_{b,0}$

for common cases:

$C_{,dir}$=1.0, 0 $C_{,season}$ = 1.0 and ;

$v_b = 1 * 1 * 25 = 25$ m.s^{-1}

Reference height:

$h = 9m \Rightarrow z_e = h = 9m$

$z_i = h = 9\ m$ (conservative).

Roughness factor:

$$C_r\,(z) = k_r * \ln\frac{Z}{Z_0}$$

Terrain category III ⇒ $z_0 = 0.3$

subsequently $z = z_e = z_i = 9\text{m} \geq z_{min} = 5.0$

Terrain factor is

$$k_r = 0.19\left(\frac{Z_0}{Z_{0II}}\right)^{0.07} = 0.19\left(\frac{0.3}{0.05}\right)^{0.07} = 0.215$$

and,

$$c_r(z) = 0.215 * \ln\frac{9}{0.3} = 0.731$$

Orography factor:

$C_0(z) = 1{,}0$

Mean wind velocity:

$V_m(z) = c_r(z).\ c_0(z).\ v_b = 0.731 * 1.0 * 25.0 = 18.275$ m/s

Peak velocity pressure:

$q_p(z) = [1+7.I_v(z)] * 0.5\,\rho * v_m^2$

- when considering turbulence intensity:

$$I_v(z) = \frac{kI}{C0\ (z).\ln\frac{z}{z0}} = \frac{1.0}{1.0*\ln\frac{9}{0.3}} = 0.294$$

And

$q_p(z) = \{1 + 7 * 0.294\} * ½ * 1.25 * 18.275^2 = 0.638\ \text{kN.m}^{-2}$.

Wind pressure on the roof structure

It is assumed placement of openable openings in the external cladding of the hall ⇒ it is necessary to consider also the internal pressure.

Resulting wind pressure on the structure:

$w_k = w_e + w_i$ (vector sum)

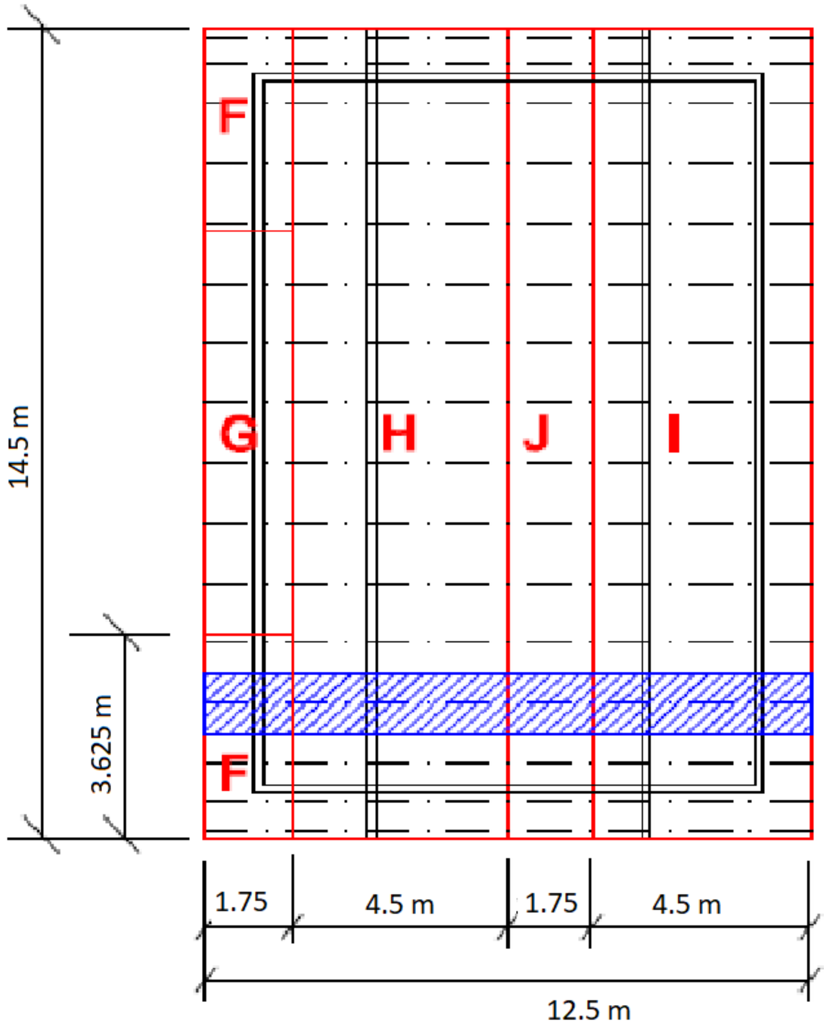
$w_k = 0.638 * (c_{pe} – c_{pi})\ \text{kN.m}^{-2}$

External pressure coefficients – wind acting perpendicular to the investigated plane

- Roof area exposed to the wind > 10,0 m^2 $\Rightarrow c_{pe,10}$
- To assessed rafter is applied load from zones F, H, J a I (see Fig.1.4.3)

$e = \min(b; 2 \cdot h) = \min(14.5; 2*9) = 14.5$ m

Pitch Angle α	Zone for wind direction $\theta = 0°$									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-0,6		-0,6		-0,8		-0,7		-1,0	-1,5
-30°	-1,1	-2,0	-0,8	-1,5	-0,8		-0,6		-0,8	-1,4
-15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2	-0,5		-0,7	-1,2
-5°	-2,3	-2,5	1,2	2,0	-0,8	-1,2	+0,2		+0,2	
							-0,6		-0,6	
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-0,6		+0,2	
	+0,0		+0,0		+0,0				-0,6	
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-0.4		-1.0	-1.5
	+0,2		+0,2		+0,2		+0,0		+0,0	+0,0
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-0,4		0,6	
	+0,7		+0,7		+0,4		+0,0		+0,0	
45°	-0,0		-0,0		-0,0		-0,2		-0,3	
	+0,7		+0,7		+0,6		+0,0		+0,0	
60°	+0,7		+0,7		+0,7		-0,2		-0,3	
75°	+0,8		+0,8		+0,8		-0,2		-0,3	



Picture 4.5: Pressure coefficient

$c_{pe,10}^{F-} = -0{,}23$

$c_{pe,10}^{G-} = -0{,}23$

$c_{pe,10}^{H-} = -0{,}09$

$c_{pe,10}^{I-} = -0{,}29$

$c_{pe,10}^{J-} = -0{,}39$

$c_{pe,10}^{F+} = +0{,}7$

$c_{pe,10}^{G+} = +0{,}7$

$c_{pe,10}^{H+} = +0{,}51$

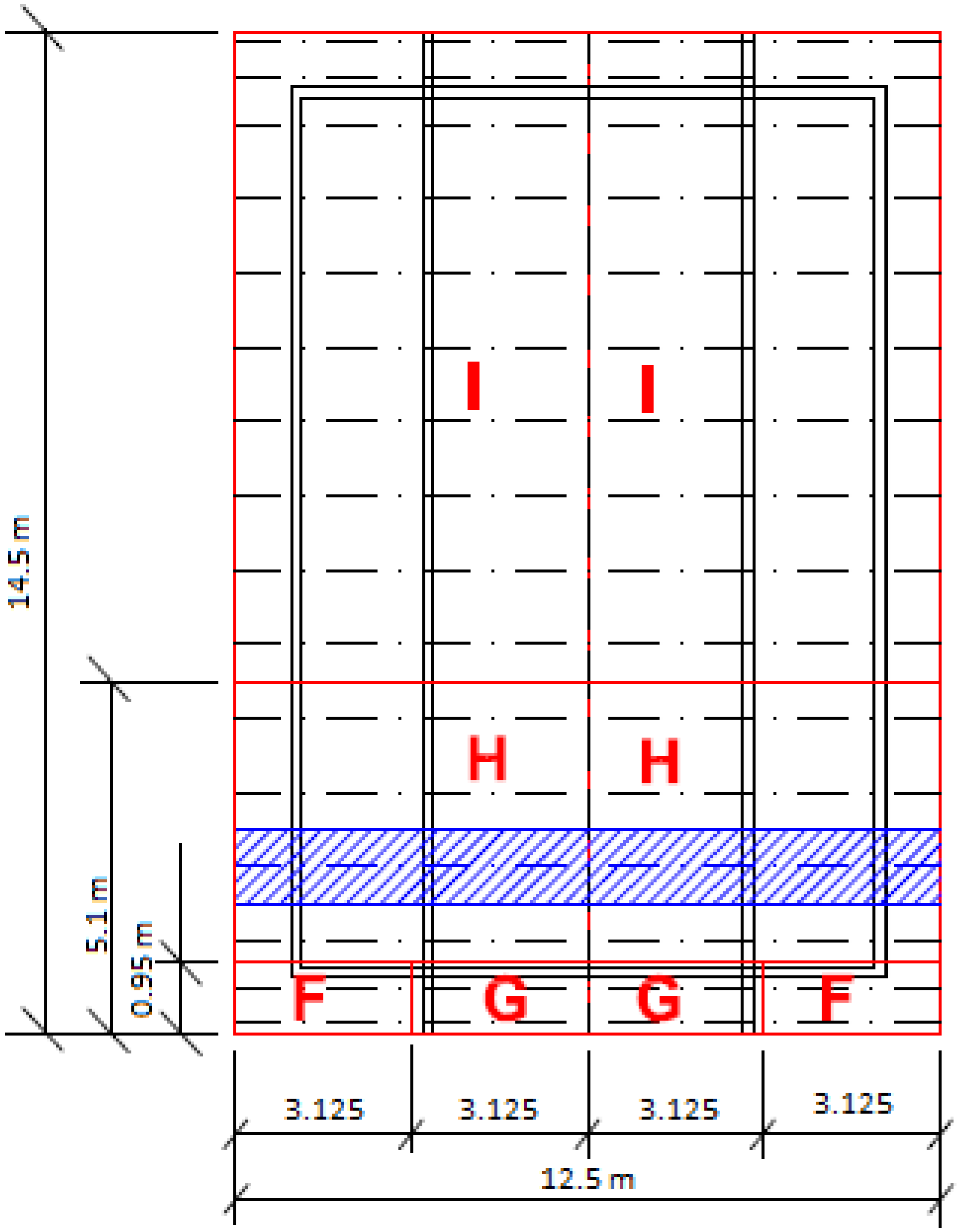
$c_{pe,10}^{I+} = 0{,}0$

$c_{pe,10}^{J+} = 0{,}0$

External pressure coefficients – wind acting in the investigated plane

- Roof area exposed to the wind > 10,0 m^2 $\Rightarrow c_{pe,10}$
- To assessed rafter is applied load from zones H (see Fig.)

$e = \min(b; 2 \cdot h) = \min(12.5; 2*9) = 12.5$ m



Picture 4.6: Pressure coefficient

Internal pressure coefficients

$C^{-}_{pi,10} = -0.3$
$C^{+}_{pi,10} = +0.2$

Resulting pressures on the roof structure

A) Perpendicular wind:

$C^{+}_{pi,10} = +0.2$

$W_k^{F\text{-}G\text{-}} = 0.638 * (-0.23-0.2) * 1m = -0.274\ kN.m^{-1} (\uparrow)$
$W_k^{F+G+} = 0.638 * (+0.7-0.2) * 1m = +0.319\ kN.m^{-1} (\downarrow)$
$W_k^{H-} = 0.638 * (-0.09-0.2) * 1m = -0.185\ kN.m^{-1} (\uparrow)$
$W_k^{H+} = 0.638 * (+0.51-0.2) * 1m = +0.197\ kN.m^{-1} (\downarrow)$
$W_k^{J-} = 0.638 * (-0.39-0.2) * 1m = -0.376\ kN.m^{-1} (\uparrow)$
$W_k^{J+} = 0.638 * (0-0.2) * 1m = -0.127\ kN.m^{-1} (\uparrow)$
$W_k^{I-} = 0.638 * (-0.29-0.2) * 1m = -0.312\ kN.m^{-1} (\uparrow)$
$W_k^{I+} = 0.638 * (0-0.2) * 1m = -0.127\ kN.m^{-1} (\uparrow)$

B) Perpendicular wind:

$C^{+}_{pi,10} = -0.3$

$W_k^{F\text{-}G\text{-}} = 0.638 * (-0.23+0.3) * 1m = +0.045\ kN.m^{-1} (\downarrow)$
$W_k^{F+G+} = 0.638 * (+0.7+0.3) * 1m = +0.638\ kN.m^{-1} (\downarrow)$
$W_k^{H-} = 0.638 * (-0.09+0.3) * 1m = +0.134\ kN.m^{-1} (\downarrow)$
$W_k^{H+} = 0.638 * (+0.51+0.3) * 1m = +0.197\ kN.m^{-1} (\downarrow)$
$W_k^{J-} = 0.638 * (-0.39+0.3) * 1m = -0.057\ kN.m^{-1} (\uparrow)$
$W_k^{J+} = 0.638 * (0+0.3) * 1m = +0.191\ kN.m^{-1} (\downarrow)$
$W_k^{I-} = 0.638 * (-0.29+0.3) * 1m = +0.0006\ kN.m^{-1} (\downarrow)$
$W_k^{I+} = 0.638 * (0+0.3) * 1m = +0.191\ kN.m^{-1} (\downarrow)$

C) Parallel wind,

$C^{+}_{pi,10} = +0.2$

$W_k^F = 0.638 * (-1.1-0.2) * 1m = -0.829\ kN.m^{-1} (\uparrow)$
$W_k^G = 0.638 * (-1.4-0.2) * 1m = -1.021\ kN.m^{-1} (\uparrow)$
$W_k^H = 0.638 * (-0.85-0.2) * 1m = -0.670\ kN.m^{-1} (\uparrow)$
$W_k^J = 0.638 * (-0.5-0.2) * 1m = -0.447\ kN.m^{-1} (\uparrow)$

D) Parallel wind,

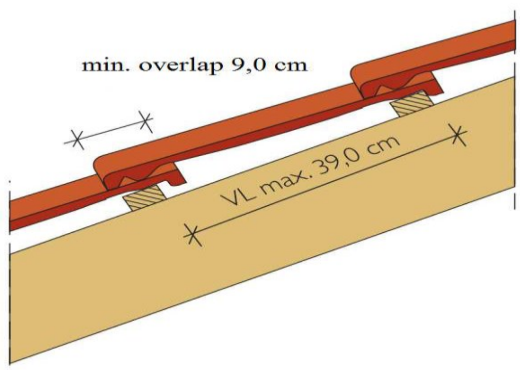
$C^{+}_{pi,10} = -0.3$

$W_k^F = 0.638 * (-1.1+0.3) * 1m = -0.510\ kN.m^{-1} (\uparrow)$
$W_k^G = 0.638 * (-1.4+0.3) * 1m = -0.702\ kN.m^{-1} (\uparrow)$
$W_k^H = 0.638 * (-0.85+0.3) * 1m = -0.351\ kN.m^{-1} (\uparrow)$
$W_k^J = 0.638 * (-0.5+0.3) * 1m = -0.128\ kN.m^{-1} (\uparrow)$

4.1.2. Design and assessment of tile battens
Action calculation

Permanent load

Description of Loads	Calculations	gk (kN/m)
Roof tiles **γ** = 50 kg.m^{-3}	0.5 * 0.3 m	0.15
Tile batten S.W **ρ** = 420 kg.m^{-3}	4.2 * 0.06 * 0.04	0.01
Total		0.16



Picture 4.6: Tile Battens

Imposed load
- $q_k = 0.75\ kN.m^{-2} * b * \cos 29° = 0.75 * (0.3 * \cos 29°) = 0.2\ kN.m^{-1}$
- $Q_k = 1$ kN

Snow load

$s_k = 1.24\ kN.m^{-2} * b * \cos 29° = 1.24 * (0.3 * \cos 29°) = 0.325\ kN.m^{-1}$

Wind load
- $w_k = 0638\ kN.m^{-2} * b = 0.638 * 0.3 = 0.191\ kN.m^{-1}$

Critical combinations

Permanent + snow + wind

$f_{Ed,y} = \gamma_G . g_k . \cos\alpha + \gamma_Q . w_k + \psi_0 . \gamma_Q . S_k . \cos\alpha =$
1.35 * 0.16 * cos (29) + 1.5 * 0.191 + 0.7 * 1.5 * 0.325 cos (29) = 0.77 $kN.m^{-1}$

$f_{Ed,z} = \gamma_G . g_k . \sin\alpha + \psi_0 . \gamma_Q . S_k . \sin\alpha =$
1.35 * 0.16 * sin (29) + 0.7 * 1.5 * 0.325 sin (29) = 0.27 $kN.m^{-1}$

Permanent + concentrated load

$f_{Ed,y} = \gamma_G . g_k . \cos\alpha =$ 1.35 * 0.16 * cos (29) = 0.189 $kN.m^{-1}$
$f_{Ed,z} = \gamma_G . g_k . \sin\alpha =$ 1.35 * 0.16 * sin (29) = 0.104 $kN.m^{-1}$
$Q_{Ed,y} = \gamma_Q . Q_k . \cos\alpha$ = 1.5 * 1 * cos (29) = 1.312 $kN.m^{-1}$
$Q_{Ed,z} = \gamma_Q . Q_k . \cos\alpha$ = 1.5 * 1 * sin(29) = 0.727 $kN.m^{-1}$

Bending moments

- tile beam is considered as continuous beam with two fields of 1 m span;
- for calculation are used static stables.

2.3.1 Permanent + snow + wind

$$M_{Ed,y} = 0.0703 * f_{Ed,y} * L^2 = 0.0703 * 0.77 * 1^2 = 0.054 \ kN.m$$
$$M_{Ed,z} = 0.0703 * f_{Ed,z} * L^2 = 0.0703 * 0.27 * 1^2 = 0.018 \ kN.m$$

Permanent + concentrated load

$$M_{Ed,y} = 0.0703 * f_{Ed,y} * L^2 + 0.2031 * Q_{Ed,y} * L =$$
$$= 0.0703 * 0.189 * 1^2 + 0.2031 * 1.312 * 1 = 0.280 \ kN.m$$

$M_{Ed,z} = 0.0703 * f_{Ed,z} * L^2 + 0.2031 * Q_{Ed,z} * L =$
$= 0.0703 * 0.104 * 1^2 + 0.2031 * 0.727 * 1 = 0.154 \ kN.m$

Calculation of stress in bending

Cross-section properties

$W_{el,y} = 1/6 * 60 * 40^2 = 16000 \ mm^3$
$W_{el,z} = 1/6 * 40 * 60^2 = 24000 \ mm^3$

Permanent + snow + wind

$\sigma_{m,y,d} = M_{ed,y}/W_{el,y} = 0.054 * 10^6/16000 = 3.375 \ Mpa$
$\sigma_{m,z,d} = M_{ed,z}/W_{el,z} = 0.018 * 10^6/24000 = 0.75 \ Mpa$

Permanent + concentrated load

$\sigma_{m,y,d} = M_{ed,y}/W_{el,y} = 0.280 * 10^6/16000 = 17.5 \ Mpa$
$\sigma_{m,z,d} = M_{ed,z}/W_{el,z} = 0.154 * 10^6/24000 = 6.42 \ Mpa$

Carrying capacity of biaxial bend

Permanent + snow + wind

$f_{m,k} = 30$ Mpa
$k_{mod} = 0.9$ (short-term, service class 2)

$f_{m,d} = k_{mod} * f_{m,k} / \gamma_M = 0.9 * 30/1.3 = 20.77 \ Mpa$

Assessment:

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

$k_m = 0.7\ldots$ for rectangular cross-sections

$(0.7 * \frac{3.75}{20.77}) + (\frac{0.75}{20.77}) = 0.150 < 1$ *Condition Fulfilled*

$(\frac{3.375}{20.77} + 0.7 * \frac{0.75}{20.77}) = 0.188 < 1$ *Condition Fulfilled*

Permanent + concentrated load

$f_{m,k} = 30$ *Mpa*
$k_{mod} = 1,1$ *(combination of permanent + concentrated load instantaneous load, service class 2)*

$f_{m,d} = k_{mod} * f_{m,k} / \gamma_M = 1.1 * 30/1.3 = 25.38$ *Mpa*

Assessment:

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1$$

$k_m = 0.7\ldots$ for rectangular cross-sections

$0.7 * \frac{17.5}{25.38} + \frac{6.42}{25.38} = 0.74\ Mpa$ *Condition Fulfilled*

$\frac{17.5}{25.38} + 0.7 * \frac{6.42}{25.38} = 0.87\ Mpa$ *Condition Fulfilled*

4.1.3 Design and assessment of Rafters

Cross-section properties

Rafter 120/180 mm
$A = 120*180 = 21600\ mm^2$
$W_y = 1/6 * 120 * 180^2 = 6.48 * 10^5\ mm^3$
$I_y = 1/12 * 120 * 180^3 = 5.832 * 10^7\ mm^4$
$i_y = \sqrt{(Iy/A)} = 52\ mm$

Material properties (C24)

$f_{c,0,k} = 21$ Mpa
$f_{m,k} = 24$ Mpa
$f_{v,k} = 4$ Mpa
$K_{mod} = 0.9$ (S.C 1, short-term load)
E0.05 = 7400 Mpa

$f_{c,0,d} = 14.54$ Mpa
$f_{m,d} = 16.62$ Mpa
$f_{v,d} = 2.77$ Mpa

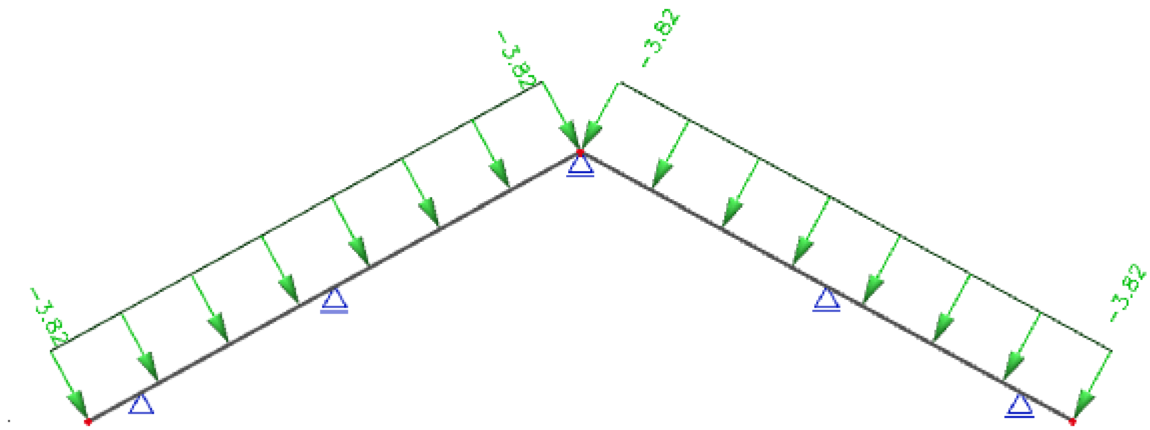
Critical combinations

Permanent + snow (leading) + wind ↓
$f_{Ed,y} = \gamma_G . g_k + \gamma_Q\ S_{k.} + \psi_0\ \gamma_Q . w_{k.} =$
$1.35 * 1 + 1.5 * 1.2 + 0.7 * 1.5 * 0.638 = 3.82\ kN.m^{-1}$

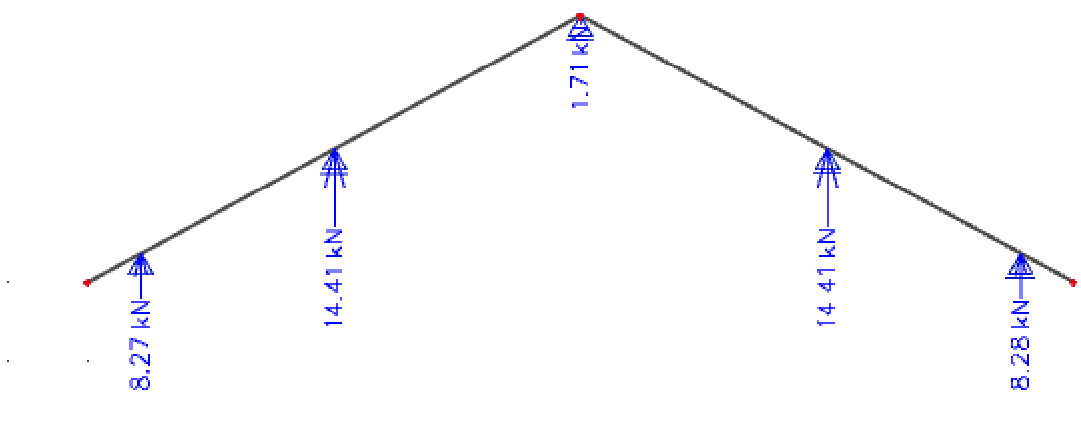
Permanent + wind ↓ (leading) + snow
$f_{Ed,y} = \gamma_G . g_k + \gamma_Q\ w_{k.} + \psi_0\ \gamma_Q . S_{k.} =$
$1.35 * 1 + 1.5 * 0.638 + 0.7 * 1.5 * 1.2 = 3.57\ kN.m^{-1}$

Permanent + concentrated load

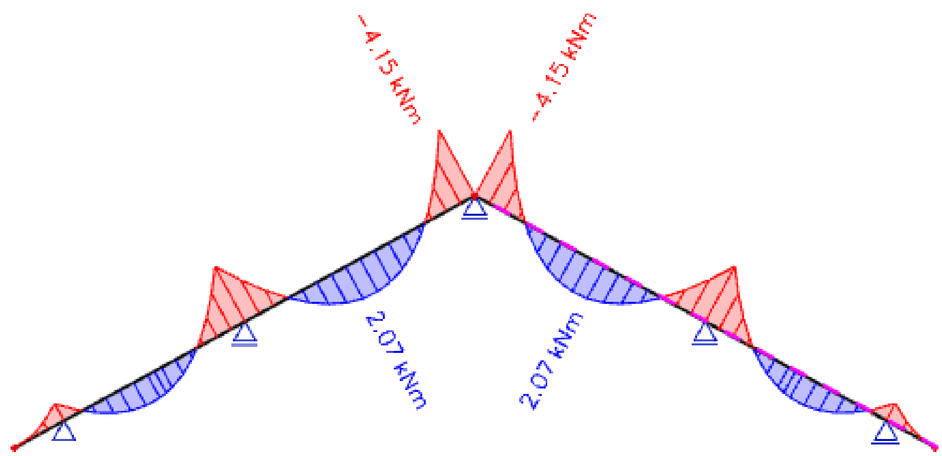
$f_{Ed,y,} = \gamma_G . g_k .+ \gamma_Q . Q_k =$
$1.35 * 1 \ +1.5 * 1 = 2.85\ kN.m^{-1}$



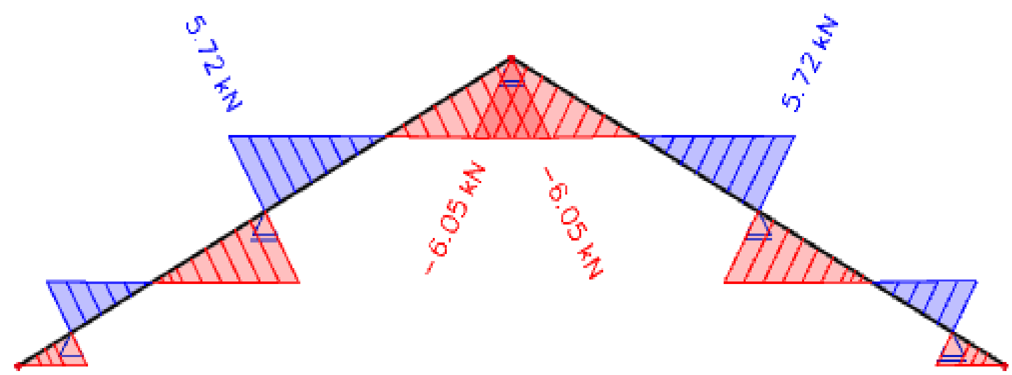
Picture 4.7: Load on Rafter



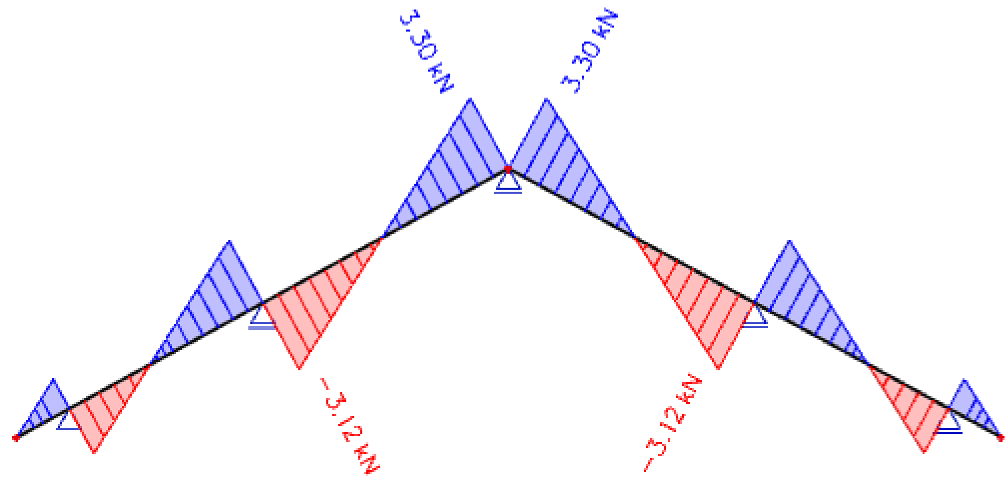
Picture 4.8: Reactions on Rafter



Picture 4.9: Bending moment diagram



Picture 4.10: Shear force diagram



Picture 4.11: Normal force diagram

Max positive moment Upper part of the rafter:

M_{ED} = 1.82 kN.m-1
V_{ED} = 0.33 kN
N_{ED} = 0.09 kN

Rafter is assessed for combination of compression axial force (buckling) and bending moment.

$$\sigma_{c,0,d} = \frac{N_{Ed}}{A} = \frac{0.09*10^3}{21600} = 0.004 \text{ Mpa}$$

$$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{1.82*10^6}{6.48*10^5}) = 2.80 \text{ Mpa}$$

$L_{crit} = L_{sys} = 3500$ mm
$i_y = 52$
$\lambda_y = L_{crit,y}/i_y = 67.3$

$$\lambda_{rel} = \lambda_y / \pi * (\frac{\sqrt{f_{c,0,k}}}{E_{0.05}}) = \frac{67.3}{3.14} * \sqrt{\frac{23}{7400}} = 1.19$$

$$k_y = 0.5 * (1+\beta_c(\lambda_{rel} - 0.3) + \lambda^2_{rel}) = 0.5\ (1 + 0.2(1.19 - 0.3) + 1.19^2)$$

$k_y = 1.29$

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y{}^2 - \lambda rel^2}} = 0.559$$

$$(\sigma_{c,0,d} / k_{c,y} * f_{c,0,d}) + (\sigma_{m,y,d} / f_{m,y,d}) < 1$$

$$(\frac{0.004}{0.559*14.54}) + \frac{2.8}{16.62} = 0.17 < 1$$

Max negative moment in connection with the purlin:

M_{ED} = -3.05
N_{ED} = -3.12
V_{ED} = 5.72
Rafter is assessed for combination of compression axial force and bending moment, where the
rafter is weakened with notch.
Notch weakens rafter about app. 40 mm.
Weakened cross-section properties

A_{osl} = 120 (180 -40) = 16800 mm^2
W_{osl} = 1/6 * 120 * (180-40)^2 = 3.92 * 10^5 mm^3

Eccentricity of the normal force due to the displaced centre of gravity of the weakened cross-section:
$e = 20mm$

$\sigma_{c,0,d} = \frac{N_{Ed}}{A_{osl}} = \frac{3.12*10^3}{16800} = 0.185\ Mpa$

$\sigma_{m,y,d} = \frac{M_{Ed}+N_{Ed}*e}{W_{osl}} = (\frac{3.05+(3.12*0.02)*10^6}{5.94*10^5}) = 7.92\ Mpa$

$(\frac{\sigma_{c,0,d}}{f_{c,0,d}})^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} < 1$

$(\frac{0.185}{14.54})^2 + \frac{7.92}{16.62} = 0.47 < 1$

Max shear force in the weakened cross section:
Weakened cross-section properties:

A_{osl} = 120 (180 -40) = 16800 mm^2
W_{osl} = 1/6 * 120 * $(180\text{-}40)^2$ = 3.92 * 10^5 mm^3
V_{Ed} = 7.03 kN

$$\tau_{v,d} = \frac{3}{2} * \frac{V_{Ed}}{A_{eff,Osl}} = (\frac{3}{2} * \frac{7.03 * 1000}{0.67 * 16800}) = 0.937 \text{ Mpa}$$

0.937 Mpa < 2.77 Mpa

Fire Resistance Design

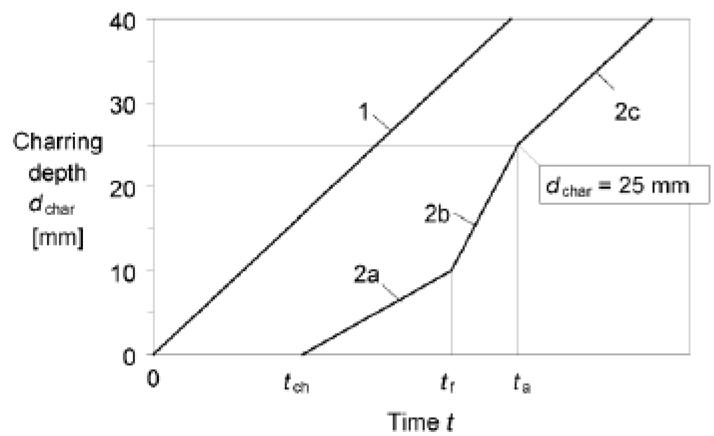
Required fire resistance t_{req} = 30 mins

Notional Charring rate β_n = 0.8 mm/min (Solid Timber)

Design Compression strength $f_{m,0,fi} = k_{fi} * f_{m,k}$ = 1.25 * 24 = 30 Mpa

Calculation of Section Modulus (3-Sides)

b_{fi} = 120 – 2(30 * 0.8 + 7) = 58 mm



Picture 4.12: Time Charring relationship

h_{fi} = 180 – (30 * 0.8 + 7) = 149 mm

A_{fi} = 58 * 149 = 8642 mm^2

$W_{fi,} = 1/6 * 58 * 149^2 = 2.14 * 10^5\ mm^3$

$M_{d,fi} = 1.82\ kN.m$

$\sigma_{m,y,fi}$ = 8.504 Mpa

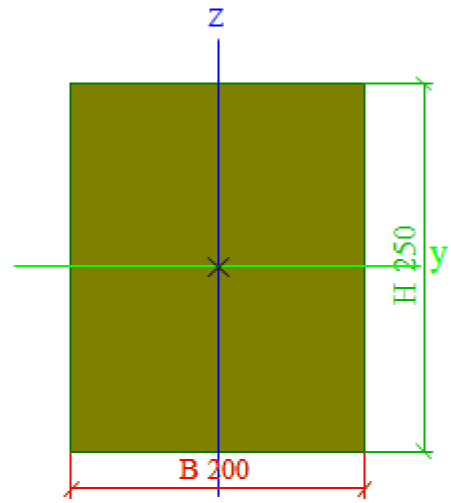
$N_{ED,fi}$ = 0.03 kN

$\sigma_{d,fi}$ = 0.003 Mpa

$\sigma_{d,fi}$ = 8.507 Mpa < 30 mpa

4.1.4. Purlin Assessment

Middle Part of the rafter



Picture 4.13: Cross-Section

Cross-section properties

A = 200 * 250 = 50000 mm^2

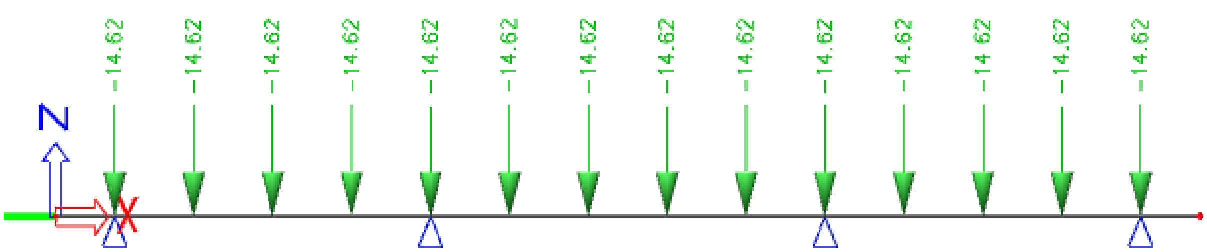
W_y = 1/6 (200) * 250^2 = 20.3 * 10^5 mm^3

Loading:

Dead Load G_k = self-weight

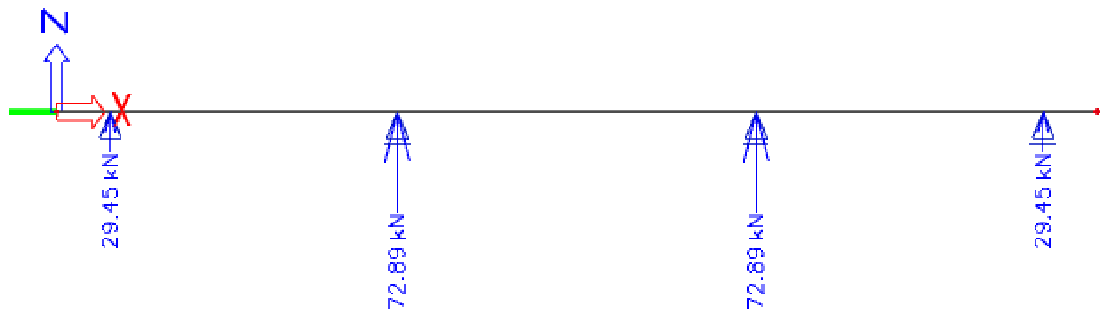
(0.25 * 0.2 * 4.6) = 0.21 kN/m

Load from the rafter 14.41 kN @ 1 m



Picture 4.14: Load on Purlin

Reactions:



Picture 4.11: Reactions on Purlin

Material properties C30

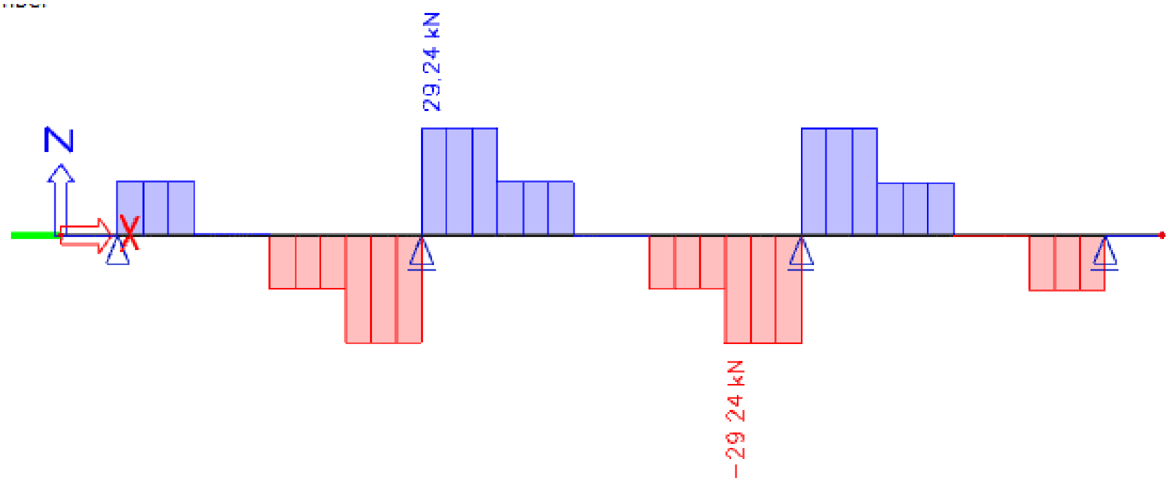
$f_{m,k}$ = 30 Mpa
$f_{v,k}$ = 3 Mpa
E0.05 = 7400 Mpa
k_{mod} = 0.7 (service class 1, long term load).

$f_{m,d} = k_{mod} * f_{m,k}/\gamma_M$ = 16.15 Mpa

$f_{v,d}$ = 1.62 Mpa

Assessment

Max Shear



Picture 4.16: Shear Force diagram

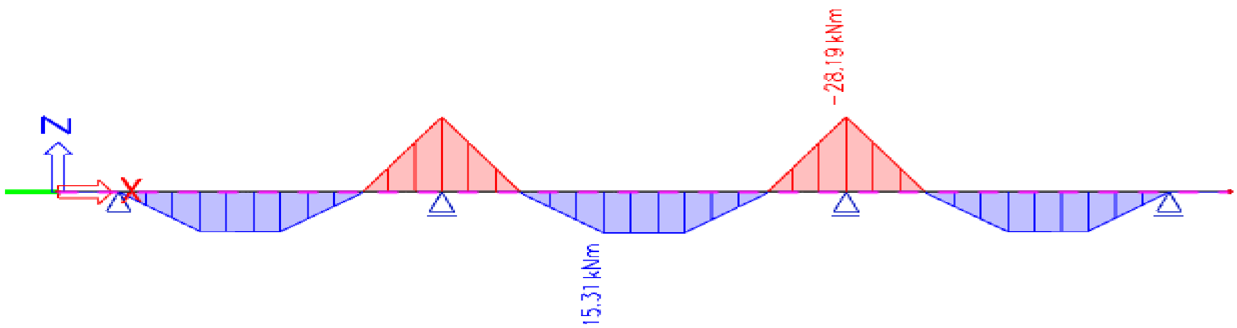
V_{Ed} = 29.24 kN

$\tau_{v,d} = \frac{3}{2} * \frac{V_{Ed}}{A_{eff}} = (\frac{3}{2} * \frac{29.24 * 1000}{0.67 * 50000})$ = 1.33 Mpa < 1.62 Mpa

$\tau_{v,d} < f_{v,d}$

Condition Fulfilled.

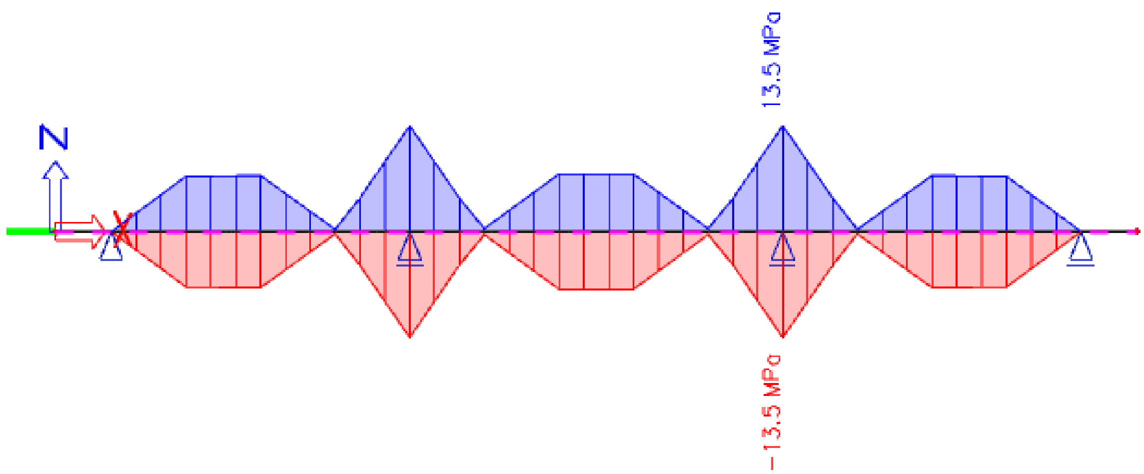
Max negative Moment



Picture 4.17: Bending moment diagram

M_{Ed} = - 28.2 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{28.2*10^6}{20.3*10^5})$ =13.9 Mpa < 16.5

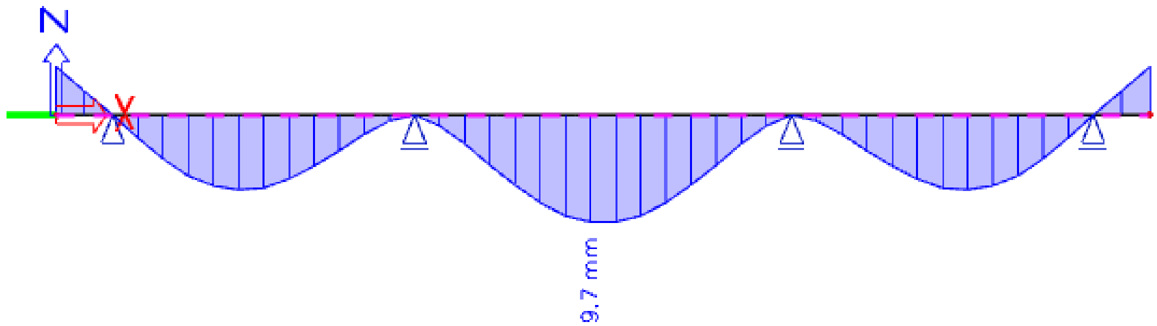


Picture 4.18: Stress diagram

$\sigma_{m,y,d} < f_{m,d}$

Condition Fulfilled.

SLS Max deformation is 9.7 mm



Picture 4.19: Purlin deflection

Fire Resistance Design

Required fire resistance t_{req} = 30 mins

Notional Charring rate β_n = 0.8 mm/min (Solid Timber)

Design Compression strength $f_{m,0,fi} = k_{fi} * f_{m,k} = 1.25 * 30 = 37.5$ Mpa

Calculation of Section Modulus (3-Sides)

$b_{fi} = 200 – (30 * 0.8 + 7) = 169$ mm

$h_{fi} = 250 – 2(30 * 0.8 + 7) = 188$ mm

$A_{fi} = 169 * 188 = 31772$ mm^2

$W_{y,fi} = 1/6 (169) * 188^2 = 9.96 *10^5$ mm^3

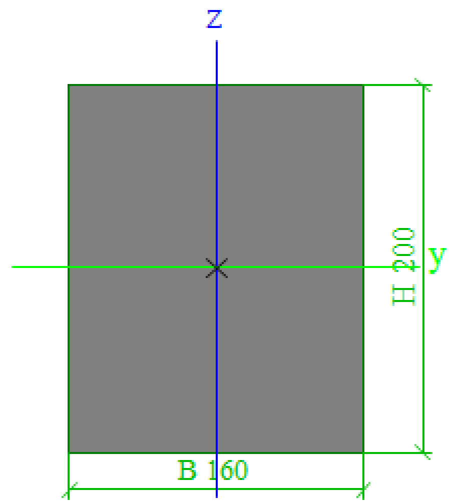
M_{Ed} = - 28.2 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{28.2*10^6}{9.96*10^5}) = 28.31$ Mpa < 34.5 Mpa

Condition fulfilled.

4.1.2.2. Purlin Assessment

Bottom part under the rafter



Picture 4.20: Purlin Cross Section

Cross-section properties

$A = 160 * 200 = 32000\ mm^2$

$W_y = 1/6\ (160) * 200^2 = 10.67 * 10^5\ mm^3$

$I_y = 1/12 * 160 * 200^3 = 10.67 * 10^7\ mm^3$

Material properties (C30)

$f_{m,k} = 30$ Mpa

$f_{v,k} = 3$ Mpa

E0.05 = 8 Gpa

$E_{mean} = 12$ Gpa

$k_{mod} = 0.7$ (service class 1, long term load).

$f_{m,d} = k_{mod} * f_{m,k}/\gamma_M = 16.15$ Mpa

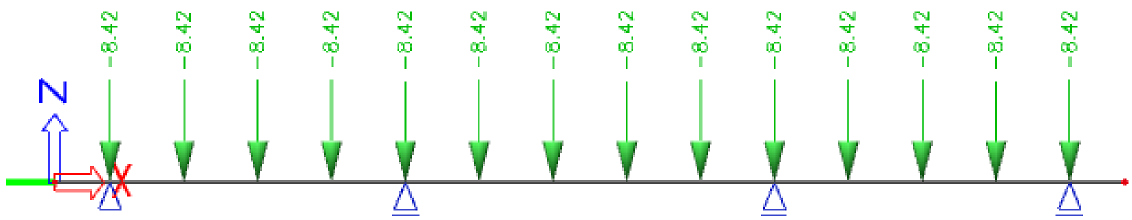
$f_{v,d} = 1.62$ Mpa

Loading:

Dead Load G_k = self-weight

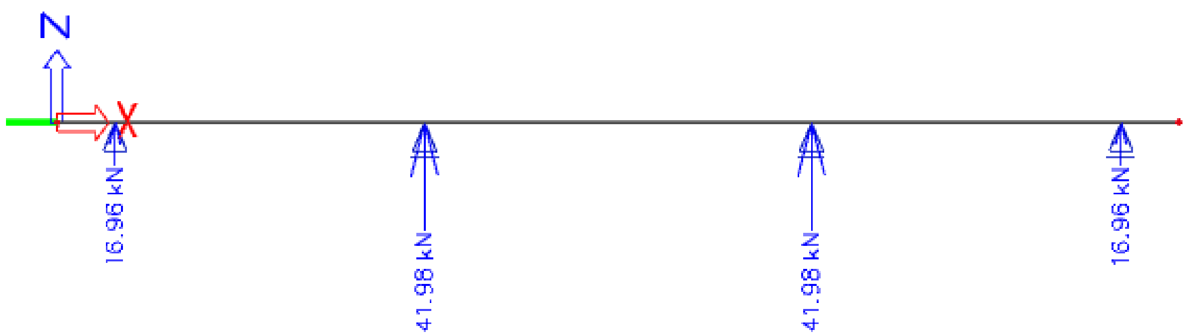
(0.16 * 0.2 * 4.6) = 0.147 kN/m

Load from the rafter 8.28 kN @ 1 m



Picture 4.21: Load on Purlin

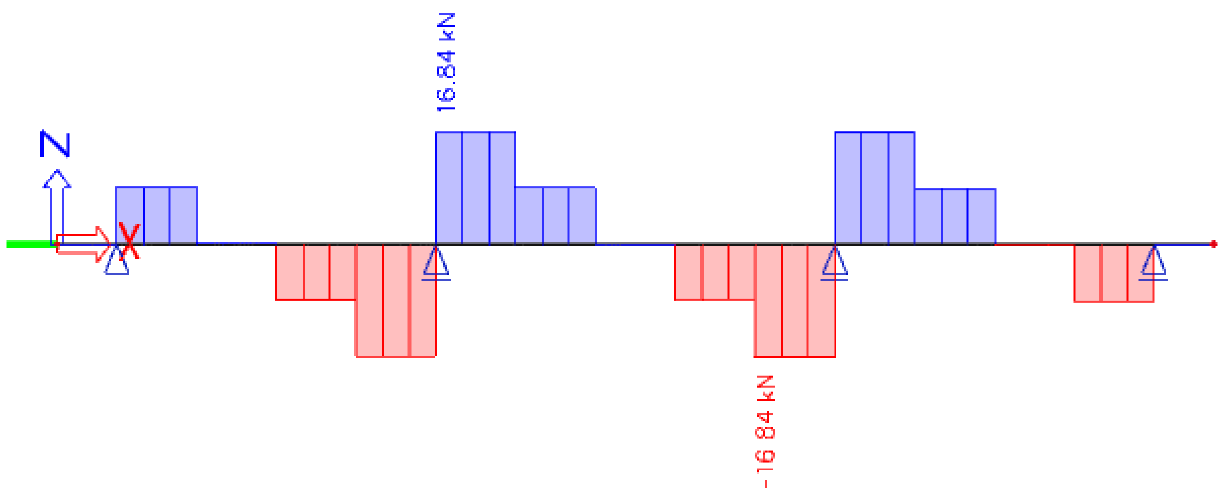
Reactions:



Picture 4.22: Reactions on Purlin

Assessment for Shear and Moment

Max Shear



Picture 4.23: Shear force diagram

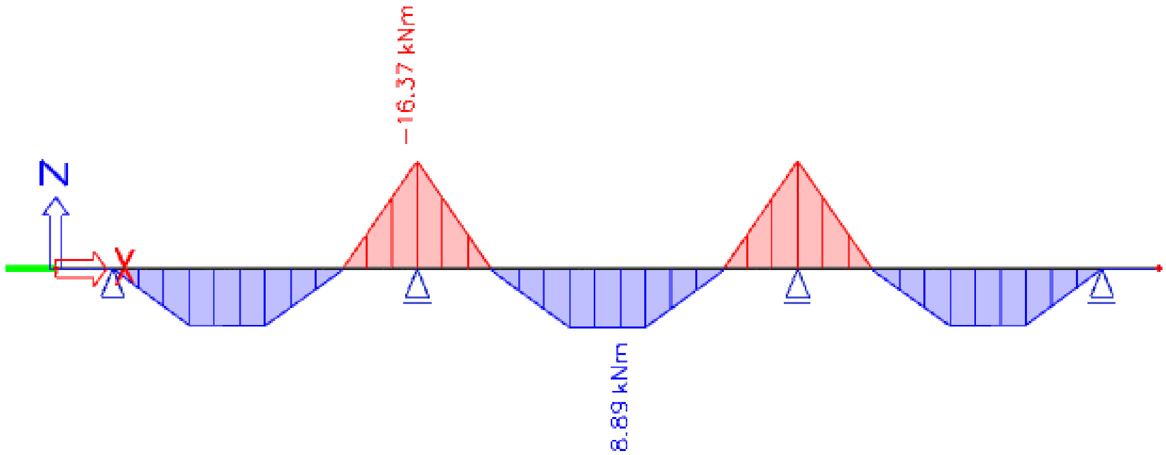
V_{Ed} = 16.84 kN

$\tau_{v,d} = \frac{3}{2} * \frac{V_{Ed}}{A_{eff}} = (\frac{3}{2} * \frac{16.84 * 1000}{0.67 * 32000})$ = 1.17 Mpa < 1.65 Mpa

$\tau_{v,d} < f_{v,d}$

Condition Fulfilled.

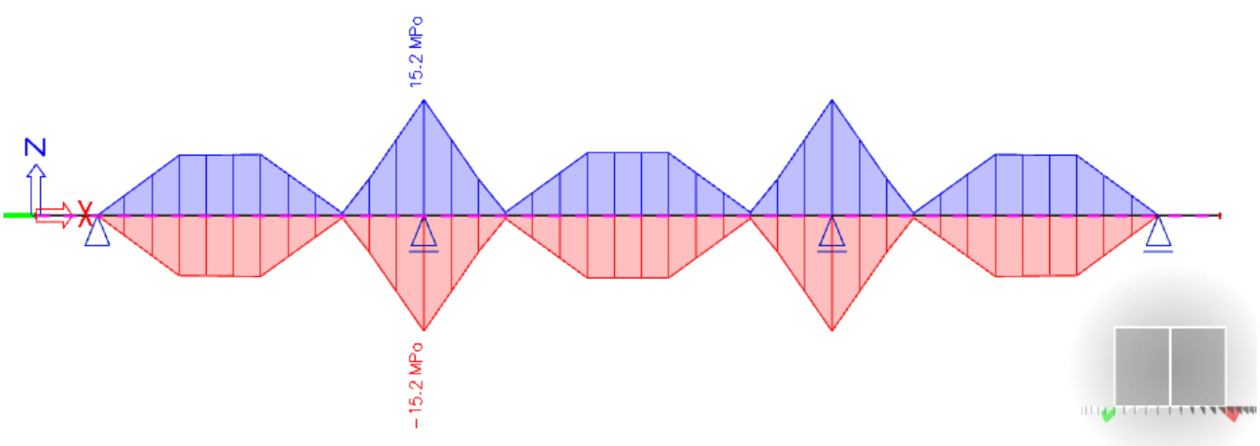
Max negative Moment



Picture 4.24: Bending moment diagram

M_{Ed} = - 16.37 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{16.37*10^6}{10.67*10^5})$ = 15.15 Mpa < 16.5



Picture 4.25: Bending Stress diagram

.

$\sigma_{m,y,d} < f_{m,d}$

Condition Fulfilled.

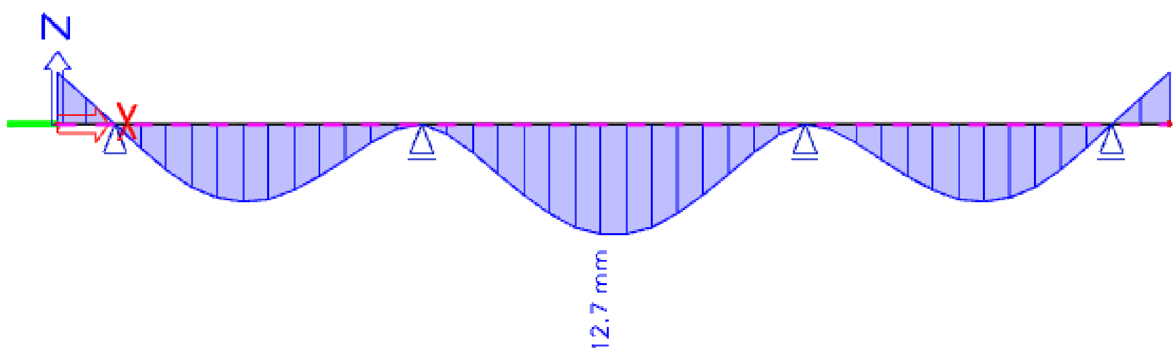
SLS Max deformation is 12.8 mm

$\delta_{inst} = \frac{5*0.012*5000^4}{384*12000*10.67*10^7}$ *(1+0.8)= 1.3 mm

$\delta_{fin} = \frac{5*1.6*5000^4}{384*12000*10.67*10^7}$ *(1+ (0.3*0.8)) = 11.4 mm

Impact deflection

δ_{inst} = l/300 = 5000/250 = 20 mm



Picture 4.26: Deflection in Purlin

Fire Resistance Design

Required fire resistance t_{req} = 30 mins

Notional Charring rate β_n = 0.8 mm/min (Solid Timber)

Design Compression strength $f_{m,0,fi} = k_{fi} * f_{m,k} = 1.25 * 30 = 37.5$ Mpa

Calculation of Section Modulus (3-Sides)

$b_{fi} = 160 – 2(30 * 0.8 + 7) = 98$ mm

$h_{fi} = 200 – (30 * 0.8 + 7) = 169$ mm

$A_{fi} = 138 * 219 = 16562$ mm^2

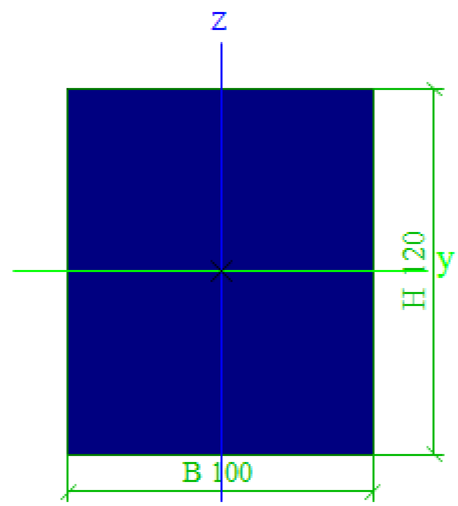
$W_{y,fi} = 1/6 (138) * 219^2 = 4.66 * 10^5$ mm^3

M_{Ed} = - 16.7 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{16.7*10^6}{4.66*10^5}) = 35.38$ Mpa < 37.5 Mpa

Condition Fulfilled

4.1.5 Post Design



Picture 4.27: Post Cross Section

Cross-section properties

A = 100 * 120 = 12000 mm^2

W_y = 1/6 * 150 * 250^2 = 15.625 * 10^5 mm^3

I_y = 1/12 * 150 * 250^3 = 19.532 * 10^7 mm^4

Iy = $\sqrt{(Iy/A)}$ = 72 mm

Material properties (c30)

$f_{c,0,k}$ = 23 Mpa

$f_{m,k}$ = 30 Mpa

$f_{v,k}$ = 3 Mpa

E0.05 = 8000 Mpa

k_{mod} = 0.7 (service class 1, long term load).

l_y = 35

l_z = 29

Design Strength Value

in compression along the grains

$f_{c,0,d}$ = 14.15 Mpa

$f_{m,d}$ = 16.5 Mpa

$f_{v,d}$ = 1.65 Mpa

Slenderness

$\lambda_y = l_{cr}/i_y = 3000/35 = 85.72$

$\lambda_z = l_{cr}/i_z = 3000/29 = 103.45$ *(higher slenderness to z-axis)*

$\lambda_{rel} = \lambda_{y/}\,\pi * (\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}) = 1.76$

$k_z = 0.5 * (1+\beta_c(\lambda_{rel} - 0.3) + \lambda^2{}_{rel}) = 0.5\ (1 + 0.2(1.84 - 0.3) + 1.84^2)$

$k_z = 2.34$

$k_{c,z} = 0.264$

Buckling coefficient

$k_z = 0.5 * \{1+\beta_c(\lambda_{rel} - 0.3) + \lambda^2{}_{rel,z}\} = 0.5\ (1 + 0.2(1.84 - 0.3) + 1.84^2)$

$k_z = 2.34$

$k_{c,z} = 0.264$

$k_{c,z} = \frac{1}{k_z + \sqrt{k^2{}_z - \lambda_z{}^2}} = \frac{1}{2.34 + \sqrt{2.34^2 - 1.84^2}} = 0.264$

$\sigma_{c,0,d} = N_{Ed}/A = 13.78 * 1000 / 12000 = 1.148$ *Mpa*

$\sigma_{c,0,d} / k_{c,z} * f_{c,0,d} < 1$

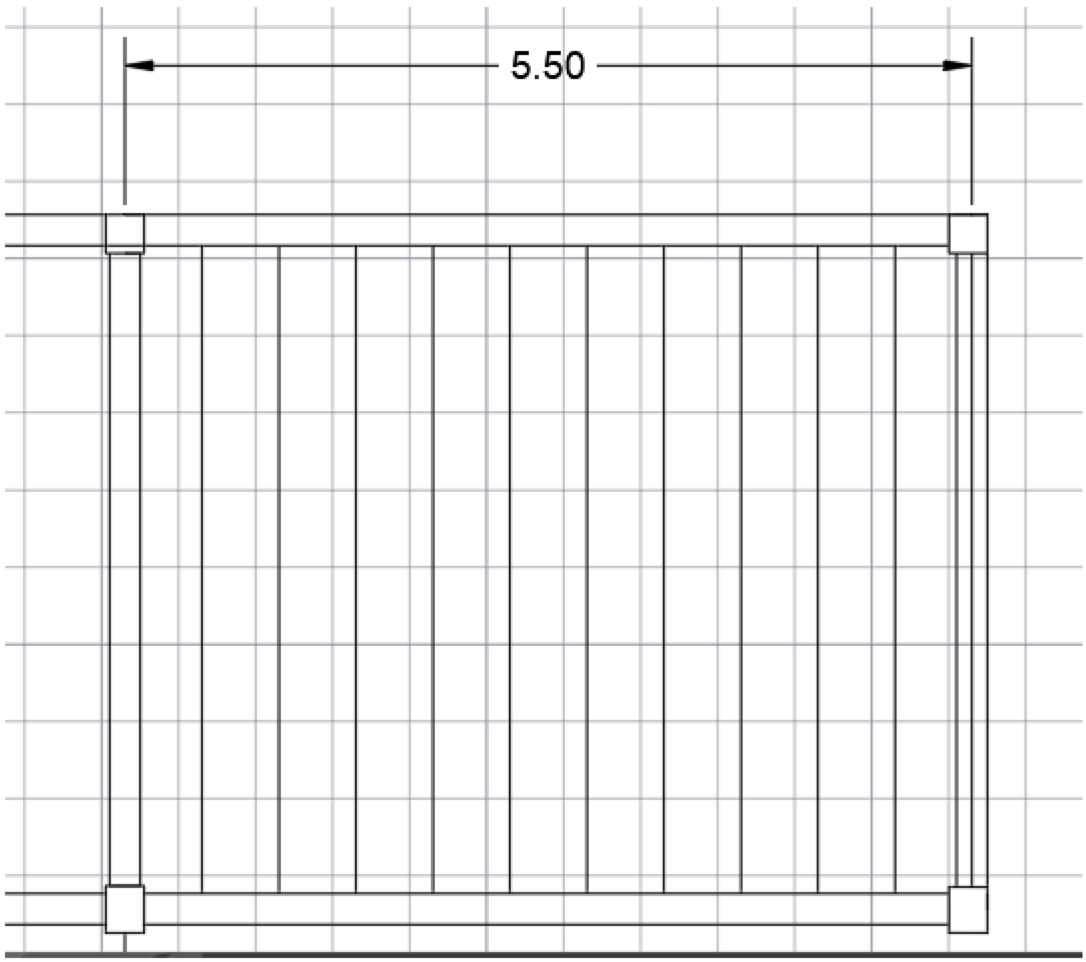
$1.148 / 3.736 < 1$

$0.317 < 1$

Post for Buckling is satisfied.

5. Floor joist design

The top floor is made up from joists 75 * 250 mm, C16 spacing at 500 mm and the clear span is 4500 mm



Picture 5.1: Floor Joist plan

Loading:

Dead Load G_k = self-weight + finishes (timber boards = 0.15 kN/m^2)

(0.25 * 0.075 * 3.7) + (0.5 * 0.15) = 0.15 kN/m

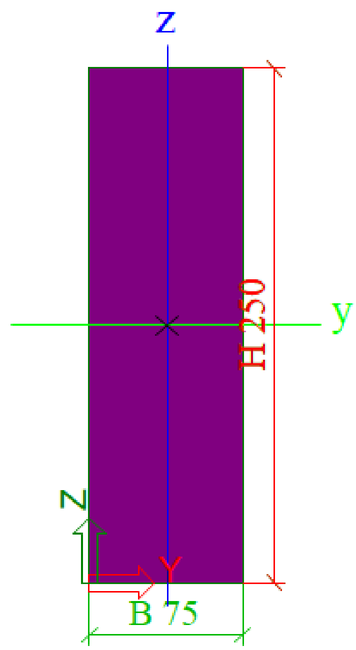
Imposed Load = Q_k = 2.5 kN/m^2 * 0.4 = 1 kN/m

UDL_{ULS} = 1.35 * 0.144 + 1.5 * 1 = 1.7 kN/m

ULD_{SLS} = 1.15 kN/m

Cross-section properties

A = 75 * 250 = 18750 mm^2
W_y = 1/6 * 75 * 250^2 = 7.81 * 10^5 mm^3
I_y = 1/12 * 75 * 250^3 = 9.76 * 10^7 mm^4
Iy = $\sqrt{(Iy/A)}$ = 72.15 mm



Picture 5.2: Joist Cross Section

Material properties (C16)

$f_{c,0,k}$ = 17 Mpa
$f_{m,k}$ = 16 Mpa
$f_{v,k}$ = 1.8 Mpa
$E_{0.05}$ = 5400 Mpa
$E_{0,mean}$ = 8000 Mpa
k_{mod} = 0.7 (service class 1, long term load).

$f_{m,d} = k_{mod} * f_{m,k}/\gamma_M$ = 8.61 Mpa

$f_{v,d}$ = 0.92 Mpa

Bending Stress check

M_{ED} = -$\frac{1.7*4.5^2}{8}$ = 4.3 kN.m

$\sigma_{m,d}$ = $\frac{4.3*10^6}{7.8*10^5}$ = 5.51 Mpa < 8.61 Mpa

Bending is Satisfied.

Shear Stress check

V_{ED} = $\frac{1.7*4.5}{2}$ = 3.83 kN

$\tau_{v,d}$ = ($\frac{3}{2}$ * $\frac{3.83*1000}{0.67*18750}$) = 0.42 Mpa < 0.92 Mpa

Shear is Satisfied

SLS Deformation

δ_{inst} = $\frac{5*0.07*4500^4}{384*8000*97.7*10^6}$ *(1+0.8)= 0.86 mm

δ_{fin} = $\frac{5*1*4500^4}{384*8000*97.7*10^6}$ *(1+ (0.3*0.8)) = 6.83 mm

impact deflection

δ_{inst} = l/300 = 4500/250 = 18 mm

Deflection is Satisfied

6. Critical Beam Design

Dead Load G_k = self-weight + finishes

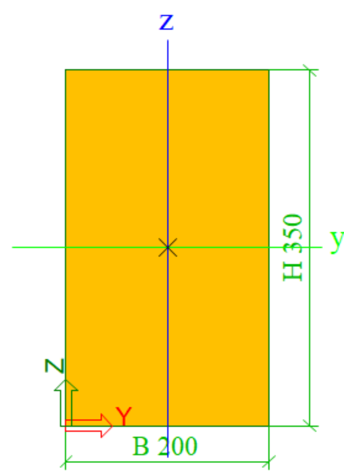
(0.2 * 0.35 * 4.2) = 0.294 kN/m

Load from the joist (3.83 @ 500 mm) = 7.66 kN/m

Cross-section properties

A = b * h = 200 * 350 = 70000 mm^2

W = 1/6 * 200 * 350^2 = 40.833 mm^3



Picture 6.1: Beam Cross Section

Material properties (C30)

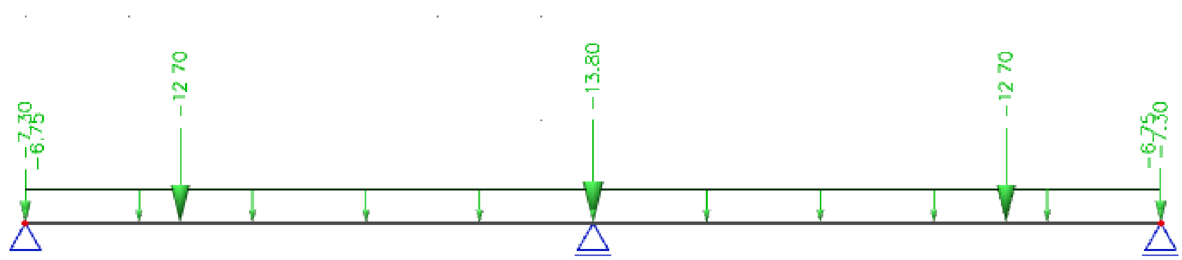
$f_{c,0,k}$ = 23 Mpa
$f_{m,k}$ = 30 Mpa
$f_{v,k}$ = 3 Mpa
E0.05 = 7400 Mpa
k_{mod} = 0.7 (service class 1, long term load).

$f_{c,0,d} = k_{mod} * f_{c,0,k}/\gamma_M$ = 12.38 Mpa

$f_{m,d} = k_{mod} * f_{m,k}/\gamma_M$ = 16.15 Mpa

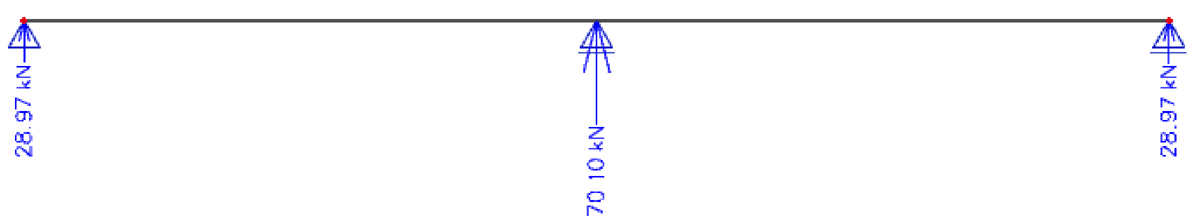
$f_{v,d}$ = 1.615 Mpa

Loading



Picture 6.2: Loading on Beam

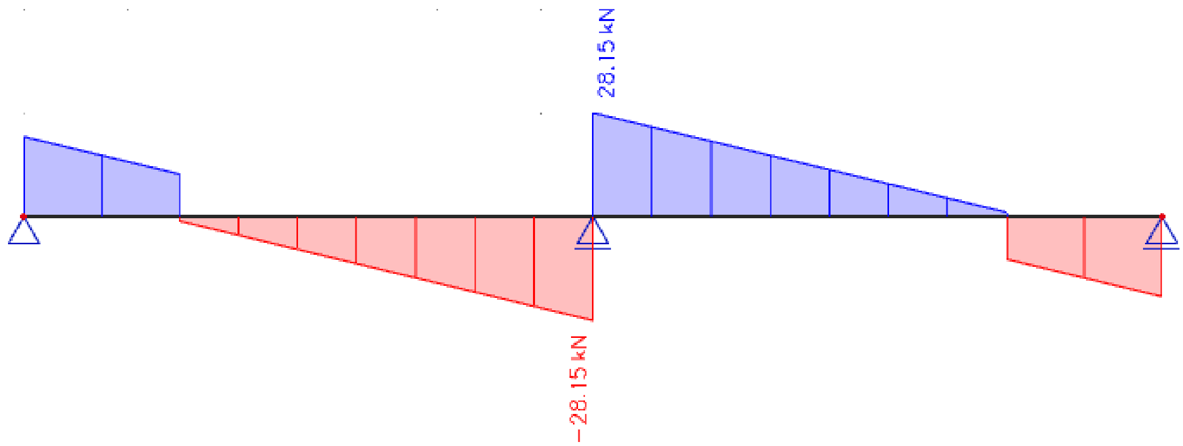
Reactions



Picture 6.3:Reactions **on** Beam

Assessment

Max Shear



Picture 6.4:Shear Force Diagram

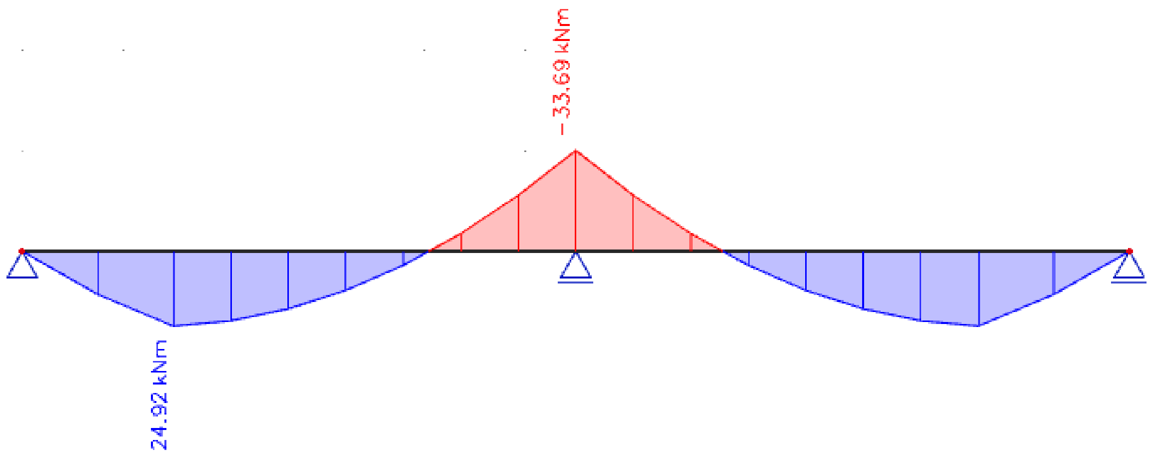
V_{Ed} = 28.15 kN

$\tau_{v,d} = \frac{3}{2} * \frac{V_{Ed}}{A_{eff}} = (\frac{3}{2} * \frac{28.15 * 1000}{0.67 * 70000})$ = 0.896 Mpa < 1.615 Mpa

$\tau_{v,d} < f_{v,d}$

Condition Fulfilled.

Max negative Moment



Picture 6.5: Bending Moment Diagram

M_{Ed} = -33.69 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{33.69*10^6}{40.83*10^5})$ = 8.25 Mpa < 16.5

.

$\sigma_{m,y,d} < f_{m,d}$

Condition Fulfilled.

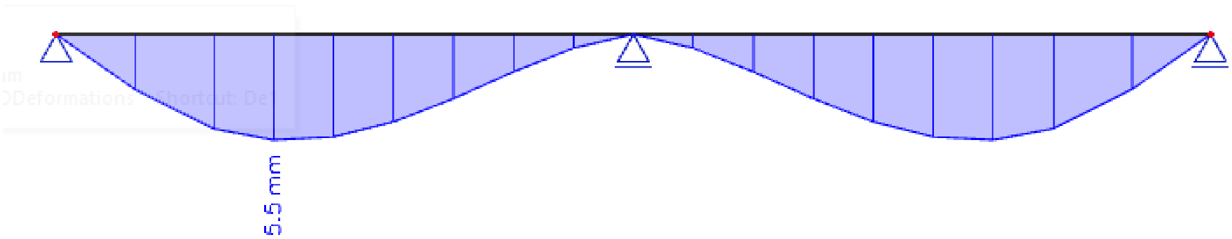
δ_{inst} = 0.9 mm

δ_{fin} = 4.6 mm

impact deflection

δ_{inst} = l/300 = 5500/250 = 22 mm

Deflection 5.5 mm



Picture 6.6: Beam Deflection

Fire Resistance Design

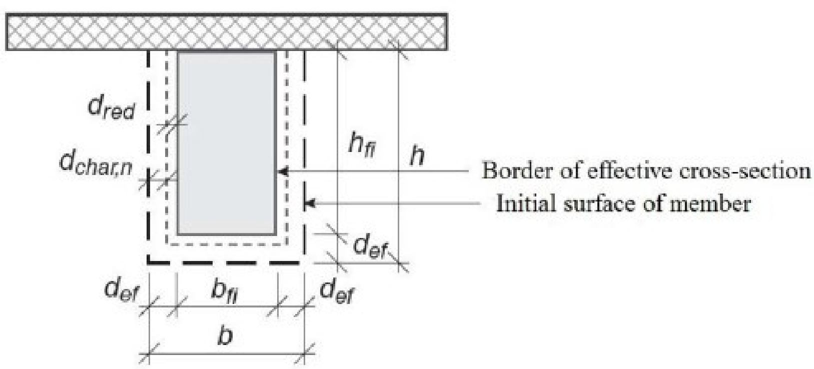
Required fire resistance t_{req} = 30 mins

Notional Charring rate β_n = 0.8 mm/min (Solid Timber)

Design Bending strength $f_{m,d,fi} = k_{fi} * f_{m,k} = 1.25 * 30 = 37.5$ Mpa

Shear Strength $f_{v,d,fi} = k_{fi} * f_{v,k} = 1.25 * 4 = 5$ Mpa

Calculation of Section Modulus (3-Sides)



Picture 6.7: Border of effective Cross Section

b_{fi} = 200 – 2(30 * 0.8 + 7) = 138 mm

h_{fi} = 350 – (30 * 0.8 + 7) = 319 mm

A_{fi} = 319 * 138 = 44022 mm^2

W_{fi} = 1/6 * 138 * 319^2 = 23.4 mm^3

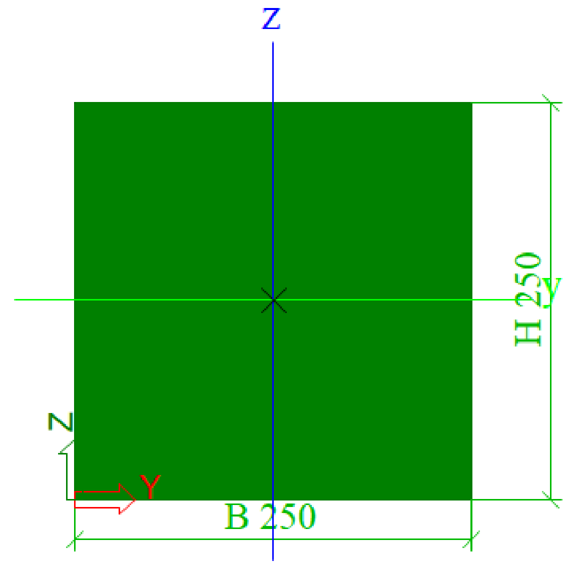
M_{Ed} = -33.69 kN.M

$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{33.69*10^6}{23.4*10^5})$ = 14.4 Mpa < 37.5

Condition fulfilled

7. Column Supporting critical beam

Load on the column 70.3 kN



Picture 7.1: Column Cross Section

Cross-section properties

A = 250 * 250 = 62500 mm^2
I_y = 1/12 * 250 * 250^3 = 32.55 * 10^7 mm^4
Iy = $\sqrt{(Iy/A)}$ = 72 mm

Material properties (C30)

$f_{c,0,k}$ = 23 Mpa
$f_{m,k}$ = 30 Mpa
$f_{v,k}$ = 3 Mpa
E0.05 = 7400 Mpa
k_{mod} = 0.7 (service class 1, long term load).

Design Strength Value

in compression along the grains
$f_{c,0,d}$ = 14.15Mpa
$f_{v,d}$ = 2.46Mpa

Slenderness

$\lambda_y = l_{cr}/i_y = 2600/72 = 36.11$

(higher slenderness to z-axis)

$\lambda_{rel} = \lambda_{y/}\ \pi * (\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}) = 0.64$

Buckling coefficient

$k_z = 0.5 * \{1+\beta_c(\lambda_{rel} - 0.3) + \lambda^2{}_{rel,z}\} = 0.5\ (\ 1 + 0.2(0.64 - 0.3) + 0.64^2)$

$k_z = 0.7388$

$k_{c,z} = 0.89$

$k_{c,z} = \frac{1}{k_z + \sqrt{k^2{}_z - \boldsymbol{\lambda_z}{}^2}} = \frac{1}{2.34 + \sqrt{2.34^2 - 1.84^{\ 2}}} = 0.89$

$\sigma_{c,0,d} = N_{Ed}/A = 70.3 * 1000 / 62500 = 1.125\ Mpa$

$\sigma_{c,0,d} / k_{c,z} * f_{c,0,d} < 1$

1.125 / 12.60 < 1

0.09 < 1

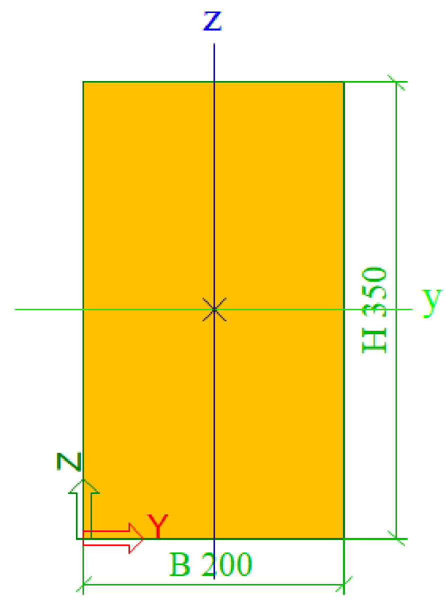
Column for Buckling is satisfied.

8. Critical Beam Design

Cross-section properties

$A = b * h = 200 * 350 = 70000\ mm^2$

$W = 1/6 * 200 * 350^2 = 40.833\ mm^3$



Picture 8.1 Beam Cross Section

Material properties (C30)

$f_{c,0,k}$ = 23 Mpa
$f_{m,k}$ = 30 Mpa
$f_{v,k}$ = 3 Mpa
E0.05 = 7400 Mpa
k_{mod} = 0.7 (service class 1, long term load).

$f_{c,0,d} = k_{mod} * f_{c,0,k}/\gamma_M = 12.38$ Mpa

$f_{m,d} = k_{mod} * f_{m,k}/\gamma_M = 16.15$ Mpa

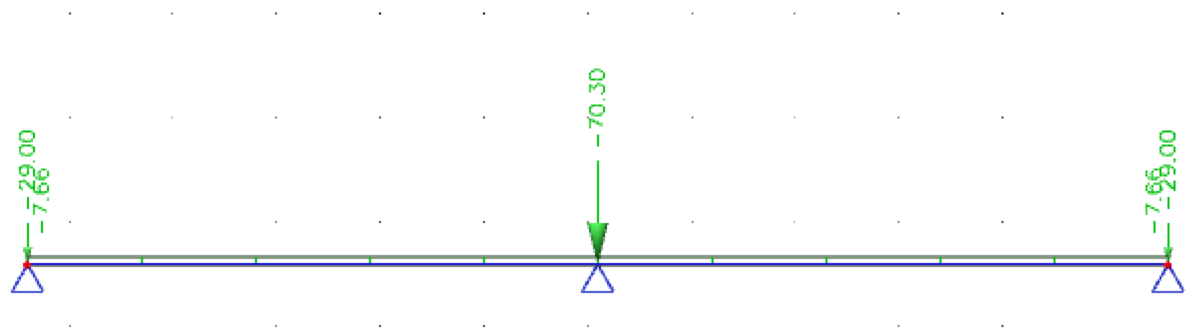
$f_{v,d}$ = 1.615 Mpa

Loading & Reactions

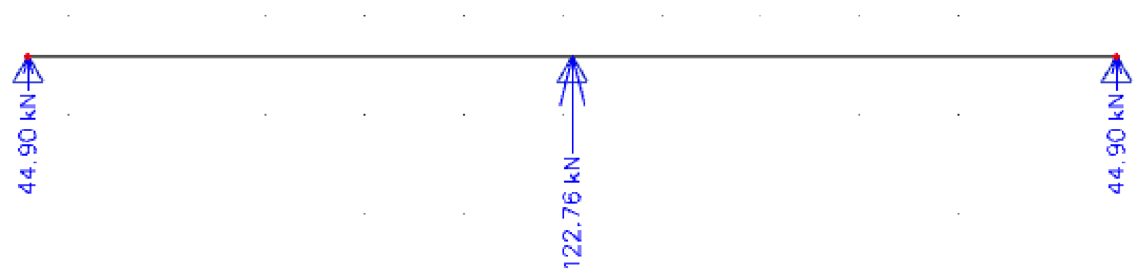
Dead Load G_k = self-weight

(0.2 * 0.35 * 4.2) = 0.294 kN/m

Load from the joist (3.83 @ 500 mm) = 7.66 kN/m



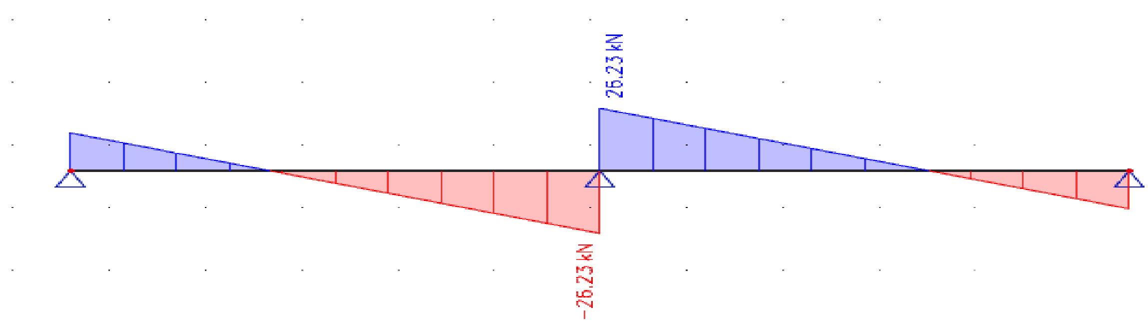
Picture 8.2 Loads on Beam



Picture 8.3 Beam Reactions

Assessment

Max Shear



Picture 8.4 Shear Force Diagram

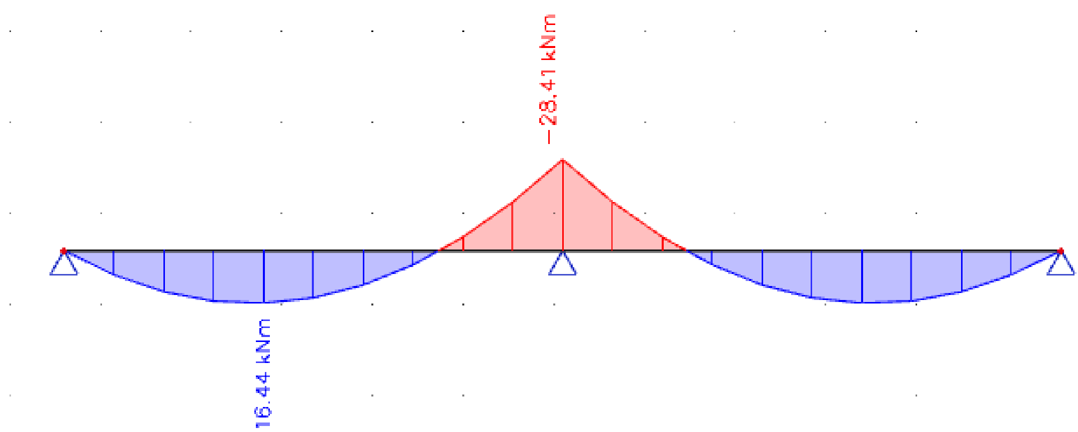
V_{Ed} = 34.24 kN

$$\tau_{v,d} = \frac{3}{2} * \frac{V_{Ed}}{A_{eff}} = (\frac{3}{2} * \frac{34.24 * 1000}{0.67 * 70000}) = 1.095 \text{ Mpa} < 1.615 \text{ Mpa}$$

$\tau_{v,d} < f_{v,d}$

Condition Fulfilled.

Max negative Moment



Picture 8.5 Bending Moment Diagram

M_{Ed} = -37.09 kN.M

$$\sigma_{m,y,d} = \frac{M_{Ed}}{W_y} = (\frac{37.09*10^6}{40.83*10^5}) = 9.08 \text{ Mpa} < 16.5$$

$\sigma_{m,y,d} < f_{m,d}$

Condition Fulfilled.

SLS Deformation

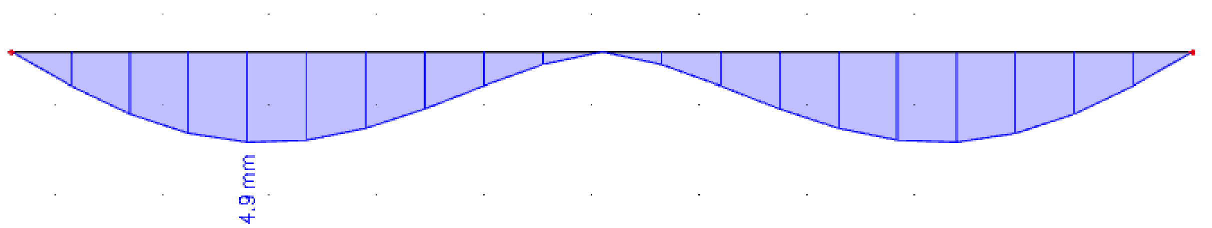
δ_{inst} = 0.9 mm

δ_{fin} = 4.1 mm

impact deflection

δ_{inst} = l/300 = 5500/250 = 22 mm

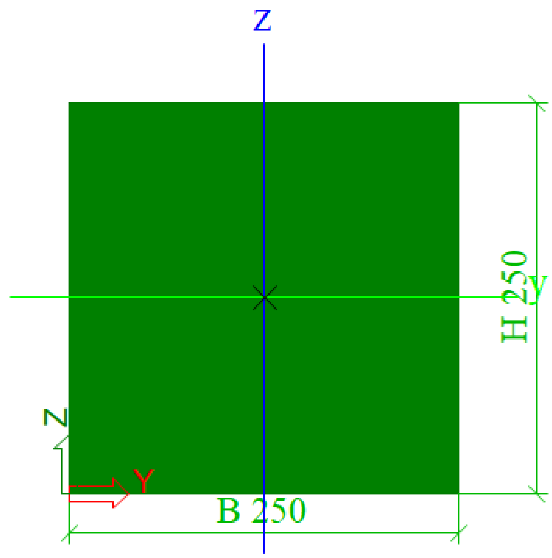
Deflection 5 mm



Picture 8.6 Beam Deflections

9. Column Supporting critical Girder C1

Load on the column N_{ED} = 123 kN



Picture9.1 Column Cross Section

Cross-section properties

A = 250 * 250 = 62500 mm^2

I_y = 1/12 * 250 * 250^3 = 32.55 * 10^7 mm^4

Iy = $\sqrt{(Iy/A)}$ = 72 mm

Material properties (C30)

$f_{c,0,k}$ = 23 Mpa

$f_{m,k}$ = 30 Mpa

$f_{v,k}$ = 3 Mpa

E0.05 = 7400 Mpa

k_{mod} = 0.7 (service class 1, long term load).

Design Strength Value

in compression along the grains

$f_{c,0,d}$ = 14.15Mpa

$f_{v,d}$ = 2.46Mpa

Slenderness

$\lambda_y = l_{cr}/i_y = 2600/72 = 36.11$

(higher slenderness to z-axis)

$\lambda_{rel} = \lambda_{y/}\,\pi * (\sqrt{\frac{f_{c,0,k}}{E_{0,05}}}) = 0.64$

Buckling coefficient

$k_z = 0.5 * \{1+\beta_c(\lambda_{rel} - 0.3) + \lambda^2_{rel,z}\} = 0.5\ (1 + 0.2(0.64 - 0.3) + 0.64^2)$

$k_z = 0.7388$

$k_{c,z} = 0.89$

$k_{c,z} = \frac{1}{k_z + \sqrt{k^2{}_z - \lambda_z{}^2}} = \frac{1}{2.34 + \sqrt{2.34^2 - 1.84^2}} = 0.89$

$\sigma_{c,0,d} = N_{Ed}/A = 139 * 1000 / 62500 = 2.24 Mpa$

$\sigma_{c,0,d} / k_{c,z} * f_{c,0,d} < 1$

2.224/ 12.60 < 1

0.09 < 1

Column for Buckling is satisfied.

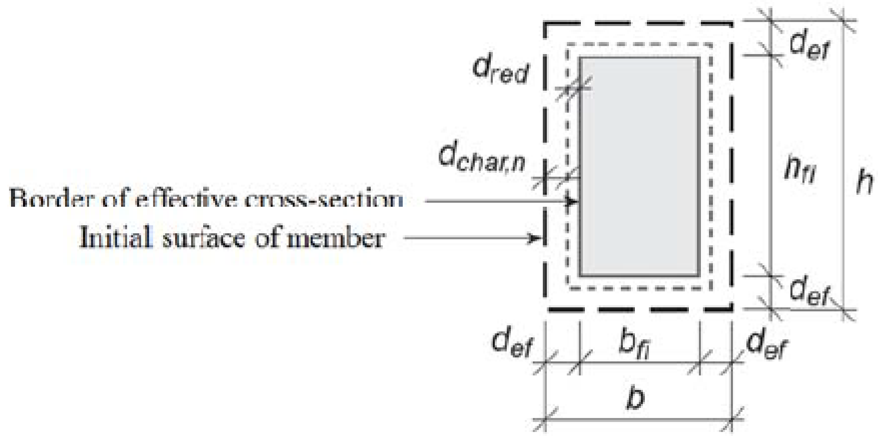
Fire Resistance Design

1- Unprotected for t_{req} = 30 mins

Notional Charring rate β_n = 0.8 mm/min (Solid Timber)

$k_{fi} = 1.25$

Calculation of Section Modulus (4-Sides)



b_{fi} = 250 – 2(30 * 0.8 + 7) =
188 mm

h_{fi} = 250 – 2(30 * 0.8 + 7) = 188 mm

A_{fi} = 188 * 188 = 35344 mm^2

$I_{z,fi}$ = 1/12 b_{fi} * h_{fi}^3 = 1/12 * 188* 188^3 = 10.4 * 10^7 mm^4

$i_{z,fi} = \sqrt{(\frac{I_{yfi}}{A_{fi}})}$ = 54.24

$\lambda_{z,fi} = l_{cr}/i_{z,fi}$ = 2600/54.24 = 47.93

$\lambda_{rel,fi} = \frac{\lambda_z,fi}{\pi} * \sqrt{\frac{f_{c,0,k}}{0.67 * E_{mean}}}$ = 1.04

$k_{c,fi}$ = 0.6

$f_{c,0,d,fi}$ = k_{fi} * $f_{c,0,fi}$ * $k_{c,fi}$

=1.25 * 23 * 0.6 = 15.87 Mpa

$\sigma_{d,fi}$ = 139*1000 / 35344 = 3.93 Mpa

$\sigma_{d,fi} < f_{c,0,d,fi}$

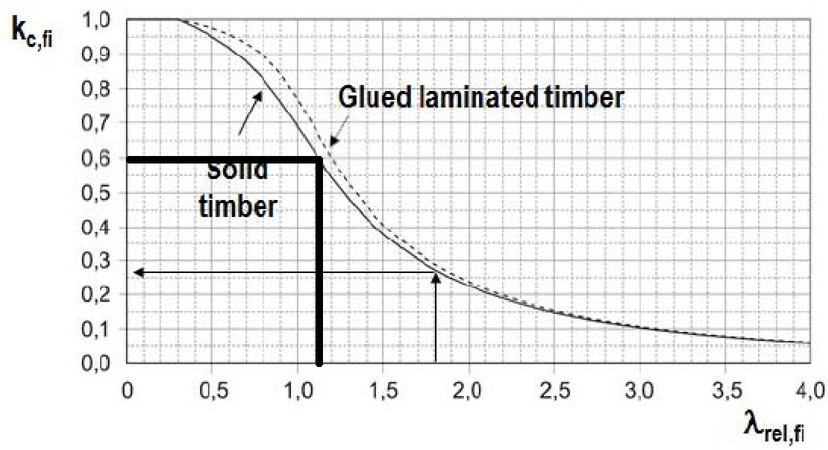


fig Buckling Curve

2- Protected element t_{req} = 60 mins

In order to increase the fire resistance, the timber column is protected by gypsum plasterboards

Protection with gypsum plasterboard type A according to EN 520, single layer, 18 mm thick.

$t_{ch} = 2.8h_p - 14 = 36$

$t_a = 36 + \frac{25}{2\,\beta_n} = 36 + \frac{25}{2*0.8} = 51$

b_{fi} = 250 – 2 (25 + (60-51) * 0.8 + 7) = 182 mm

h_{fi} = 250 – 2 (25 + (60-51) * 0.8 + 7) = 182 mm

A_{fi} = 182.4 * 182.4 = 33269 mm^2

$I_{z,fi}$ = 1/12 b_{fi} * h_{fi}^3 = 1/12 * 182* 182^3 = 9.14 * 10^7 mm^4

$i_{z,fi} = \sqrt{(\frac{I_{yfi}}{A_{fi}})}$ = 52. 4

$\lambda_{z,fi} = l_{cr}/i_{z,fi} = 2600/52.4 = 49.60$

$\lambda_{rel,fi} = \frac{\lambda_z,fi}{\pi} * \sqrt{\frac{f_{c,0,k}}{0.67 * E_{mean}}} = 0.84$

$k_{c,fi} = 0.8$

$f_{c,0,d,fi} = k_{fi} * f_{c,0,fi} * k_{c,fi}$

$= 1.25 * 23 * 0.8 = 23$ Mpa

$\sigma_{d,fi} = 139*1000 / 35344 = 3.93$ Mpa

$\sigma_{d,fi} < f_{c,0,d,fi}$

The Column is fire resistance for both cases

10.Connections

1- Rafter to Rafter

(Nailed Connection d ≤ 8 mm)

d = 8 mm

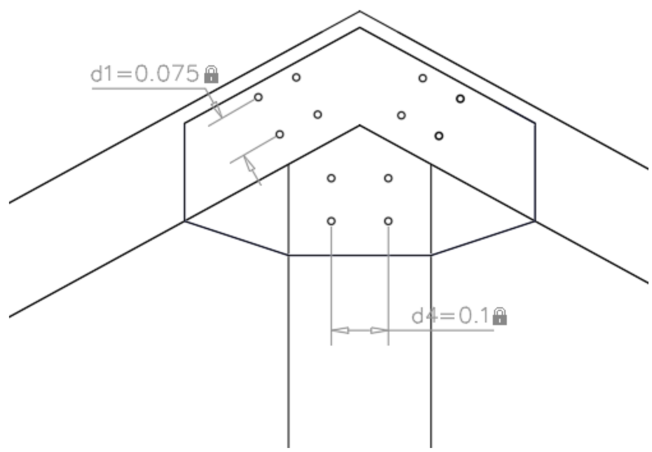
$F_{v,Rk} = \sqrt{2 \,.\, M_{y,Rk} \,.\, f_{h,1,k} \,.\, d}$

Predrilled holes

$t_{req} = 9d$

$f_{h,k} = 0.082 * (1 - 0.01d)\, \rho_k$

$f_{h,k} = 26.4\ Mpa$



Pic 10.1 Rafter to rafter connection

$M_{y,Rk} = 0.3\, f_{u,k}\ d^{2.6}$

$M_{y,Rk} = 0.3 * 600 * 8^{2.6} * = 40144\ N.mm$

$F_{v,Rk} = 4118\ N$

$F_{v,Rd} = \frac{k_{mod} \,.\, F_{v.Rk}}{\gamma_M} = 0.6 * 4118/1.1 = 2246.2\ N$

Tension at the connection = 3300 N
Use 2 dowels 8 mm in the direction of the grain at spacing 75 mm

2- Girder to Column connection G1,C1

(Nailed Connection d ≥ 8 mm)

Connection between the column and beam is made through steel sheet, it's a double shear joint.

Material (C30) solid wood

Steel sheet S275

k_{mod} = 0.8 (medium term, service class 1)

γ_M = 1.3

γ_M = 1.15 safety factor of steel

$f_{y,k}$ = 275 Mpa

$f_{y,d} = \frac{f_{y,k}}{\gamma_M} = \frac{275}{1.15}$ = 239.13 Mpa

Bolt class 3.6

Bolt strength $f_{u,k}$ = 300 Mpa

Bolt diameter d = 16 mm

Plate thickness t_0 = 8 mm

Depth of bolt into the wood t = 106 mm

Hole diameter d_0 = 18 mm

Number of bolts in row n = 2

Wood density ρ_k = 380 kg/m^3

$M_{y,Rk} = 0.3\ f_{u,k}\ d^{2.6w}$

$M_{y,Rk} = 0.3 * 300 * 16^{2.6} * = 121605.84$ N.mm

$F_{h,0,k} = 0.082 * (1 - 0.01 * d)\ \rho_k$

= 0.082 * (1 – 0.01*16) * 380 =26.17 Mpa

$$F_{v,r,k} = \min \begin{Bmatrix} f_{h0k},* t_1,* d \\ f_{h0k},* t_1,* d \left[\sqrt{2 + \frac{4\, M_{y,Rk}}{f_{h,0,k} * d * {t_1}^2}}\right] + \frac{F_{ax,Rk}}{4} \\ 2.3 * \sqrt{M_{yRk},* f_{h0k},* d} + \frac{F_{ax,Rk}}{4} \end{Bmatrix}$$

$$F_{y,r,k} = \min \begin{Bmatrix} 44384.32 \\ 19986.57 \\ 16412.15 \end{Bmatrix} = 16412.15 \text{ N}$$

$$F_{v,Rd} = 2 * n * \frac{k_{mod} * F_{v,Rk}}{1.3} = 2*2*\frac{0.8*16412.15}{1.3} = 40398.76 \text{ N}$$

V_{max} = 34240 N

34240 < 40398.76

$V < F_{v,Rd}$

Condition Fulfilled

Assessment of the steel sheet

Thickness of the sheet t = 8 mm

Height h = 200

Area of the steel sheet = 8 * 200 = 1600 mm^2

Height of the weakened cross section h_{osl} = h – (n * d_0) = 200 – (2*18) = 164 mm

Area of the weakened cross section A_{osl} = 8 * 164 = 1312 mm^2

Shear stress $\tau_{v,d} = \frac{\sqrt{3} * V_{Ed}}{A_{osl}} = \frac{\sqrt{3}*34240}{1312}$ = 45.20 Mpa

45.20 < 239.13

$\tau_{v,d} < f_{y,d}$

Condition fulfilled

N_{Ed} = 1.88 kN

$\sigma_{c,d} = \frac{\sigma_{cd}}{A_{osl}} = \frac{1.88*10000}{1312}$ = 1.43 Mpa

1.43 < 239.13

Condition fulfilled

Spacing Assessment

Angle to the grains α =0 α_2 = 4d = 64 mm

Loaded end α =0 $\alpha_3 = \max\{7d, 80\}$ = 112 mm

Loaded edge α =180 $\alpha_4 = \max\{(2 + 2\sin\alpha)d, 3d\}$

α_4 = 48 mm

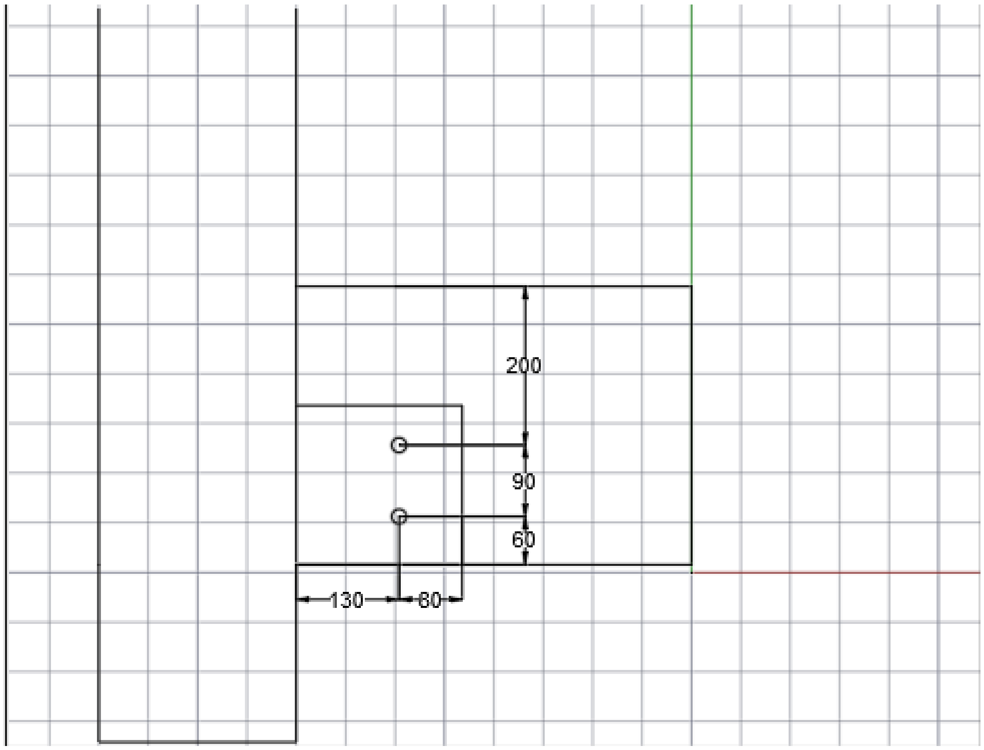
Unloaded edge α_4 = 3d = 48 mm

α_2 = 90 > 64 mm Condition fulfilled

$\alpha_{3,t}$ = 130 > 112 mm Condition fulfilled

$\alpha_{4,t}$ = 60 > 48 mm Condition fulfilled

$\alpha_{4,c}$ = 60 > 48 mm Condition fulfilled



Pic 10.2 Beam to Column Connection

11.Conclusion

In this diploma thesis, a two-storey house + roof made of wood materials was designed according to the assignment.

The main supporting structure; columns and girders, was designed from solid timber C30 size 250x250 mm for columns and 200x350 for girders.

C16 class solid wood was used for the design of floor joists and C24 for rafters.

Steel sheets were used to connect and transfer forces between individual load-bearing structures. The work also included an assessment of the fire resistance of the load-bearing structure.

It is recommended to use composite material (steel-timber) for the long spans.

For fire resistance 30 minutes complied. Calculation found that all unprotected elements used in the building are enough to carry the load bearing.

SCIA Engineer was used to calculate internal forces. The model was then loaded with everything constant and variable loads. Permanent loads include self-weight of load-bearing elements, self - weight of ceiling and floor constructions, roof construction.

The variable loads include roof structures, snow and wind.

12.Tables

All the tables and figures are from Eurocode 5.1 and 5.2

Table 2.2 – Examples of load-duration assignment

Load-duration class	Examples of loading
Permanent	self-weight
Long-term	storage
Medium-term	imposed floor load, snow
Short-term	snow, wind
Instantaneous	wind, accidental load

Service Class	Average moisture content u_m	Environmental conditions
1	$u \leq 12\%$	20°C und 65% rel. humidity
2	$u \leq 20\%$	20°C und 85% rel. humidity
3	$u > 20\%$	Higher humidity compared to SC 2

Table 2.3 – Recommended partial factors γ_M for material properties and resistances

Fundamental combinations:	
Solid timber	1,3
Glued laminated timber	1,25
LVL, plywood, OSB,	1,2
Particleboards	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections	1,3
Punched metal plate fasteners	1,25
Accidental combinations	1,0

Table 3.1 – Values of k_{mod}

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90

Table 6.1 – Effective length as a ratio of the span

Beam type	Loading type	ℓ_{ef}/ℓ [a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1,0 0,9 0,8
Cantilever	Uniformly distributed load Concentrated force at the free end	0,5 0,8

[a] The ratio between the effective length ℓ_{ef} and the span ℓ is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by $2h$ and may be decreased by $0,5h$ for a load at the tension edge of the beam.

Table 2.1 — Values of k_{fi}

	k_{fi}
Solid timber	1,25
Glued-laminated timber	1,15
Wood-based panels	1,15
LVL	1,1
Connections with fasteners in shear with side members of wood and wood-based panels	1,15
Connections with fasteners in shear with side members of steel	1,05
Connections with axially loaded fasteners	1,05

Table 6.1 Effective length as a ratio of the span

Beam type	**Loading type**	ℓ_{ef}/ℓ [a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1,0 0,9 0,8
Cantilever	Uniformly distributed load Concentrated force at the free end	0,5 0,8

[a] The ratio between the effective length ℓ_{ef} and the span ℓ is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by $2h$ and may be decreased by $0,5h$ for a load at the tension edge of the beam.

Table 7.2 Examples of limiting values for deflections of beams

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	$\ell/300$ to $\ell/500$	$\ell/250$ to $\ell/350$	$\ell/150$ to $\ell/300$
Cantilevering beams	$\ell/150$ to $\ell/250$	$\ell/125$ to $\ell/175$	$\ell/75$ to $\ell/150$

Table 4.1 — Determination of k_0 for unprotected surfaces with t in minutes (see figure 4.2a)

	k_0
$t < 20$ minutes	$t/20$
$t \geq 20$ minutes	1,0

Web Pages Sources of Pictures

[1] Modern Log House http://colsonagency.com/listing/modern-log-cabin-with-shop-and-land/

[2] Mosbach House https://en.wikipedia.org/wiki/Mosbach#/media/File:Mosbach_kickelhain.jpg

[3] Contemporary House https://nachhaltigwirtschaften.at/en/hdz/projects/timber-passive-house-at-muehlweg-1210-vienna.php

[4] https://www.pinterest.com/pin/213639576042887304/

[5] https://www.pinterest.com/pin/215898462211145756/

[6] Steinhaus https://www.nemetschek.com/en/reference-brands/largest-timber-building-in-switzerland

[7] https://www.designbuild-network.com/projects/e3-leaf/

[8] Hoho Wien https://smartcity.wien.gv.at/site/en/hoho-vienna/

[9] Snow load map https://www.haly-polak.cz/en/basic-parameters-333.html

[10] Wind Load Maphttps://www.haly-polak.cz/en/basic-parameters-333.html#gallery-2

References

EN 1990:2002 Eurocode – Basis of structural design

EN 1991-1-1:2002 Eurocode 1: Actions on structures – Part 1-2: General actions – Densities, self-weight and imposed loads

EN 1991-1-3 Eurocode 1: Actions on structures – Part 1-3: General actions – Snow loads

EN 1991-1-4 Eurocode 1: Actions on structures – Part 1-4: General actions – Wind loads

EN 1991-1-5 Eurocode 1: Actions on structures – Part 1-5: General actions – Thermal actions

EN 1991-1-6 Eurocode 1: Actions on structures – Part 1-6: General actions – Actions during execution

EN 1991-1-7 Eurocode 1: Actions on structures – Part 1-7: General actions – Accidental actions due to impact and explosions

EN 10147:2000 Specification for continuously hot-dip zinc coated structural steel sheet and strip – Technical delivery conditions

EN 13271:2001 Timber fasteners – Characteristic load-carrying capacities and slip moduli for connector joints

EN 13986 Wood-based panels for use in construction – Characteristics, evaluation of conformity and marking

EN 14080 Timber structures – Glued laminated timber – Requirements

EN 14081-1 Timber structures – Strength graded structural timber with rectangular cross-section – Part 1, General requirements

EN 14250 Timber structures. Production requirements for fabricated trusses using punched metal plate fasteners

EN 14279 Laminated veneer lumber (LVL) – Specifications, definitions, classification and requirements EN 14358 Timber structures – Fasteners and wood-based products – Calculation of characteristic 5-percentile value and acceptance criteria for a sample

EN 14374 Timber structures – Structural laminated veneer lumber – Requirements EN 14544 Strength graded structural timber with round cross-section – Requirements

EN 14545 Timber structures – Connectors – Requirements

EN 14592 Timber structures – Fasteners – Requirements

EN 26891:1991 Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics

Literature

Porteous, J., Kermani, A structural Timber Design to Eurocode, 2013, John Wiley and Sons Ltd, ISBN 978-0-470-67500-7

Larsen, H., Enjily, V., Practical Design of Timber Structures to Eurocode 5, UCE Publishing, 2009, ISBN-10: 0727736940, ISBN-13:978-0.727736093