Design of an office building with two variants of the roof structure

BRUNO ALEXANDRE ALVES GONÇALVES

Thesis submitted for partial fulfillment of the degree requirements

MASTER IN CIVIL ENGINEERING — SPECIALIZATION IN STRUCTURES DESIGN

Supervisors:

Dr. Łukasz Sadowski (Wrocław University of Technology)
Dr. Rui Manuel Meneses Carneiro de Barros (FEUP)

June, 2015

Department of Civil Engineering (Porto)

Tel. +351-22-508 1901
Fax +351-22-508 1446
Rua Dr. Roberto Frias
4200-465 PORTO
http://www.fe.up.pt
✉ feup@fe.up.pt
miec@fe.up.pt

Department of Civil Engineering (Wrocław)

11 Grunwaldzki Sq.
50-377 Wrocław
Building C-7
Dean's Office: room 04, 205, 206
tel./fax +48 71 320 34 54
Dean's Office Manager: Maria Kucharska
room 502B
tel. +48 71 320 46 52
✉ wydz.bud@pwr.wroc.pl
www.wbliw.pwr.wroc.pl

To my organization, persistence and hard work

After climbing a very high mountain, we find that there are many other mountains to climb

Nelson Mandela

Acknowledgement

A special thanks for Dr. Łukasz Sadowski for the availability, support and interest shown over my period abroad and for accepting this challenge being my super-visor which gave me an enriching experience in various aspects.

I thank to Dr. Rui Barros by taking responsibility as advisor in my thesis under mobility program and for the material provided.

To Poland which gave me a fantastic life experience and well welcomed me.

A thank you to all my family, especially to my mother, father and sister for the support, patience and understanding shown in the difficult times.

To my friends, comrades in the struggle in good and bad moments.

To my country which formed me as an engineer and as a man.

For all the people that I didn't mention here that helped me making this very important step in my life.

Abstract

With the introduction of the Eurocodes civil engineering becomes more and more a globalized area, standardizing the design of structures and facilitating the mobility of civil engineers across the globe. Yet there are some differences in the way of conceiving projects from country to country due to the different systems that each university works with for example the construction in timber structures is rare in Portugal and very common in Poland. In Portuguese universities wood is not an exploited material justified by their little use in the country itself but still it is important to have a minimal knowledge of this material. During the academic course the student acquires a deep theoretical knowledge in various subjects and it is of utmost importance to see the connection between theory and practice in order to understand every step of a project.

This project has the objective to demonstrate a part of designing timber structures according to Eurocode 5 and to understand how the different types of wood react to different actions during its life time, to demonstrate the usually way to design (drawings) family houses in Poland and to demonstrate the whole picture of a project´s design for a single-family house linking the knowledge acquired during the academic route.

An extension of an office building with two different variants of roof structure is designed in this project, one horizontal and another vertical. Are calculated Teriva floors constituted by pre-fabricated elements. Reinforcement concrete beams are calculated in detail demonstrating all the steps for ultimate limit states and serviceability limit states according to Eurocode 2. The roof structure in timber is also designed, according to Eurocode 5, and also the study of insulation and of condensations. The dissertation ends with a brief conclusion comparing the two variants in terms of wood mass used in each one.

CONTENTS

1 INTRODUCTION 22
1.1. GENERAL CONSIDERATIONS 22
1.2. OBJECTIVES 22
1.3. DESCRIPTION 23

2 PREFABRICATED TERIVA BEAMS (BOTH VARIANTS) 28
2.1. MAIN ASPECTS 28
2.2. STRUCTURAL ANALYSIS – ULTIMATE LIMIT STATES 30
2.2.1. **LOADS** 30
2.2.2. **DIMENSION SELECTION** 31

3 REINFORCED CONCRETE BEAM (EUROCODE 2) 34
3.1. DATA 34
3.2. STRUCTURAL ANALYSIS – ULTIMATE LIMIT STATES 35
3.2.1. **LOADS** 36
3.2.2. **DIAGRAMS** 37
3.2.3. **BENDING REINFORCEMENT IN THE BEAMS** 38
3.2.4. **SHEAR REINFORCEMENT IN THE BEAMS** 40
3.2.5. **DESIGN ANCHORAGE LENGTH IN TENSION FOR THE BEAM** 43
3.3. STRUCTURAL ANALYSIS – SERVICEABILITY LIMIT STATES 45
3.3.1. **LOADS** 45
3.3.2. **DIAGRAMS** 48
3.3.3. **CRACKING IN THE BEAM** 51
3.3.4. **MINIMUM REINFORCEMENT AREAS** 53
3.3.5. **DEFLECTION** 54

4 EUROCODE 5 (EC5) – PART 1-1 REVIEW 57
4.1. PHYSICAL AND MOLECULAR CHARACTERISTICS OF TIMBER 57
4.2. FIRE RESISTANCE 59
4.3. FIRE RESISTANCE 59
4.4. MECHANICAL CHARACTERISTICS 59
4.4.1. AXIAL FORCE 60

4.4.2. Bending 60
4.4.3. Shear 60
4.4.4. Fluency 60
4.5. Examples of Eurocode 5 Notation 61
4.6. Strength classes of timber 62
4.7. Design values of material properties 65
4.8. Load duration 66
4.9. Moisture Content and Temperature 68
4.10. Depth Factor – Solid Timber 69
4.11. Ultimate Limit States 70
4.11.1. Bending 73
4.11.2. Shear 75
4.11.3. Torsion 77
4.11.4. Complex stress states 79
4.11.5. Lateral Buckling 80
4.12. Serviceability limit states 80
4.12.1. Deflection 81
4.12.2. Vibrations 83
4.13. Connections 84
4.13.1. Metal dowel type connections 87
4.13.2. Plate based connections 88
4.13.3. Ring based connections, split-ring 89
4.13.4. Connection used in the present project 90
4.14. Glue Laminated Beams 97
4.14.1. Fabrication 98
4.14.2. Design 98

5 LOADS ACCORDING TO EUROCODE 1 (EC1) 102
5.1. Snow 102
5.2. Wind 104

6 STRUCTURAL ANALYSIS 107
6.1. First Variant 107
6.1.1. **Ultimate limit states** 107

6.1.2. SERVICEABILITY LIMIT STATES 113
6.1.3. CONNECTIONS 114
6.2. SECOND VARIANT 117
6.2.1. ULTIMATE LIMIT STATES 117
6.2.2. SERVICEABILITY LIMIT STATES 125
6.2.3. CONNECTIONS 125
6.3. INSULATION (BOTH VARIANTS) 128
6.4. WALL 129
6.5. ROOF 130
6.5.1. FIRST VARIANT 130
6.5.2. SECOND VARIANT 131
6.6. CONDENSATION 132

7 CONCLUSION 133

8 BIBLIOGRAPHY 134

9 BIBLIOGRAPHIC REFERENCES 136

10 ANNEXES 137

List of Figures

Figure 1.1 Original Architectural Plan, Author: Dr. Eng. Łukasz Sadowski

Figure 1.2 Posts directly to the ground [28]

Figure 1.3 Plinth wall [28]

Figure 1.4 Rafter Roof [28]

Figure 1.5 Truss Roof [28]

Figure 1.6 Aisled construction [28]

Figure 1.7 First variant, horizontal extension

Figure 1.8 Second variant, vertical extension

Figure 2.1 Teriva ceiling [1]

Figure 2.2 Teriva Beams, Project

Figure 2.3 Teriva Beams, Loads

Figure 2.4 Teriva Beams, Bending Moment

Figure 2.5 Teriva Ceiling, Project

Figure 2.6 Teriva Ceiling, Pillar Support, Project

Figure 3.1 Beam B-3, Project, First Variant

Figure 3.2 Beam B-3, Project, Second Variant

Figure 3.3 Teriva Beams, ULS Loads

Figure 3.4 Teriva Beams Bending, Moment

Figure 3.5 Beam, ULS Load, First Variant

Figure 3.6 Beam, ULS Load, Second Variant

Figure 3.7 Beam, ULS Bending Moment, First Variant

Figure 3.8 Beam, ULS Bending Moment, Second Variant

Figure 3.9 Beam, ULS Shear Force, First Variant

Figure 3.10 Beam, ULS Shear Force, Second Variant

Figure 3.11 Beam cross-section (M>O), Rectangular Stress Distribution, ULS [3]

Figure 3.12 Beam Cross-Section, Project

Figure 3.13 Beam Cross-Section, Project

Figure 3.14 Teriva Beams, Frequent Combination Bending Moment

Figure 3.15 Loads, Frequent Combination, First Variant

Figure 3.16 Loads, Frequent Combination, Second Variant

Figure 3.17 Bending, Frequent Combination, First Variant

Figure 3.18 Bending, Frequent Combination, Second Variant

Figure 3.19 Shear, Frequent Combination First Variant

Figure 3.20 Shear, Frequent Combination, Second Variant

Figure 3.21 Teriva Beams, Quasi Permanent Combination Shear Force

Figure 3.22 Quasi Permanent Combination, First Variant

Figure 3.23 Quasi Permanent Combination, First Variant

Figure 3.24 Bending, Quasi Permanent, First Variant

Figure 3.25 Bending, Quasi Permanent, Second Variant

Figure 3.26 Shear, Quasi Permanent First Variant

Figure 3.27 Shear, Quasi Permanent Second Variant

Figure 4.1 Tree trunk cross-section [16]

Figure 4.2 Examples of Eurocode5 Notation

Figure 4.3 Examples of Eurocode5 Notation

Figure 4.4 Examples of Eurocode5 Notation

Figure 4.5 Hardwood [6] [14]

Figure 4.6 Softwood [6] [13]

Figure 4.7 Timber beam, ULS [9]

Figure 4.8 Direction of Grain [7]

Figure 4.9 Limits Compression Widths [5]

Figure 4.10 Effective area of compression [5]

Figure 4.11 Compressive stresses at an angle [5]

Figure 4.12 $f_{c,a,d} / f_{c,0,d}$ & a 。

Figure 4.13 Stress in different axis [5]

Figure 4.14 Shear parallel to the grain [10]

Figure 4.15 Shear perpendiculars to the grain [10]

Figure 4.16 Variation of cross-section[5]

Figure 4.17 Torsion [5]

Figure 4.18 Components of deflection [18]

Figure 4.19 Variable Height [5]

Figure 4.20 Timber to timber connection

Figure 4.21 Timber to timber connection – nail plate

Figure 4.22 Timber to plywood connection

Figure 4.23 Timber to heavy gauge steel connection

Figure 4.24 Timber to dowelled fin

Figure 4.25 Half-lap joint

Figure 4.26 Frame joint

Figure 4.27 Tenon joint Figure

Figure 4.28 Cogging joint

Figure 4.29 Connection group 1

Figure 4.30 Connection group 2

Figure 4.31 Friction force [19]

Figure 4.33 Metal bolts cross-section

Figure 4.34 Metal bolts 3d view

Figure 4.35 Examples of plates [22]

Figure 4.36 Ring connection [23]

Figure 4.38 –Single shear connection

Figure 4.39 – Double shear connection

Figure 4.40 - Overlapping nails [5]

Figure 4.41 – (a) Nailing perpendicular to grain and (b) slant nailing [5]

Figure 4.42 Bridge in Sneek (Netherlands) [24]

Figure 4.43 Glulam timber cycle [25]

Figure 4.44 Horizontal and vertical glulam cross-section

Figure 4.45 Glued Laminate cross-section in same class and different classes [26]

Figure 4.46 Glulam timber cross-section [5]

Figure 5.47 Snow Map of Poland [4]

Figure 5.48 Exposure categories coefficients [4]

Figure 6.49 Horizontal Extension, First Variant, Project

Figure 6.50 Roof Battens, ULS Loads

Figure 6.51 Roof Battens, ULS Bending Moment

Figure 6.52 Loads ULS, First Variant

Figure 6.53 Diagram of Bending Moment ULS, First Variant

Figure 6.54 Diagram of Axial Force ULS, First Variant

Figure 6.55 Connection 1, First Variant

Figure 6.56 Connection 2, First Variant

Figure 6.57 Vertical Extension, Second Variant, Project

Figure 6.58 Roof Battens, ULS Bending Moment

Figure 6.59 Roof Battens, ULS Bending Moment

Figure 6.60 Loads ULS, Second Variant

Figure 6.61 Diagram of Bending Moment ULS, Second Variant

Figure 6.62 Diagram of Axial Force ULS, Second Variant

Figure 6.63 Connection 1, Second Variant

Figure 6.64 Connection 1, Second Variant

Figure 6.65 Insulation, External Wall Isolation, Project

Figure 6.66 Insulation, Roof Insulation, First Variant, Project

Figure 6.67 Insulation, Roof Insulation, Second Variant, Project

Figure 6.68 Psychometric diagram [27]

List of Tables

Table 2.1 Balance of loadings ULS, Teriva beams

Table 2.2 Maximum values of the bending moments per one rib in the roof Teriva [2]

Table 2.3 Teriva dimensions [2]

Table 3.1 Concrete Data

Table 3.2 Balance of loadings ULS, Teriva Beams

Table 3.2 Actions Categories [4]

Table 3.3 Balance of loadings, Frequent Combination, Teriva Beams

Table 3.4 Actions Categories [4]

Table 3.5 Balance of loadings, Frequent Combination, Teriva Beams

Table 3.6 Balance of loadings, Quasi Permanent Combination, Teriva beams

Table 3.7 Recommended values of $W_{\max}$ [3]

Table 3.8 Maximum bar diameters for crack control [3]

Table 3.9 Maximum bar spacing for crack control [3]

Table 3.10 *K* values [3]

Table 4.1 Strength Classes of Timber [7]

Table 4.2 Strength Types of Timber [18]

Table 4.3 Recommended cross-sections dimensions according to PN-EN 1313-1 [8]

Table 4.4 Recommended Partial Factors γ_M (Fundamental Combinations) [5]

Table 4.5 Values of K_{mod} [5]

Table 4.6 Values of K_{def} [5]

Table 4.7 Percentage of Moisture Content & Strength [5]

Table 4.8 Service Classes of Timber [5]

Table 4.9 Limits Compression Widths [5]

Table 4.10 k_{cr} values [5]

Table 4.11 Examples of permissible deflections for beams [5]

Table 4.12 Permissible deflections w_{fin} [5]

Table 4.13 Types of nails and screws [20]

Table 4.14 Spacing parallel to grain in a row and perpendicular to grain between rows [5]

Table 4.15 Edge and end distances [5]

Table 4.16 Angle between nails and edges/ends [5]

Table 4.17 Minimum spacing or end/edge distance [5]

Table 4.18 Properties of glulam [15]

Table 5.19 Recommended values of C_e for different topographies [4]

Table 5.20 Snow load shape coefficients [4]

Table 5.21 s_K values [4]

Table 6.22 Balance of loadings ULS, First variant of roof structure

Table 6.23 $\mathrm{K_{mod}}$ values [5]

Table 6.24 Load & K_{def}, u_{inst}, u_{fin} values [5]

Table 6.25 Balance of loadings ULS, Second variant of roof structure

Table 6.26 $\mathrm{K_{mod}}$ values [5]

Table 6.27 Load & K_{def}, u_{inst}, u_{fin} values [5]

Table 7.1 Total Wood Mass, First Variant

Table 7.2 Total Wood Mass, Second Variant

Symbols and abbreviations

A_s - Area of beam

b - Breadth of beam

h - Height of beam

L - Length of member

i - Radius of gyration

I - Second Moment of Area

W_y, W_z - Elastic modulus about y-y and z-z respectively

l – Span

M_d - Design moment

G - Permanent action

Q - Variable action

$\sigma_{m,d}$ - Design normal bending stress

$f_{m,k}$ - Characteristic bending strength

$f_{m,d}$ - Design bending strength

γ_G - Partial coefficient for permanent actions

γ_Q - Partial coefficient for variable actions

γ_M - Partial factor for material properties, modelling uncertainties and geometric variations

k_{mod} - Modification factor to strength values, allowing for load duration and moisture content

k_{sys} - Load sharing factor

k_{inst} - Instability factor for lateral buckling

$E_{0,05}$ - Fifth percentile value of modulus of elasticity

E_{mean} - Mean value of modulus of elasticity (parallel) to grain

u_{inst} - Instantaneous deformation

$u_{inst,G}$ - Instantaneous deformation due to a permanent action G

$u_{inst,Q,1}$ - Instantaneous deformation for the leading variable action Q1

u_{fin} - Final deformation

$u_{fin,G}$ - Final deformation due to a permanent action G

$u_{fin,Q,1}$ - Final deformation for the leading variable action Q1

k_{def} - Deformation factor

w_{creep} - Creep deflection

w_c - Camber deflection

w_{inst} - Instantaneous deflection

$w_{net,fin}$ - Net final deflection

w_{fin} - Final deflection

u_m - Bending deflection

u_v - Shear deflection

$F_{90,d}$ - Design bearing force

L - Length of bearing

$\sigma_{c,90,d}$ - Design compression stress perpendicular to grain

$f_{c,90,k}$ - Characteristic compression strength perpendicular to grain

L_{ef} - Effective length of column

λ_y, λ_z - Slenderness ratios about y–y and z–z axes

$\lambda_{rel,y}$, $\lambda_{rel,z}$ - Relative slenderness ratios about y–y and z–z axes

N - Design axial force

$\sigma_{c,0,d}$ - Design compression stress parallel to grain

$f_{c,0,k}$ - Characteristic compression strength parallel to grain

$f_{c,0,d}$ - Design compression strength parallel to grain

$\sigma_{m,y}$,$\sigma_{m,z,d}$ - Design bending stresses parallel to grain

$f_{m,y,d}$, $f_{m,z,d}$ - Design bending strengths parallel to grain

k_c - Compression factor

1
INTRODUCTION

1.1. GENERAL CONSIDERATIONS

The objective of this thesis is to design two different extensions of an existing house, a horizontal extension corresponding to the first variant (see figure 1.7) and a vertical extension corresponding to the second variant (see figure 1.8).

Both extensions are projected from an initial architectural plan provided by Dr. Łukasz Sadowski as shown in Figure 1.1. However, it was necessary to make some initial arrangements of the compartments and measures of the house in order to be considered as a family/office building (see design drawings at the end- annexes).

Once the plant is completed some decisions respectively to the materials that will be forming the house´s structure have to be made. A prefabricated Teriva ceiling, very common in Poland, is supporting the roof structure in the first variant and supporting the second floor in the second variant. This ceiling is formed by prefabricated beams that support several hollow blocks made of concrete that posteriorly are covered by a concrete layer in order to make a smooth surface. The Teriva ceiling is supported by a secondary beam also supported by a continuous wall formed of Ytong blocks. As for the roof, it is designed with timber elements making it possible explore timber structures according to Eurocode 5. Timber is also a very common material in Poland especially for roofs.

1.2. OBJECTIVES

The objectives of this thesis are to explore the main aspects of designing timber elements according to Eurocode 5 for ultimate limit states and serviceability limit states, to design in detail a reinforcement concrete beam according to Eurocode 2 in such way that all the aspects necessary to analyze any concrete structure are covered as for ultimate limit states and for serviceability limit states. To design a prefabricated Teriva ceiling (very common in Poland) using polish catalogs, to determinate the wind and snow actions on the structure according to Eurocode 1 taking into account the location (Warsaw, Poland) and to calculate the isolation with the polish limit values of transfer heating and the verification of condensation inside the house represent the main design tasks developed under this project. This thesis also has the objective to design the whole project of a family home, such as the foundations plants, ceilings plants, roof plants, cross-sections views, details of particular areas, quantities and types of steel and wood and description of the materials used to build the houses.

1.3. DESCRIPTION

Respectively to the ceiling (chapter 2), an initial consideration of live and dead loads is made according to Eurocode 1 and to the permanent loads of materials which form the ceiling. Once the necessary resistance values are calculated, the dimensions of the prefabricated elements are selected from a Polish catalog.

Due to the architectural plan, a reinforced concrete beam is designed in order to overcome a span of 4,30 m for both variants (see design drawings at the end). With this in chapter three, it is analyzed in detail according to Eurocode 2 the ultimate limit states (ULS) and serviceability limits states (SLS) for the reinforced concrete beams. To prevent the beam from collapse (ULS), it is calculated the steel necessary to resist to bending and shear force and the respective anchorage length in tension. To ensure a good behavior of the beam in service (SLS) it is controlled the cracking, deflection and minimum reinforcement in such way that unpleasant problems will appear during the life time of the structure. These analyze cover all the fundamental areas of concrete structures, according to Eurocode 2.

Related to the roof structure in timber, two different variants of roof are considered. In pre-1200 the building's structures gained rigidity by placing their timber post directly to the ground. However, over the years it was developed a new technique where the timber posts jointed to a timber beam supported by a plinth wall. This became known as "framed wall".

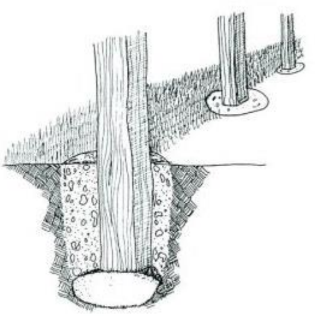


Figure 1.2 Posts directly to the ground [28]

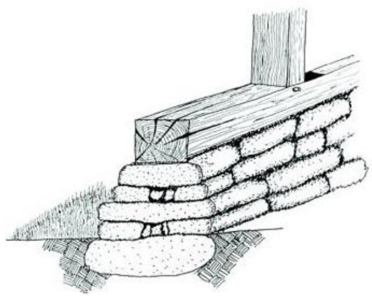


Figure 1.3 Plinth wall [28]

A "box frame" consists of framed walls, connected at bay intervals by cross tie beams designed to support the load of the rafter roof. Sometimes, instead of a rafter roof there is a truss roof where the elements are structurally united within each cross frame and purlins are used in this case. The truss roof allows for a better distribution of loads along the framed walls.

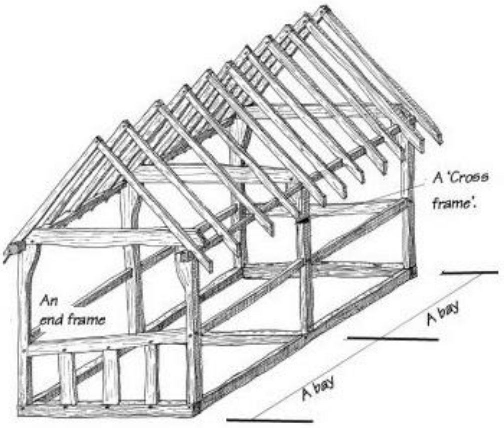


Figure 1.4 Rafter Roof [28]

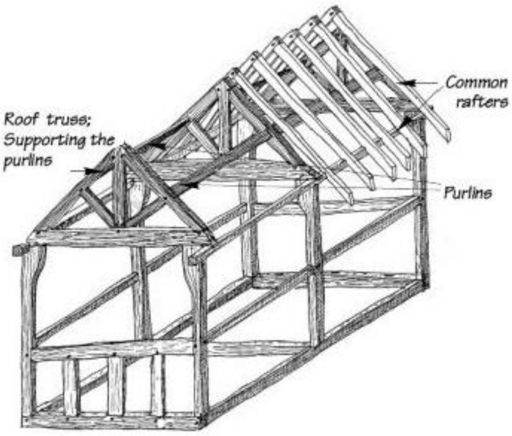


Figure 1.5 Truss Roof [28]

In order to provide a better utility of the space, the aisled constructions are very often used where the wall posts become internal and the aisles are roofed over with lean to roofs at an angle matching the main roof. In these situations, “window frames” and “door frames” were a part of the timber frame itself different from the way it is build nowadays.

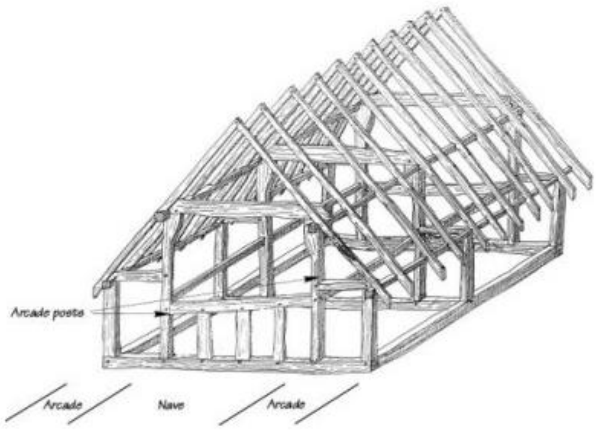


Figure 1.6 Aisled construction [28]

In this particular project the case which is studied is a rafter roof related for the first roof variant (horizontal extension) and the aisled constructions related to the second roof variant (vertical extension).

In chapter four it is explored the construction in timber elements. It is made the respective calculations for the roof in study and it is presented the description of the loads derived by wind and snow, according to Eurocode 1. Related to timber structures, an introduction about physical and molecular characteristics of wood is described in order to give a basic concept about types of timber, how timber elements react to fire, to bending, axial and shear force and to fluency. It is demonstrated some examples of notations according to Eurocode 5, some design values of the material properties and the different classes of timber strength depending on softwood or hardwood. Some important coefficients are explored as the K_{mod} and K_{def} coefficients related to the duration of the loads, the coefficient K_{temp} related to the moisture content of the wood and related to the temperature which affects the mechanical properties of the structure elements and the depth factor

K_h related to the volume of the elements (the larger the volume of a timber element the bigger it is its loss resistance). In order to prevent timber structures from collapse it is explored the ultimate limit states according to Eurocode 5 for torsion, bending, shear, bending plus axial forces simultaneously and lateral buckling. For serviceability limit states deflections and vibrations are analyzed. Different types of connections are shown as metal dowel type connections (screws and nails), plate based connections and ring based connections. A deeper look is made to connections by screws because these are the ones used in this particular project. It is demonstrated how to calculate its moment, axial, shear and shear plus axial resistance including the definition of the correct spaces and angles between edges/ends and screw’s spaces. An introduction for glue laminated timber elements it´s also demonstrated. It is presented how glulam timber is made and the basic concept to calculate these elements. In the last part of this chapter it is demonstrated all the calculations for the roof structure in each variant.

In chapter five it is calculated the isolation for the wall, roof and foundation in each variant and also it´s made the verification of condensation in order to prevent from serviceability problems.

The project culminates with a brief conclusion making the most affordable choice between the two roof structure variants.

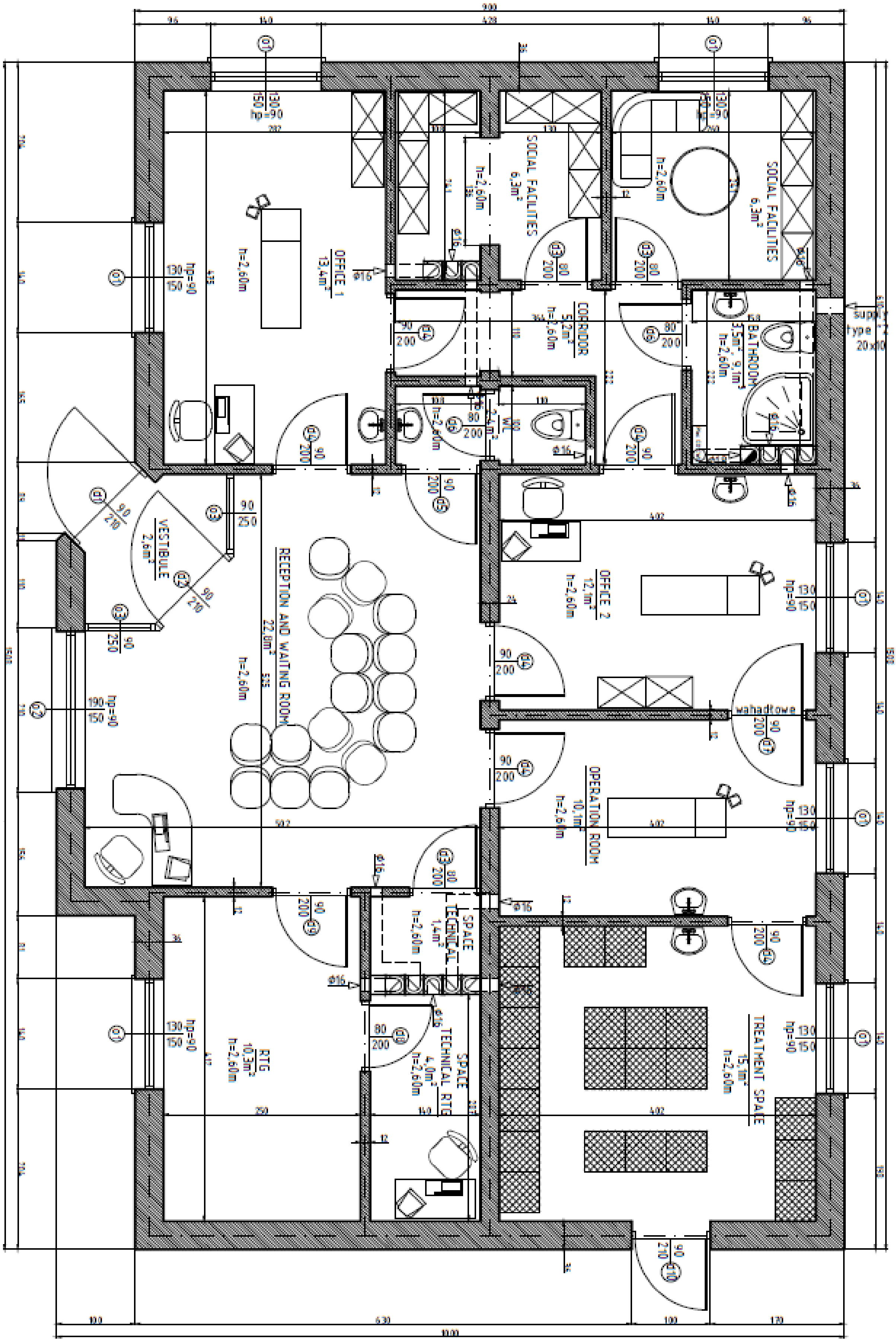


Figure 1.1 Original Architectural Plan, Author: Dr. Eng. Łukasz Sadowski

Figure 1.7 First variant, horizontal extension

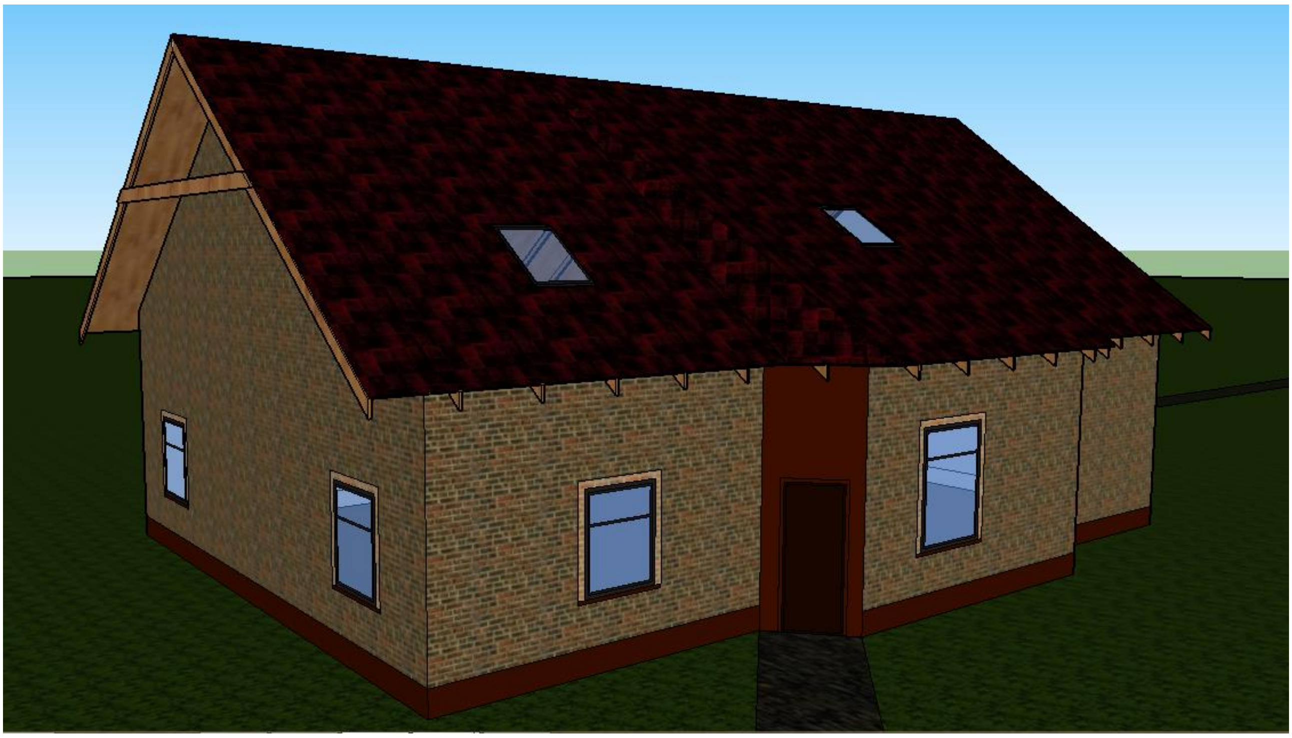


Figure 1.8 Second variant, vertical extension

2
PREFABRICATED TERIVA BEAMS (BOTH VARIANTS)

2.1. Main Aspects

Teriva ceilings are very common in Poland. They are constituted by hollow blocks supported by reinforced concrete beams-ribs. The beams-ribs are load bearing structural elements from the ceiling, and then to the wall.

These ceilings have capacity to support live loads around 4,0, 6,0 and 8,0 kN/m^2, depending of the dimensions of the blocks and beams, depending on the amount of steel used on the beams and of the span. The constructions of Teriva ceilings are performed without the need of heavy equipment which minimizes the cost of the building, with high efficiency of the works due to the prefabricated elements that are used. It doesn´t need all formwork (only in the support beams) what also it decreases the final cost and it is characterized for its good qualities related to thermal and acoustic insulators. On the other hand, Teriva ceilings are characterized by their own high thickness due to the pre-fabricated elements like the hollow blocks.

Figure 2.1 Teriva ceiling [1]

The beam-ribs that support the hollow blocks are supported in another continuous beam (made in the local) which lays in the structural wall. This support in the wall should have at least 8 cm to ensure the minimum effective contact area due to the stress created by the loads. In order to the stress to be distributed evenly to the wall it is recommended a prefabricated element called edge profiles which supports the continuous beam in the wall. All must be concreted together to ensure a healthy behavior of the ceiling. Before putting the concrete over the ceiling, all contamination must be removed and all the components should be cleaned with water, such as hollow blocks elements and joists. The ceiling concreting must be done in the direction of the beams and if the delivering is made by means of wheelbarrows it is necessary to place platforms laid out at the right angle to the beams in such way that all empty space must be full filled for a good concrete consolidation and it´s also important that during these task samples are taken, in order to the concrete quality. For the platform´s cross-section sizes it´s recommended at least 3,8 cm thickness and 20 cm of width and it´s also important to refer that the edges of the platform must be prevented with skirting boards for a good handling of the wheelbarrow.

During the construction process of the ceiling there should exist temporary supports (braces) until the concrete reaches its own characteristics as projected in such way that unpleasant deflections won´t appear. These supports normally are placed with 2 m distance (maximum) and must be under the beams and not under the blocks. It´s important to refer that for spans bigger than 6,4 m a reverse deflection of 15 mm is necessary to support its own weight.

It´s recommended to use reinforcement in the upper fibers of the cross-section to absorb tensions of temperature changes and avoid cracking. The reinforcement must be formed by a grid of steel A-IIIN welded in the connections and normally spacing 20 cm.

Relating to the project itself there are some aspects important to take into account. Each floor must be designed by the manufacturer and should be present in the project the drawings of the ceiling elements. Any change that may be made in the project must be informed and discussed with the constructer. Otherwise it´s a risk that can result in several problems during many years. Consequently the design must specify all the dimensions of the ceiling elements, such as the blocks and beams, and a special attention for the edges near the structural wall which depending on the distance of the last beam it´s possible to adopt various solutions. In this project, it was designed a extension of the supporting beam located along the structural wall in such way that makes it possible to fit all the elements one to each other with the correct designed dimensions. Another important aspect that must be present in the project is the way of making holes in the ceiling installations. This may be made in the blocks and not on the beams and if the holes are small then just the block itself is perforated and it won´t be necessary to pay any special attention but often the holes are spaced along the beams and in this case it´s necessary additional beams to support the ceiling in this area. The design also must specify the way of making the wall beams which support the ceiling. Here it's important to transfer the stress created by the loads in uniform way to the structural wall. The ceiling can´t be directly supported on the wall otherwise the top of the wall will break. A beam normally with 4 steel rods with 12 mm of diameter must be designed for this place.

2.2. STRUCTURAL ANALYSIS – ULTIMATE LIMIT STATES

In this project, there are different spans relating to the Teriva beams. The biggest span has 5,3 m like presented in the figure 2.2.

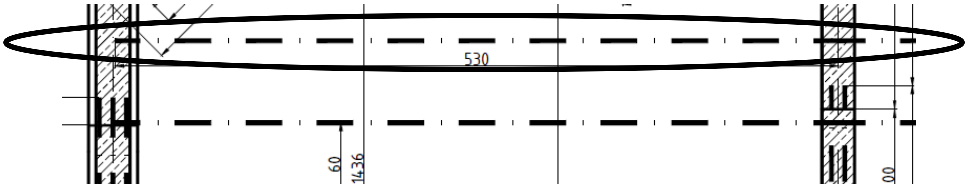


Figure 2.2 Teriva Beams, Project

2.2.1. LOADS

Below are presented the typical balance of permanent and live loadings

Table 2.1 Balance of loadings ULS, Teriva beams

Dead (permament loading)	**Calculation**	**gk**	**γGj, sup**	**gd $\left[\frac{KN}{m}\right]$**
Self-leveling cementious underlayment (3 cm)	γ_{slcu} [KN/m^3]* 0,03 m*0,6 m	0,308	1,35	0,51
Protective layer - PCV foli	0,02 kN/m^2*0,6 m	0,012	1,35	0,02
Isolation - mineral wool (35 cm)	γ_{pvc} * 0,35 m*0,6 m	0,005	1,35	0,006
Hollow Block $q_k = 0{,}16$ KN/unit	$q_k = 0{,}9$ KN/m	0,9	1,35	1,3
Reinforced concrete slab+beam (41,25 cm^2)	γ_c [KN/m^3]* 0.004125*0,6 m	0,072	1,35	0,084
Cementious plaster work (1,5 cm)	γ_{cpw} [kN/m^3] * 0,015 m*0,6 m	0,19	1,35	0,23
SUM		1,61		**2,31**
Live (variable loading)		**qk**	**γQ,1**	**qd**
		1,5	1,5	**2,25**

2.2.2. DIMENSION SELECTION

Below there are the permanent + variable load for the TERIVA beams in the project

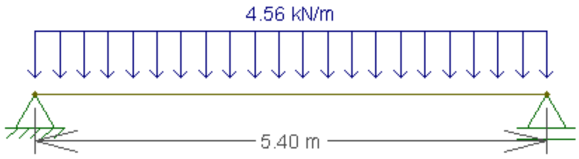


Figure 2.3 Teriva Beams, Loads

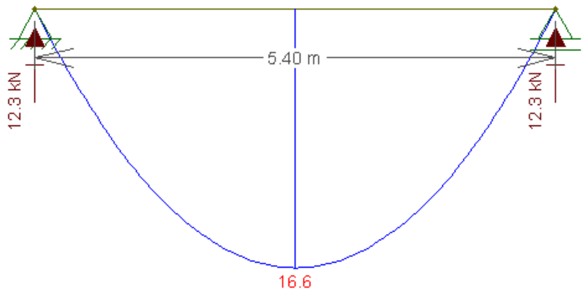


Figure 2.4 Teriva Beams, Bending Moment

$$M^{MAX} = 16.6 \text{ KNm}$$

$$V^{MAX} = 12.3 \text{ KN}$$

Once Teriva ceiling is constituted by prefabricated elements there is also a catalog with the bending moment resistance of each ceiling according to the span, size of elements and loading which must be consulted to select the correct one. On the table 2.2, it is presented a part of the catalog for Teriva 1 type and it´s possible to verify that it has the necessary resistance for a bending moment of 16.6 KNm with 5,4 m of span.

Table 2.2 Maximum values of the bending moments per one rib in the roof Teriva 1 [2]

Modular Span	Design Span	Moment of the Design Load [KNm]	Shear of the Design Load [KNm]
2,4	2,37	6,843	13
2,8	2,77	6,843	13
3,0	2,97	6,843	13
3,4	3,37	6,843	13
3,6	3,57	6,843	13
3,8	3,77	8,059	13
4,2	4,17	9,374	13
4,4	4,37	10,789	13
4,8	4,77	12,303	13
5,0	4,97	13,917	13
5,4	5,37	15,630	13
5,6	5,57	17,442	13
6,0	5,97	19,354	13

Once chosen the type of Teriva ceiling (Teriva 1) to the project, it follows the selection of dimensions of the ceiling like is presented in the table below

Table 2.3 Teriva 1 dimensions [2]

Ceiling type	Ceiling span [m]	Joists gauge [m]	Ceiling construction height [m]	Top concrete layer thickness [mm]	Ceiling construction weight [kN/m2]
TERIVA 4,0/1	2,4 ÷ 7,2 *	0,60	0,24	30	2,68
TERIVA 4,0/2	2,4 ÷ 8,0	0,60	0,30	40	3,15
TERIVA 4,0/3	2,4 ÷ 8,6	0,60	0,34	40	3,40
TERIVA 6,0	2,4 ÷ 7,8	0,45	0,34	40	4,00
TERIVA 8,0	2,4 ÷ 7,2	0,45	0,34	40	4,00

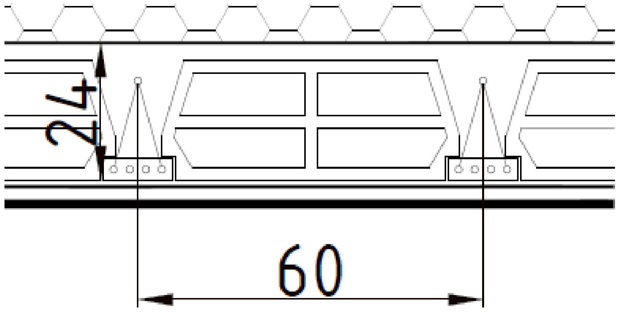


Figure 2.5 Teriva Ceiling, Project

Important Note: In order to give support to the timber pillar of the second variant, it is necessary to locate three Teriva beams under each pillar to give the proper support.

$$\mathrm{M}^{\mathrm{MAX}} = 50{,}2\ \mathrm{KNm} \rightarrow \mathrm{M}^{\mathrm{Resis.}} = 17{,}422 * 3 = 52\ \mathrm{KNm}$$

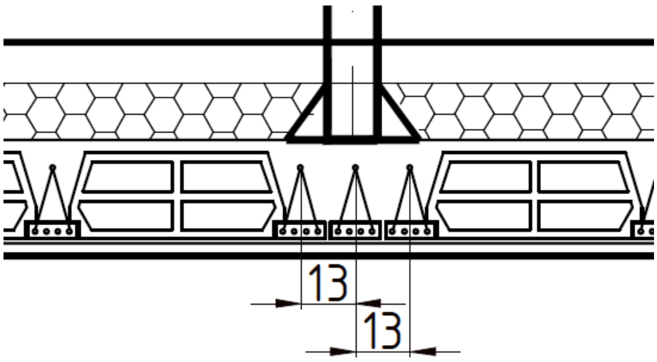


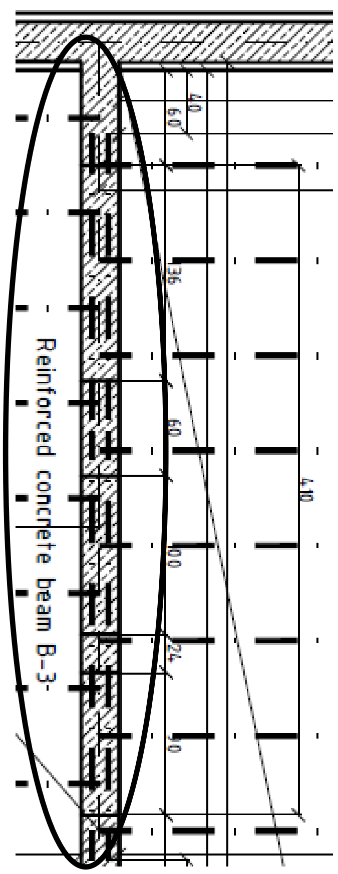
Figure 2.6 Teriva Ceiling, Pillar Support, Project

3
REINFORCED CONCRETE BEAM (EUROCODE 2)

Apart from the fact that in this project the concrete is just related for a simple supported beam, it is important to refer that in the past few years the construction in concrete structures has been developing and increasing. The quality and durability of the concrete have been explored in such way to predict unpleasant problems in the concrete structures. The causes of concrete problems can be several as excess of the loads comparing from the loads design in the project, errors of units and calculations, corrosion of the steel, thermal variations, biological attacks, incompatibility of the materials, among other reasons. The consequences of this errors may put in risk the own stability of the structure and sometimes driving to collapse overpassing the ultimate limit states and also may result some serviceability problems, such as cracking and deflections in the concrete. In order to avoid these problems a detailed design is made for the reinforced concrete beam in study.

3.1. DATA

Due to the architectural plan it is necessary to design a beam with 25 cm of width and 25 cm of height like presented on figure 3.1 (B-3). This reinforced concrete beam supports the Teriva beams.



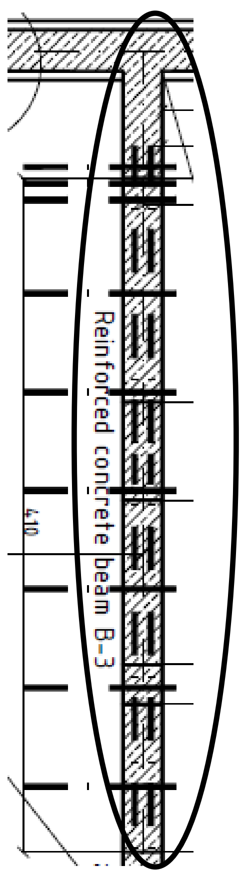


Figure 3.1 Beam B-3, Project, First Variant Figure 3.2 Beam B-3, Project, Second Variant

From Eurocode 2 and Eurocode 1, the following data was selected (see table 3.1).

Table 3.1 Concrete Data

$f_{ck,cube}$ [MPa]	f_{ctm} [MPa]	f_{yk} [MPa]	q_k [KN/m^2]	$\xi_{eff,lim}$ [AIIIN]	l [m]	h [m]	b [m]
25	2.2	500	1.5	0.5	4.2	0.25	0.25

For the ultimate limit states, according to Eurocode 2, it´s adopted 1,5 as a safety factor value for concrete and 1,15 for steel. Improper compaction, inappropriate curing, uncertainty in the properties of the ingredients when they are mixed are some of the reasons that explain the biggest safety factor of the concrete comparing with the safety factor of the steel. As for the steel, the uncertainties of variation in strength of the reinforcement are small so it´s not necessary such a large safety factor like for the concrete.

Applying the safety factors from Eurocode 2, the design values which will be taken into account are determinate:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 434.78\ \mathrm{MPa} \rightarrow f_{yd} = 434.78\ \mathrm{Mpa} \qquad \text{eq. (3.1)}$$

$$f_{ck,cube} = 25\mathrm{MPa} \rightarrow f_{ck} = 20\mathrm{MPa}$$

$$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c} = 1 * \frac{20}{1.5} = 13.33\mathrm{MPa} \rightarrow f_{cd} = 13.33\mathrm{Mpa} \qquad \text{eq. (3.2)}$$

3.2. STRUCTURAL ANALYSIS – ULTIMATE LIMIT STATES

Limit states is a state from which it is considered when the structure is no longer able to perform the duties that are assigned to. The calculations to ensure the safety of structures must be made based in certain limit states which stabilizes a limit value for the applying forces. Posteriorly the limit states must be compared with the actual forces applied on the structure in such way that the resistance stress won´t be exceeded. Apart from the fact that in this particular project it will not be analyzed all parts of the limit states, it is important to refer all the areas related to SLS as bending moment resistance (simply and composed), shear resistance, torsion resistance, puncture resistance, buckling, balance and fatigue.

3.2.1. LOADS

Eurocode 2 divides the loads in three groups as the permanent loads, live loads and accidental loads. The permanent loads have constant values or almost constant during the whole life of the structure as the own weight of structural elements (structural or not), fixed equipment weight, earth pressures, prestressing, retraction, creep of concrete, settlements of the supports among others. The live loads are variable during the structure´s life as overloads and their dynamic effects, wind, earthquake and temperature variations, among others. Finally, the accidental loads have a low probability of occurring during the life of the structure and the loads are such as explosions, fires, vehicle crashes among others.

Table 3.2 Balance of loadings ULS, Teriva Beams

Balance of loadings (typical) – Teriva Beams supported on the beam to be calculated (l=5,4m)				
Dead (permanent loading)	**Calculation**	**gk**	**γGj, sup**	**gd $\left[\frac{KN}{m}\right]$**
Self-leveling cementious underlayment (3 cm)	γ_{slcu} [KN/m^3]* 0,03 m*0,6 m	0,308	1,35	0,51
Protective layer - PCV foli	0,02 kN/m^2*0,6 m	0,012	1,35	0,02
Isolation - mineral wool (35 cm)	γ_{pvc} * 0,35 m*0,6 m	0,005	1,35	0,006
Hollow Block $q_k = 0{,}16$ KN/unit	$q_k = 0{,}9$ KN/m	0,9	1,35	1,3
Reinforced concrete slab+beam (41,25 cm^2)	γ_c [KN/m^3]* 0.004125*0,6 m	0,072	1,35	0,084
Cementious plaster work (1,5 cm)	γ_{cpw} [kN/m^3] * 0,015 m*0,6 m	0,19	1,35	0,23
SUM		1,61		**2,31**
Live (variable loading)		**qk**	**γQ,1**	**qd**
		1,5	1,5	**2,25**

3.2.2. DIAGRAMS

Below there are the permanent + variable load for TERIVA beams supported on the beam to be calculated

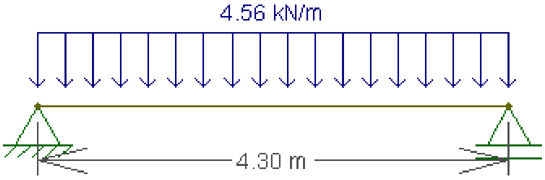


Figure 3.3 Teriva Beams, ULS Loads

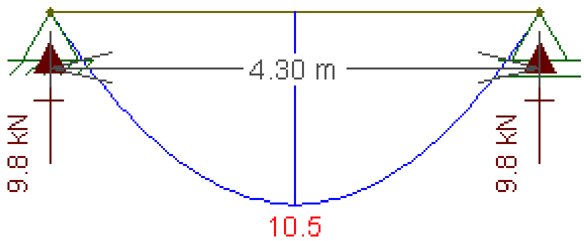


Figure 3.4 Teriva Beams Bending, Moment

$$M^{MAX} = 10.5 \text{ KNm}$$

$$R^{MAX} = 9{,}8 \text{ KN}$$

Below there are the loads for the beam (B-3) to be calculated.

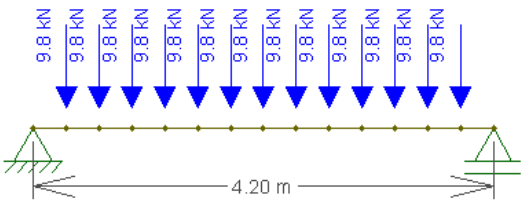


Figure 3.5 Beam B-3, ULS Load, First Variant

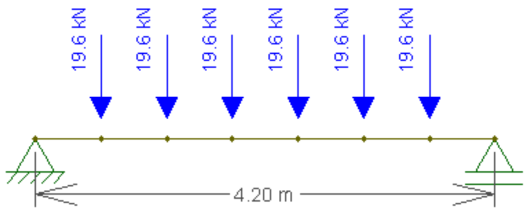


Figure 3.6 Beam B-3, ULS Load, Second Variant

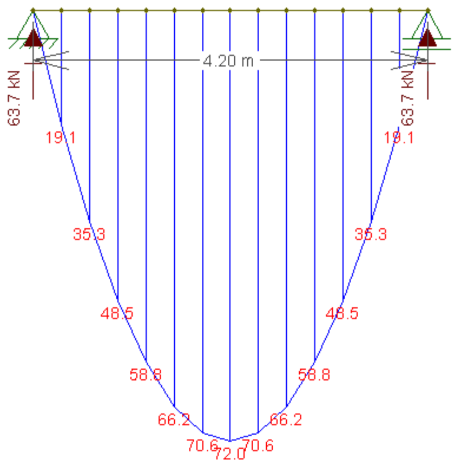


Figure 3.7 Beam B-3, ULS Bending Moment, First Variant

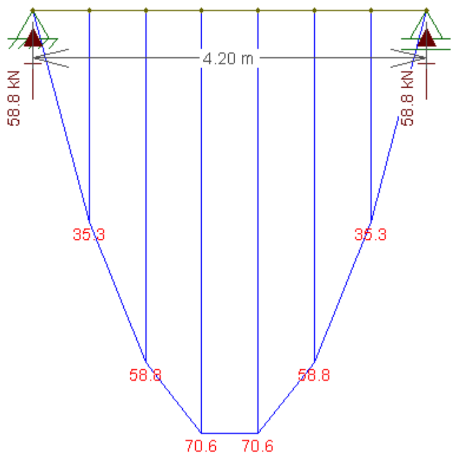


Figure 3.8 Beam B-3, ULS Bending Moment, Second Variant

$$M^{MAX} = 72.0 \text{ KNm}$$

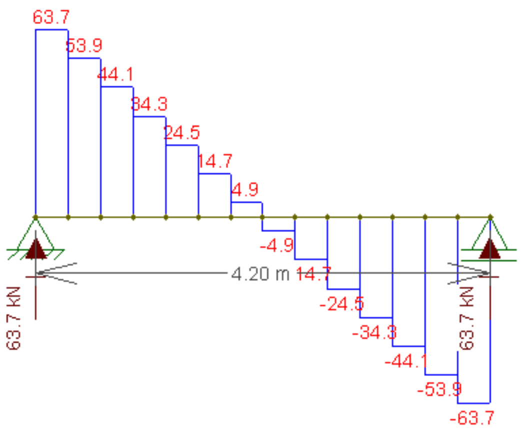


Figure 3.9 Beam B-3, ULS Shear Force, First Variant

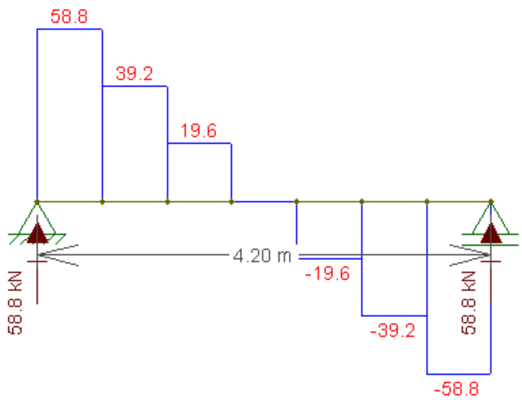


Figure 3.10 Beam B-3, ULS Shear Force, Second Variant

$$V^{MAX} = 63.7 \text{ KNm}$$

The span of the beam B-3 is 4,20 m due to the architectural plants. Like it is possible to verify on the drawings (annexes), in the plant is located a missing space to support the Teriva beams. The span of 4.20 m was chosen in such way to find two possible supports to be analysed as a simple supported beam.

3.2.3. BENDING_REINFORCEMENT IN THE BEAMS

The calculation of the bending moment resistance of the reinforced concrete sections is based in some principles such as the geometry of the sections that should remain flat before and after the load, the tension on the steel is the same extension that exist in the concrete surrounding it, the resistance of the concrete on the traction fibers is not taken into account due the weak behavior of the concrete when submitted to traction, the stress in the compressed area of the concrete cross-section is calculated using the stress-strain design diagram and the tension on the steel that reinforce the concrete is obtained also through the design diagrams.

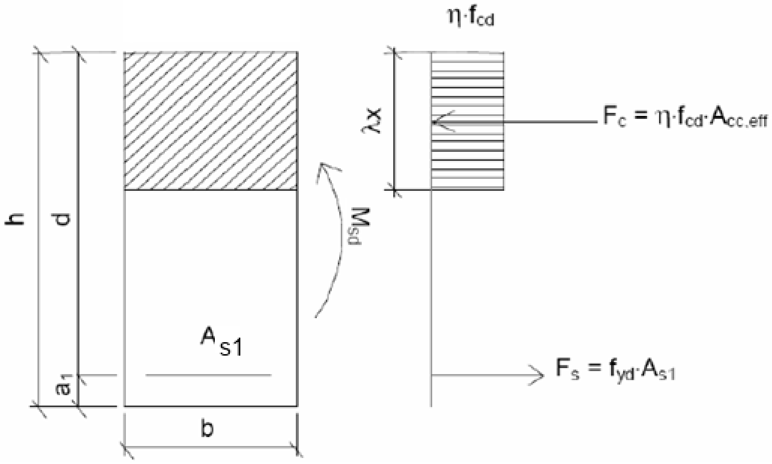


Figure 3.11 Beam cross-section (M>O), Rectangular Stress Distribution, ULS [3]

$$\mathrm{M^{MAX}} = 72\ \mathrm{KNm}$$

$$\mathrm{a_1} = 15\ \mathrm{mm}, \mathrm{b} = 250\ \mathrm{mm}, \mathrm{h} = 250\ \mathrm{mm}, \mathrm{d} = 235\ \mathrm{mm}$$

$$\mathrm{m_s} = \frac{\mathrm{M_{Sd}}}{\mathrm{f_{cd}\ b\ d^2}} \qquad \text{eq. (3.3)}$$

$$= \frac{72}{13.3 * 10^3 * 0.25 * 0.235^2} = 0.36671$$

$$\xi_{\mathrm{eff}} = 1 - \sqrt{1 - \frac{2}{\eta}\mathrm{m_s}} \qquad \text{eq. (3.4)}$$

$$= 1 - \sqrt{1 - \frac{2}{1}0.36671} = 0.4837$$

$$\xi_{\mathrm{eff}} = 0.4837 < \xi_{\mathrm{eff,lim}} = 0.5 \ \ (\mathrm{OK})$$

In case of collapse a ductile behavior of the beam B-3 is guaranteed.

As verified, no compression reinforcement is required. Proceeding to the determination of the necessary area of reinforcement in the concrete cross-section, it comes:

$$As_1 = \xi_{eff}\ \eta\ \frac{f_{cd}\ b\ d}{f_{yd}} \qquad \text{eq. (3.5)}$$

$$= 0.4837 * 1 * \frac{13.33 * 10^3 * 0.25 * 0.235}{434.78 * 10^3} = 8.8\ cm^2$$

Checking the minimum reinforcement

$$As_1 > \begin{cases} 0.26 * f_{ctm} * b_t * \dfrac{d}{f_{yk}} & \text{eq. (3.6)} \\ \quad = 0.26 * 2.2 * 10^3 * 0.25 * \dfrac{0.235}{500 * 10^3} = 0.67cm^2 & \\ 0.0013 * b_t * d & \text{eq. (3.7)} \\ \quad = 0.0013 * 0.25 * 0.235 = 0.76cm^2 & \end{cases}$$

In the cross section $As_{1required} = 8.8\ cm^2$. For this reason it is placed 5$\boldsymbol{\phi}$16 ($10.05\ cm^2$) for the beam (both variants) as shown on figure 3.12

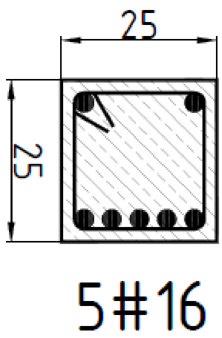


Figure 3.12 Beam Cross-Section, Project

3.2.4. SHEAR REINFORCEMENT IN THE BEAMS

In elements submitted to bending forces usually the calculation of the reinforcement is made in such way that the shear force resistance is not the first condition factor for collapse but where the bending resistance must be the condition factor. In other words, the collapse must occur by bending in first place and not by shear to ensure a ductile behavior from the concrete element. In this way it is possible to anticipate the collapse through the symptoms of a normal bending as the cracking in the concrete due to the high tension in the fibers and the deformations which are possible to see in naked eye.

Eurocode 2 considers three resistances of shear force which must be verified: resistance of the shear force without reinforcement, $V_{Rd,c}$, maximum value of the shear force that can be resisted without crush of fictitious concrete strut, Vrd,max, and the resistance of the shear force with reinforcement, $V_{Rd,s}$.

Sometimes the shear resistance of the own concrete, $V_{Rd,c}$, is enough to ensure the necessary resistance of the element. However, every concrete beam must have the minimum reinforcement for shear resistance, Vrd,cmin, to ensure the ductile behavior like described before.

In this project it is assumed $\theta = 45°$.and $\alpha = 90°$.

At first, it is necessary to check the shear load bearing capacity of the cross section related to concrete failure $V_{Rd,max} > V_{Ed,d}$ design shear force.

$$V_{Rd,max} = \alpha_{cw}\, b_w\, f_{cd}\, z\, \upsilon \frac{1}{ctg\theta + tg\theta} \qquad \text{eq. (3.8)}$$

$$\upsilon = 0.6\left(1 - \frac{f_{ck}}{250}\right), \qquad \alpha_{cw} = 1, \qquad z = 0.9\, d$$

Checking the self-resistance of the beam to the design shear force $V_{Rd,c} < V_{Ed,d}$.

$$V_{Rd,c} \;=\; \left[C_{Rd}, c \;*\; k\,(100\,\rho_l f_{ck})^{\frac{1}{3}} \;+\; 0.15\sigma_{cp}\right] bw\, d \qquad \text{eq. (3.9)}$$

$$\geq \left(0.0035\, k^{\frac{3}{2}}\; f_{ck}^{\frac{1}{2}} +\; 0.15\,\sigma_{cp}\right) b_w d$$

$$k \;= 1 + \sqrt{\frac{200}{d} <\; 2} \qquad C_{Rd} = \frac{0.18}{\gamma_c} = \frac{0.18}{1.5} \qquad \rho_l = \frac{A_{sl}(tensile)}{s\, d} < 0.02$$

The spacing between the stirrups is calculated in so that shear load bearing capacity of cross-section related to the transversal reinforcement $V_{Rd,s} = V_{Ed,d}$ design shear force.

$$V_{Rd,s} = \frac{A_{sw}}{s}\, z\, f_{ywd} ctg\theta \qquad \text{eq. (3.10)}$$

Furthermore is necessary to check that $\rho_{sw} > \rho_{sw,min}$

$$\rho_{sw} = \frac{A_{sw}}{s\ b_w}\ ,\quad \rho_{sw,min} = \frac{0.08\sqrt{f_{cd}}}{f_{yk}} \qquad \text{eq.}(3.11)$$

And check the maximum spacing in transversal direction $s < s_{max}$

$$s_{max} = 0.75\ d\ (1 + \text{ctg}\alpha) \leq 600\text{mm}\ \text{ for stirrups} \qquad \text{eq.}(3.12)$$

In order to calculate $V_{Rd,c,}$ table 3.3 shows the calculations of the measure of the longitudinal designed tensile reinforcement in mm^2 of the beam for both variants.

Table 3.3 Stirrups calculation

DATA			ASSUMPTION STIRRUPS	
fck (Mpa)	20		2 arms ϕ6	
bw (mm)	250		Asw=2ϕ6 [mm^2]	56,54
d (mm)	235		fywd [Mpa]	434,78
σ_{cp}	0			
Asl (tensile) [mm^2]	1005		**CALCUTATION S**	
ctgθ	1		s rqd [mm]	247
VEd,d [KN]	63.7		It is assumed that the effective	
			trasversal spacing is	
CHECK Vrd,max and Vrd,c			s [mm]	200
Vrd,max [KN]	129.72			
Vrd,max > VEd,d	true		**CHECKING LIMIT**	
Vrd,c [KN]	27,1		Checking Smax	
Vrd,cmin [KN]	8		S<Smax	true
Vrd,c > Vrd,cmin	true		s<600mm	true
			check ρsw	
k (<=2)	1,922	true	ρsw<ρsw,min	true
ρl (<0,02)	0,013697	true	ρsw	0,005381
CRd,c	0,12	(γc=1,5)	ρsw,min	0,000366

The design values are stirrups ϕ6 with 2 arms and transversal spacing equal to 20 cm (figure 3.13).

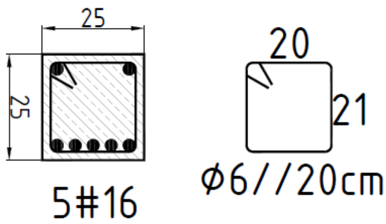


Figure 3.13 Beam Cross-Section, Project

3.2.5. DESIGN ANCHORAGE LENGTH IN TENSION FOR THE BEAM

[3] According to Eurocode 2, the design anchorage length l_{bd} is characterized for the effect of the form of the bars assuming adequate cover, α_1, for the effect of concrete minimum cover, α_2, for the effect of confinement by transverse reinforcement, α_3, for the influence of one or more welded transverse bars along the design anchorage length, α_4, and for the effect of the pressure transverse to the plane of splitting along the design anchorage length. The product $\alpha_2\alpha_3\alpha_5 \geq 0{,}7$.

In this project, it is considered α_1=1 for straight anchorage, α_2=1 in a simplified and safety approach, α_3=0,965 because of K=0,05, α_4=0,7 and α_5=1, because of the transversal pressure p=0. For the values of these coefficients:

$$l_{bd} = 0.6755\, l_{b,req} \geq l_{bmin} \qquad \text{eq. (3.13)}$$

Determination of the required length for anchoring the force $A_s f_{yd}$ in the bar

$$l_{b,req} = \frac{\phi}{4}\frac{\sigma_{sd}}{f_{bd}} \qquad \text{eq. (3.14)}$$

where

$\sigma_{sd} = \dfrac{M_{sd}}{z\, A_s}$ is the design stress of the bar at the position where the anchorage is measured at the ultimate limit state and

$$f_{bd} = 2.25\, \eta_1\eta_2 f_{ctd} \qquad \text{eq. (3.15)}$$

$$z = 0.8\, d \qquad \eta_1 = \eta_2 = 1$$

A_s tensile longitudinal reinforcement in the cross section

$$f_{ck} = 20\text{MPa}$$

$$\text{fctk} = 0.70 \text{ fctm} = 0.70 * 2.2 = 1.54 \text{ MPa}$$

$$\text{fctd} = \frac{f_{ctk}}{\gamma_c} = \frac{1.54}{1.5} = 1.03\text{MPa} \qquad \text{eq.}(3.16)$$

$$l_{b,req} = \frac{\phi}{4}\frac{\sigma_{sd}}{f_{bd}} = \frac{\phi}{4}\frac{M_{sd}}{0.8\text{d } A_s}\frac{1}{2.25\eta_1\eta_2 f_{fctd}} = \frac{\phi}{4}\frac{M_{sd}}{0.8\text{d } A_s}\frac{1}{2.25 f_{ctd}} = \qquad \text{eq.}(3.17)$$

$$= \frac{0.016}{4}\frac{72}{0.8 * 0.235 * 10.05 * 10^{-4}}\frac{1}{2.25 * 1.35} = 502 \text{ mm}$$

For the evaluation of l_{bmin} it is used the formula:

$$l_{bmin} = \max\left(0.3\, l_{brqu}, 10\phi, 100\text{mm}\right) = 160 \text{ mm} \qquad \text{eq.}(3.18)$$

Therefore

$$l_{bd} = 0.6755\, l_{b,req} = 0.6755 * 502 = 340\text{mm} \geq l_{bmin} = 160\text{mm} \qquad \text{eq.}(3.19)$$

The l_{bd} anchorage length on the beam is equal to340mm .

3.3. STRUCTURAL ANALYSIS – SERVICEABILITY LIMIT STATES

A serviceability limit state is related to the behavior of the structure or its structure elements under normal conditions in service with the objective to give more comfort to the people. SLS is also related to construction aspects as for example the good performance of the tasks which were conceived to last the expected period of life, without necessarily need to spend maintenance costs and not to need unexpected repairs.. For this, it is necessary to ensure a good behavior of the structure in service of current situation controlling the level of cracking, deformation, vibration, shrinkage and creep effects. Here the actions take real values and the medium properties of the materials are the ones taking into account and not the design values like it is used for example in the case of ultimate limit states.

3.3.1. LOADS [4]

For serviceability limit states, the combinations used are according to different probabilities of occurrence:

-Rare combination with small probability of occurrence indicating some hours in the life time of the structure

$$Gm + Qk + \Sigma\psi_1 Qik \qquad \text{eq.(3.20)}$$

-Frequent combination with probability of occurrence superior or equal to 5% of the life time of the structure, it is consider a limit state of short term

$$Gm + \psi_1 Qk + \Sigma\, \psi_2 Qik \qquad \text{eq.(3.21)}$$

-Quasi-permanent combination with the probability of occurrence superior or equal to 50% of the life time of the structure, it is consider a limit state of long term

$$Gm + \Sigma\psi_2 Qik \qquad \text{eq.(3.22)}$$

Where:

Gm – Medium value of the permanent actions

Qk – Characteristic value of the base live actions

Qik – Characteristic value of the rest of the live actions

Table 3.4 Action Categories [4]

Action	ψ_0	ψ_1	ψ_2
Category A: Domestic, residential areas	0,7	0,5	0,3
Category B: Office areas	0,7	0,5	0,3
Category C: Congregation areas	0,7	0,7	0,6
Category D: Shopping areas	0,7	0,7	0,6
Category E: Storage areas	1,0	0,9	0,8
Category F: Traffic area, vehicle weight ≤ 30 KN	0,7	0,7	0,6
Category G: Traffic area 30 KN ≤ vehicle weight ≤ 160 KN	0,7	0,5	0,3
Category H: Roofs	0	0	0

In this particular project category B is the one to be selected:

$$\psi_0 = 0{,}7 \quad \psi_1 = 0{,}5 \quad \psi_2 = 0{,}3$$

Once the coefficients are chosen, they should follow the loads for serviceability limit states (see table 3.5 and 3.6)

Table 3.5 Balance of loadings, Frequent Combination, Teriva Beams

Dead (permament loading)	**Calculation**	**gk**	**γGj, sup**	**gd $\left[\frac{KN}{m}\right]$**
Self-leveling cementious underlayment (3 cm)	γ_{slcu} [KN/m^3]* 0,03 m*0,6 m	0,308	0,7	0,22
Protective layer - PCV foli	0,02 kN/m^2*0,6 m	0,012	0,7	0,008
Isolation - mineral wool (35 cm)	γ_{pvc} * 0,35 m*0,6 m	0,005	0,7	0,004
Hollow Block $q_k = 0{,}16$ KN/unit	$q_k = 0{,}9$ KN/m	0,9	0,7	0,63
Reinforced concrete slab+beam (41,25 cm^2)	γ_c [KN/m^3]* 0.004125*0,6 m	0,072	0,7	0,06
Cementious plaster work (1,5 cm)	γ_{cpw} [kN/m^3] * 0,015 m*0,6 m	0,19	0,7	0,14
SUM		1,61		**1,2**
Live (variable loading)		**qk**	**γQ,1**	**qd**
		1,5	0,5	**0,75**

Table 3.6 Balance of loadings, Quasi Permanent Combination, Teriva beams

Dead (permanent loading)	**Calculation**	**gk**	**γGj, sup**	**gd $\left[\frac{KN}{m}\right]$**
Self-leveling cementious underlayment (3 cm)	γ_{slcu} [KN/m^3]* 0,03 m*0,6 m	0,308	1	0,308
Protective layer - PCV foli	0,02 kN/m^2*0,6 m	0,012	1	0,012
Isolation - mineral wool (35 cm)	γ_{pvc} * 0,35 m*0,6 m	0,005	1	0,005
Hollow Block $q_k = 0{,}16$ KN/unit	$q_k = 0{,}9$ KN/m	0,9	1	0,9
Reinforced concrete slab+beam (41,25 cm^2)	γ_c [KN/m^3]* 0.004125*0,6 m	0,072	1	0,072
Cementious plaster work (1,5 cm)	γ_{cpw} [kN/m^3] * 0,015 m*0,6 m	0,19	1	0,19
SUM		1,61		1,61
Live (variable loading)		**qk**	**γQ,1**	**qd**
		1,5	0,3	**0,45**

3.3.2. DIAGRAMS

Below there are the permanent + variable load for TERIVA beams supported on the beam to be calculated for frequent combination

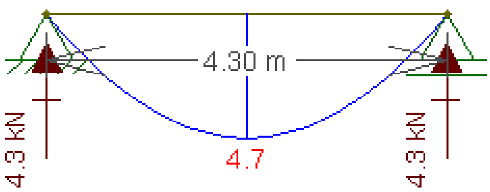


Figure 3.14 Teriva Beams, Frequent Combination Bending Moment

$$M^{MAX} = 4.7 \text{ KNm}$$

Below there are the loads for the beam to be calculated applied for the Teriva beams of the frequent combination

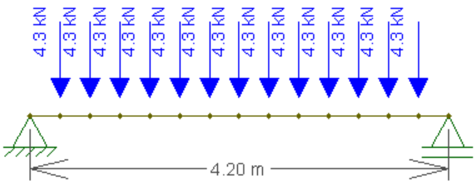


Figure 3.15 Loads, Frequent Combination, First Variant

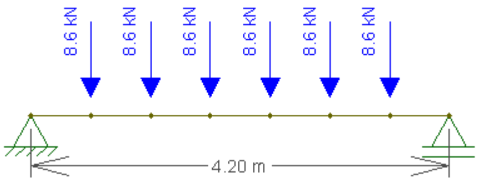


Figure 3.16 Loads, Frequent Combination, Second Variant

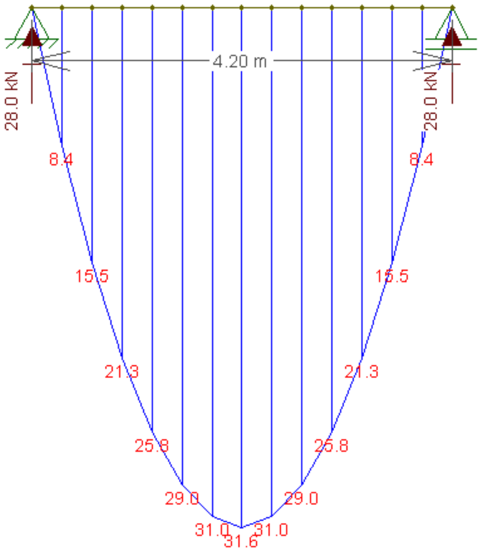


Figure 3.17 Bending, Frequent Combination, First Variant

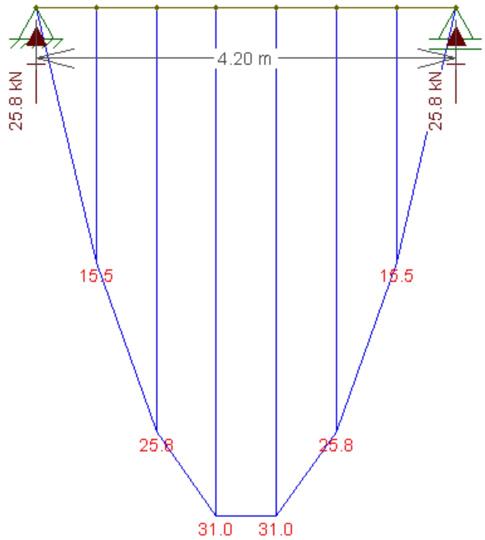


Figure 3.18 Bending, Frequent Combination, Second Variant

$$M^{MAX} = 31.6 \text{ KNm}$$

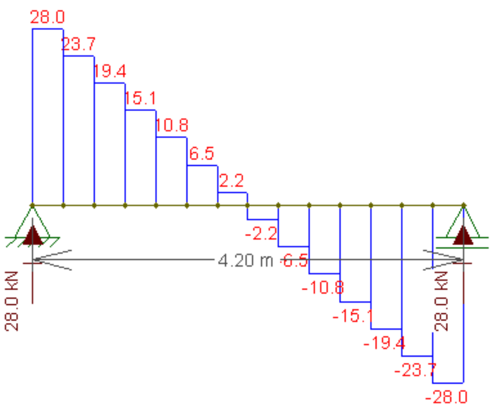


Figure 3.19 Shear, Frequent Combination First Variant

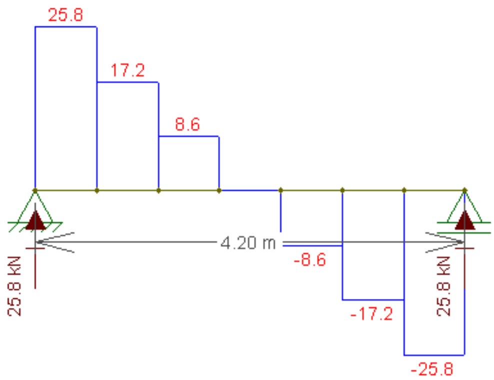


Figure 3.20 Shear, Frequent Combination, Second Variant

$$V^{MAX} = 28 \text{ KNm}$$

Below there are the permanent + variable load for TERIVA beams supported on the beam to be calculated for quasi permanent combination

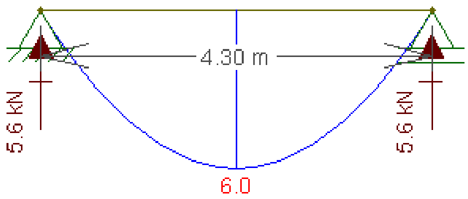


Figure 3.21 Teriva Beams, Quasi Permanent Combination Shear Force

$$M^{MAX} = 6 \text{ KNm}$$

Below there are the loads for the beam to be calculated applied for the Teriva beams for the quasi permanent combination.

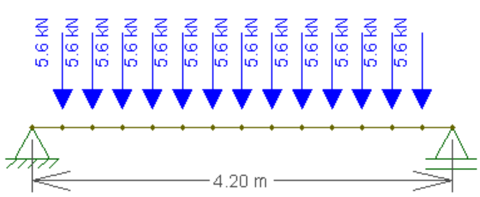


Figure 3.22 Quasi Permanent Combination, First Variant

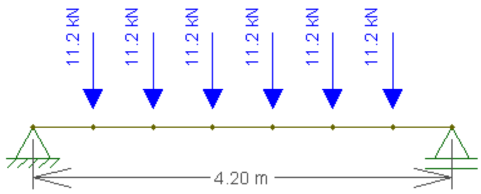


Figure 3.23 Quasi Permanent Combination, First Variant

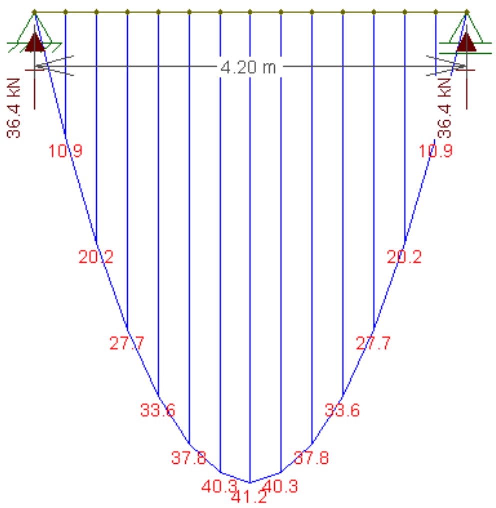


Figure 3.24 Bending, Quasi Permanent, First Variant

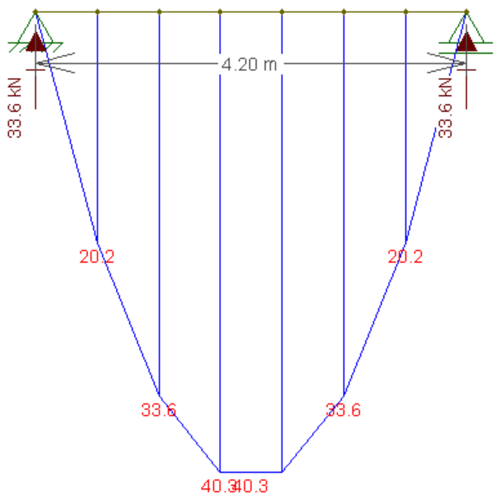


Figure 3.25 Bending, Quasi Permanent, Second Variant

$$M^{MAX} = 40.3 \text{ KNm}$$

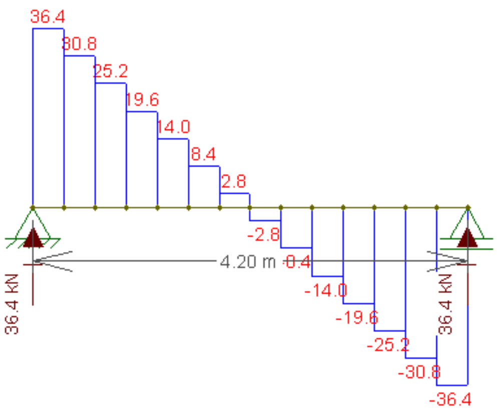


Figure 3.26 Shear, Quasi Permanent First Variant

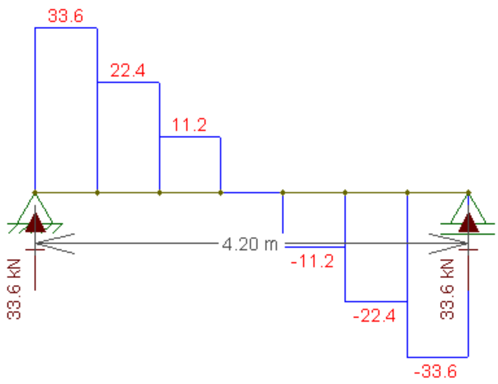


Figure 3.27 Shear, Quasi Permanent Second Variant

$$V^{MAX} = 36.4 \text{ KNm}$$

3.3.3. Cracking in the beam

Cracking is a common problem in reinforced concrete structures which can drive to an uncomfortable behavior of the structure or element. This situation of cracking occurs when the resistance of the concrete in tension is exceeded by loads or by internal forces. Here, it is analyzed the internal forces in order to prevent the excess of tension on the beam. The temperature variations and a poor design of the expansion joints can also cause cracking in the reinforcement concrete. One of the important reasons to occur cracking is the shrinkage of the concrete, when the concrete´s volume decreases in different states and in different ages.

Shrinkage is formed by plastic shrinkage, thermal shrinkage and chemical shrinkage. The plastic shrinkage occurs on the early ages of the concrete and due to the loss of water by evaporation or by capillary absorption of the aggregates the concrete´s volume decreases generating internal tensions. The thermal shrinkage occurs during the cure of the concrete until the time to remove the formwork and it is related to the cement which release heat during its hydration and cooling which also causes volume variation in the concrete and consequently internal tensions. The chemical shrinkage occurs during the hydration of the cement and is more accentuated in the first ages of the concrete. The generation of the internal stress by the chemical shrinkage is due to the fact that the volume of the cement is smaller than the volume of water and cement together.

Apart from the shrinkage, there is also the fluency in the concrete which is able to drive the element to excessive deformations and after to cracking. Usually the cracking appears due to design mistakes as the deficient details in the drawings for example, the coverage to be used and the details in the reinforcement. It is also common the occurrence of mistakes related to the material, for example a bad compression and curing of the concrete, the bad disposition of the steel located by the workers and wrong placing of the anchors.

To avoid the cracking in the concrete it is necessary to calculate the reinforcement to dispose along the element in a way that the internal tensions of the element instead being applied in the concrete are transferred to the steel which is more competent to absorb tension stresses.

Table 3.7 Recommended values of W_{max} [3]

Exposure Class	Reinforced members and prestressed members with unbounded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4	0,2
XC2, XC3, XC4	0,3	0,2
XD1, XD2, XS1, XS2, XS3		Descompression

This particular project belongs to XC1 exposure class which drives for a maximum cracking value equal to 0,4 mm, in quasi-permanent load combination.

$$\sigma_{sd} = \frac{M_{sd}}{z\, A_s} \qquad \text{eq. (3.23)}$$

$$z = 0.8\, d$$

σ_{sd} is the stress in the steel for the quasi permanent combination and A_s is the cross section of the reinforcement. In order to consider the maximum σ_{sd}, the cross-section on the middle of the span was considered.

$$M_{Ed} = 40.3\ KNm \quad A_s = 10.05 cm^2$$

$$\sigma_{sd} = \frac{40.3}{0.8 * 0.235 * 10.05 * 10^{-4}} = 213.3 MPa \qquad \text{eq. (3.23)}$$

Table 3.8 Maximum bar diameters for crack control [3]

Steel Stress	Maximum bar size [mm]		
[MPa]	w_k = 0,4mm	w_k = 0,3mm	w_k = 0,2mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

The stress in the reinforcement is 213,3 MPa and 0,4 mm is the limit value for cracking. Looking to the table 3.8, it is possible to verify that the more close value is 240 MPa, which limits the maximum bar size to 20 mm.

Table 3.9 Maximum bar spacing for crack control [3]

Steel Stress	**Maximum bar spacing [mm]**		
[MPa]	$w_k = 0{,}4$**mm**	$w_k = 0{,}3$**mm**	$w_k = 0{,}2$**mm**
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

The limit for the maximum bar spacing is 250 mm (see table 3.9). The maximum bar diameters and the maximum bar spacing for crack control are satisfied.

3.3.4. MINIMUM REINFORCEMENT AREAS

Due to the imposed deformations it is important to calculate the minimum reinforcement for the beam. When the first crack occurs in the concrete, this is not a reason to be worried. However, as the imposed deformation increases, if the reinforcement hasn´t enough strength to drive the concrete to a second cracking all the forces will be concentrated in the first crack which will drive for unacceptable values of stress.

Imposed deformations are related to the serviceability limit states and according with Eurocode 2 the minimum reinforcement is given by the following equation:

$$A_{smin} = \frac{k_c \, k \, f_{ct,eff} \, A_{ct}}{\sigma_s} \qquad \text{eq. (3.24)}$$

Where:

$A_{s\,min}$ - minimum reinforcement for the tension zone of the concrete cross-section

A_{ct} - area of the concrete cross-section where the tension is installed

$f_{ct,ef}$ - average of concrete resistance for tension for the time which is expected to be formed the first cracks. In this particular project $f_{ct,ef}$ is equal to f_{ctm}

k - coefficient connected to thick walls related to not uniform tensions along the length of the wall however in this particular project the value k is equal to 1 because the element in analyze is a beam and not a wall

k_c - coefficient which takes into account the distribution of stress along the concrete cross-section just before be formed the first crack. In this particular project due the situation is a simple bending the value for this coefficient is 0,4

σ_s - maximum stress which is pretended to achieve in the reinforcement, must by smaller than f_{yk}, value which will be consider for this project.

In this case, for only bending moment,

$$A_{smin} = \frac{k_c\, k\, f_{ct,eff} A_{ct}}{\sigma_s} = \frac{0.4 * 1 * 2.2 * 10^3 * 0.5 * 0.125 * 0.25}{213.3 * 10^3} = 0.65\ \mathrm{cm}^2 \quad \text{eq.}(3.24)$$

In the beam, in all the tensile zones the check is satisfied.

$$A_{s,tensile} > A_{smin} \text{ in all the tensile zone.}$$

3.3.5. DEFLECTION

When a structure is submitted to loads, a deflection occurs and it is important to keep these deformations in acceptable values to avoid a bad view of the structure and to avoid damage in non-structural elements. For example visible deformations in the floor, clearly visible cracks in the internal and external walls which allow the penetration of the moisture are not acceptable behaviors for a good functionality of the structure.

To analyze deflections in the concrete, it is necessary to study its behavior in service before the cracking occurs and take into account that for short term the young's modulus is important and another aspect is that when the cracks appear a loss of rigidity occurs affecting the final deformation.

According to Eurocode 2, the maximum deflection values for offices, commercial buildings and family houses is L/250 for the total deformation according to the quasi-permanent combination loads and L/500 for the additional deflection after being built the masonry walls. It is important to refer that this values are focused in the space between the support and the zone of maximum deflection and a good distance must be kept from those limits.

Considering aspects as the slenderness, cracking, concrete creep and concrete shrinkage, Eurocode 2 limits the deflections by relating the slenderness/span of the element with the height l/d.

a) Stress in steel

In the cross section the $As_{1required} = 8.8\ cm^2$ (see calculations of bending, ULS). Below there are the calculations of the necessary values concerning the span:

$$\sigma_{sd} = \frac{M_{sd}}{z\ A_s} = \frac{40.3}{0.8 * 0.235 * 10.05 * 10^{-4}} = 189\ \text{MPa} \qquad \text{eq. (3.25)}$$

$$z = 0.8\ d, \quad A_s\ \text{cross section of reinforcement}$$

σ_{sd} is the stress in the steel for the quasi permanent combination on the beam.

The span chosen takes the value of 5.4 m because it´s the biggest span of both extensions (see drawings in annexes).

b) Control of the relation span/height of the beam

$$\rho = \frac{As_{1required}}{b_w\ d} \qquad \text{eq. (3.26)}$$

$$= \frac{880}{80 * 660} = 0.0166$$

$$\rho_0 = \frac{\sqrt{f_{ck}}}{10^3} \qquad \text{eq. (3.27)}$$

$$= 0.00447$$

$$\rho' = 0 \qquad l = 5.4\ \text{m}$$

$$\frac{l}{d} = k\left[11 + 1.5\ \sqrt{f_{ck}}\ \frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\ \sqrt{f_{ck}}\ \sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if}\ \rho > \rho_0 \qquad \text{eq. (3.28)}$$

Where:

l/d - limit span/depth

k - factor to take into account the different structural systems

ρ_0 - reference reinforcement ratio

ρ - required tensions reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)

ρ' - required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)

Table 3.10 *K* values [3]

Structural System	**k**	**Concrete highly stressed $\boldsymbol{\rho = 1,5\%}$**	**Concrete highly stressed $\boldsymbol{\rho = 0,5\%}$**
Simply supported beam	1,0	14	20
End span of continuous beam	1,3	18	26
Interior span of the beam	1,5	20	30
Slab supported on columns without beams (flat slab)	1,2	17	24
Cantilever	0,4	6	8

In this particular project the case in study is a simply supported beam consequently k=1

It's possible to calculate the limit span/depth ratio on the beam using the formula for the case $\rho > \rho_o$

$$\left(\frac{l}{d}\right)_{limit} = \frac{310}{\sigma_s}\left[11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] = \qquad \text{eq. (3.29)}$$

$$= \frac{310}{213.3}\left[11 + 1.5\sqrt{20}\frac{0.00447}{0.012} + \frac{1}{12}\sqrt{20}\sqrt{0}\right]$$

$$= 19.61$$

Now it's necessary to check the ratio between the effective and the limit value:

$$\frac{l_{eff}}{d} = \frac{4200}{235} = 17.87 < \left(\frac{l}{d}\right)_{limit} = 19.61 \qquad \text{eq. (3.30)}$$

It's satisfied the check of the limit ratio length/depth on the beam.

4
EUROCODE 5 (EC5) – PART 1-1 REVIEW

Some materials used in construction are able to be recycled but timber is the only material that can be renewable. In Scandinavia the planting of trees are controlled to ensure a healthy equilibrium between the wood that the society needs and the wood that is growing. In these countries there are more trees standing than any other country. This cycle of felling and planting is very beneficial to the atmosphere, only when the tree is growing the CO^2 is absorbed and vital oxygen is given in return. Once a tree is mature this process virtually stops.

Timber is not very often used in Portugal due to the high temperatures that exist in the country which provide biological corrosion of the material. However in a time where the globalization of engineering is present in our daily day it is necessary to have the minimum knowledge on timber structures. In some countries timber has a very strong presence in the construction of single-family houses for like is the example of United States, Canada, Poland, Scotland and Nordic Countries.

In Europe engineering is increasingly being conditioned by the Environmental Impacts and wood is a current option that overcomes this obstacle. Being formed by organic chemical compounds consisting of 50% carbon and 43% oxygen, the environmental impact of wood is reduced compared with materials like concrete and steel, as a renewable material, having a cleaner construction process, by low energy consumption and carbon fixation during their growth.

4.1. PHYSICAL AND MOLECULAR CHARACTERISTICS OF TIMBER

Root, stem and cup are parts of the tree structure where the wood is extracted. However referring more specifically to wood which is used build, normally only the wood of the stem is used. Mostly wood is consisted by cellulose, hemicellulose, lignin and other substances. Cellulose fills about 45% of the element, these molecules attract water and the connections between its own molecules are very strong due its high mechanical strength (example of traction). Hemicellulose fills about 25% of wood element and basically has the function to connect the fibers. Lignin also fills about 25% of the wood and its influence is very important for the links of the fibers, shear strength, compressive strength and stiffness of the wood element. The remaining 5% are elements that have a less significant in the wood structure.

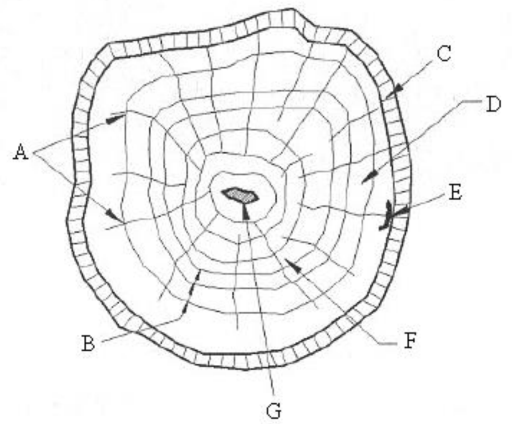


Figure 4.1 Tree trunk cross-section [16]

A- Medullary rays

B- Annual growth rings

C- Peel

D Sapwood or White

E- Exchange or Liber

F- Cerne or Durâmen

G- Marrow

Regarding physical characteristics of the wood it is important to understand how this material behaves in sound conductivity, thermal conductivity, moisture, shrink, density and durability

About sound conductivity of the wood, this material has a good capacity to absorb sounds and providing a good acoustic isolation and reducing reverberation problems. The same can be said in relation to thermal conductivity, wood is a good insulator and a good choice for cold zones. About the moisture, it is important to be controlled because has an important influence on the mechanical strength of the structure especially for shrink. Shrink ability is directly connected with the humidity, in other words, the shrink is the variation of volume due to the variation of wood moisture and so could contract or expand the timber. Another physical characteristic to be considered is durability. Durability depends on how the element is vulnerable to biological attacks such as insects, parasites, fungi and rot. These biological attacks are more or less often depending on weather conditions, air humidity and temperature which it´s necessary to take into account.

4.2. Fire Resistance

A very important aspect in timber characteristics is the fire resistance of this material. Timber is a good material to resist to the fire compared to steel and reinforced concrete elements. Despite the fact that wood is more flammable than steel however when exposed to fire the wood creates a carbonized insulating layer which protects the remainder cross-section of being affected by the flames. This carbonized insulating layer of protection against flames also prevents the temperature from rising in the interior part of the element. Yet steel when subjected to high temperatures despite being a much less flammable than timber element has large losses of mechanical resistance after a few minutes exposed to fire unlike the wood that has a slow combustion. It is concluded that timber when expose to the fire it doesn´t collapse because of the high temperature but because of the loss of cross-section area. To avoid the collapse or at least to give the regular time before the collapse usually is added additional 2 or 3 cm in the cross-section sizes as a protective layer of the remaining section.

4.3. Fire Resistance

There are different types of wood which can be divided in two groups: industrial wood and hardwoods. In hardwood group exists the gross wood or round wood normally used in axial forces serving as piles, anchors and pillars, the trimmed wood normally used for driven piles and curtains and for last the lumber wood which is the most used in the construction and however suffer from some drawbacks as their geometric limitations. In industrialized woods group exists plywood, it is the oldest wood. It is formed by gluing sheets, laminated, glued wood (very often used in Europe which consists of blades glued under pressure) and the recompense wood usually used for plate structures originated by the rests of wood.

4.4. Mechanical Characteristics

Timber is a material which has different resistances depending on the type of loads and the direction which the loads are applied in relation to the fibers. Fiber´s wood structure in the trees is developed in a way to resist to the natural actions that are subjected during its lifetime like for example wind and gravity forces that produce bending and compression stress.

4.4.1. AXIAL FORCE

During a lifetime, the tree is submitted to gravitational compressive forces parallel to the fibers driving the tree for a need to possess higher capacity to resist the loads in this direction. As for the traction stress, wood has less resistance when stress is applied perpendicular to the fibers because it tends to separate the fibers decreasing the cohesion of the element.

4.4.2. BENDING

As for the bending, the timber resistance has a similar value as the traction resistance and has the advantage of its own weight being lighter when compared to concrete or steel. Despite the fact that steel responds better to bending then timber, the resistance values achieved by timber structures are satisfactory facing the normal loads applied to current structures.

4.4.3. SHEAR

As for the shear strength of timber, this stress can act in different directions as perpendicular, parallel or oblique to the fibers. Timber has a lower resistance facing shear forces when applied parallel to the fibers because the fibers are disposed in vulnerable way making possible the sliding in this direction. However it is the opposite case when the stress is applied in perpendicularly direction to the fibers, sliding will not be a problem.

4.4.4. FLUENCY

Fluency occurs when the element it´s faced with long-term loads. Besides the resistance of the element is enough to face the load however due to the long-term of the load the material will suffer continuous deformations in long term driving the element for a plastic state and consequently collapse. The fluency depends on air temperature, air humidity and the water content of the element. In order to prevent problems usually the solution is to design counter arrows, to pay attention to the variations of temperature which the structure will be exposed and a good control of the water content of the wood.

4.5. Examples of Eurocode 5 Notation

Here is presented some of the Eurocode 5 notions very often used in the calculations as it is the example of:

- Design value of compressive strength perpendicular to the grain direction

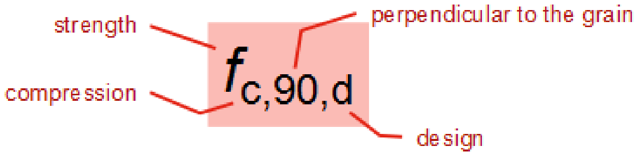


Figure 4.2 Examples of Eurocode5 Notation

- Mean modulus of elasticity parallel to the grain direction

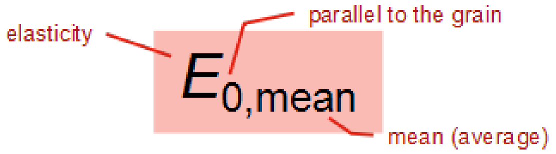


Figure 4.3 Examples of Eurocode5 Notation

- Instantaneous deformation caused by bending moment

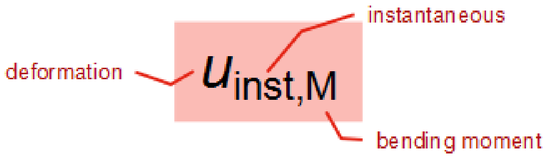


Figure 4.4 Examples of Eurocode5 Notation

4.6. STRENGTH CLASSES OF TIMBER

To analyze the strength classes of timber it is important to understand that they are divided in two family groups to softwood and hardwood. Hardwood is not necessarily a harder material (more dense) and softwood is not necessarily a softer material (less dense). For example, balsa wood is one of the lightest, least dense woods there is, and it's considered a hardwood.

The distinction between hardwood and softwood actually has to do with plant reproduction. All trees reproduce by producing seeds, but the seed structure varies. Hardwood trees are angiosperms, plants that produce seeds with some sort of covering. This might be a fruit, such as an apple, or a hard shell, such as an acorn. Hardwoods instead of tracheid contain pores ranging in size and shape allowing water to flow from the roots to the top of the tree. (e.g. Hickory)

Figure 4.5 Hardwood [6] [14]

Softwoods, on the other hand, are gymnosperms. These plants let seeds fall to the ground as is, with no covering. Pine trees, which grow seeds in hard cones, fall into this category. In conifers like pines, these seeds are released into the wind once they mature. This spreads the plant's seed over a wider area. At the structural level, softwoods are constituted by a tracheid system used to transport water and produce sap. (e.g. Cypress)

Figure 4.6 Softwood [6] [13]

For the most part, angiosperm trees lose their leaves during cold weather while gymnosperm trees keep their leaves all year round. So, it's also accurate to say evergreens are softwoods and deciduous trees are hardwoods.

The hardwood/softwood terminology does make some sense. Evergreens do tend to be less dense than deciduous trees, and therefore easier to cut, while most hardwoods tend to be denser, and therefore sturdier [17].

In the table below is presented the strength class timber.

Table 4.1 Strength Classes of Timber [7]

Species type	**Softwood**									**Hardwood**					
Class	**C14**	**C16**	**C18**	**C22**	**C24**	**C27**	**C30**	**C35**	**C40**	**D30**	**D35**	**D40**	**D50**	**D60**	**D70**
Strength properties (N/mm^2)															
$f_{m,k}$	14	16	18	22	24	27	30	35	40	30	35	40	50	60	70
$f_{t,0,k}$	8	10	11	13	14	16	18	21	24	18	21	24	30	36	42
$f_{t,90,k}$	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.6	0.6	0.6	0.6	0.7	0.9
$f_{c,0,k}$	16	17	18	20	21	22	23	25	26	23	25	26	29	32	34
$f_{c,90,k}$	4.3	4.6	4.8	5.1	5.3	5.6	5.7	6.0	6.3	8.0	8.4	8.8	9.7	10.5	13.5
$f_{v,k}$	1.7	1.8	2.0	2.4	2.5	2.8	3.0	3.4	3.8	3.0	3.4	3.8	4.6	5.3	6.0
Stiffness properties (kN/mm^2)															
$E_{0,mean}$	7	8	9	10	11	12	12	13	14	10	10	11	14	17	20
$E_{0,05}$	4.7	5.4	6.0	6.7	7.4	8.0	8.0	8.7	9.4	8.0	8.7	9.4	11.8	14.3	6.8
$E_{90,mean}$	0.23	0.27	0.30	0.33	0.37	0.40	0.40	0.43	0.47	0.64	0.69	0.75	0.93	1.13	1.33
G_{mean}	0.44	0.50	0.56	0.63	0.69	0.75	0.75	0.81	0.88	0.60	0.65	0.70	0.81	1.06	1.25
Density (kg/m^3)															
ρ_k	290	310	320	340	350	370	380	400	420	530	560	590	650	700	900
ρ_{mean}	350	370	380	410	420	450	460	480	500	640	670	700	780	840	1080

Table 4.2 Strength Types of Timber [18]

Strength Class	Tree Species
C14	Western Red Cedar
C16	Hem Fir
C22	British Pine, Spruce
C24	Spruce, Poplar, Larch
C30	Spruce, Poplar, Larch, Pine (redwood)
C35	Douglas Fir
D30	Oak
D35	Beech
D40	Beech, Teak, Eucalyptus
D50	Keruing, Karri, Opepe
D60	Kapur
D70	Balau, Greenheard, Ekki

Table 4.3 Recommended cross-sections dimensions according to PN-EN 1313-1 [8]

Thickness [mm]	Width [mm]						
	75	100	125	150	175	200	225
38		x	x	x			
50		x	x	x	x	x	x
63		x	x	x			
75				x	x	x	x
100						x	

4.7. Design values of material properties [5]

In order to prevent collapse or unacceptable behavior of the structure limiting the values applied to the material, this is used the partial factor method where the design values are calculated according to characteristics actions & characteristics strength influenced by safety factors. In this order the design value of a material property must be determinate by the following equation:

$$X_d = k_{\mathrm{mod}} \cdot \frac{X_k}{\gamma_M} \qquad \text{eq. (4.1)}$$

where:

X_k - characteristic value of strength property

k_{mod} - coefficient modifying strength property due to time under load and moisture content

γ_M - partial coefficient for material property

When two elements are made of different materials k_{mod} is determinate by the following expression:

$$k_{\mathrm{mod}} = \sqrt{k_{\mathrm{mod},1} \cdot k_{\mathrm{mod},2}} \qquad \text{eq. (4.2)}$$

The design value of material elasticity and the design value of material rigidity is given in the following equations

$$E_d = \frac{E_{mean}}{\gamma_M} \qquad \text{eq. (4.3)}$$

$$G_d = \frac{G_{mean}}{\gamma_M} \qquad \text{eq. (4.4)}$$

Where:

E_{mean} - average modulus of elasticity

G_{mean} - average modulus of rigidity

γ_M - partial coefficient for material property

Table 4.4 Recommended Partial Factors γ_M (Fundamental Combinations) [5]

Design situation	**Partial factor γ_M**
Solid timber	1.3
Glued laminated timber	1.25
Laminated veneer lumber (LVL),plywood, OSB	1.2
Particle board	1.3
Fibreboard hard	1.3
Fibreboard medium	1.3
Fibreboard, MDF	1.3
Fibreboard , soft	1.3
Connections	1.3
Punched metal plate fasteners	1.25
Accidental combinations	1.0
Serviceability limit states	1.0

4.8. LOAD DURATION

Load duration is an important aspect to be analyzed in timber structures. Wood has the capacity to resist better to loads applied instantaneously than when the loads are applied tor long duration of time. To prevent collapse and unacceptable deflections Eurocode suggest the application of the factor K_{mod} related to the resistance of the material and K_{def} related to the deflections of the material.

Table 4.5 Values of K_{mod} [5]

Load duration class	**Service class**		
	1	**2**	**3**
Permanent (> 10 years)	0.60	0.60	0.50
Long-term (6 months - 10 years)	0.70	0.70	0.55
Medium-term (1 week - 6 months)	0.80	0.80	0.65
Short-term (< 1 week)	0.90	0.90	0.70
Instantaneous	1.10	1.10	0.90

Table 4.6 Values of K_{def} [5]

Load duration class	**Service class**		
	1	**2**	**3**
Solid Timber	0.60	0.80	2,00
Glued laminated timber	0.60	0.80	2,00
LVL	0.60	0.80	2,00
Plywood type	0.80	-	-
OSB	2,25	-	-
Particleboard	2,25	-	-
Fibreboard hard.	2,25	-	-
Fibreboard med.	3,00	-	-
Fibreboard	2,25	-	-

4.9. MOISTURE CONTENT AND TEMPERATURE [5]

One of the elements of wood is water, this quantity of water is defined as moisture content. It is important to take into account this phenomenon because it has direct influence in the mechanical properties of timber and in order to prevent the moisture excess in wood Eurocode distinguishes three service classes:

- Service class 1: The structure is submitted to a temperature of 20 °C and the air only reaches 65 % of humidity few weeks per year. In this conditions normally the wood as a maximum moisture content value of 12 %.

- Service class 2: The structure is submitted to a temperature of 20 °C and the air only reaches 85 % of humidity few weeks per year. In this conditions normally the wood as a maximum moisture content value of 20 %.

- Service class 3: The structure is submitted for bigger humidity and consequently moisture content in the element than service class 2. (Usually cross-sections with thickness or height equal or higher than 10cm belong to service class three because this elements are very difficult to dry)

The temperature which the timber structure is exposed must be above 60°C however if the temperature is higher than 60°C and above 75°C a coefficient $K_{temp} = 0.80$ must be used in the calculations to decrease the resistance expected.

Table 4.7 Percentage of Moisture Content & Strength [5]

Strength for moisture content between 8 and 20%		**Strength drop with 1% moisture content**
Compressive strength parallel to grain	$f_{c,0}$	6,0%
Bending strength	f_m	4,0%
Tensile strength parallel to grain	$f_{t,0}$	2,5%

Table 4.8 Service Classes of Timber [5]

Service class	Environmental conditions
1	Timber in buildings with heating and protected from damp conditions. Examples are internal walls, internal floors (other than ground floors) and warm roofs.
2	Timber in covere buildings. Examples are ground floor structures where free moisture is present cold roofs, the inner leaf of cavity walls and external single leaf walls with eternal cladding
3	Timber fully exposed to the weather. Examples are the exposed parts of open buildings and timber used in marine structures

4.10. DEPTH FACTOR – SOLID TIMBER

Timber elements possess an important phenomenon related to its resistance and volume. It is proved that the resistance of timber elements decreases according to the proportion of volume of the element. By another words, as bigger the volume of the element more a decrease of resistance the element suffers. Eurocode suggest to apply the coefficient K_h for cross-sections with small sizes (solid timber with sizes below 150 mm and glued laminated timber with sizes bellow 600 mm). With this coefficient for small sizes of cross-sections the gain of strength can reach a maximum value of 30% forb solid timber and 10% for glued laminated timber. Characteristic strength values $f_{m,k}$ and $f_{t,0,k}$ should be multiplied by K_h coefficient. The coefficient K_h for solid timber is calculated following the equation bellow:

$$K_\mathrm{h} = \min \begin{cases} \left(\frac{150}{\mathrm{h}}\right)^{0,2} \\ 1,3 \end{cases} \qquad \text{eq. (4.5)}$$

The coefficient K_h for glued laminated timber is calculated following the equation bellow:

$$K_h = \min\begin{cases}\left(\frac{600}{h}\right)^{0,1} \\ 1,1\end{cases} \qquad \text{eq. (4.6)}$$

Where:

h – height of bended element or width of an element under tension

4.11. ULTIMATE LIMIT STATES

The ultimate limit state prevents the structure from collapse limiting the loads which will be applied on the structured and ensuring its respective resistance with influence of safety factors. This following condition must be verified:

$$E_d \leq R_d \qquad \text{eq. (4.7)}$$

E_d corresponds the design value of the actions applied in the element and R_d the design value corresponding the resistance of the element

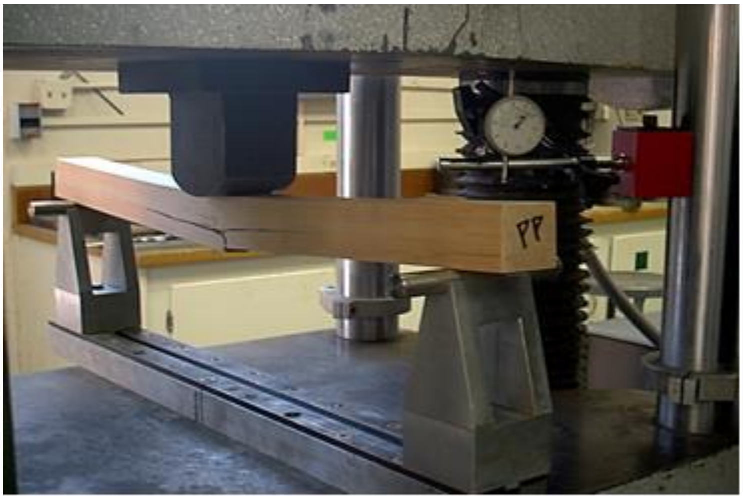


Figure 4.7 Timber beam, ULS [9]

As mentioned before the direction which the loads are applied to timber elements in relation to the grain as direct influence in the mechanics properties of the timber and also tits own resistance. Usually the tension is applied parallel to the grain because the timber responds better in this direction. Considering the axes of the figure 4.8, the following condition must be verified tor tensions parallel to the grain:

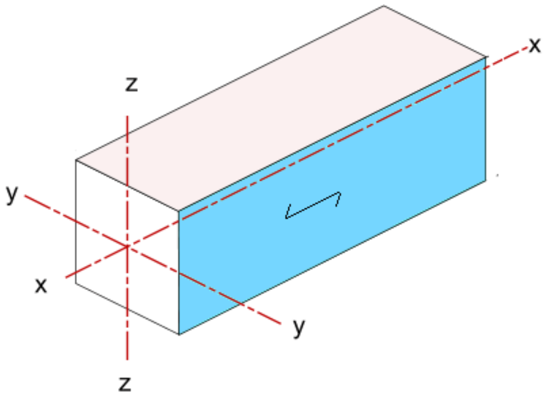


Figure 4.8 Direction of Grain [7]

$$\sigma_{t,0,d} \leq f_{t,0,d} \qquad \text{eq. (4.8)}$$

$$\sigma_{t,0,d} = \frac{N_d}{A_{net}} \qquad \text{eq. (4.9)}$$

Where:

N_d - design value of axial force [N]

$f_{t,0,d}$ - design strength parallel to grain [MPa]

A_{net} - cross-sectional area $[mm^3]$

The following condition corresponds for compression perpendicular to the grain however it is important to make reference to the coefficient $k_{c,90}$. When a timber element is submitted for a load of compression perpendicular to the grain its own resistance depends on the geometry and extension of the loaded zone. Taking in account this factor Eurocode suggest the application of the $k_{c,90}$ coefficient. To be noted that when is used high values for $k_{c,90}$, this can drive the element to significant transversal deformations due the compression what can cause problems in service.

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \qquad \text{eq. (4.10)}$$

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} \qquad \text{eq. (4.11)}$$

Where:

$F_{c,90,d}$ - design value of compressive force

A_{ef} - effective area of compression

$k_{c,90}$ - coefficient increasing strength when length of the section under load does not exceed value specified in the table below

Table 4.9 Limits Compression Widths [5]

	$l_1 \leq$ 150 mm	l_1 < 150mm	
		α ≥ 100mm	α < 100mm
l ≥ 150mm	1	1	1
150 mm > l ≥ 15mm	1	1+(150-l)/170	1+ α(150-l)/17000
l < 15mm	1	1,8	1+ α/125

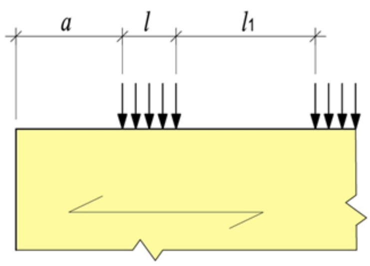


Figure 4.9 Limits Compression Widths [5]

The effective area submitted to stress of a timber element when submitted to compression perpendicular to the grain is bigger than the area in touch with the surface of the load like is shown in the figure 4.10 below:

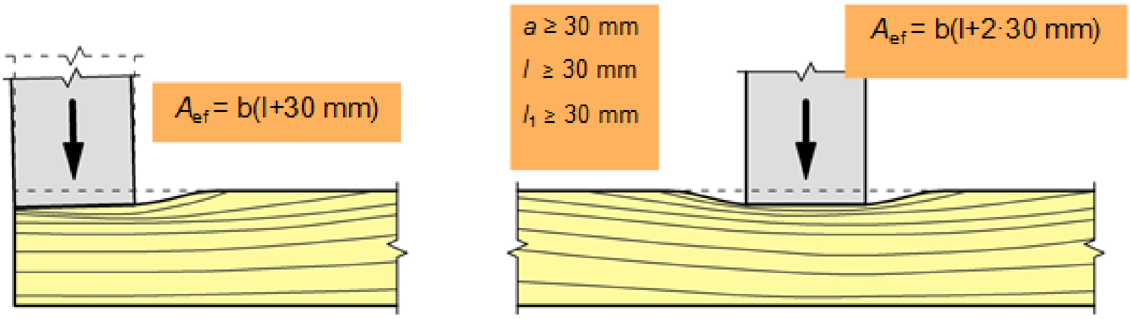


Figure 4.10 Effective area of compression [5]

However when the element is subjected to a compression with a certain angle to the grain the following condition must be verify:

$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \qquad \text{eq. (4.12)}$$

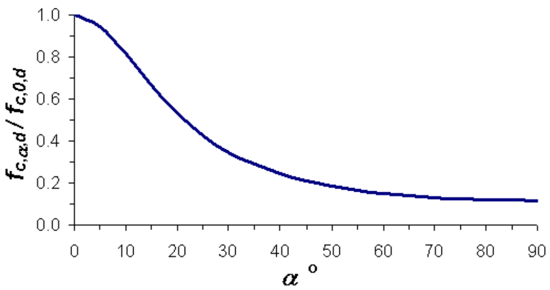


Figure 4.12 $f_{c,a,d} / f_{c,0,d}$ & a°[15]

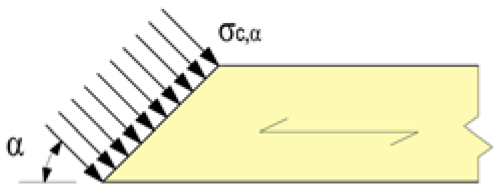


Figure 4.11 Compressive stresses at an angle [5]

4.11.1. BENDING

When a timber beam is submitted to bending stress usually it is necessary to verify the condition of resistance for the bending which is applied, instability of the beam, deformations, vibrations, resistance of the connections, resistance of the joins and resistance to the shear force.

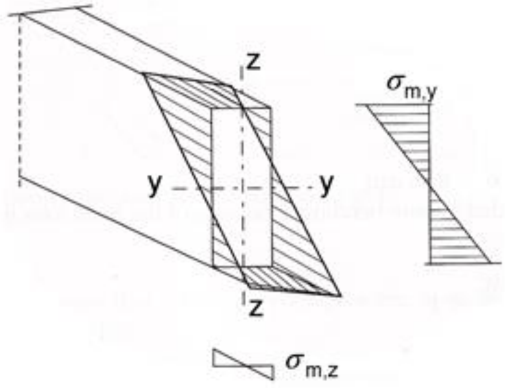


Figure 4.13 Stress in different axis [5]

To verify the resistance to the bending is necessary verify the following equations:

$$k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.13)}$$

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.14)}$$

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_y} \qquad \text{eq. (4.15)}$$

$$\sigma_{m,z,d} = \frac{M_{z,d}}{W_z} \qquad \text{eq. (4.16)}$$

Where:

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ - design bending stresses about
the principal axes strengths [MPa]

K_m - factor taking into account possibility of stress redistribution

$f_{m,y,d}$ and $f_{m,z,d}$ - design bending strengths [MPa]

Normally for rectangular cross-sections K_m = 0,7 and for other cross-section K_m = 1,0.

4.11.2. SHEAR

Shear resistance depends on the direction which is applied relating to the grain. When the action is applied in the same direction of the grain the resistance is minimum because the fibers slide between in each other however if the shear force is applied in transversal way relating to the fibers the force needed to cut the timber is much more higher.

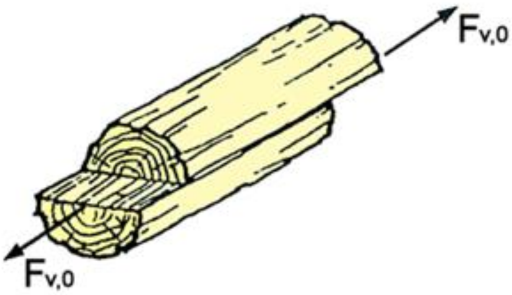


Figure 4.14 Shear parallel to the grain [10]

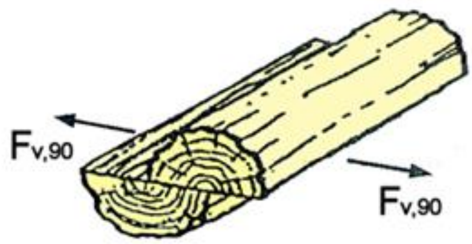


Figure 4.15 Shear perpendicular to the grain [10]

In order to prevent the beam from a shear force collapse the following condition must be verify:

$$\tau_d \leq f_{v,d} \qquad \text{eq. (4.17)}$$

Where:

τ_d - design shear stress [MPa];
$f_{v,d}$ - design shear strength [MPa];

In cases of a rectangular cross-section in order to prevent the beam from a collapse the following condition must be verify:

$$\tau_d = 1{,}5\frac{V}{b \cdot h} \leq f_{v,d} \qquad \text{eq. (4.18)}$$

Where:

τ_d - design shear stress [MPa];
$f_{v,d}$ - design shear strength [MPa];
V - design shear force [KN];
b, h - width and height of rectangular cross - section [m];

For beams which the cross-section is not constant in the ends like the figure 4.16 shows bellow, it is necessary to verify the following condition:

$$\tau_d = \frac{V \cdot S}{I \cdot b} \leq f_{v,d} \qquad \text{eq. (4.19)}$$

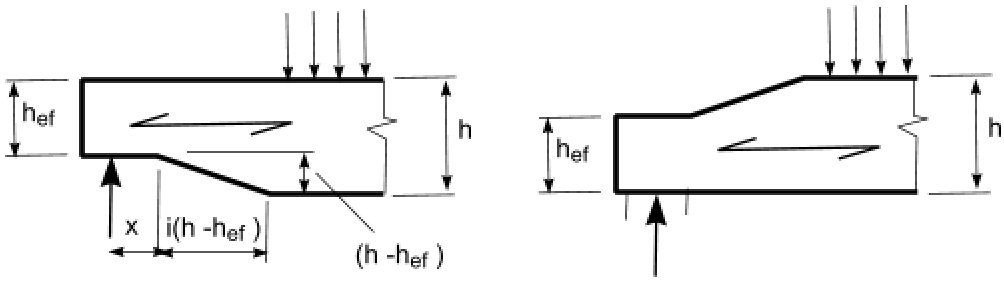


Figure 4.16 Variation of cross-section [5]

Where:

i - notched inclination
h - beam depth
x - distance between the line of action to the corner

Eurocode take into account the cracks and fractures in the wood when related to ultimate limit states and in order to prevent this problem the k_{cr} coefficient must be applied to the width

$$b_{ef} = k_{cr} \cdot b \qquad \text{eq. (4.20)}$$

Where:

b - width of cross - section

k_{cr} - factor taking into account possibility of cracking due to shear:

Table 4.10 k_{cr} values [5]

Types	k_{cr}
Solid timber	0,67
Glulam timber	0,67
Wood – based materials	1,0

4.11.3. TORSION

Timber torsion resistance is not one of the strongest points of this material due to the sliding of the longitudinal fibers however Eurocode gives a condition to verify torsional stresses

$$\tau_{tor,d} \leq k_{shape} \cdot f_{v,d} \qquad \text{eq. (4.21)}$$

$$\tau_{tor,d} = \frac{M_{tor,d}}{W_{tor}} \qquad \text{eq. (4.22)}$$

$$M_{tor} = M_x = F \cdot e \qquad \text{eq. (4.23)}$$

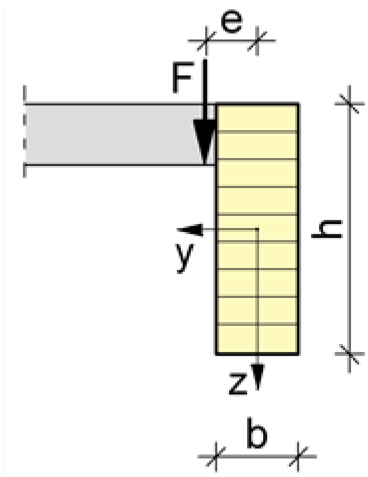


Figure 4.17 Torsion [5]

k_{shape} - coefficient takes into account the shape of the cross-section

$$k_{shape} = \begin{cases} 1{,}2 \\ \min\begin{cases} 1 + 0{,}15\dfrac{h}{b} \\ 2{,}0 \end{cases} \end{cases} \qquad \text{eq. (4.24)}$$

where:

$\tau_{tor,d}$ - design torsional stress [MPa]
$f_{v,d}$ - design shear strength [MPa]
k_{shape} - factor concerning shape of the cross - section
h - longer side of the cross – section
b - shorter side of the cross – section

For rectangular cross - section:

$$W_{tor} = \frac{h \cdot b^2}{3\eta} \qquad \text{eq. (4.25)}$$

For round cross - section:

$$W_{tor} = \frac{\pi \cdot d^3}{16} = \frac{\pi \cdot r^3}{2} \qquad \text{eq. (4.26)}$$

4.11.4. COMPLEX STRESS STATES

When the element is submitted to bending plus tension the following conditions must be verify:

$$\frac{\sigma_{t,0,d}}{f_{t.0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.27)}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.28)}$$

When the element is submitted to bending plus compression the following conditions must be verify:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.29)}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.30)}$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \qquad \text{eq. (4.31)}$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \qquad \text{eq. (4.32)}$$

4.11.5. LATERAL BUCKLING

When a beam/column is submitted to axial and bending forces it is necessary to verify the buckling condition in such way that the lateral displacement doesn´t exceed the limit. In order to verify this situation the following condition must be verify:

$$\sigma_{m,d} \leq k_{crit} f_{m,d} \qquad \text{eq. (4.33)}$$

Where:

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

$f_{m,d}$- design bending strength

k_{crit}- factor which takes into account the reduction in bending strength due to lateral buckling and is given by

$k_{crit} = 1$ for $\lambda_{rel,m} \leq 0,75$

$k_{crit} = 1,56 – 0,75$ for $\lambda_{rel,m}$ for $0,75 \leq \lambda_{rel,m} \leq 1,4$

$k_{crit} = 1 / \lambda_{rel,m}^2$ for $1,4 < \lambda_{rel,m}$

$\lambda_{rel,m}$ - relative slenderness ratio

4.12. SERVICEABILITY LIMIT STATES

Serviceability limit state has in view a good behavior of the structure when submitted to diverse loads during its life. Usually a symptom that the serviceability limit state was exceeded is the difficulty to open doors, windows or cracks in walls and ceilings due the deformations of the structure. Eurocode take into account the durability, global stability, fire resistance, cracking, vibrations and fire resistance.

$$E_d \leq C_d \qquad \text{eq. (4.34)}$$

E_d corresponds the value of the actions applied in the element and C_d the value corresponding the resistance of the element

4.12.1. DEFLECTION [5]

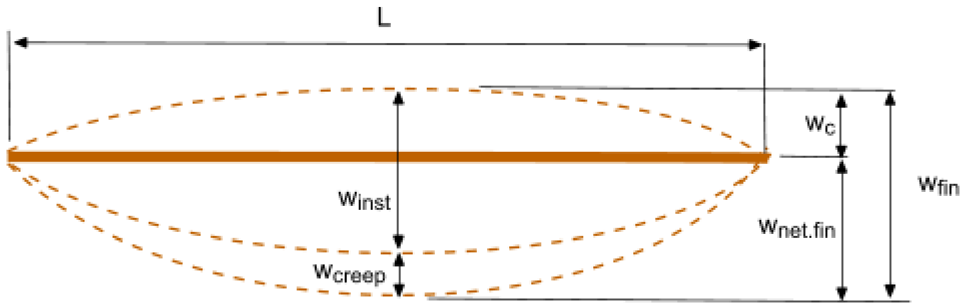


Figure 4.18 Components of deflection [18]

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \quad \text{eq. (4.35)}$$

where:

w_c - precamber (if applied);

w_{inst} - instantaneous deflection;

w_{creep} - deflection due to effect of creep;

w_{fin} - final deflection,

$w_{net,fin}$ - net value of final deflection

Table 4.11 Examples of permissible deflections for beams [5]

Support type	w_{inst}	$w_{net,fin}$	w_{fin}
Beam supported at both ends	l/300 – l/500	l/250 – l/350	l/150 – l/300
Cantilever	l/150 – l/250	l/125 – l/175	l/75 – l/150

Table 4.12 Permissibile deflections w_{fin} [5]

Structure type	w_{fin}
Trusses	l/500 (simplified calculations) l/300 (exact calculations)
Floor beams	l/250, l/300 (vulnerable to deflection, f.e. gypsum plasterboard ceiling)
Roof rafters	l/200
Cantilevers	l/150

It is allowed to increase the permissible deflections value by 50% for historical, renovated elements. In case of structures composed of elements, components and connectors with the same creep behavior, and assuming a linear relationship between actions and the corresponding displacements, final displacement u_{fin} can be calculated in a simplified manner, using the following formula:

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + \sum u_{fin,Qi} \qquad \text{eq. (4.36)}$$

$$u_{fin} = u_{inst} + u_{creep} \qquad \text{eq. (4.37)}$$

Where:

$u_{fin,G} = u_{fin,G}(1+k_{def})$ - for permanent loads G

$u_{fin,Q1} = u_{fin,Q1}(1+\Psi_{2,1}k_{def})$ - for variable load Q_i

$u_{fin,Qi} = u_{fin,Qi}(\Psi_{0,i} + \Psi_{2,i}k_{def})$ - for variable actions $Q_i (i \geq 1)$

$u_{inst,Gi}, u_{inst,Q1}, u_{inst,Qi}$ - Instantaneous deflections

$\Psi_{2,1}, \Psi_{2,i}$ - Coefficients for quasi-permanent variable combination

$\Psi_{0,i}$ - Coefficients of variable actions combinations

k_{def} - Deflection coefficient

For beams with l / h ratio > 20

$$u_{inst} = u_M = \frac{5q \cdot l^4}{384 E_{0,mean} \cdot I} \qquad \text{eq. (4.38)}$$

For beams with l / h ratio < 20 and constant cross – section

$$u_{inst} = u_M + u_V = u_M \left[1 + 19{,}2 \left(\frac{h}{l} \right)^2 \right] \qquad \text{eq. (4.39)}$$

For beams with l / h ratio < 20 and variable cross – section

$$u_{inst} = u_M \left[\frac{1 + 19{,}2 \left(\frac{h_{ap}}{l} \right)}{0{,}15 + 0{,}85 \frac{h_e}{h_{ap}}} \right] \qquad \text{eq. (4.40)}$$

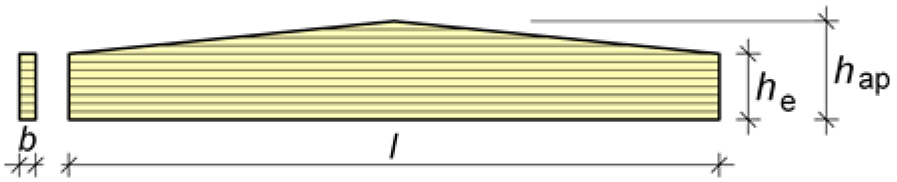


Figure 4.19 Variable Height [5]

4.12.2. VIBRATIONS

Eurocode limits the vibrations to safety levels for the serviceability limit states. The fundamental frequency of vibration of a rectangular residential floor, f_1, is estimated using the following expression and usually should not exceed 8 Hz.

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{(EI)}{m}} \qquad \text{eq. (3.79)}$$

Where:

m - mass of the floor [kg m^{-2}]
L - span of the floor [m]
(EI) - bending stiffness in the beam direction [N.m^2 / m]

When the fundamental frequency of the floor is bigger than 8 Hz, the following condition must be verified

$$\frac{\omega}{F} \leq a\,\mathrm{mm/kN} \quad \text{and} \quad \upsilon \leq L^{(f_1,\xi-1)}\, m/(N/s^2) \qquad \text{eq. (3.80)}$$

Where:

ξ - modal damping coefficient, normally taken as 0.02

ω - maximum vertical deflection caused by a concentrated static force F = 1.0 KN

ν - unit impulse velocity

a - deflection of floor under a 1 KN point load

(must not exceed 1,8mm for L ≤ 4 000mm and must not exceed 16 500/ $L^{1,1}$ mm for L > 4 000mm)

b - velocity response constant

(b = 180 - 60.a if a ≤ 1mm and b = 160 - 40.a if a > 1mm)

4.13. CONNECTIONS

Joints are the most complex area in timber structures to be calculated due its concentrated tensions in critical zones. Usually the joints are a specific zone of the structure where the tension is transfer to another element with a certain angle between them and with a discontinuity of the element which brings a special attention in these situations.

In a brief introduction it is important the understanding of some basic meanings related to this matter as the words “connector” and “connection”. A single bolt or a single nail can be described as connector because it is refer to a single one. In other hand connection is a combination of a group of connectors and together have the enough resistance to transfer the tensions to the next element (joint = connection). The interaction between the timber and the metal connection is a very complex matter which includes important aspects as the strength of timber, the geometry of the connection, the type and stiffness of the fastener. In the figures bellow are presented the examples of connections.

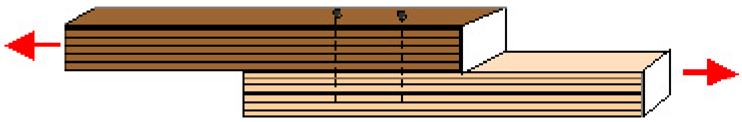


Figure 4.20 Timber to timber connection

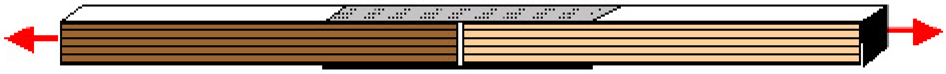


Figure 4.21 Timber to timber connection – nail plate

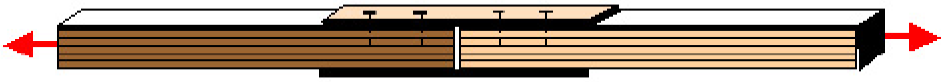


Figure 4.22 Timber to plywood connection

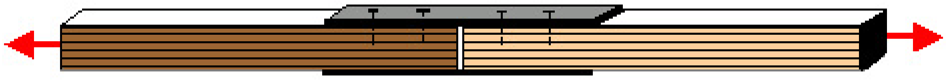


Figure 4.23 Timber to heavy gauge steel connection

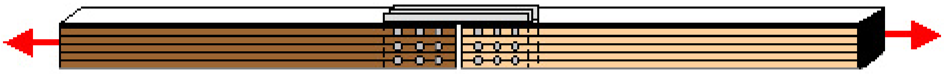


Figure 4.24 Timber to dowelled fin

Also the timber can connect itself with other element of timber considering just the geometry in way that one fits into another

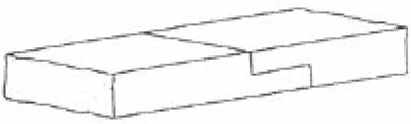


Figure 4.25 Half-lap joint

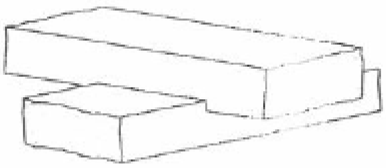


Figure 4.26 Frame joint

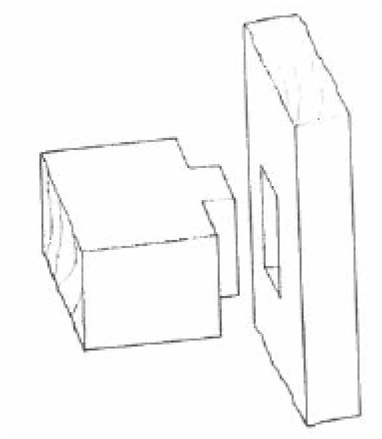
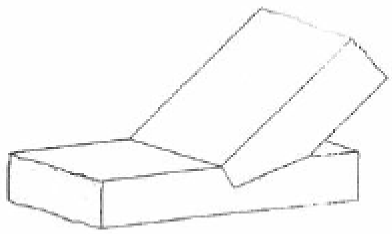



Figure 4.27 Tenon joint Figure 4.28 Cogging joint

The type of connection depends on the type of the action which influences the design. Basically there are two groups of connections depending in the type of the stress: groups 1 and group 2. Group 1 is related to connections that are submitted to shear stress as shown in the following figures 4.29

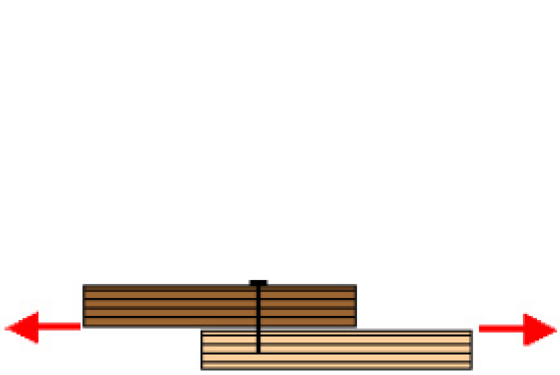


Figure 4.29 Connection group 1

Group 2 is related to connections that are submitted to axil stress as shown in the following figure 4.30

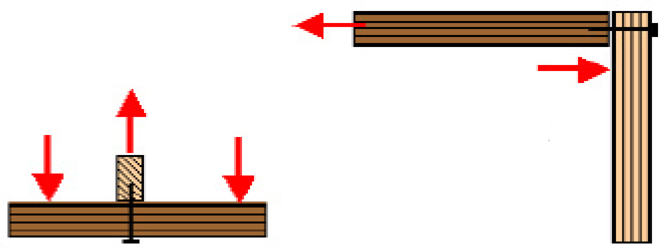


Figure 4.30 Connection group 2

4.13.1. METAL DOWEL TYPE CONNECTIONS

Very often the wood elements are connected by nails and it is verified an existence of a friction force between the surface of the wood and the nails. Density of the wood, the depth of penetration and the diameter of the nail are important aspects to take into consideration to measure the resistance of a nail shank.

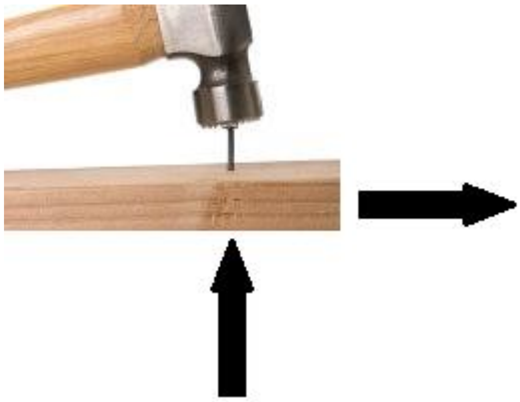
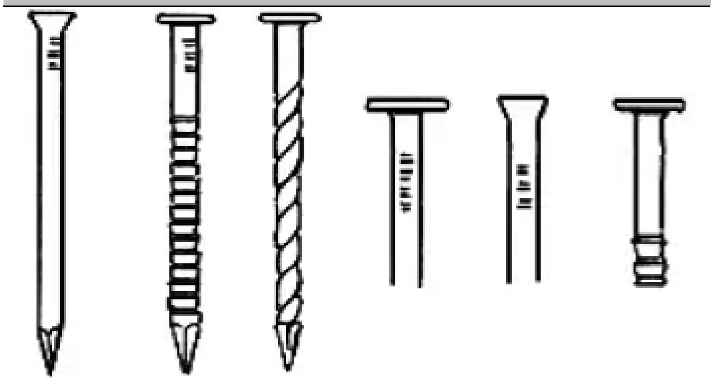


Figure 4.31 Friction force [19]

In the table 4.13 bellow it is presented the some types of nails

Table 4.13 Types of nails and screws [20]

Shanks			Heads		
Straight	**Ring**	**Spiral**	**Box**	**Casing**	**Siding**



Screws are installed into a drilled hole, by turning the screw and allowing the flutes on the thread of the screw to draw it in. The hole diameter is the same as the root diameter of the threads. The thread cuts into the fiber structure and forms a mechanical bond with the wood [21]

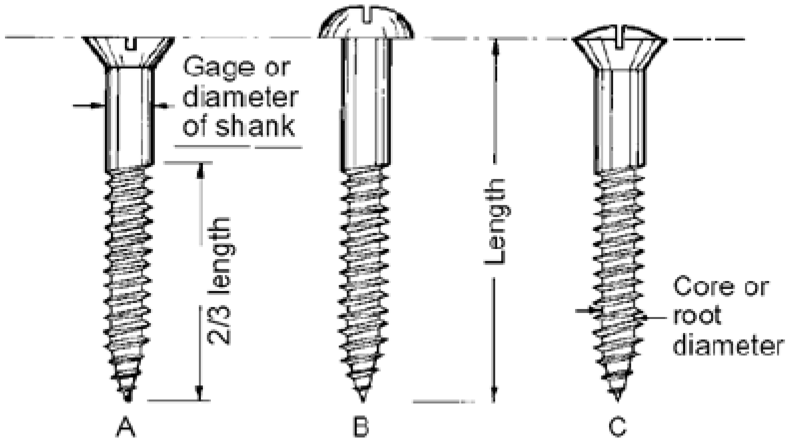


Figure 4.32 Common types of wood screws [20]

A- Flathead

B- Roundhead

C- Ovalhead

Similar to the screws also exist the bolts which are metal connectors that are installed into pre-drilled holes in the timber. The nut holds the connected elements together and the bolt shank gives a bearing surface to transfer force from the timber to the metal and vice versa. A washer under the head of the bolt and under the nut ensures that there is adequate bearing on the timber to transfer the forces from the bolt to the timber without crushing. The cost of installing bolts is much higher than that of nails. [21]

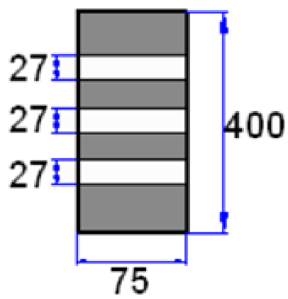


Figure 4.33 Metal bolts cross-section

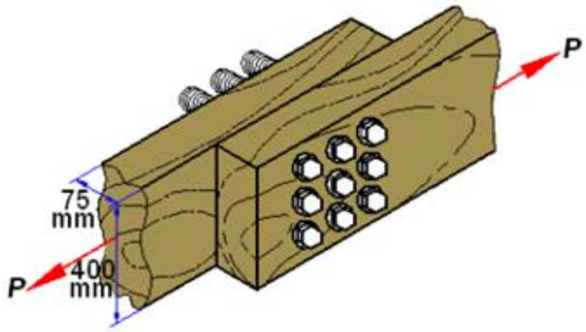


Figure 4.34 Metal bolts 3d view

4.13.2. PLATE BASED CONNECTIONS

These types of connections are also known as truss plates and are very often used in trussed rafters and joists. The stress is transfer by teeth, plugs or nails depending of the manufacturer. As a disadvantage related to the process to be installed which requires a hydraulic press or other heavy equipment but some can be installed without the heavy equipment.

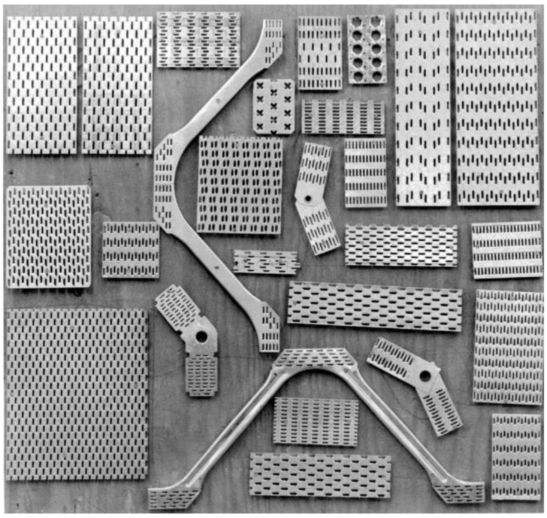


Figure 4.35 Examples of plates [22]

The strength of the connection is evaluated making the relation between slip and load for each teeth, plugs or nails. .

4.13.3. RING BASED CONNECTIONS, SPLIT-RING

The connection split-ring is able to resist to large shear forces due its bearing area however must be located in the zones of the structure where the tensions are smaller due the loss of fibers. This connections con only be chosen when the elements to connect are both timber.

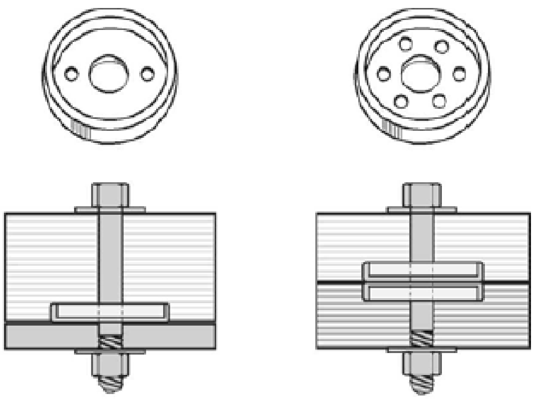


Figure 4.36 Ring connection [23]

When the connection requires putting together timber and steeling the toothed-plate is a good choice to take into account. This connection exists as single-slide toothed-plate or double-sided toothed plates as shown in the following figure 4.37 bellow. Double-sided toothed are used for non-demountable connections between timbers. Toothed-plate connections exist in various shapes as circular, square and octagonal.

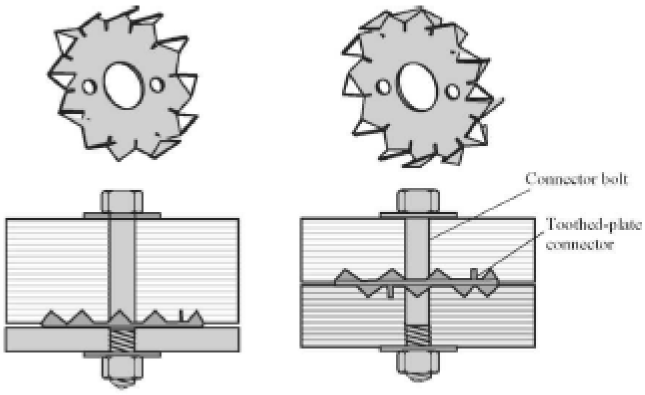


Figure 4.37 Toothed-plate connection [23]

4.13.4. CONNECTION USED IN THE PRESENT PROJECT

In this particular project the connection used to link the timber elements of the roof structure of booth variants is by nails. The following figure 4.38 and 4.39 present a single and double shear connection made by nails.

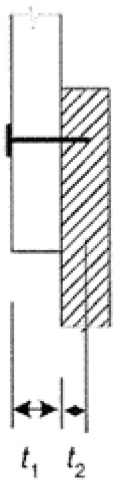


Figure 4.38 –Single shear connection [5]

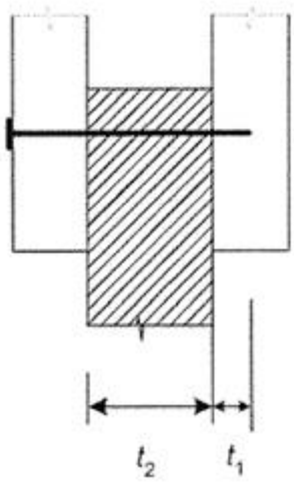


Figure 4.39 – Double shear connection [5]

Where:

$t_{1,}$ t_2 – minimum penetration size of the nail for each element

Penetration size of the nail in each timber element is fundamental to calculate its resistance to the shear force. This penetration must be at least eight times the diameter of the nail in cases of smooth nails and six times the diameter of the nail for another nails not smooth. There should be at least two nails for each connection. When the density of the timber is bigger than 500 kg/m^3 band when the diameter of the nail is bigger than 6 mm it is recommended to pre-drill the timber elements to be connected however it is not the case in this project (ρ = 340 kg / m^3). When the cross-section of the

nail has a square shape the diameter to take into account is the diameter of the square´s size. [5]
For the moment resistance, for smooth nails which can resist to a minimum value of 600 N/mm^2 in traction the yield moment resistance in characteristic value can be calculated by the following expression:

$$M_{y,Rk} = \begin{cases} 0.3\ f_u\ d^{2,6} & \text{eq. (4.41)} \\ 0.45\ f_u\ d^{2,6} & \text{eq. (4.42)} \end{cases}$$

The equation eq. (4.41) is for round nails and the equation eq. (4.42) is for square and grooved nails

Where:

$M_{y,Rk}$ - characteristic value for the yield moment

d - nail diameter

f_u - tensile strength of the wire

When the diameter of the nail is bigger than 8mm and without predrilled holes the tensile strength is calculated by the following equations:

$$f_{h,k} = 0{,}082\ \rho_k\ d^{-0,3}\ N/mm^2 \qquad \text{eq. (4.43)}$$

When the diameter of the nail is bigger than 8mm and with predrilled holes the tensile strength is calculated by the following equations:

$$f_{h,k} = 0{,}082(1-0{,}01\ d)\rho_k\ N/mm^2 \qquad \text{eq. (4.44)}$$

Where:

ρ_k - characteristic timber density

d - nail diameter

In cases which is necessary to connect three timber elements it is allowed the overlap of the nails like is shown in the figure 4.40 bellow:

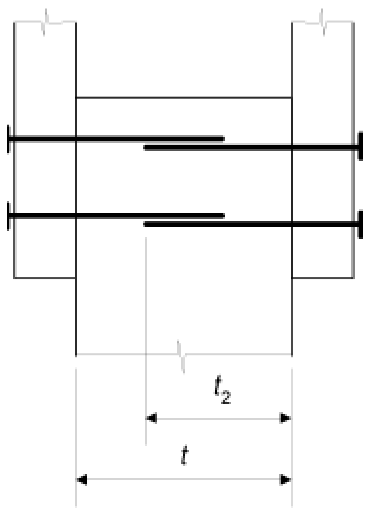


Figure 4.40 - Overlapping nails [5]

Timber should be pre-drilled when the thickness of the timber members is smaller than:

$$t = \max \begin{cases} 7d & \text{eq. (4.45)} \\ (13d - 30)\dfrac{\rho_k}{400} & \text{eq. (4.46)} \end{cases}$$

Where:

t - minimum thickness of timber member to avoid pre-drilling

ρ_k - characteristic timber density

d - nail diameter

The minimum space between the nail and the edge of the element must be taking it into account because it is important to possess a necessary effective area to distribute the stress applied in the connection. The following tables 4.14, 4.15, 4.16 and 4.17 present the various dimensions for spacing and angles in the connections by nails

Table 4.14 – Spacing parallel to grain in a row and perpendicular to grain between rows [5]

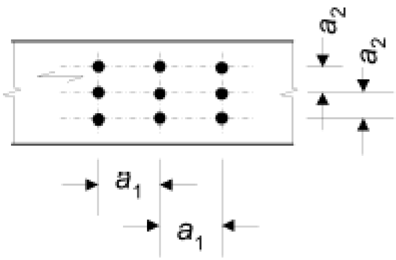
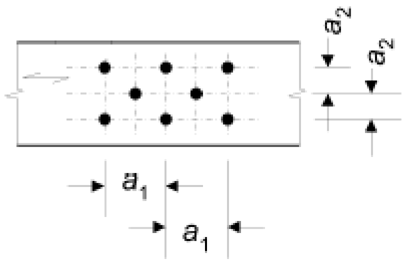
Nails in row	Nails not in row
	

Table 4.15 – Edge and end distances [5]

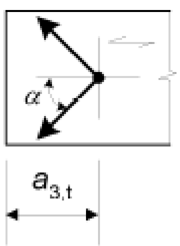
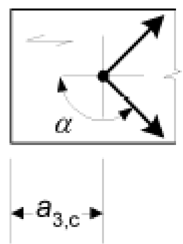
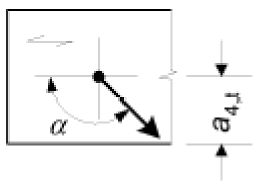
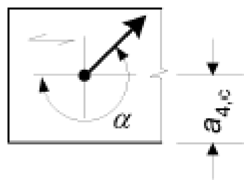
Loaded End	Unloaded End	Loaded Edge	Unloaded Edge
			
$-90° \le \alpha \le 90°$	$90° \le \alpha \le 270°$	$0° \le \alpha \le 180°$	$180° \le \alpha \le 360°$

Table 4.16 Angle between nails and edges/ends [5]

Spacing or distance	Angle α
Spacing a_1 (parallel to grain)	$0° \le \alpha \le 360°$
Spacing a_2 (perpendicular to grain)	$0° \le \alpha \le 360°$
Distance $a_{3,t}$ (loaded end)	$-90° \le \alpha \le 90°$
Distance $a_{3,c}$ (unloaded end)	$90° \le \alpha \le 270°$
Distance $a_{4,t}$ (loaded edge)	$0° \le \alpha \le 180°$
Distance $a_{4,c}$ (unloaded edge)	$180° \le \alpha \le 360°$

Table 4.17 Minimum spacing or end/edge distance [5]

Spacing or distance	Without pre-drilled holes		With pre-drilled holes
	$\rho_k \leq 420$ kg/m^3	420 kg/m^3 < $\rho_k \leq$ 500 kg/m^3	
Spacing a_1 (parallel to grain)	d < 5 mm: (5+5|cos α |)d d ≥5 mm: (5+7|cos α|)d	(7+8|cos α|)d	(4+|cos α|)d
Spacing a_2 (perpendicular to grain)	5d	7d	(3+|sin α|)d
Distance $a_{3,t}$ (loaded end)	(10+ 5 cos α) d	(15 + 5 cos α) d	(7+ 5cos α) d
Distance $a_{3,c}$ (unloaded end)	10d	15d	7d
Distance $a_{4,t}$ (loaded edge)	d < 5 mm: (5+2 sin α) d d ≥ 5 mm: (5 + 5 sin α) d	d < 5 mm: (7+2 sin α) d d ≥ 5 mm:(7 + 5 sin α) d	d < 5 mm: (3 + 2 sin α) d d ≥ 5 mm: (3 + 4 sin α) d
Distance $a_{4,c}$ (unloaded edge)	5d	7d	3d

Where:

a_1 - spacing of nails with one row parallel to grain

a_2 - spacing of rows of nails perpendicular to grain

$a_{3,c}$ - distance between nail and unloaded end

$a_{3,t}$ -distance between nail and loaded end

$a_{4,c}$ -distance between nail and unloaded edge

$a_{4,t}$ - distance between nail and loaded edge

α - angle between the force and the grain direction

To resist to permanent axil loads the nail must be threaded to offer more stability and safety to the structure. In this type of nails only the threaded side must be take into account for the resistance however the ending of the threaded nail should not be considered for the resistance capacity. The characteristic resistance for smooth nails perpendicular and slant to the grain it is defined as $F_{ax,Rk}$ and it must be consider the smaller value of the following equations.

$$F_{\mathrm{ax,R,k}} = \begin{cases} f_{\mathrm{ax,k}}\, \mathrm{d}\, \mathrm{t_{pen}} & \text{eq. (4.47)} \\ f_{\mathrm{ax,k}}\, \mathrm{d}\, \mathrm{t} + f_{\mathrm{head,k}}\, \mathrm{d_h}^2 & \text{eq. (4.48)} \end{cases}$$

The characteristic resistance for another type of nails but not smooth nails perpendicular and slant to the grain it is defined as $F_{ax,Rk}$ and it must be consider the smaller value of the following equation.

$$F_{\mathrm{ax,R,k}} = \begin{cases} f_{\mathrm{ax,k}}\, \mathrm{d}\, \mathrm{t_{pen}} & \text{eq. (4.49)} \\ f_{\mathrm{ax,k}}\, \mathrm{d_h}^2 & \text{eq. (4.50)} \end{cases}$$

Where:

$f_{ax,k}$ - characteristic pointside withdrawal strength

$f_{head,k}$ - characteristic headside pull-through strength

d - nail diameter

t_{pen} - pointside penetration length or the length of the threaded part in the pointside member

t - thickness of the headside member

d_h - nail head diameter

For smooth nails with a pointside penetration of at least $12d$, the characteristic values of the withdrawal and pull-through strengths should be found from the following expressions[5]

$$f_{\mathrm{ax,k}} = 20 * 10^{-6}\, \rho_{\mathrm{k}}^{2} \qquad \text{eq. (4.51)}$$

$$f_{\mathrm{head,k}} = 70 * 10^{-6}\, \rho_{\mathrm{k}}^{2} \qquad \text{eq. (4.52)}$$

Where:

ρ_k - characteristic timber density

[5] Some rules to take into account:

- For smooth nails, the pointside penetration t_{pen} should be at least 8d

- For nails with a pointside penetration smaller than 12d the withdrawal capacity should be multiplied by (t_{pen}4d− 2)

- For threaded nails, the pointside penetration should be at least 6d

- For nails with a pointside penetration smaller than 8d the withdrawal capacity should be multiplied by (t_{pen}/2d − 3).

- For structural timber which is installed at or near fiber saturation point, and which is likely to dry out under load, the values of $f_{ax,k}$ and $f_{head,k}$ should be multiplied by 2/3

- For slant nailing the distance to the loaded end should be at least 10d. There should be at least two slant nails in a connection

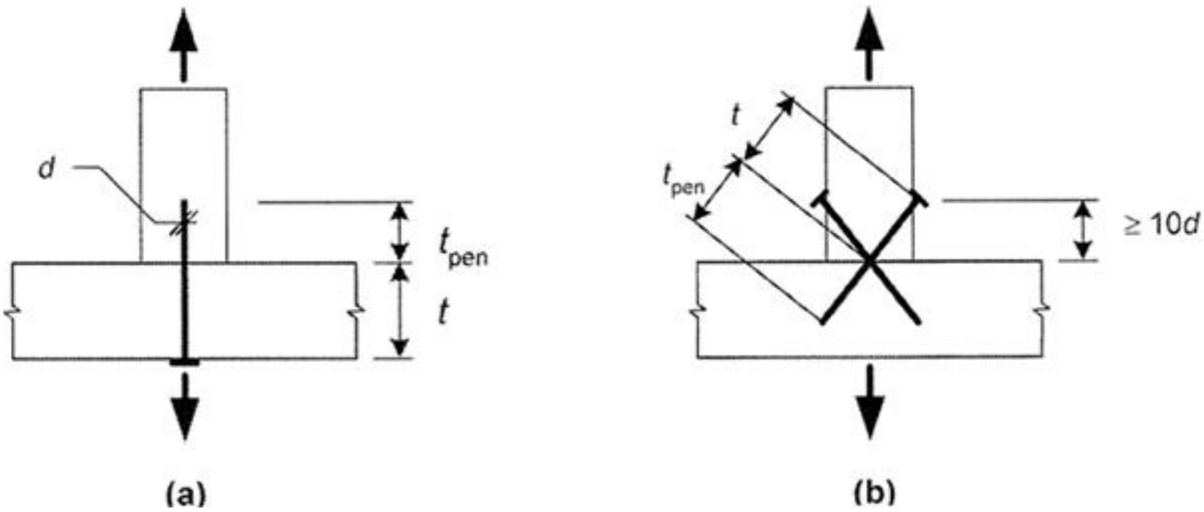


Figure 4.41 – (a) Nailing perpendicular to grain and (b) slant nailing [5]

For smooth nails when the load is combined in lateral and axial loads the following condition must be verify:

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1 \qquad \text{eq. (4.53)}$$

For other than smooth nails when the load is combined in lateral and axial loads the following condition must be verify

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1 \qquad \text{eq. (4.54)}$$

Where:

$F_{ax,Ed}$ - axial load applied to the nail

$F_{v,Ed}$ - lateral load applied to the nail

4.14. GLUE LAMINATED BEAMS

Glulam beam consist in a group of several timber layers (lamellas) glued one to each other, it´s very common in Western Europe and USA and it allows the designer to explore bigger spans and several shapes of the elements which can give a very good looking for the structure with arcs, break arcs, different kind of curves among other options. Glulam beam is 20% lighter than a steel beam and 600% than a reinforcement concrete beam which makes the construction more economical related to the foundations, transport and erection. Glulam is also efficient in energy due its good quality as insulator and delete the risk of the common problem known as “thermal bridges” which allows to reduce the fuel necessary to heat the spaces inside of the house. Another advantage for using glulam it´s the good resistance for biological attacks what makes the glulam an often choice.

Figure 4.42 Bridge in Sneek (Netherlands) [24]

4.14.1. Fabrication

The fabrication process of glulam timber elements begins with the creation of a sustainable forest to ensure the growth exceeds harvested quantities. From the tree the structural elements are cut proceeding for a controlled drying until the moisture reaches 12 % and then planed. When the wood is cut there is a special attention to the knots and other wood-defects that must be cut it out. This defects existence in wood decrease its mechanical properties of resistance and must be deleted. Then the elements are submitted for a strength grading to check its mechanical properties in such way that the expected resistance can be verified as its visual appearance. Once strength grading is conclude finger joints are made connecting the elements and glued them together. After the glue is dry and the finger joints have been hardened the lamellas are planed until it reaches the thicknesses of 45 mm and then the glue is applied on the broader edge of the lamella and joined to the other lamellas to be pressed and curing in strait line or curved line depending of the design. After the glue lines the edges of the glulam timber elements are hardened and then planed.

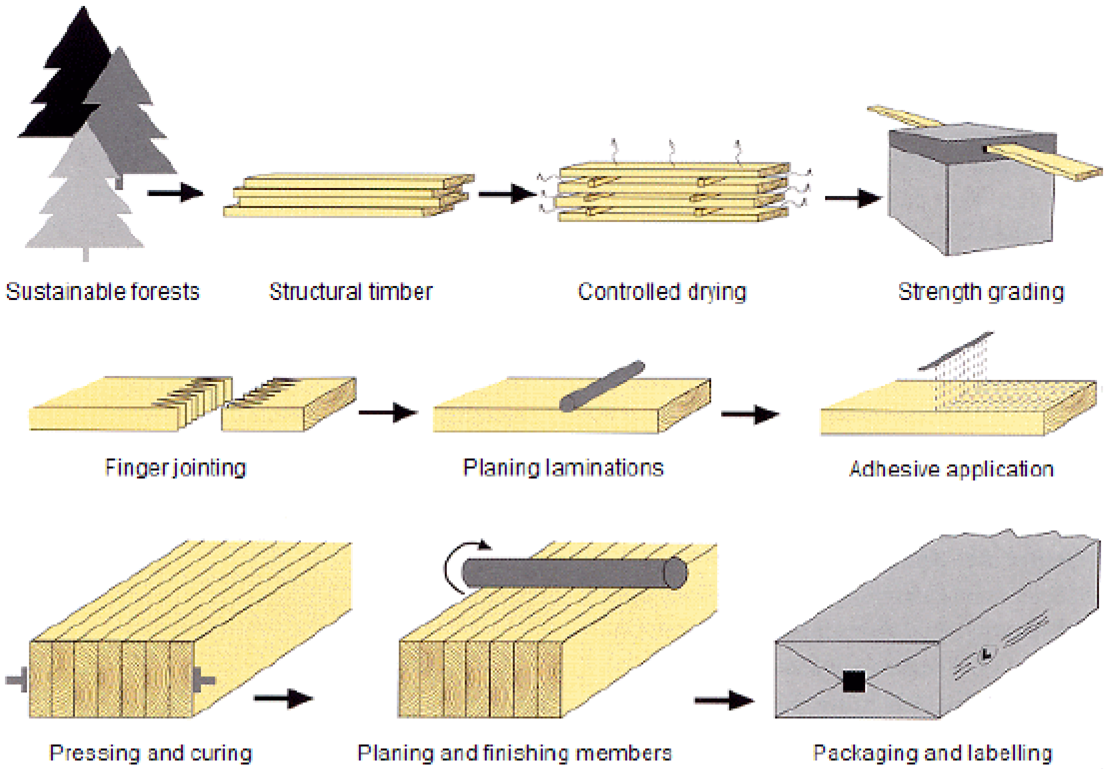


Figure 4.43 Glulam timber cycle [25]

4.14.2. Design

Lamellas can be glued in vertical or horizontal way and in different types of timber as shown in the figures 4.44 and 4.45 bellow:

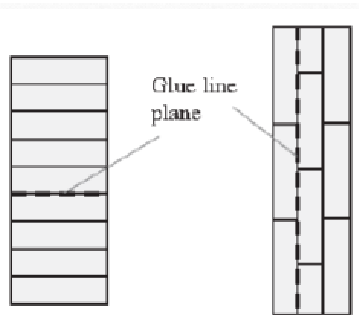


Figure 4.44 Horizontal and vertical glulam cross-section

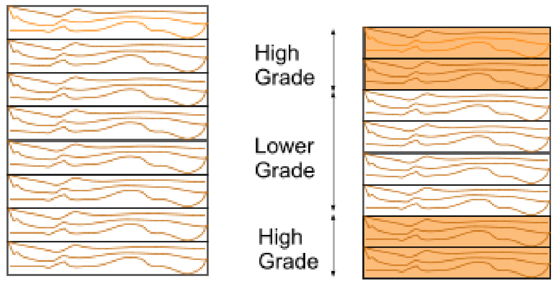


Figure 4.45 Glued Laminate cross-section in same class and different classes [26]

The following table 4.18 presents the properties of glulam timber. For example the glulam timber GL24h is the values that are related to the characteristic resistance to bending moment parallel to the grain with 24 MPa of strength.

Table 4.18 Properties of glulam [15]

Properties	**Glulam strength class**	**Homogeneous glulam**			**Combined glulam**		
		GL24h	**GL28h**	**GL32h**	**GL24c**	**GL28c**	**GL32c**
Bending (N/mm^2)	$f_{m,g,k}$	24	28	32	24	28	32
Tension (N/mm^2)	$f_{t,0,g,k}$	16,5	19,5	22,5	14	16,5	19,5
	$f_{t,90,g,k}$	0,4	0,45	0,5	0,35	0,4	0,5
Compression (N/mm^2)	$f_{c,0,g,k}$	24	26,5	29	21	24	29
	$f_{c,90,g,k}$	2,7	3	3,3	2,4	2,7	3,3
Shear (N/mm^2)	$f_{v,g,k}$	2,7	3,2	3,8	2,2	2,7	3,8
Modulus of Elasticity (KN/mm^2)	$G_{0,g,mean}$	11,6	12,6	13,7	11,6	12,6	13,7
	$G_{0,g,05}$	9,4	10,2	11,1	9,4	10,2	11,1
	$G_{90,g,mean}$	0,39	0,42	0,46	0,32	0,39	0,46
Shear Modulus (KN/mm^2)	$G_{g,mean}$	0,72	0,78	0,85	0,59	0,72	0,85
	$\rho_{g,k}$	380	410	430	350	380	430

The characteristic bending strength and the tensile strength, $f_{m,g,k}$ and $f_{t,0,g,k}$, is for cross-sections with a minimum depth and thickness of 600 mm and 150 mm respectively. However it is possible to increase this resistance for the cases in which the cross-section of the glulam timber is smaller than the referred. For this Eurocode has a factor for size effect that only ca be applied to the depth for bending and the width for tension and makes no adjustment for thickness.

$$K_{\mathrm{h}} = \min \begin{cases} \left(\dfrac{600}{\mathrm{h}}\right)^{0,1} \\ 1,1 \end{cases} \qquad \text{eq. (4.55)}$$

In order to verify the safety of the timber structures it is necessary to verify the following conditions:

$$\sigma_{\mathrm{f,c,max,d}} \leq f_{\mathrm{m,d}} \qquad \text{eq.(4.56)}$$

$$\sigma_{\mathrm{f,t,max,d}} \leq f_{\mathrm{m,d}} \qquad \text{eq.(4.57)}$$

$$\sigma_{\mathrm{f,c,d}} \leq k_{\mathrm{c}}\, f_{\mathrm{c,0,d}} \qquad \text{eq.(4.58)}$$

$$\sigma_{\mathrm{f,t,d}} \leq f_{\mathrm{t,0,d}} \qquad \text{eq.(4.59)}$$

Where:

$\sigma_{\mathrm{f,c,max,d}}$ - extreme fiber flange design compressive stress

$\sigma_{\mathrm{f,t,max,d}}$ - extreme fiber flange design tensile stress

$\sigma_{\mathrm{f,c,d}}$ - mean flange design compressive stress

$\sigma_{\mathrm{f,t,d}}$ - mean flange design tensile stress

k_{c}- factor which takes into account lateral instability

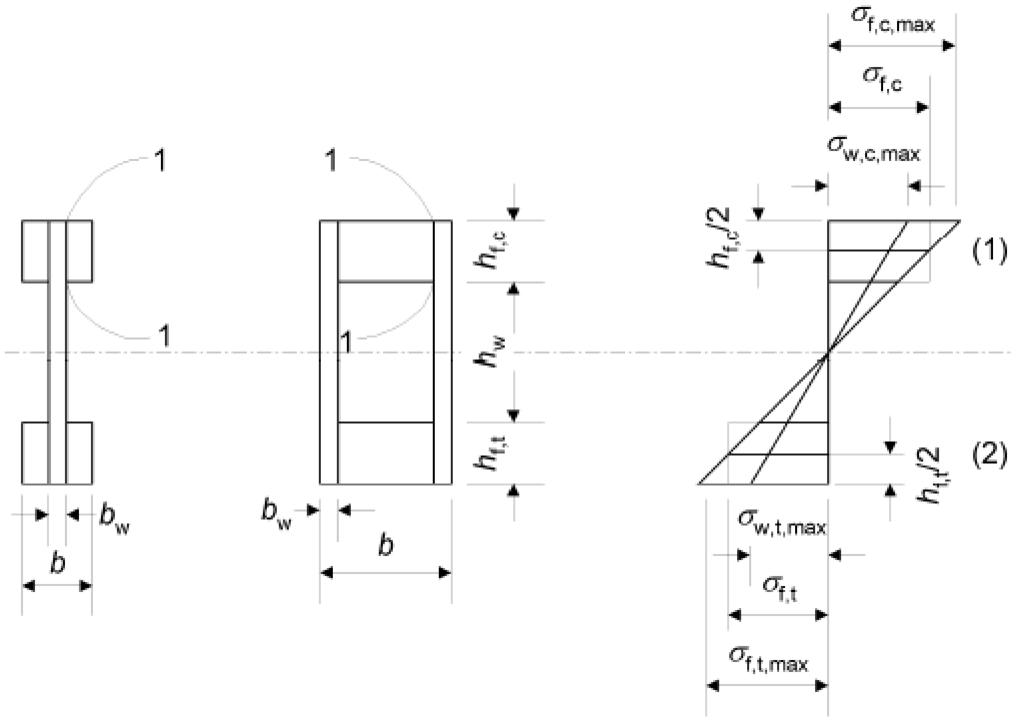


Figure 4.46 Glulam timber cross-section [5]

Where:

(1)- compression

(2)- tension

5
LOADS ACCORDING TO EUROCODE 1 (EC1)

5.1. SNOW

In countries like Poland, snow is an important load to be considered in the calculations. According to Eurocode 1 it is necessary to take into account a persistent design situation and a transient design situation for this load. The persistence design case is related to the snow without suffering any slips (staying in the same position) and the transient design situation is related to the slipping snow which takes different forms after moving from the initial position. For both cases the loads on roofs shall be determined as follows:

$$s = u_i . C_e . C_t . s_K \qquad \text{eq. (5.60)}$$

Where:

u_i is the snow load shape coefficient

s_K is the characteristic value of snow load on tile ground

C_e is the exposure coefficient

C_t is the thermal coefficient (=1,0)

The exposure coefficient C_e should be used for determining the snow load on the roof. The choice for C_e should consider the future development around the site. C_e should be taken as 1,0 unless otherwise specified for different topographies.

Table 5.19 Recommended values of C_e for different topographies [4]

Topography	C_e
Windswept	0,8
Normal	1,0
Sheltered	1,2

Windswept topography: flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees.

Normal topography: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.

Sheltered topography: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.

In this case, the value C_e =1,0 should be considered.

Table 5.20 Snow load shape coefficients [4]

Angle of pitch of roof "α"	$0^0 \leq \alpha \leq 30^0$	$30^0 \leq \alpha \leq 60^0$	$\alpha \geq 60^0$
u_1	0,8	0,8(60 - α)/30	0,0
u_3	0,8 + 0,8 α /30	1,6	–

In this particular project the angle of the roof, in both variants, is 40^0 corresponding in the table to the interval $30^0 \leq \alpha \leq 60^0$

Figure 5.47 Snow Map of Poland [4]

Table 5.21 s_K values [4]

Zone	$s_K \quad KN/m^2$
1	0,007 A - 1,4; $s_K \geq 0,7$
2	0,9
3	0 ,006A - 0,6; $s_K \geq 1,2$
4	1,6
5	0,93exp(O,00134A); $s_K \geq 2,0$
NOTE: A = Site altitude above sea level (m)	

In this particular project the house is located in Warsaw corresponding to zone 2 and following the table it is possible to conclude that $s_K = 0,9\,KN/m^2$

5.2. WIND

Wind is generated by variations of temperature in atmospheric masses. For example when the cold air crosses with the hot air, wind is created. Wind can apply different types of actions in the structures: pressure and suction. The pressure occurs when the wind acts in the forefront of the building and makes pressure in the same direction and way from which the wind came from. However, suction occurs when the wind acts on a forefront of the building but, due to its contour the force is applied in the same direction but on opposite way from which the wind came from. The wind always acts perpendicular to the surface which is facing.

The pressure applied by the wind on buildings is given by the following equation:

$$w_e = c_{pe} q_p (z_e) \qquad \text{eq. (5.61)}$$

Where:

z_e - reference heights for the wind pressure

c_{pe} - pressure coefficient

q_p - maximum value of dynamic pressure

The dynamic pressure is determinate by:

$$q_b = \frac{1}{2}\rho V_b^{\ 2} \qquad \text{eq. (5.62)}$$

Where:

ρ - air density (=1,25 Kg/ m^2)

The reference value for the wind velocity is given by the following equation:

$$V_b = c_{dir} c_{season} V_{b,0} \qquad \text{eq. (5.63)}$$

Where:

$V_{b,0}$ - characteristic value of wind velocity (=22 m/s)

c_{season} - season coefficient (=1)

c_{dir} - Coefficient depending on the wind direction (=1)

The average of wind speed at a certain height is determined by the following expression:

$$V_m(z) = c_r(z) c_0(z) V_b \qquad \text{eq. (5.64)}$$

Where:

$c_0(z)$ - Coefficient depending on the topography of the land (=1)

$c_r(z)$ - Roughness coefficient (=0,84)

The reference value of dynamic pressure is given by:

$$q_p(z) = c_e(z) q_b \qquad \text{eq. (5.65)}$$

$c_e(z)$ - Exposure coefficient is taken from the following graphic (=2, category III)

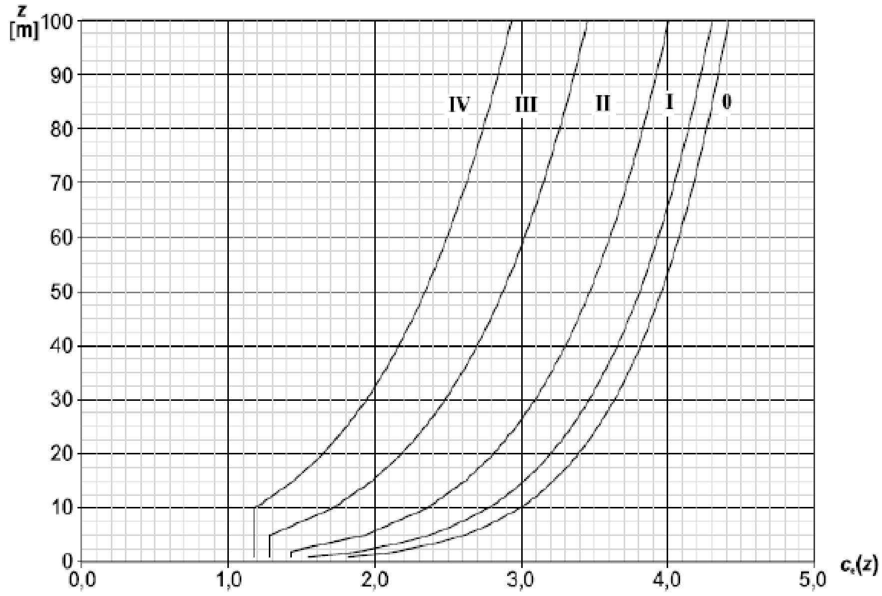


Figure 5.48 Exposure categories coefficients [4]

6
STRUCTURAL ANALYSIS

6.1. First Variant

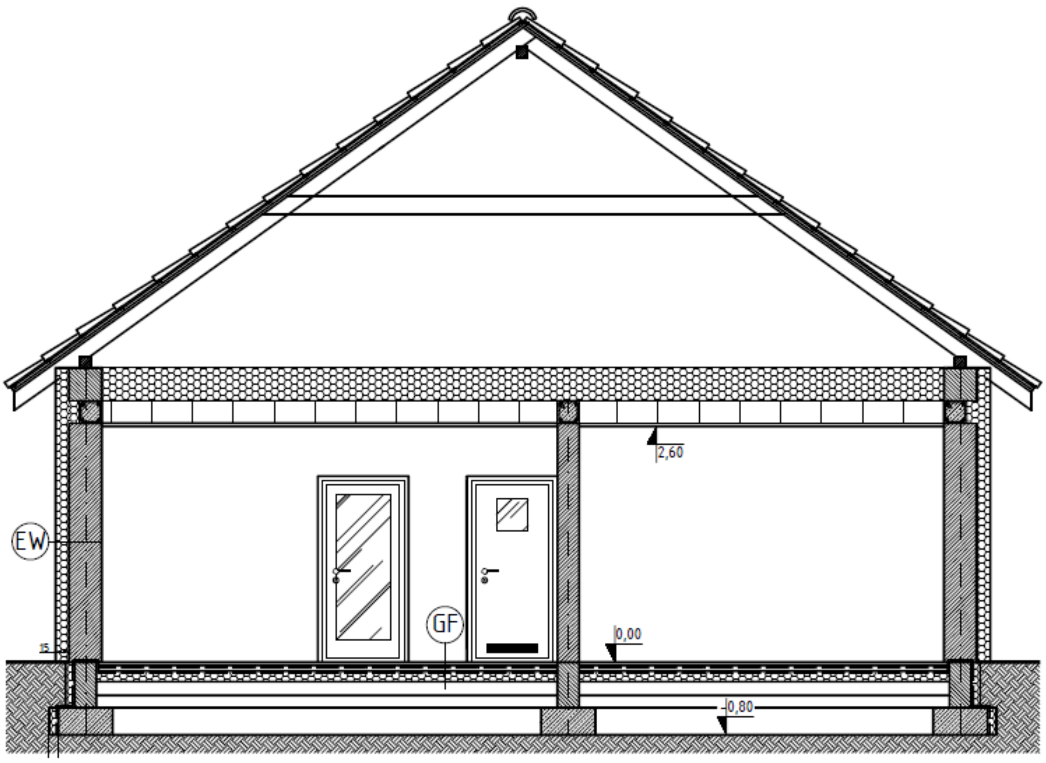


Figure 6.49 Horizontal Extension, First Variant, Project

6.1.1. Ultimate limit states

For the ultimate limit states the respective coefficients are applied to the own weight and coverage, snow, wind and variable forces according to Eurocodes as it is possible to verify on the table 4.12

Table 6.22 Balance of loadings ULS, First variant of roof structure

Load	Characteristic Value (KN/m)	Load Factor (γ_M)	Design Value (KN/m)	Vertical Components Load Values (KN/m)		Horizontal Components Load Values (KN/m)	
				Characteristic Value	Design Value	Characteristic Value	Design Value
Own weight and coverage	0,8	1,35	1,08	0,613	0,827	0,514	0,694
Snow	0,19	1,5	0,285	0,146	0,218	0,122	0,183
Wind	0,065	1,5	0,097	0,050	0,074	0,0418	0,063
Variable	0,5	1,5	0,750	0,383	0,575	0,322	0,482
Total	1,9		2,212	1,192	1,695	1,0	1,422
Concentrated load	1,00	1,5	1,50	0,766	1,149	0,643	0,965

6.1.1.1. Roof battens

Below are shown the critical combinations for the second variant of the roof battens (w-1). On the right combination it was considered the concentrated load + the permanent loads and on the left side it was considered the permanent + variable loads

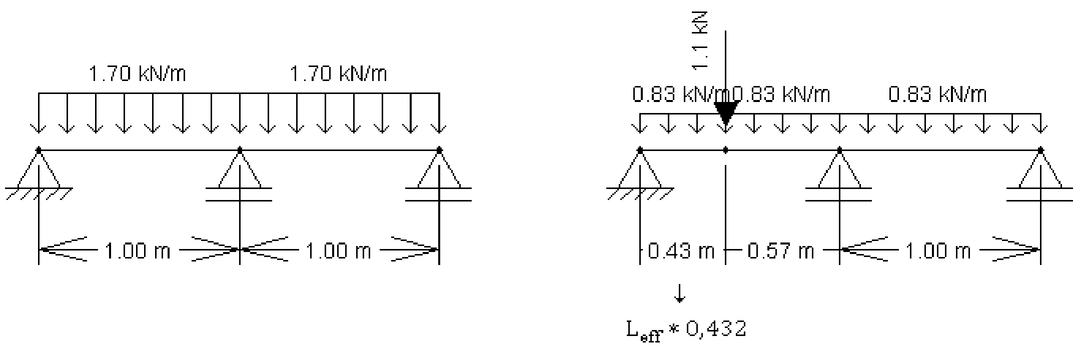


Figure 6.50 Roof Battens, ULS Loads

Note: $L_{eff} * 0.432$ is an approximation of the distance where the concentrated force can be applied to generate the maximum bending moment in this case.

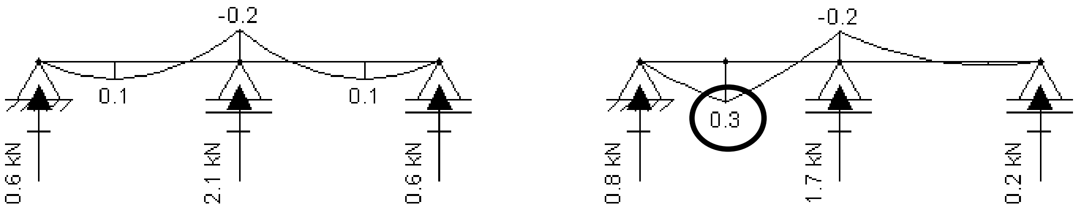


Figure 4.51 Roof Battens, ULS Bending Moment

$$W_y = \frac{b.h^2}{6} = \frac{0,05 * 0,02^2}{6} = 0,0000333\ m^3 \qquad \text{eq. (4.66)}$$

Following eq. 4.15:

$$\sigma_{m,y,d} = \frac{M_y}{W_y} = \frac{0,3}{0,0000333} = 9009,01\ \text{KPa} = 9,009\ \text{MPa} \qquad \text{eq. (4.15)}$$

(Softwood)

$$f_{m.y.d} = \frac{K_{mod} * f_{m.y.k}}{\gamma_M} = \frac{0,7 * 22}{1,3} = 11,85\ MPa \qquad \text{eq. (4.67)}$$

6.1.1.2. Roof beams, Bending, Axial

According to Eurocode 5, following the eq. 4.14:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq.(4.14)}$$

$$\frac{9,009}{11,85} + 0 = 0,76 < 1$$

Below are shown the critical combination for the first variant of the roof structure, on the right side there are just the permanent loads and on the left side there are the permanent + variable + concentrated loads.

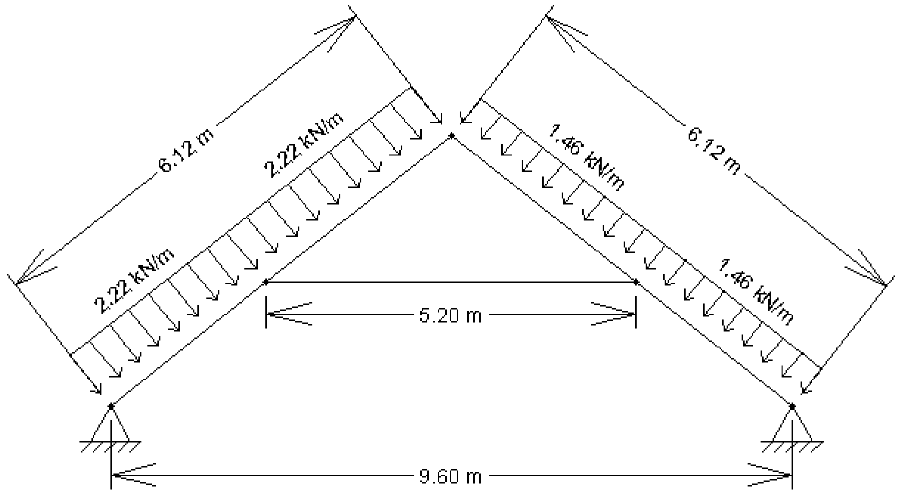


Figure 6.21 Loads ULS, First Variant

Note: The diagrams were obtained from the software Ftool. The forces were introduced with horizontal and vertical components like calculated before but the software (Ftool) represents them the oblique forces.

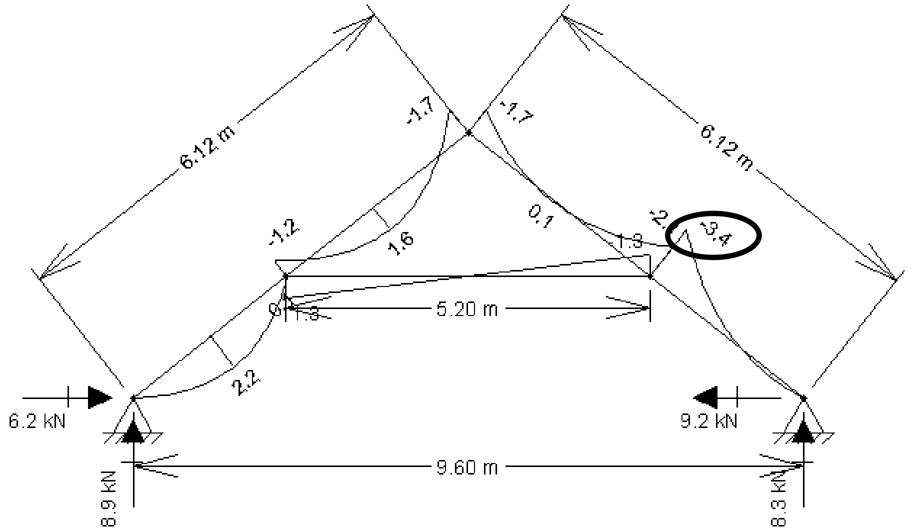


Figure 6.53 Diagram of Bending Moment ULS, First Variant

$$M = 3.4 \text{ KNm}$$

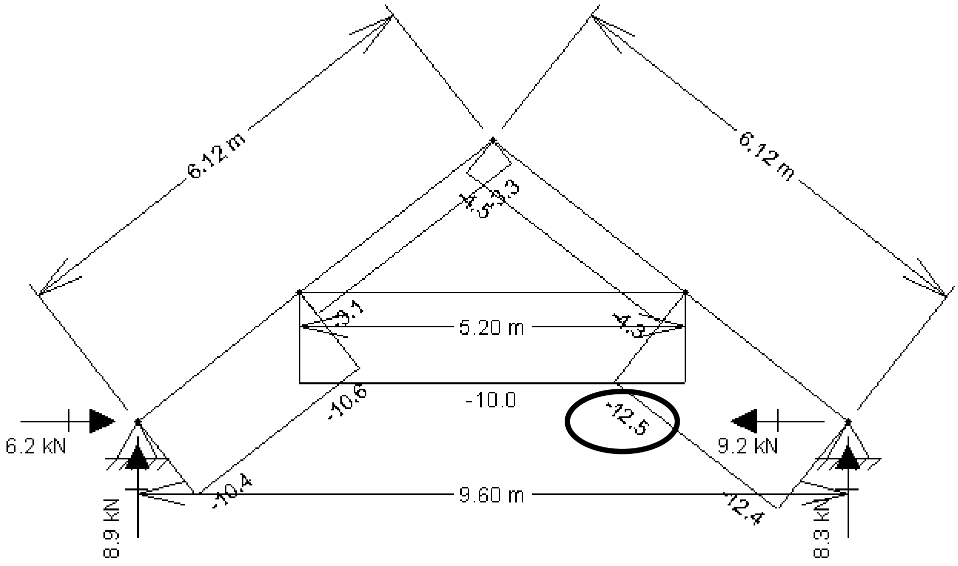


Figure 6.54 Diagram of Axial Force ULS, First Variant

$$\mathrm{N} = 12.5\ \mathrm{KN}\ (\text{Compression})$$

$$W_y = \frac{b.h^2}{6} = \frac{0{,}08 * 0{,}2^2}{6} = 0{,}000533\ m^3 \qquad \text{eq. (4.68)}$$

$$W_z = \frac{h.b^2}{6} = \frac{0{,}20 * 0{,}08^2}{6} = 0{,}0002133\ m^3 \qquad \text{eq. (4.69)}$$

Following eq.4.15:

$$\sigma_{m,y,d} = \frac{M_y}{W_y} = \frac{3{,}4}{0{,}000533} = 6378{,}98\ \mathrm{KPa} = 6{,}3789\ \mathrm{MPa} \qquad \text{eq. (4.15)}$$

$$\sigma_{c,0,d} = \frac{N}{A} = \frac{12{,}5}{0{,}08 * 0{,}2} = 781{,}25\ \mathrm{KPa} = 0{,}7813\ \mathrm{MPa} \qquad \text{eq. (4.70)}$$

Table 6.23 K_{mod} values [5]

Load class for the duration of solid wood, laminated and plywood.	**Class of Use**		
	1	**2**	**3**
Permanent	0,6	0,6	0,5
Long-Term	0,7	0,7	0,55
Medium-Term	0,8	0,8	0,65
Short-Term	0,9	0,9	0,7
Instantaneous	1,1	1,1	0,9

$$f_{c.0.d} = \frac{K_{mod} * f_{c.0.k}}{\gamma_M} = \frac{0,7 * 20}{1,3} = 10,76\ MPa \qquad \text{eq. (4.71)}$$

$$f_{m.y.d} = \frac{K_{mod} * f_{m.y.k}}{\gamma_M} = \frac{0,7 * 22}{1,3} = 11,85\ MPa \qquad \text{eq. (4.67)}$$

For the situation of bending and compression according to Eurocode 5, following eq. 4.29:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.29)}$$

$$\left(\frac{0,7813}{10,76}\right)^2 + \frac{6,3789}{11,86} + 0 = 0,5431 < 1$$

6.1.1.3. <u>Lateral Buckling</u>

$$\lambda_{rel,m} \leq 0,75$$

For the respective slenderness value it comes $K_{crit} = 1$

$$I = \frac{0{,}04.(0{,}2)^3}{12} = 0{,}000027\text{m}^4 \qquad \text{eq. (4.72)}$$

$$\sigma_{m,d} = \frac{3.4}{0{,}000027} * 0{,}1 + \frac{12.5}{0{,}2 * 0{,}04} = 14.15 \text{ MPa} \qquad \text{eq. (4.73)}$$

$$K_{crit} * f_{m,d} = \frac{22}{1{,}3} = 16.93 \text{ MPa} > 14{,}15 \text{ MPa} \qquad \text{eq. (4.33)}$$

6.1.2. SERVICEABILITY LIMIT STATES

According to the table 4.12 it is concluded that $u_{net,fin} = \frac{L}{200}$

$$u_{net,fin} = \frac{L}{200} = \frac{270}{200} = 13{,}5 \; mm \qquad \text{eq. (4.74)}$$

Table 6.24 Load & K_{def}, u_{inst}, u_{fin} values [5]

Load	K_{def}	Components of Load [mm]	
		u_{inst}	u_{fin}
Weight own (class load duration = permanent, the class of use = 2)	0,8	3,1	5,58
snow (Class duration load = medium-term, class of use = 2)	0,25	1,8	2,25
wind (Class duration load = short-term, class of use = 2)	0	1,2	1,2
Deflections summary $u_{fin} = u_{fin1} + u_{fin2} + u_{fin3}$		9,03	

$$u_{fin} = 9{,}03 \text{ mm} < u_{net,fin} = 13{,}5 \text{ mm} \qquad \text{eq. (4.75)}$$

6.1.3. CONNECTIONS

The nails used in this particular project are threaded, pre-drilled, M24 nails from 4.6 class (S275). In each connection two nails are used.

$$f_{u,k} = 400\text{N/mm}^2 \quad f_{y,k} = 240\text{N/mm}^2$$

a) Connections 1 – Fisrt Variant

In this particular connection there is two timber beams with cross-section sizes of 8x20 cm connecting an oblique timber beam also with the cross-section sizes of 8x20 cm like is shown in the figure 6.55. There is four nails, two in one side and another two in the opposite side.

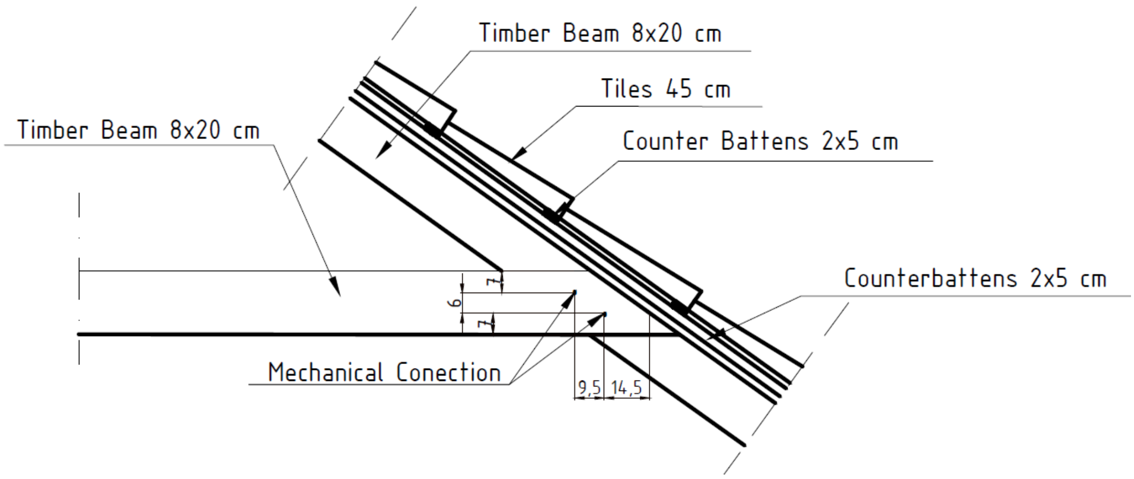


Figure 6.55 Connection 1, First Variant

$$f_{h,k} = 0{,}082(1\text{-}0{,}01\ d)\rho_k\ \text{N/mm}^2 \qquad \text{eq. (4.44)}$$

$$f_{h,k} = 0{,}082(1\text{-}0{,}01{*}24)340 = 21{,}19\ \text{N/mm}^2 \qquad \text{eq. (8.44)}$$

$$M_{y,Rk} = 0.3\ f_u\ d^{2,6} = 0{,}3 * 400 *\ 24^{2,6} = 465297\ \text{Nmm} \qquad \text{eq. (4.41)}$$

$$M_{y,Rk} = 465297\ \text{Nmm}$$

$$f_{head,k} = 70 * 10^{-6}\ \rho_k{}^2 = 70 * 10^{-6}\ 340^2 = 8{,}092\ \text{N/mm}^2 \qquad \text{eq. (4.52)}$$

$$f_{\mathrm{ax,k}} = 0{,}52 * \mathrm{d}^{-0{,}5}\, l_{\mathrm{eff}}{}^{-0{,}1}\, \rho_{\mathrm{k}}{}^{0{,}8} = 0{,}52 * 24^{-0{,}5} * 60^{-0{,}1} * 340^{0{,}8} = 7{,}46\ \mathrm{N/mm^2}$$

$$F_{\mathrm{ax,R,k}} = \begin{cases} f_{\mathrm{ax,k}}\, \mathrm{d}\, \mathrm{t_{pen}} = 7.46 * 24 * 60 = 10742{,}4\ \mathrm{N} & \text{eq. (4.50)} \\ f_{\mathrm{ax,k}}\, \mathrm{d_h}{}^2 = 7{,}46 * 30^2 = 6714\ \mathrm{N} & \text{eq. (4.49)} \end{cases}$$

Resistance of the nail to the shear force:

$$\mathrm{F_{v,Rd}} = \frac{\mathrm{K} * f_{\mathrm{ub}} * A_{\mathrm{s}}}{\gamma_{\mathrm{m}}} = \frac{0.6 * 400 * 452{,}16}{1{,}25} = 86814{,}72\ N$$

$$\mathrm{F_{v,Ed}} = 3.3\ \mathrm{KN}$$

$$\frac{\mathrm{F_{ax,Ed}}}{\mathrm{F_{ax,Rd}}} + \frac{\mathrm{F_{v,Ed}}}{\mathrm{F_{v,Rd}}} = \frac{(3.3 * 10^3)/2}{86814.72} = 0.02 \leq 1 \qquad \text{eq. (4.53)}$$

The distances and angles from the nails to the edges/ends of the timber beams were checked according to the tables 4.14, 4.15, 4.16 and 4.17. It was concluded that all the distances and angles are in accordance with the limits.

b) Connection 2 – First Variant

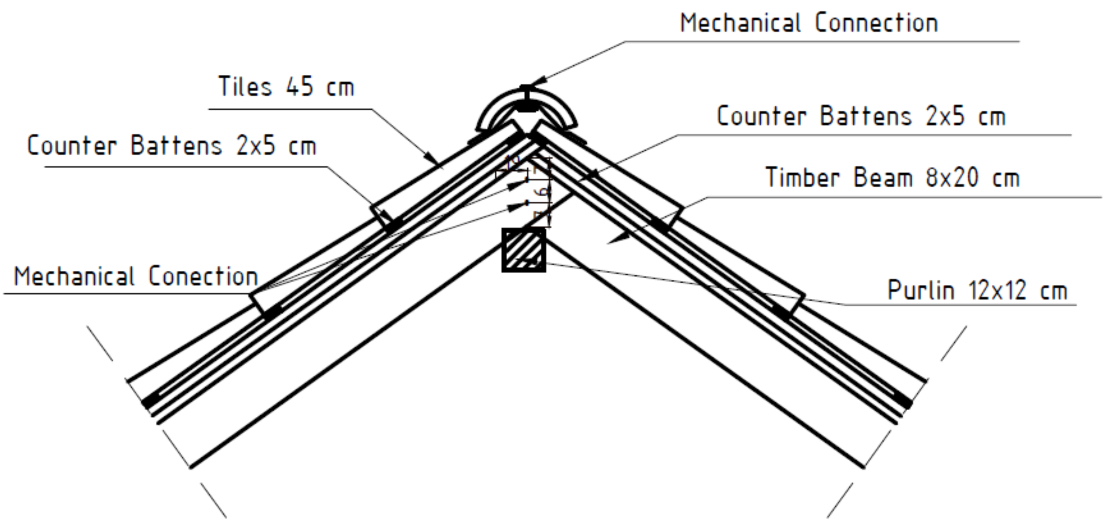


Figure 6.56 Connection 2, First Variant

$$f_{h,k} = 21{,}19 \text{ N/mm}^2$$

$$M_{y,Rk} = 465297 \text{ Nmm}$$

$$f_{head,k} = 8{,}092 \text{ N/mm}^2$$

$$f_{ax,k} = 7{,}46 \text{N/mm}^2$$

$$F_{ax,R,k} = 6714 \text{ N}$$

$$F_{v,Rd} = 86814{,}72\ N$$

$$F_{v,Ed} = 2.3 \text{ KN}$$

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{(2.3 * 10^3)}{86814.72} \leq 1 \qquad \text{eq. (4.53)}$$

The distances and angles from the nails to the edges/ends of the timber beams were checked according to the tables 4.14, 4.15, 4.16 and 4.17. It was concluded that all the distances and angles are in accordance with the limits.

6.2. SECOND VARIANT

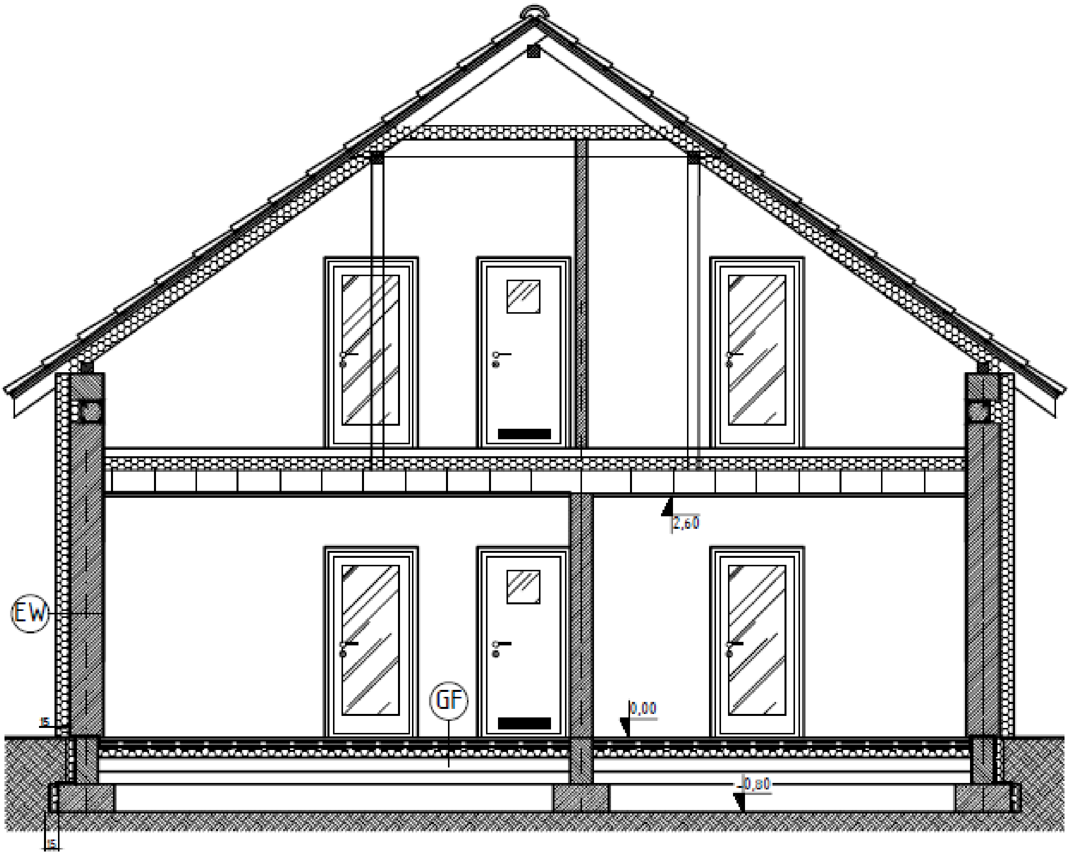


Figure 6.57 Vertical Extension, Second Variant, Project

6.2.1. ULTIMATE LIMIT STATES

For the ultimate limit states the respective coefficients are applied to the own weight and coverage, snow, wind and variable forces according to Eurocodes as it is possible to verify on the table 4.12

Table 6.25 Balance of loadings ULS, Second variant of roof structure

Load	Characteristic Value (KN/m)	Load Factor (γ_M)	Design Value (KN/m)	Vertical Components Load Values (KN/m)		Horizontal Components Load Values (KN/m)	
				Characteristic Value	Design Value	Characteristic Value	Design Value
Own weight and coverage	0,8	1,35	1,08	0,613	0,827	0,514	0,694
Snow	0,19	1,5	0,285	0,146	0,218	0,122	0,183
Wind	0,065	1,5	0,097	0,050	0,074	0,0418	0,063
Variable	0,5	1,5	0,750	0,383	0,575	0,322	0,482
Total	1,9		2,212	1,192	1,695	1,0	1,422
Concentrated load	1,00	1,5	1,50	0,766	1,149	0,643	0,965

6.2.1.1. Roof battens

Below is shown the critical combinations for the second variant of the roof battens (w-2). On the right combination it was considered the concentrated load + the permanent loads and on the left side it was considered the permanent + variable loads

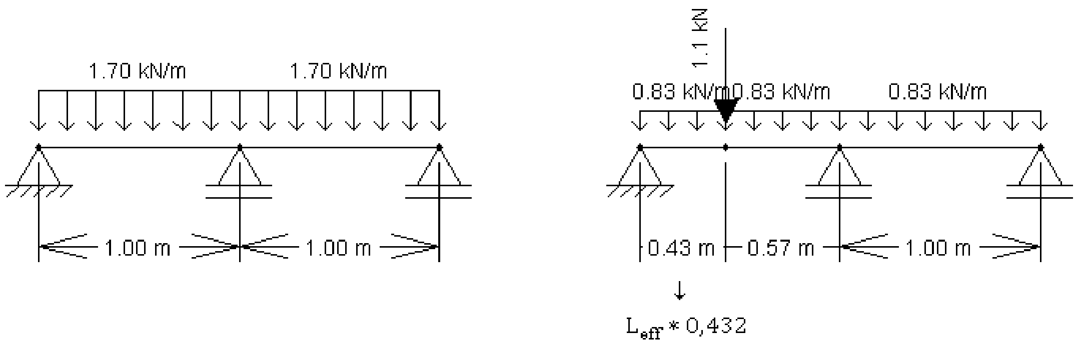


Figure 6.58 Roof Battens, ULS Bending Moment

Note: $L_{eff} * 0.432$ is an approximation of the distance where the concentrated force can be applied to generate the maximum bending moment in this case.

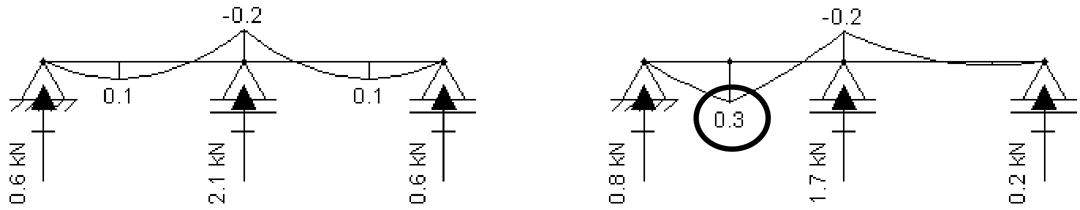


Figure 6.59 Roof Battens, ULS Bending Moment

$$W_y = \frac{b.h^2}{6} = \frac{0{,}05 * 0{,}02^2}{6} = 0{,}0000333\ m^3 \qquad \text{eq. (4.76)}$$

Following eq.4.15:

$$\sigma_{m,y,d} = \frac{M_y}{W_y} = \frac{0{,}3}{0{,}0000333} = 9009{,}01\ KPa = 9{,}009\ MPa \qquad \text{eq. (4.15)}$$

(Softwood)

$$f_{m.y.d} = \frac{K_{mod} * f_{m.y.k}}{\gamma_M} = \frac{0{,}7 * 22}{1{,}3} = 11{,}85\ MPa \qquad \text{eq. (4.67)}$$

According to Eurocode 5, following eq.4.14:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.67)}$$

$$\frac{9{,}009}{11{,}85} + 0 = 0{,}76 < 1$$

,

6.2.1.2. Roof beams, Bending, Axial

Below is shown the critical combination for the first variant of the roof structure, on the right side there are just the permanent loads and on the left side there are the permanent + variable + concentrated loads

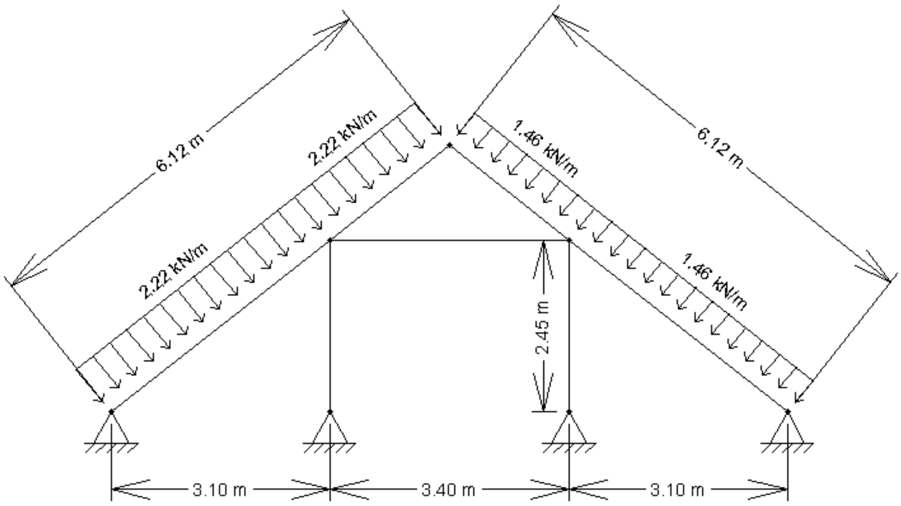


Figure 6.60 Loads ULS, Second Variant

Note: The diagrams were obtained from the software Ftool. The forces were introduced with horizontal and vertical components like calculated before but the software (Ftool) represent them like oblique forces.

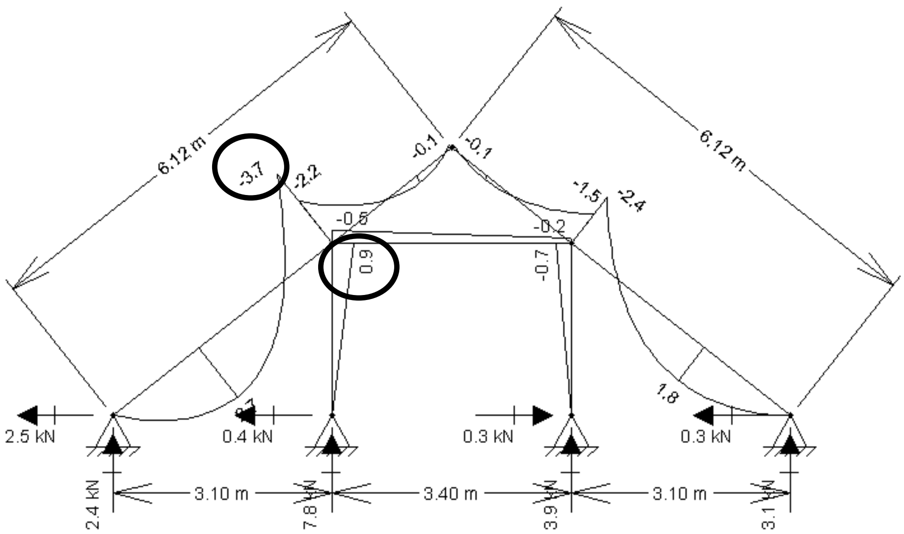


Figure 6.61 Diagram of Bending Moment ULS, Second Variant

$$M = 3.7\ \text{KNm}$$

$$M^{\text{Pillar}} = 0.9\ \text{KNm/m}$$

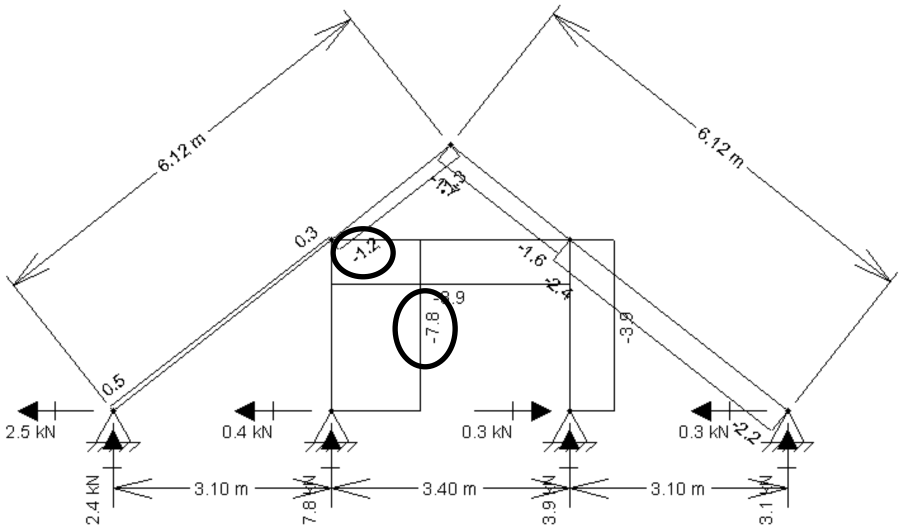


Figure 6.62 Diagram of Axial Force ULS, Second Variant

$$\mathrm{N} = 1.2\ \mathrm{KN\ (Compression)}$$

$$\mathrm{N^{Pillar}} = 7.8\ \mathrm{KN/m\ (Compression)}$$

Table 6.26 $\mathrm{K_{mod}}$ values [5]

Class load for the duration of solid wood, laminated and plywood.	**Class of Use**		
	1	**2**	**3**
Permanent	0,6	0,6	0,5
Long-Term	0,7	0,7	0,55
Medium-Term	0,8	0,8	0,65
Short-Term	0,9	0,9	0,7
Instantaneous	1,1	1,1	0,9

Following the eq.4.15:

$$\sigma_{m,y,d} = \frac{M_y}{W_y} = \frac{3{,}7}{0{,}000533} = 6941{,}84\ \text{KPa} = 6{,}9418\ \text{MPa} \qquad \text{eq. (4.15)}$$

$$\sigma_{c,0,d} = \frac{N}{A} = \frac{1{,}2}{0{,}08 * 0{,}2} = 75\ \text{KPa} = 0{,}075\ \text{MPa} \qquad \text{eq. (4.70)}$$

$$f_{c.0.d} = \frac{K_{mod} * f_{c.0.k}}{\gamma_M} = \frac{0{,}7 * 20}{1{,}3} = 10{,}76\ \text{MPa} \qquad \text{eq. (4.71)}$$

$$f_{m.y.d} = \frac{K_{mod} * f_{m.y.k}}{\gamma_M} = \frac{0{,}7 * 22}{1{,}3} = 11{,}85\ \text{MPa} \qquad \text{eq. (4.67)}$$

For the situation of bending and compression according to Eurocode 5, following eq. 4.29:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.29)}$$

$$\left(\frac{0{,}075}{11}\right)^2 + \frac{6{,}9418}{18{,}62} + 0 = 0{,}3729 < 1$$

For the Column:

$$W_y = \frac{b.h^2}{6} = \frac{0{,}12 * 0{,}12^2}{6} = 0{,}000288\ \text{m}^3 \qquad \text{eq. (4.76)}$$

$$W_z = \frac{h.b^2}{6} = \frac{0{,}12 * 0{,}12^2}{6} = 0{,}000288\ \text{m}^3 \qquad \text{eq. (4.76)}$$

Following the eq.4.12:

$$\sigma_{m,y,d} = \frac{M_y}{W_y} = \frac{0{,}9 * 4{,}8}{0{,}000288} = 1500\ \text{KPa} = 1{,}5\ \text{MPa} \qquad \text{eq. (4.15)}$$

$$\sigma_{c,0,d} = \frac{N}{A} = \frac{7{,}8 * 4{,}8}{0{,}12 * 0{,}12} = 2600\ \text{KPa} = 2{,}6\ \text{MPa} \qquad \text{eq. (4.71)}$$

$$f_{c.0.d} = \frac{K_{mod} * f_{c.0.k}}{\gamma_M} = \frac{0{,}7 * 20}{1{,}3} = 10{,}76\ \text{MPa} \qquad \text{eq. (4.71)}$$

$$f_{m.y.d} = \frac{K_{mod} * f_{m.y.k}}{\gamma_M} = \frac{0{,}7 * 22}{1{,}3} = 11{,}85\ \text{MPa} \qquad \text{eq. (4.67)}$$

For the situation of bending and compression according to Eurocode 5, following eq. 4.29

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \qquad \text{eq. (4.29)}$$

$$\left(\frac{2{,}6}{10{,}76}\right)^2 + \frac{1{,}5}{11{,}86} + 0 = 0{,}368 < 1$$

6.2.1.3. Lateral Buckling

$$\lambda_{rel,m} \leq 0{,}75$$

For the respective slenderness value it comes $K_{crit} = 1$

$$K_{crit} = 1$$

$$\mathrm{I} = \frac{0{,}04.\,(0{,}2)^3}{12} = 0{,}000027\mathrm{m}^4 \qquad \text{eq. (4.77)}$$

$$\sigma_{m,d} = \frac{3.7}{0{,}000027} * 0{,}1 + \frac{1.2}{0{,}2 * 0{,}04} = 13.85 \text{ MPa} \qquad \text{eq. (4.78)}$$

$$\mathrm{K}_{crit} * f_{m,d} = \frac{22}{1{,}3} = 16.93 \text{ MPa} > 13{,}85 \text{ MPa} \qquad \text{eq. (4.33)}$$

Verifying lateral buckling for the column:

$$\lambda_{\mathrm{rel,m}} \leq 0{,}75$$

For the respective slenderness value it comes $\mathrm{K}_{crit} = 1$

$$\mathrm{K}_{crit} = 1$$

$$\mathrm{I} = \frac{0{,}16.\,(0{,}16)^3}{12} = 5{,}46^{-5}\mathrm{m}^4 \qquad \text{eq. (4.78)}$$

$$\sigma_{m,d} = \frac{3.7}{5{,}46^{-5}} * 0{,}1 + \frac{1.2}{0{,}16 * 0{,}16} = 6.83 \text{ MPa} \qquad \text{eq. (4.78)}$$

$$\mathrm{K}_{crit} * f_{m,d} = \frac{22}{1{,}3} = 16.93 \text{ MPa} > 6{,}83 \text{ MPa} \qquad \text{eq. (4.33)}$$

6.2.2. SERVICEABILITY LIMIT STATES

According to the table 4.12 it is concluded that $u_{net,fin} = \frac{L}{200}$

$$u_{net,fin} = \frac{L}{200} = \frac{370}{200} = 1{,}38\ mm \qquad \text{eq. } (4.74)$$

Table 6.27 Load & K_{def}, u_{inst}, u_{fin} values [5]

Load	K_{def}	Components of Load [mm]	
		u_{inst}	u_{fin}
Weight own (class load duration = permanent, the class of use = 2)	0,8	2,89	5,21
Snow (Class duration load = medium-term, class of use = 2)	0,25	1,29	1,61
Wind (Class duration load = short-term, class of use = 2)	0	1,06	1,06
Deflections summary $u_{fin} = u_{fin1} + u_{fin2}+u_{fin3}$		7,88	

$$u_{fin} = 7{,}88\text{ mm} < u_{net,fin}=13{,}8\text{ mm} \qquad \text{eq. } (4.75)$$

6.2.3. CONNECTIONS

The nails used in this particular prooject are threaded, pre-drilled, M24 nails from 4.6 class (S275). In each connection is used two nails.

$f_{u,k} = 400\text{N/mm}^2 \qquad f_{y,k} = 240\text{N/mm}^2$

a) Connection 1 – Second variant

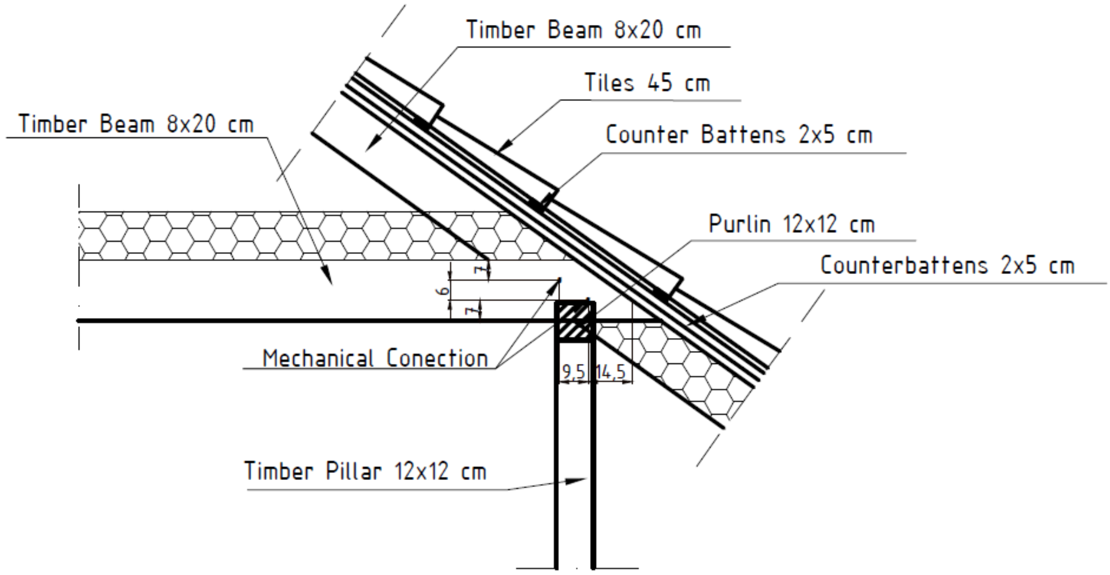


Figure 6.63 Connection 1, Second Variant

$$f_{h,k} = 0{,}082(1\text{-}0{,}01\ d)\rho_k\ \mathrm{N/mm^2} \qquad \text{eq.(4.44)}$$

$$f_{h,k} = 0{,}082(1\text{-}0{,}01{*}24)340 = 21{,}19\ \mathrm{N/mm^2} \qquad \text{eq.(4.44)}$$

$$M_{y,Rk} = 0.3\ f_u\ d^{2,6} = 0{,}3 * 400 * 24^{2,6} = 465297\ \mathrm{Nmm} \qquad \text{eq. (4.41)}$$

$$M_{y,Rk} = 465297\ \mathrm{Nmm}$$

$$f_{head,k} = 70 * 10^{-6}\ \rho_k{}^2 = 70 * 10^{-6}\ 340^2 = 8{,}092\ \mathrm{N/mm^2} \qquad \text{eq. (4.52)}$$

$$f_{ax,k} = 0{,}52 * d^{-0,5}\ l_{eff}{}^{-0,1}\ \rho_k{}^{0,8} = 0{,}52 * 24^{-0,5} * 60^{-0,1} * 340^{0,8} = 7{,}46\mathrm{N/mm^2}$$

$$F_{ax,R,k} = \begin{cases} f_{ax,k}\ d\ t_{pen} = 7.46 * 24 * 60 = 10742{,}4\ \mathrm{N} & \text{eq. (4.50)} \\ f_{ax,k}\ d_h{}^2 = 7{,}46 * 30^2 = 6714\ \mathrm{N} & \text{eq. (4.51)} \end{cases}$$

Resistance of the nail to the shear force:

$$F_{v,Rd} = \frac{K * f_{ub} * A_s}{\gamma_m} = \frac{0.6 * 400 * 452{,}16}{1{,}25} = 86814{,}72\ N$$

$$F_{v,Ed} = 3.5\ \mathrm{KN}$$

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{(3.5 * 10^3)/2}{86814.72} = 0.021 \leq 1 \qquad \text{eq. } (4.53)$$

The distances and angles from the nails to the edges/ends of the timber beams were checked according to the tables 4.14, 4.15, 4.16 and 4.17. It was concluded that all the distances and angles are in accordance with the limits.

b) Connection 2 – Second Variant

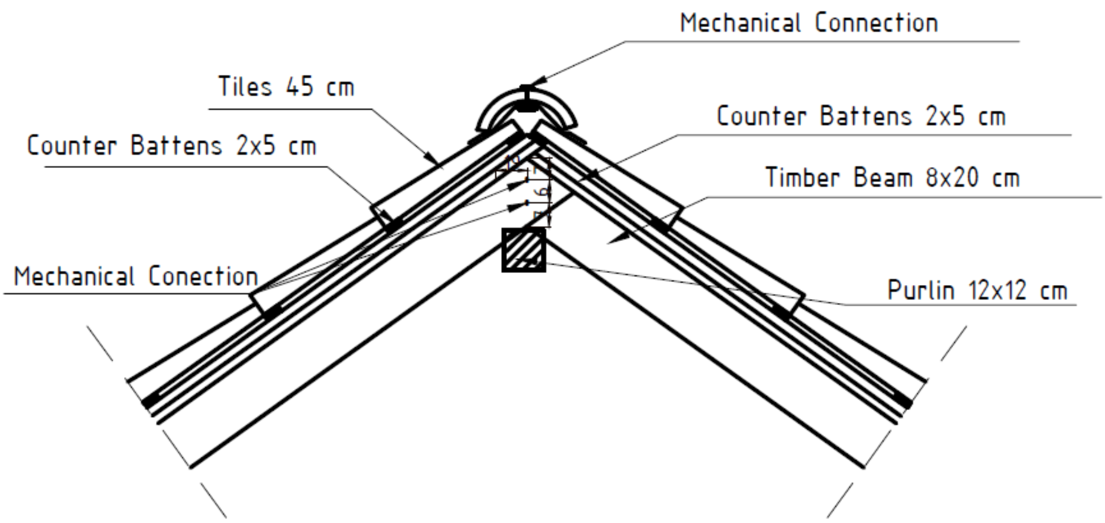


Figure 6.64 Connection 1, Second Variant

$$f_{h,k} = 21{,}19\ \mathrm{N/mm^2}$$

$$M_{y,Rk} = 465297\ \mathrm{Nmm}$$

$$f_{head,k} = 8{,}092\ \mathrm{N/mm^2}$$

$$f_{\mathrm{ax,k}} = 7{,}46\mathrm{N/mm^2}$$

$$F_{\mathrm{ax,R,k}} = 6714\ \mathrm{N}$$

$$\mathrm{F_{v,Rd}} = 86814{,}72\ N$$

$$\mathrm{F_{v,Ed}} = 1\ \mathrm{KN}$$

$$\frac{\mathrm{F_{ax,Ed}}}{\mathrm{F_{ax,Rd}}} + \frac{\mathrm{F_{v,Ed}}}{\mathrm{F_{v,Rd}}} \leq 1 \qquad \text{eq. (4.53)}$$

The distances and angles from the nails to the edges/ends of the timber beams were checked according to the tables 4.14, 4.15, 4.16 and 4.17. It was concluded that all the distances and angles are in accordance with the limits.

6.3. INSULATION (BOTH VARIANTS)

The heat transmission between different locations is one of the most common physical phenomena. Heat can be transmitted in three distinct ways: conduction, convection and radiation.

The heat conduction occurs between two bodies that are in contact with each other, or between an area of a body to another area of the same body, and it is a mechanism that involves the transfer of kinetic energy molecules at higher temperature to the molecules of lower temperature. The laws of heat conduction can be expressed mathematically and analyzed in analytically way.

Convection heat is transmitted by movements of the air in means of particles (liquids and gases) moving from one location to another. Natural convection or free convection occurs without action of external agents. The movement is given by the difference of temperature between particles.

Transmission by radiation occurs without physical contact between the bodies. Heat is transferred through electromagnetic waves.

Very often, heat transmission takes occurs with the simultaneous actions of the three cases. The heat transfer coefficient and the thermal resistance is given by the following equations:

$$U= \frac{1}{\Sigma R} \ [W/m^2.\,°C] \qquad eq.(6.1)$$

$$\Sigma R= \Sigma \frac{e}{\lambda} \ [m^2.\,°C/W] \qquad eq.(6.2)$$

Where:

e- Thickness [m]

λ- Thermal conductivity [W/m. °C]

6.4. WALL ($U_{limit} = 0,25$ m^2. °C/W [15])

[10] Thermal conductivity and resistance values:

Interior plaster cement-lime (ρ=1900 Kg/m^3) → λ =1,3 W/m. °C , *e* = 1,5 cm

External plaster cement-lime (ρ=1900 Kg/m^3) → λ =1,3 W/m. °C, *e* = 1,5 cm

Type blocks YTONG (ρ ≤ 1000 Kg/m^3) → λ =0,34 W/m. °C, *e* = 36 cm

Isolation mineral wool (ρ= 25 - 40 Kg/m^3) → λ =0,037 W/m. °C, *e* =15 cm (to be calculated)

$R_{superficial\ interior} = 0{,}13$ m^2. °C/W

$R_{superficial\ external} = 0{,}04$ m^2. °C/W

$U \leq 0,25$ m^2. °C/W → Isolation polystyrene XPS, *e* = 11,28 cm → *e* = 15 cm

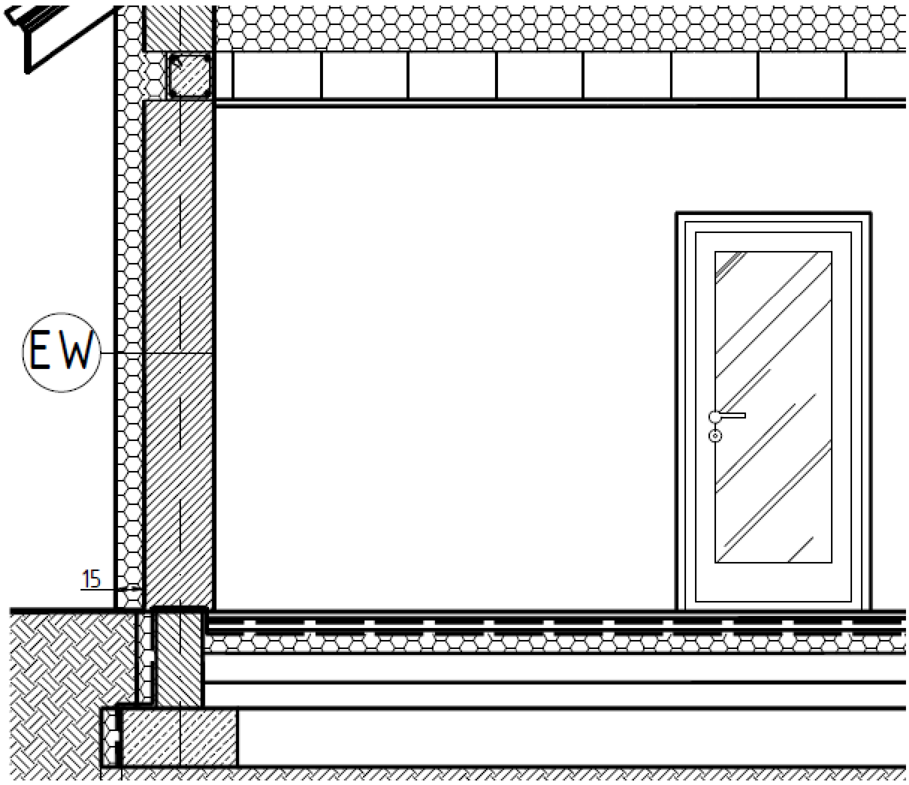


Figure 6.65 Insulation, External Wall Isolation, Project

6.5. ROOF ($U_{limit} = 0,20$ m^2.°C/W [15])

6.5.1. FIRST VARIANT

[10] Thermal conductivity and resistance values:

Isolation mineral wool (ρ= 25 - 40 Kg/m^3) → λ =0,037 W/m.°C, *e* = *35* cm (to be calculated)

All elements from Teriva Slab → $U = 2,1$ m^2.°C/W → $R = 0,476$ m^2.°C/W

$U \leq 0,20$ m^2.°C/W → Isolation polystyrene, *e* = 18,25 cm → *e* = 20 cm

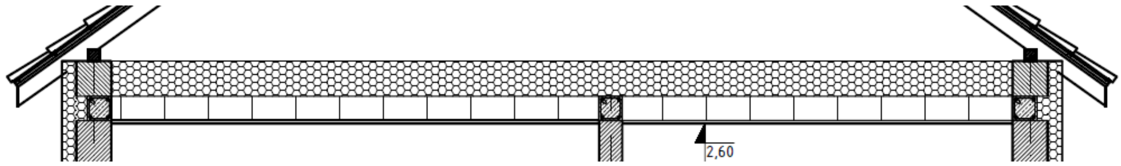


Figure 6.66 Insulation, Roof Isolation, First Variant, Project

Note: Despite the fact that the necessary thickness for the isolation is 20 cm of isolation, it is decided to design 35 cm to fulfill the empty space in a way that the YTONG blocks and the isolation are located in the same quota.

6.5.2. SECOND VARIANT

[10] Thermal conductivity and resistance values:

Isolation mineral wool (ρ= 25 - 40 Kg/m^3) → λ =0,037 W/m. ℃, e = X cm

Ceramic tiles (ρ ≤ 1000 Kg/m^3) → λ =0,34 W/m. ℃, e = 3 cm

$U \leq 0,25$ m^2. ℃/W → Isolation polystyrene XPS, e = 19,34 cm → e = 20 cm

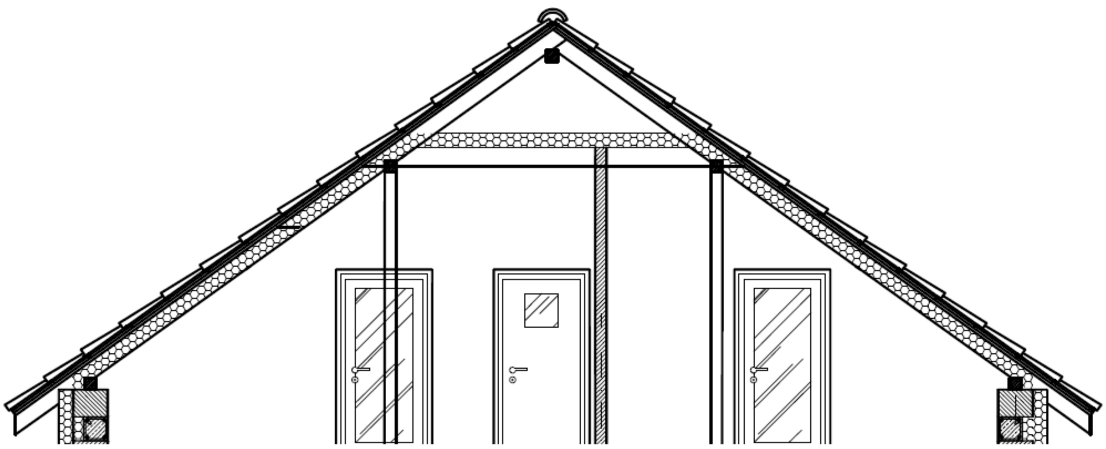


Figure 6.67 Insulation, Roof Isolation, Second Variant, Project

6.6. CONDENSATION

Considering the normal temperature inside the house of $t_i = 20°C$, external temperature $t_e = 0°C$ and relative humidity RH=70% throw the graphic bellow it is possible to determine the dry bulb temperature $t_{db} = 14°C$. The dry bulb temperature is the temperature where the air is saturated and has the equal numbers of grams by m^2 than the degree inside. To avoid condensation it is necessary that the interior superficial temperature (t_{si}) is above the dry bulb temperature (t_{db}). By another words, when the air touches the wall it gets the temperature of the wall and in the situation where the temperature of the wall is very low and the temperature inside the house is much higher, the water that the air possess as vapour will become liquid (condensation).

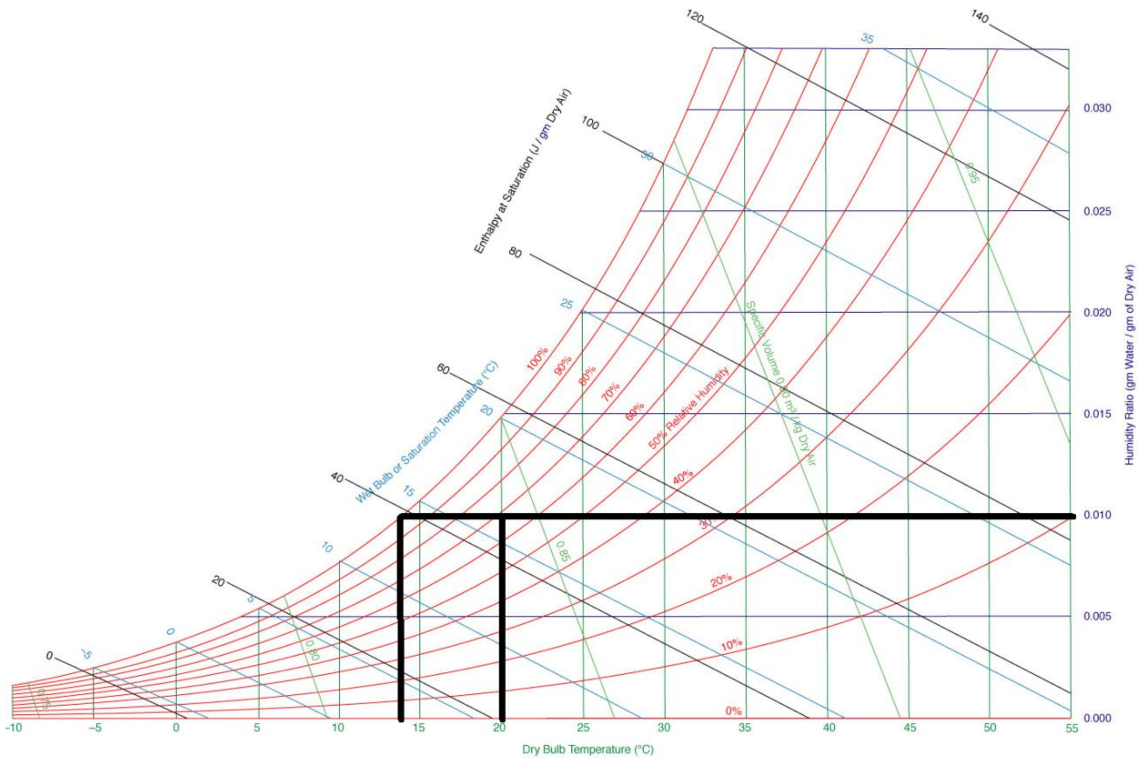


Figure 6.68 Psychometric diagram [27]

$$t_{si} = t_i - R_{si}.U(ti - te) = 20 - 0{,}13 * 0{,}25(20 - 0) \qquad eq.(6.3)$$

$$= 19{,}35°C > t_{db} = 14°C \qquad \text{OK}$$

7
CONCLUSION

Table 7.1 Total Wood Mass, First Variant

First Variant				
Element Description	**Amount (units)**	**Length**	**Cross-Section Area [m^2]**	**Total Mass [340 Kg/m^3]**
Timber Beam K-1, K-2	6 un. →K-2 42 un. →K-1	42 un. → 6,12 m 6 un. → 7,20 m	0,04*0,2 =0,008 m^2	0,008*(42*6,12+6*7,2)*340= 816,65 kg / m^3
Timber Beam K-5	48 un.	5,83 m	0,04*0,2 =0,008 m^2	0,008*5,83*48*340= 761,17 kg / m^3
Counter battens	85 un.	30 un.→ 23,08 m 55 un.→ 10 m	0,02*0,05 =0,003 m^2	0,003*(30*23,08+55*10)*340= 1267,3 kg / m^3
Purlin K-3, K-4	3 un.	23.08 m	0,12*0,12 =0,0144 m^2	0,0144*23,08*3*340= 329,58 kg / m^3
				Σ=3175 kg

Table 7.2 Total Wood Mass, Second Variant

Second Variant				
Element Description	**Amount (units)**	**Length [m]**	**Cross-Section Area [m^2]**	**Total Mass [340 Kg/m^3]**
Timber Beam K-1, K-2	7 un. →K-2 31 un. →K-1	31 un. → 6,12 m 7 un. → 7,20 m	0,04*0,2= 0,008 m^2	0,008*(31*6,12+7*7,2)*340= 653,13 kg / m^3
Timber Beam K-5	38 un.	4 m	0,04*0,2= 0,008 m^2	0,008*4*38*340= 413,44 kg / m^3
Counter Battens	70 un.	30 un. → 17,08 m 40 un.→ 10 m	0,02*0,05= 0,003 m^2	0,003*(30*17,08+40*10)*340= 930,65 kg / m^3
Purlin K-3, K-4, K-6	5 un.	17,08 m	0,12*0,12= 0,0144 m^2	0,0144*17,08*5*340= 418,119 kg / m^3
Column S-1	8 un.	3,37 m	0,16*0,16= 0,0256 m^2	0,0256*3,37*8*340= 234,66 kg / m^3
				Σ=2650 kg

Considering total mass as the condition factor, it´s conclude that the best option is the vertical extension (second variant).

8
BIBLIOGRAPHY

[1] http://serwer1416797.home.pl/durskidomy/index.php/artykuly/48-konstrukcja-stropu/94-strop-gestozebrowy-teriva

[2] http://www.uciechowski.com.pl/poradnik_stropy_dla_projektantow.pdf

[3] EN 1992-1-1_2010 (Eurocode 2)

[4] EN 1991-1-1_2009 (Eurocode 1)

[5] EN 1995-1-1_2004 (Eurocode 5)

[6] http://www.armstrong.com/flooring/hardwood-and-softwood-differences.html

[7] Handbook *2 – Design of Timber Structures According to EC5* , 2008 - Leonardo da Vinci Pilot Project, Educational Materials for Designing and Testing of Timber Structures - TEMTIS

[8] PN – EN 1313, Round And Sawn Timber - Permitted Deviations And Preferred Sizes

[9] http://www.learneasy.info/MDME/focus/materials/enmat/LECTURES/Lecture-03/images/

[10] http://www4.hcmut.edu.vn/~dantn/Matter/Wood.html

[11] University of Porto, FEUP, PowerPoints - Bridges- Practical classes - 2014/2015

[12] Coeficientes de transmissão térmica de elementos da envolvente dos edifícios, LNEC

[13] http://www.mrwallpaper.com/cypress-trees-wallpaper/

[14] http://www.arborday.org/shopping/trees/treeImages.cfm?ID=102

[15]http://www.architektura.info/index.php/prawo/warunki_techniczne_budynki/zalacznik_2_wymagania_izolacyjnosci_cieplnej_i_inne_wymagania_zwiazane_z_oszczednoscia_energii

[16] Estruturas de madeira, Walter Pfeil, Michèle Pfeil – Sixth Edition, LTC Editor

[17] http://science.howstuffworks.com/life/genetic/question598.htm

[18] http://www.roymech.co.uk/Related/Construction/Timber.html

[19] http://www.bigstockphoto.com/video/search/hammer/]

[20] http://secrets-of-self-sufficiency.com/types-of-nails-and-screws

[21] *Structure Timber Design*, Abdy Kermani, Blackwell Science

[22] http://www.valentazt.sk/dosky-uholniky.html

[23] http://www.diygardenprojects.com/page/5/

[24] http://www.solaripedia.com/713/104/material.html

[25] Power points of Timber structure classes 2014/2015, University of Wroclaw

[26] http://www.roymech.co.uk/Useful_Tables/Timber/Timber_Glued_laminate.html

[27] http://www.instructables.com/id/Who-Needs-a-Meteorologist-Measure-Humidity-the-Ol/step8

[28] http://fet.uwe.ac.uk/conweb/house_ages/timber/print.htm

9
BIBLIOGRAPHIC REFERENCES

The following set of bibliographic elements was used generally as query for implementation of this work.

Obliczenie konstrukcji budynków wznoszonych tradycyjnie - Jerzy Hola, Piotr Pietraszek, Krzysztof Schabowłcz

Materiały do ćwiczeń projektowych z budownictwa ogólnego - K.Schabowicz, T.Gorzelańczyk

EN 1991-1-1_2009 (Eurocode 1)

EN 1992-1-1_2010 (Eurocode 2)

EN 1995-1-1_2004 (Eurocode 5)

Handbook 2 – Design of Timber Structures According to EC5, 2008

Coeficientes de transmissão térmica de elementos da envolvente dos edifícios, LNEC

PN – EN 1313-1, Round And Sawn Timber - Permitted Deviations And Preferred Sizes

PowerPoints – Bridges- Practical classes - 2014/2015

http://serwer1416797.home.pl/durskidomy/index.php/artykuly/48-konstrukcja-stropu/94-strop-gestozebrowy-teriva

http://www.armstrong.com/flooring/hardwood-and-softwood-differences.html

http://www.learneasy.info/MDME/focus/materials/enmat/LECTURES/Lecture-03/images/

http://www4.hcmut.edu.vn/~dantn/Matter/Wood.html

http://www.uciechowski.com.pl/poradnik_stropy_dla_projektantow.pdf

EN1194:2003

Structure Timber Design, Abdy Kermani

Blass, H., Aune, P., et al, *Timber Engineering STEP 1*

Blass, H., Aune, P., et al, *Timber Engineering STEP 2*

Estruturas de madeira, Walter Pfeil

Acetatos aulas Betão Armado 1 2013/2014, FEUP

Esforço Transverso, Torção, Punçoamento, Rui Manuel Faria e Nelson Vila Pouca, Março 1997, FEUP

Dissertação "*Considerações sobre análise e dimensionamento de estruturas de madeira para habitações uni e plurifamiliares*", João Fernando Sá Laranjeira, Julho 2009, FEUP

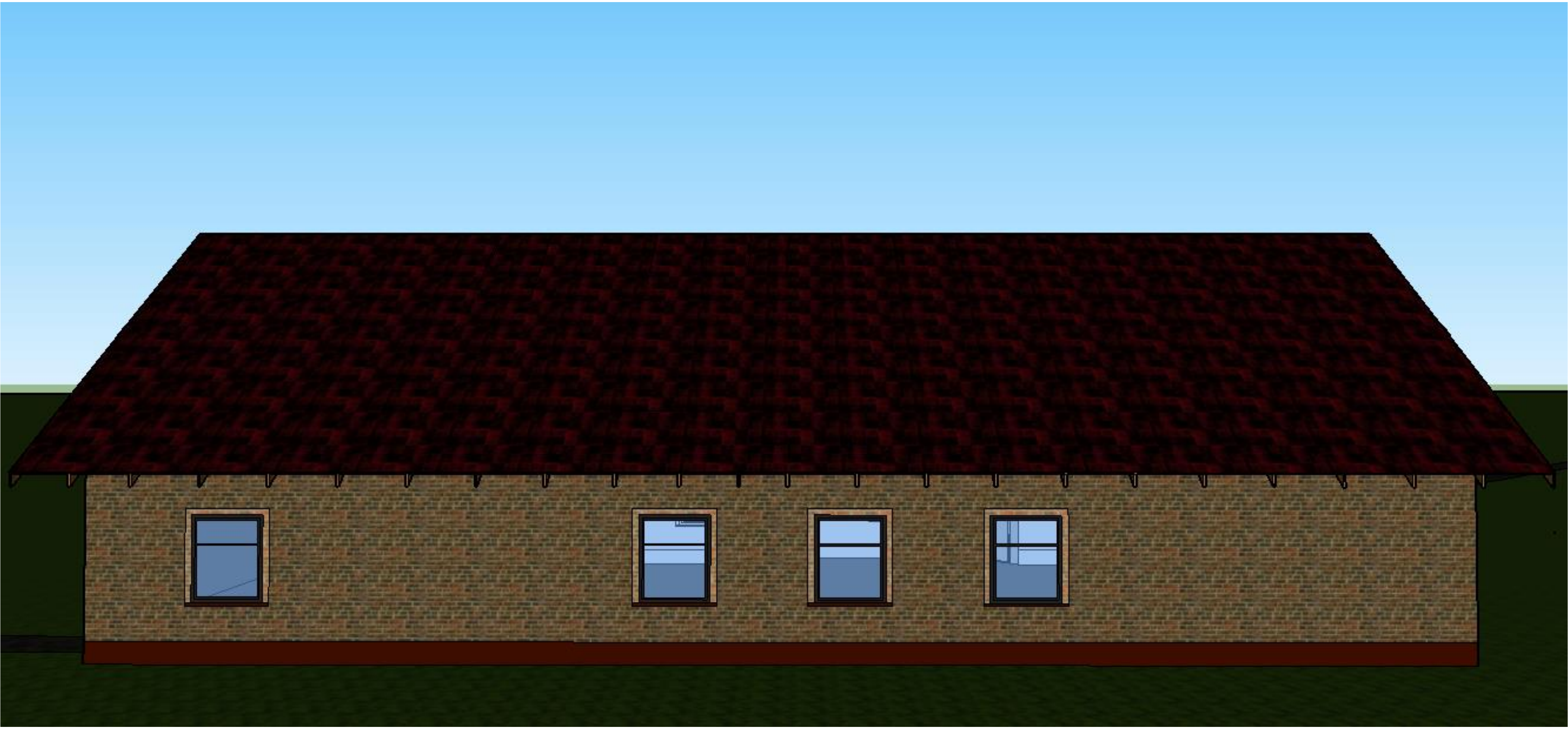
http://fet.uwe.ac.uk/conweb/house_ages/timber/print.htm

10 ANNEXES

First Variant

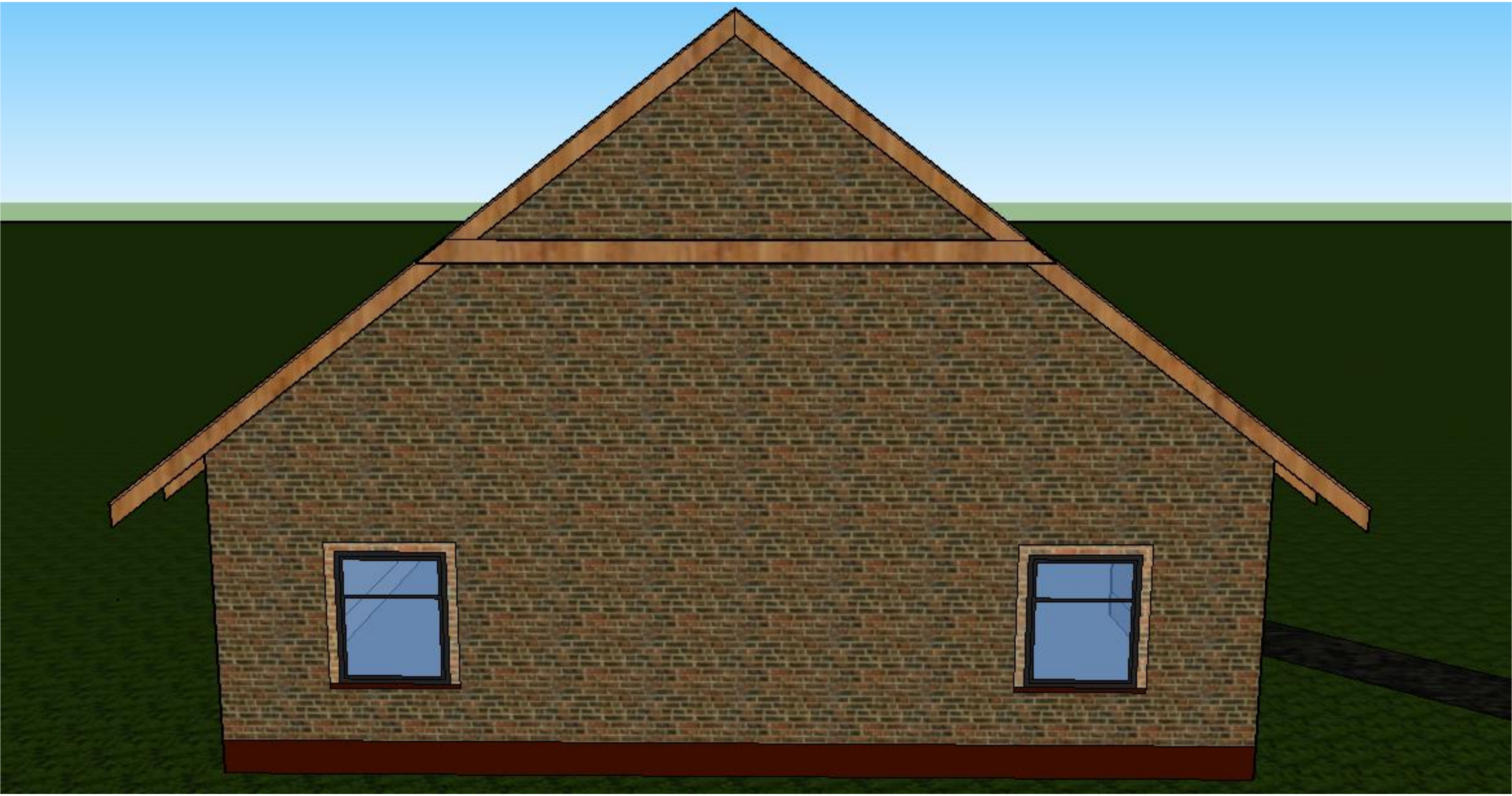
First Variant, Horizontal Extension

First Variant, Horizontal Extension

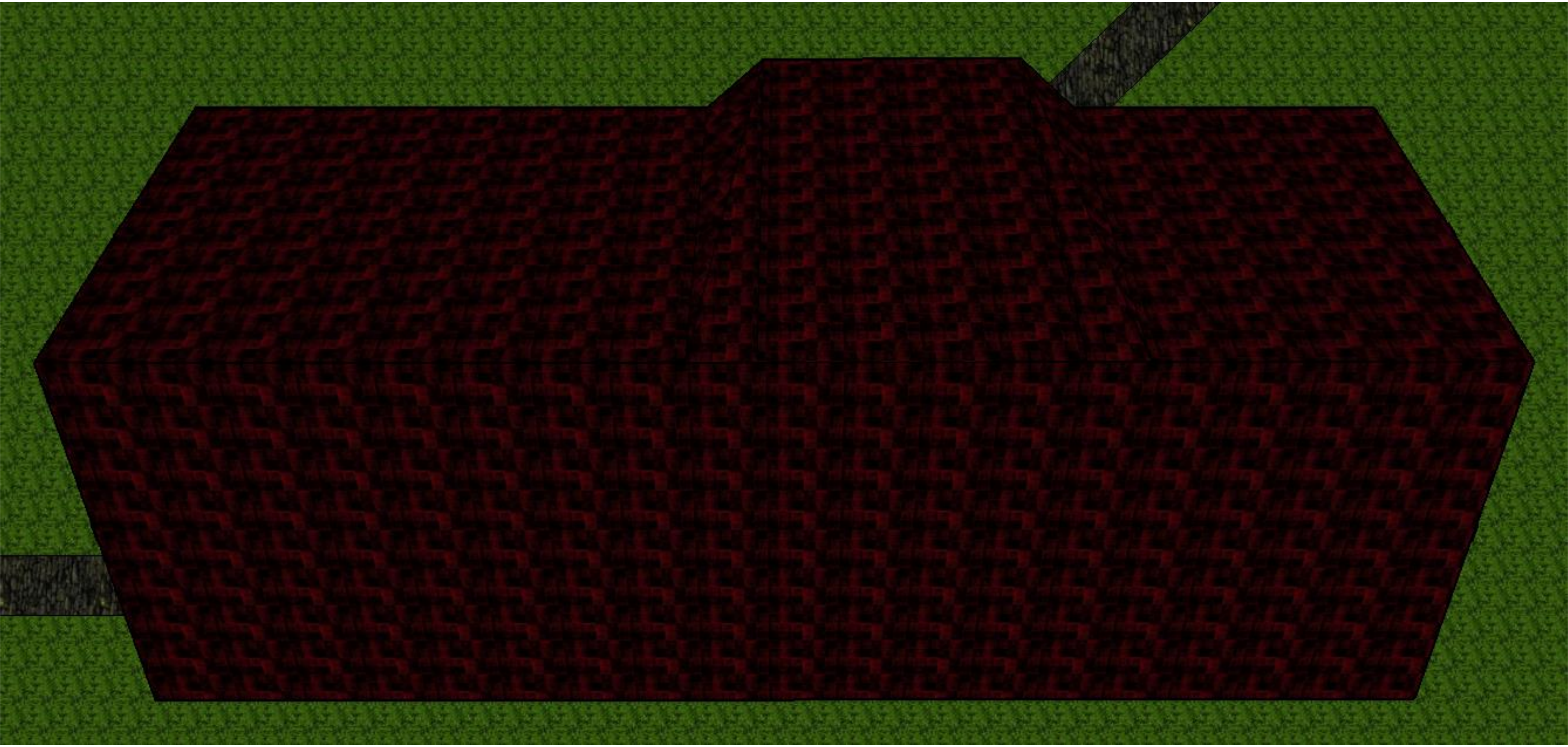


First Variant, Horizontal Extension

First Variant, Horizontal Extension



First Variant, Horizontal Extension

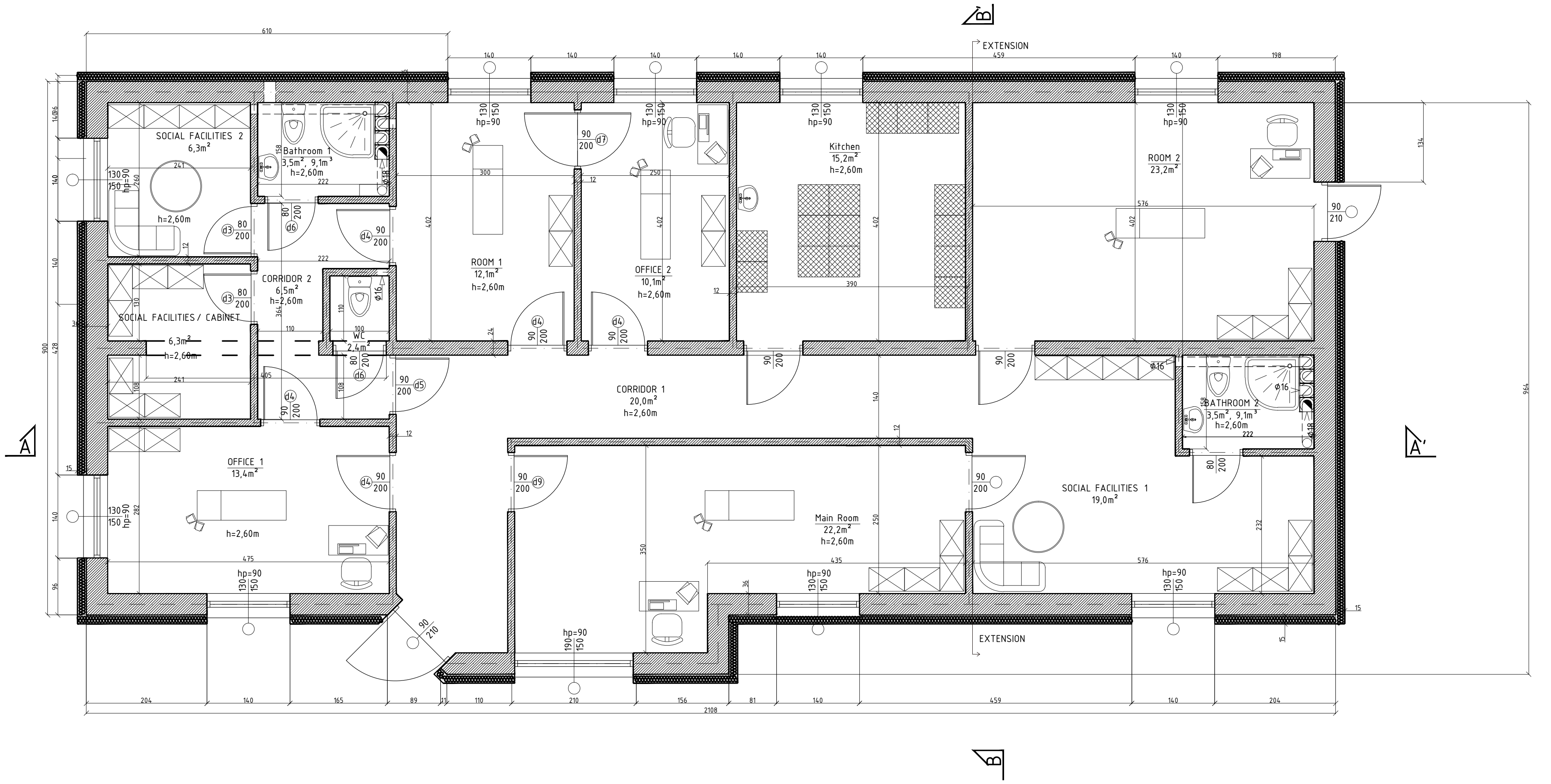


First Variant, Horizontal Extension

Compartment Areas

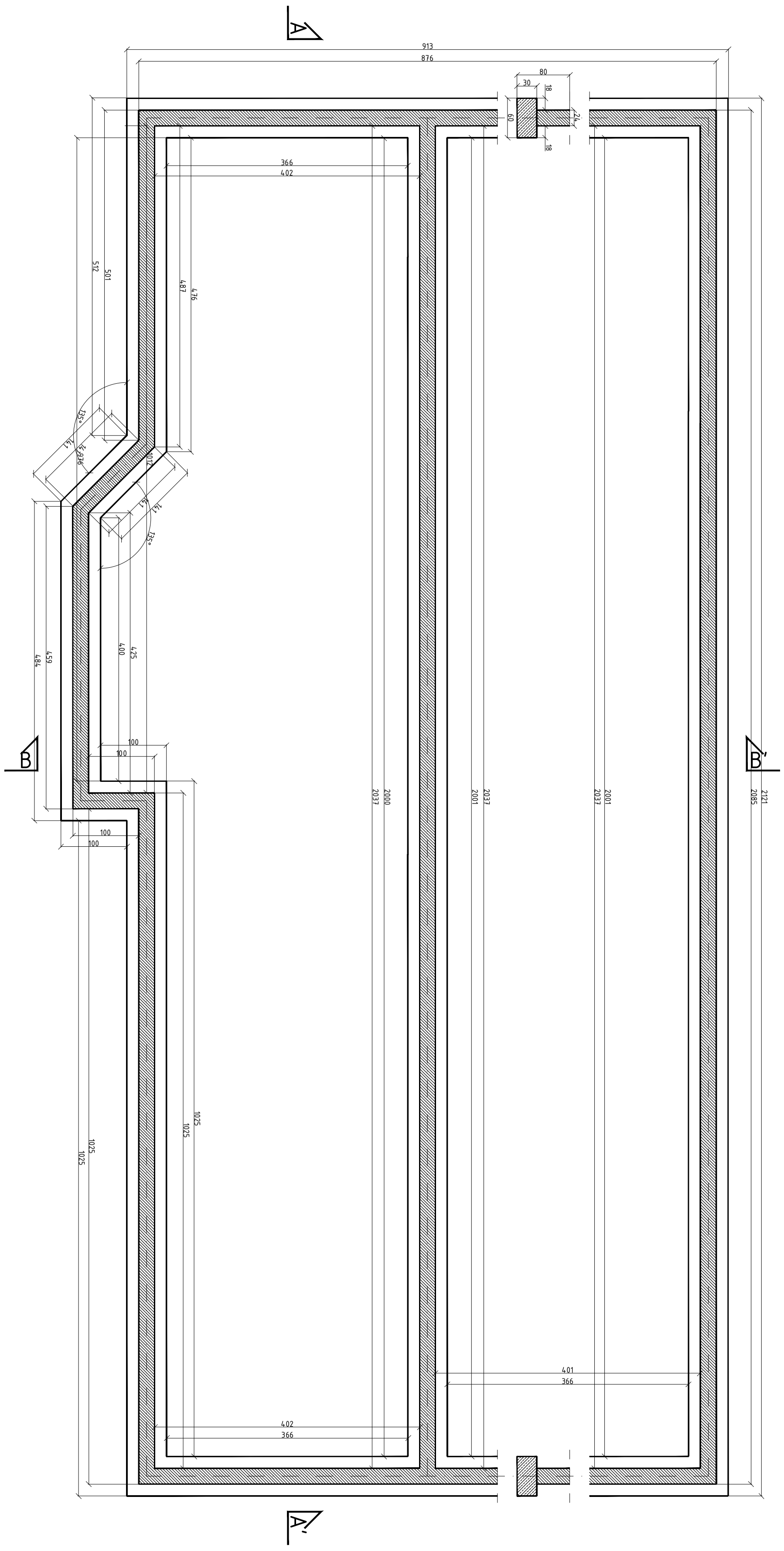
Compartment Name	Area [m²]
Office 1	13,4
Office 2	10,1
Social Facilities 1	19,0
Social Facilities 2	6,3
Social Facilities/Cabinet	6,3
Bathroom 1	3,5
Bathroom 2	3,5
Corridor 1	20,0
Corridor 2	6,5
WC	1,1
Room 1	12,1
Room 2	23,2
Main Room	22,2
Kitchen	15,2
Σ	156,1

USEFULL AREA = 162,4m²
BUILDING AREA = 196m²

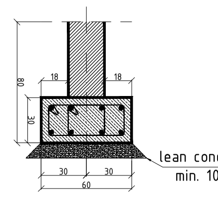


Wroclaw University of Technology	
Architectural Plan – First Variant	
Student	Bruno Gonçalves
Super-Visor	Dr. Łukasz Sadowski
OBJECT: Office Building	ADDRESS: Warsaw
Data:	Stage:
Signature:	Scale: 1/50
Drawing Number	1

Steel	Function	Flength (m)	Quantite (units)
#12	Longitudinal Reinforcement	12	61
#8	Stirrups	1.2	680



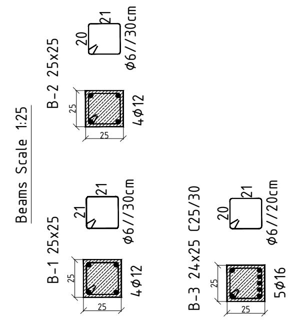
FOUNDATION SCALE 1:25

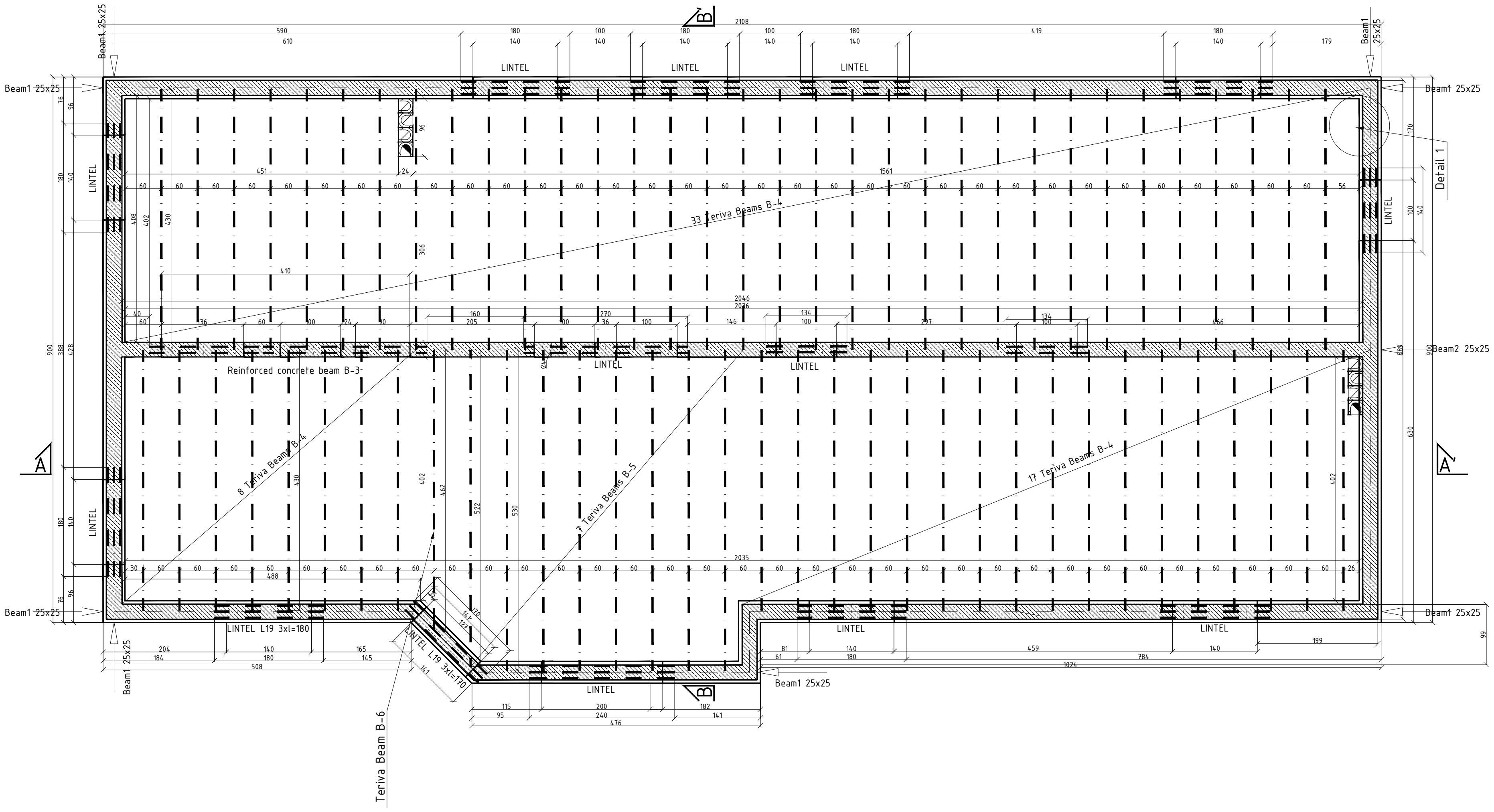


Wroclaw University of Technology			
Foundations – First Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 2

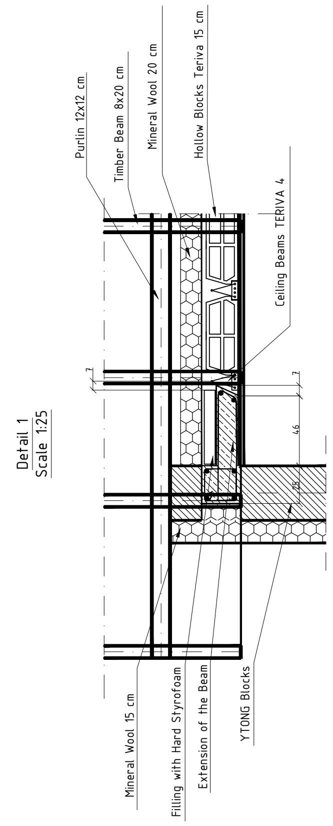
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Element	Amount (units)	Length (m)
B-1	6	21,05/9/9/10,25/4,75/5,1
B-2	1	21,05
B-3	1	4,1
B-4	58	4,3
B-5	7	5,4
B-6	1	5,15





8 Teriva Beams 4,3 m

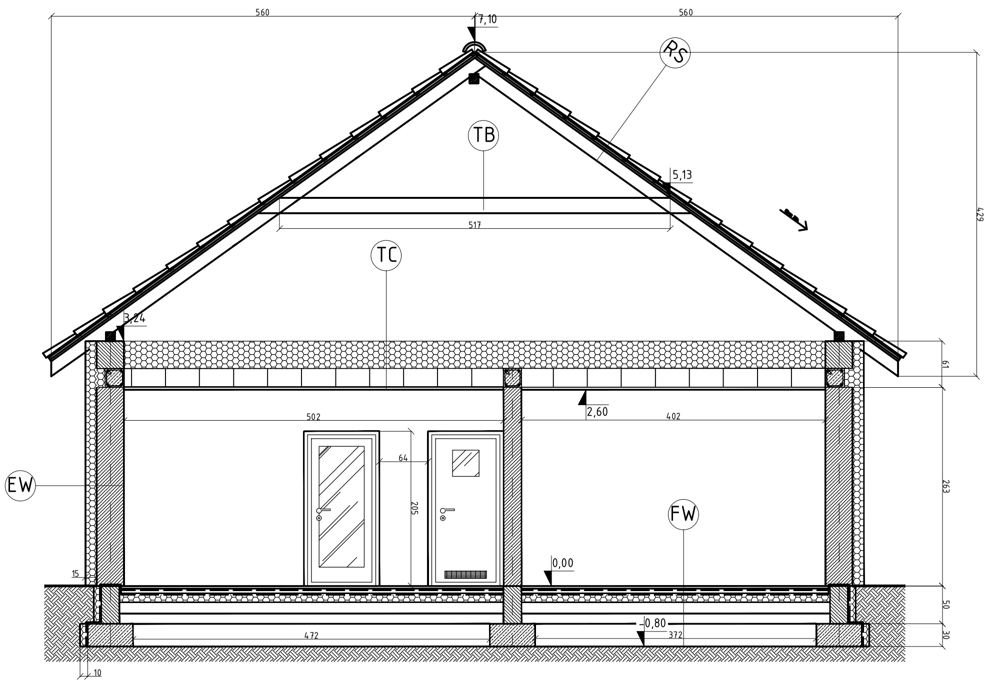


Detail 1
Scale 1:25

Wroclaw University of Technology	
Ceilind/Beams - First Variant	
Student: Bruno Gonçalves	Data:	Stage:
Super-Visor: Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw	Drawing Number 3

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Section B-B



STATEMENT OF LAYERS

FW FOUNDATION BARRIER

1. Ceramic
2. Primer monolithic concrete (3.5cm)
3. Isolation - Mineral Woll
4. Lean concrete gravel faded smoothly (10cm)
5, Interior plaster cement-lime
6. Coarse sand or gravel compacted stabilized (15cm)
7. Tamped earth

EW EXTERIOR WALL

1. Interior plaster cement-lime
2. Structural Wall outer monolayer
3. Blocks YTONG type
4. Outside mineral plaster on a grid of fiberglass

TC TERIVA CEILLING

1. Self leveling cementious underlayment (3 cm)
2. Protective layer - PCV foli
3. Isolation - mineral wool (35 cm)
4. Hollow Block
5. Reinforced concrete slab+beam
6. Cementious plaster work (1,5 cm)

TB TIMBER BEAM

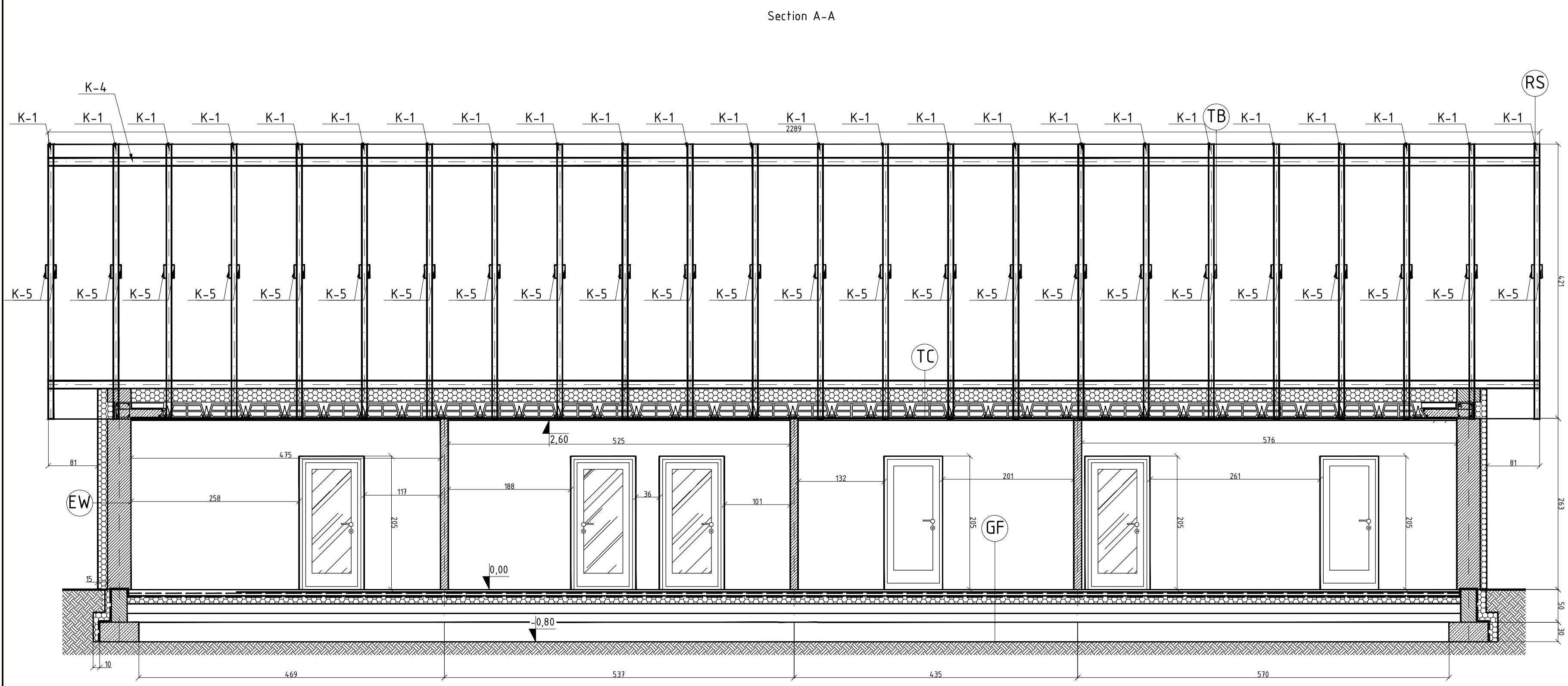
1. Timber Beam (8x20 cm), Class C22

RS ROOF STRUCTURE

1. Timber Beam 8x20 cm, Class C22
2. Tiles (45 cm)
3. Counter Battens (2x5 cm)

Wroclaw University of Technology			
Section B-B - First Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 4

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



STATEMENT OF LAYERS

GF GROUND FLOOR

1. Ceramic
2. Primer monolithic concrete (3.5cm)
3. Isolation - mineral wool
4. Lean concrete gravel faded smoothly (10cm)
5. Interior plaster cement-lime
6. Coarse sand or gravel compacted stabilized (15cm)
7. Tamped earth

EW EXTERIOR WALL

1. Interior plaster cement-lime
2. Structural Wall outer monolayer
3. Blocks YTONG type
4. Outside mineral plaster on a grid of fiberglass
5. Isolation - mineral wool (15 cm)

TC TERIVA CEILLING

1. Self leveling cementious underlayment (3 cm)
2. Protective layer - PCV foli
3. Isolation - mineral wool (20 cm)
4. Hollow Block
5. Reinforced concrete slab+beam
6. Cementious plaster work (1,5 cm)

TB TIMBER BEAM

1. Timber Beam (8x20 cm), Class C22
2. Isolation - mineral wool (20 cm)

RS ROOF STRUCTURE

1. Timber Beam 8x20 cm, Class C22
2. Tiles (45 cm)
3. Counter Battens (2x5 cm)

Element	Amount (units)	Length (m)	Cross-section Dimensions (m*m)	Area (m^2)	Total Mass 340 Kg/m^3
K-1	42	6,12	0,04*0,2	0,008	699
K-2	6	7,2	0,04*0,2	0,008	118
K-3	2	23,08	0,12*0,12	0,014	220
K-4	1	23,08	0,12*0,12	0,014	110
K-5	48	5,83	0,04*0,2	0,008	762
Counter Battens	85	30 uni.-23,08 55 uni.-10	0,02*0,05	0,003	1267
				Σ	3176

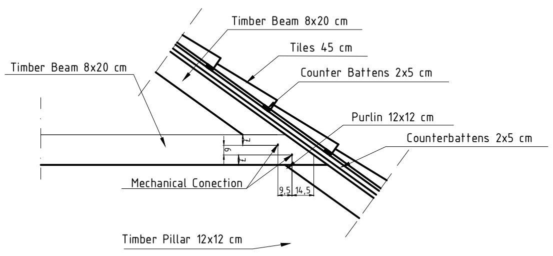
Wroclaw University of Technology			
Section A-A - First Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 5

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

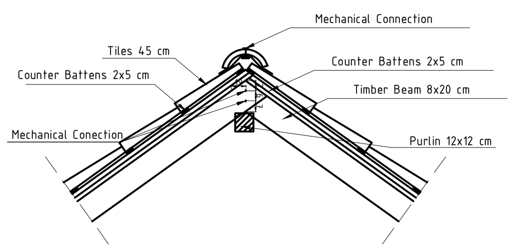
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

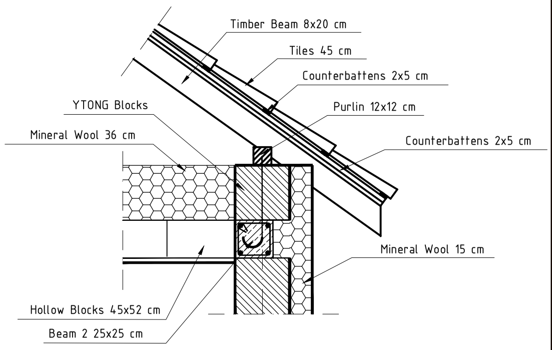
Detail 4
Scale 1:25

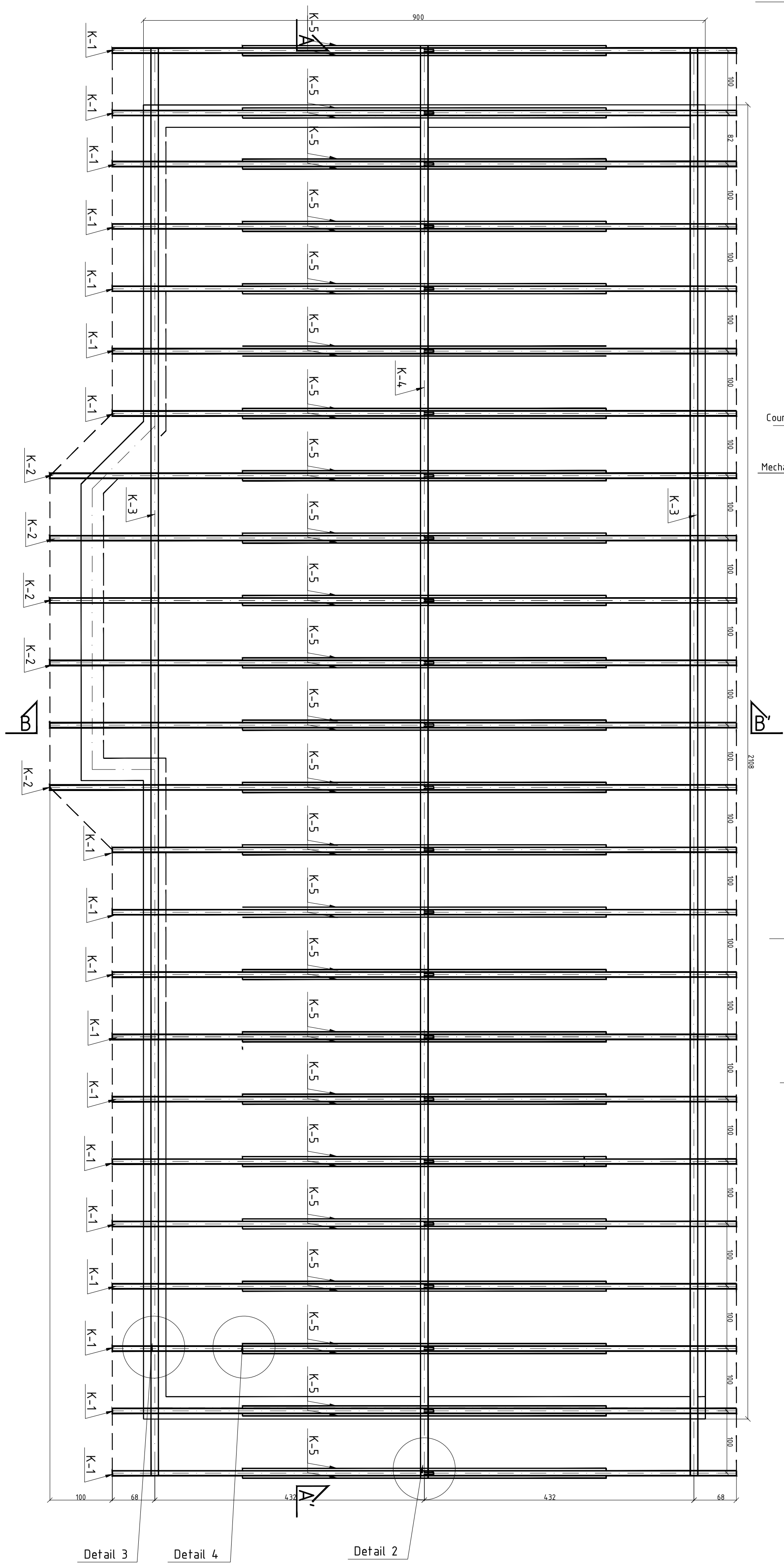


Detail 2
Scale 1:25



Detail 3
Scale 1:25





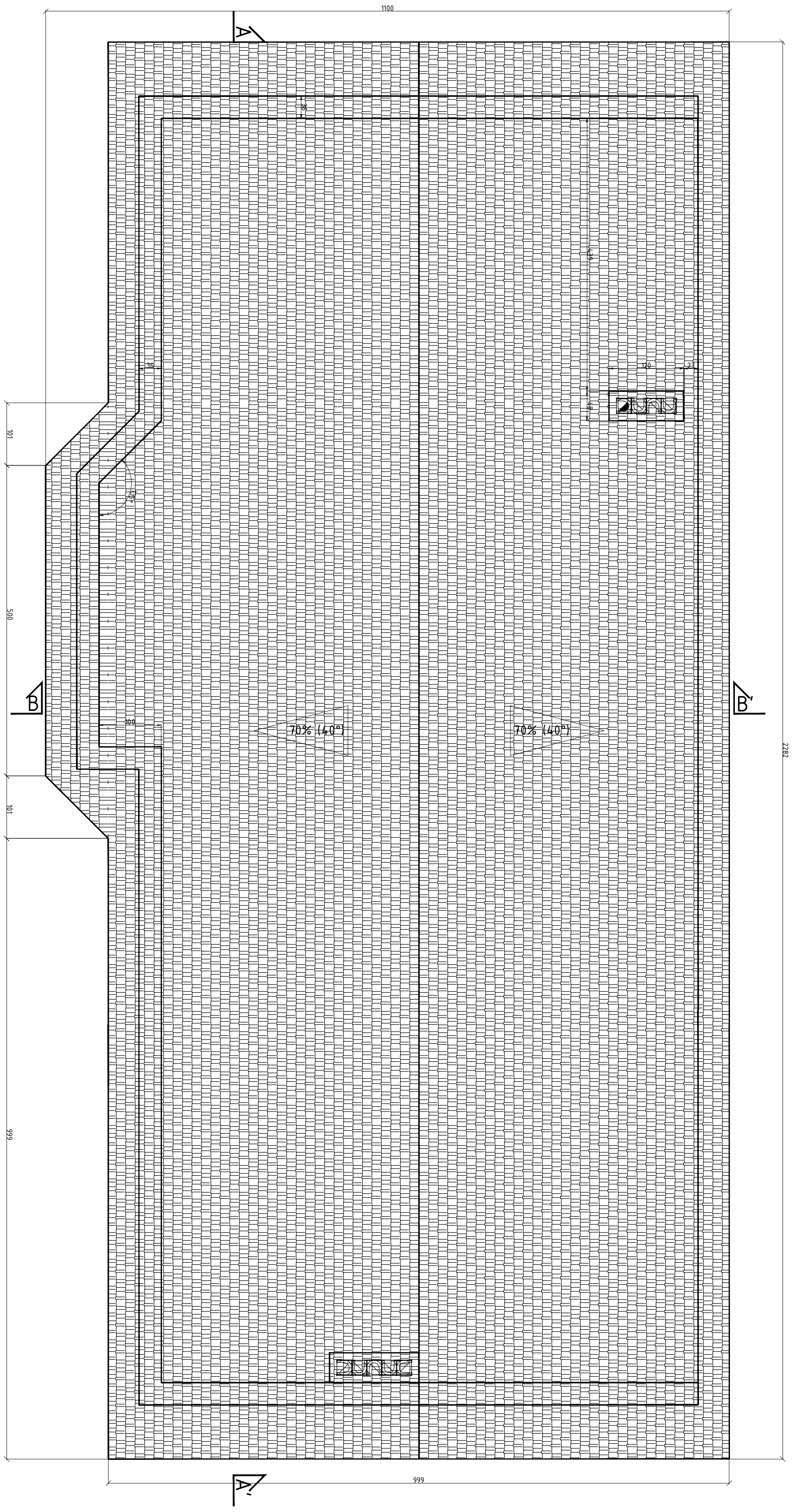
Element	Amount (units)	Length (m)	Cross-section Dimensions (m*m)	Area (m^2)	Total Mass 340 Kg/m^3
K-1	42	6,12	0,04*0,2	0,008	699
K-2	6	7,2	0,04*0,2	0,008	118
K-3	2	23,08	0,12*0,12	0,014	220
K-4	1	23,08	0,12*0,12	0,014	110
K-5	48	5,83	0,04*0,2	0,008	762
Counter Battens	85	30 uni.-23,08 55 uni.-10	0,02*0,05	0,003	1267
				Σ	3176

ROOF AREA = 185m²
SLOPE OF ROOF = 70% (40 °)

SOFTWOOD ELEMENTS:
- Construction - timber class C22,
- Formwork - timber class C18,

Wroclaw University of Technology			
Roof Structure - First Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 6

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



ROOF AREA = 185m²
SLOPE OF ROOF = 70% (40 °)

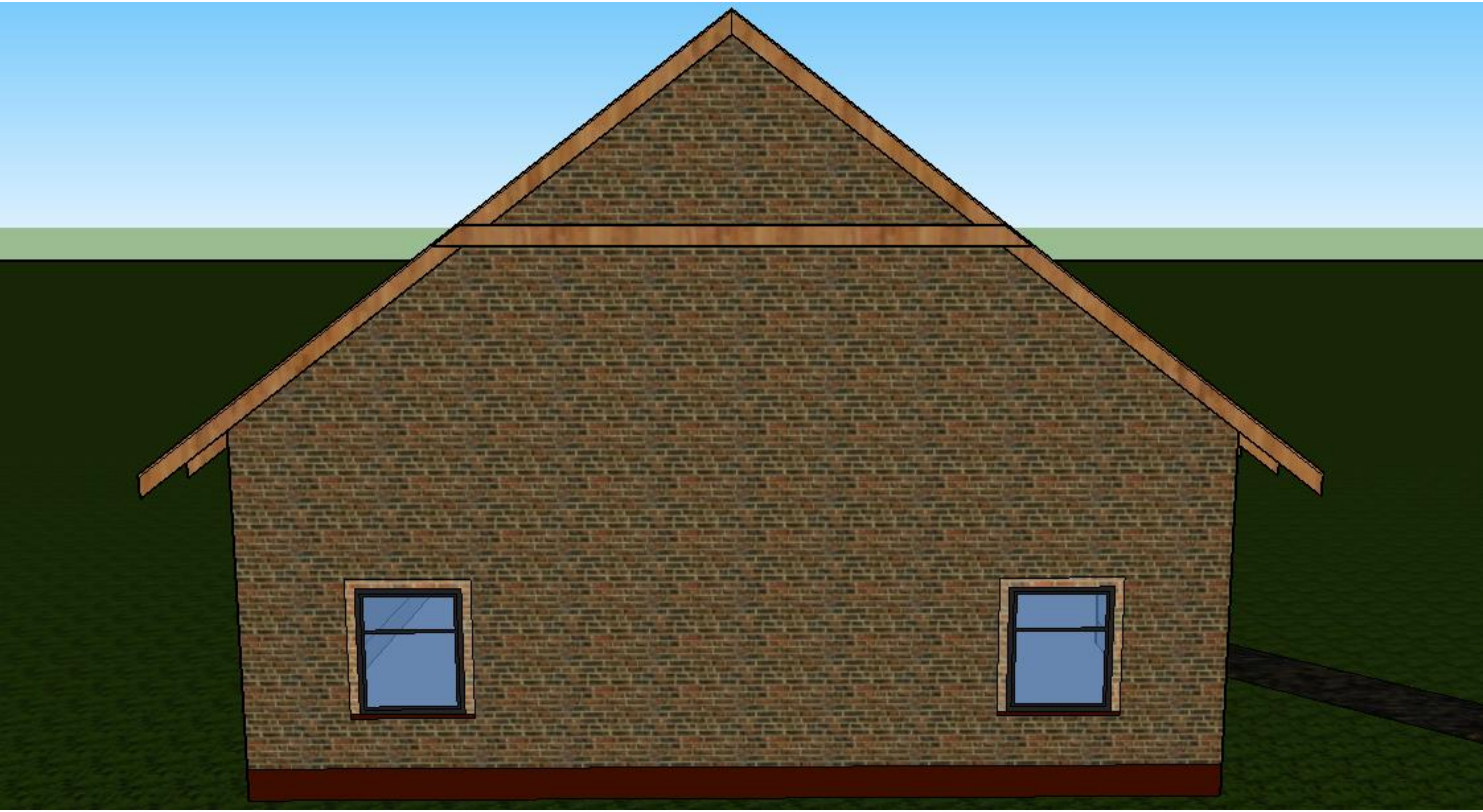
WOODEN ELEMENTS:
- Construction - timber class C22,
- Formwork - timber class C18,

Wroclaw University of Technology			
Roof Top View - First Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 7

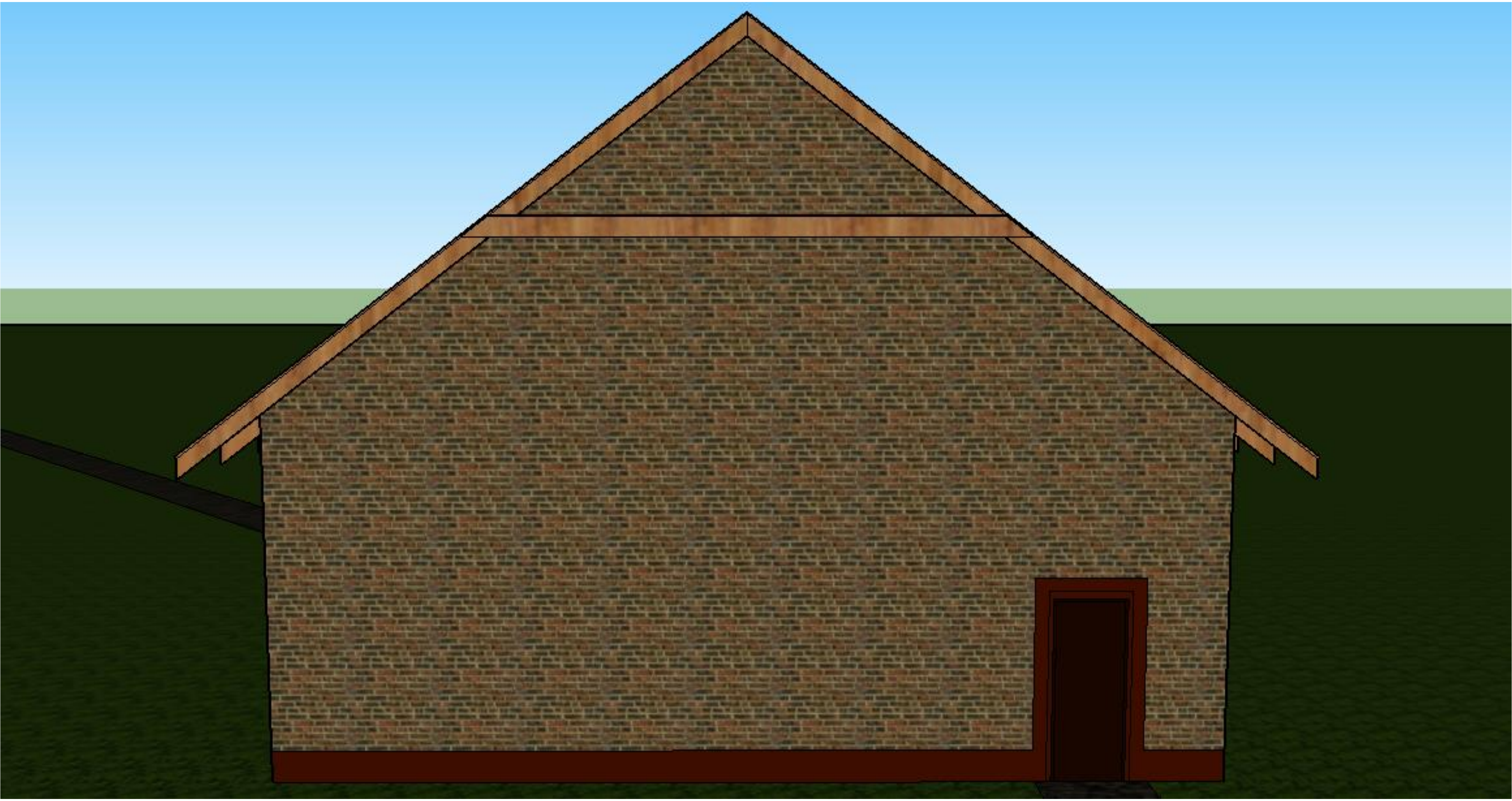
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Second Variant

Second Variant, Vertical Extension



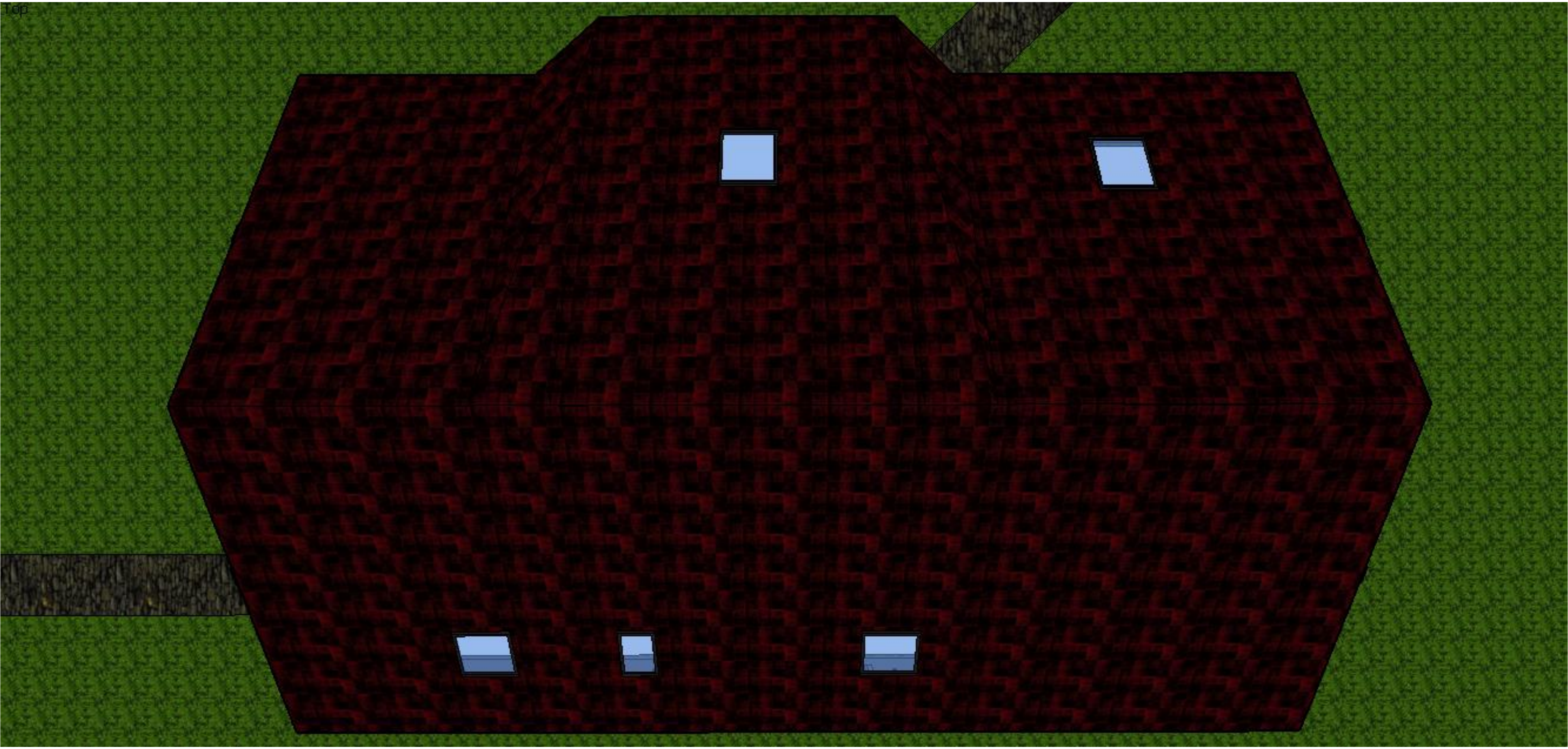
Second Variant, Vertical Extension



Second Variant, Vertical Extension

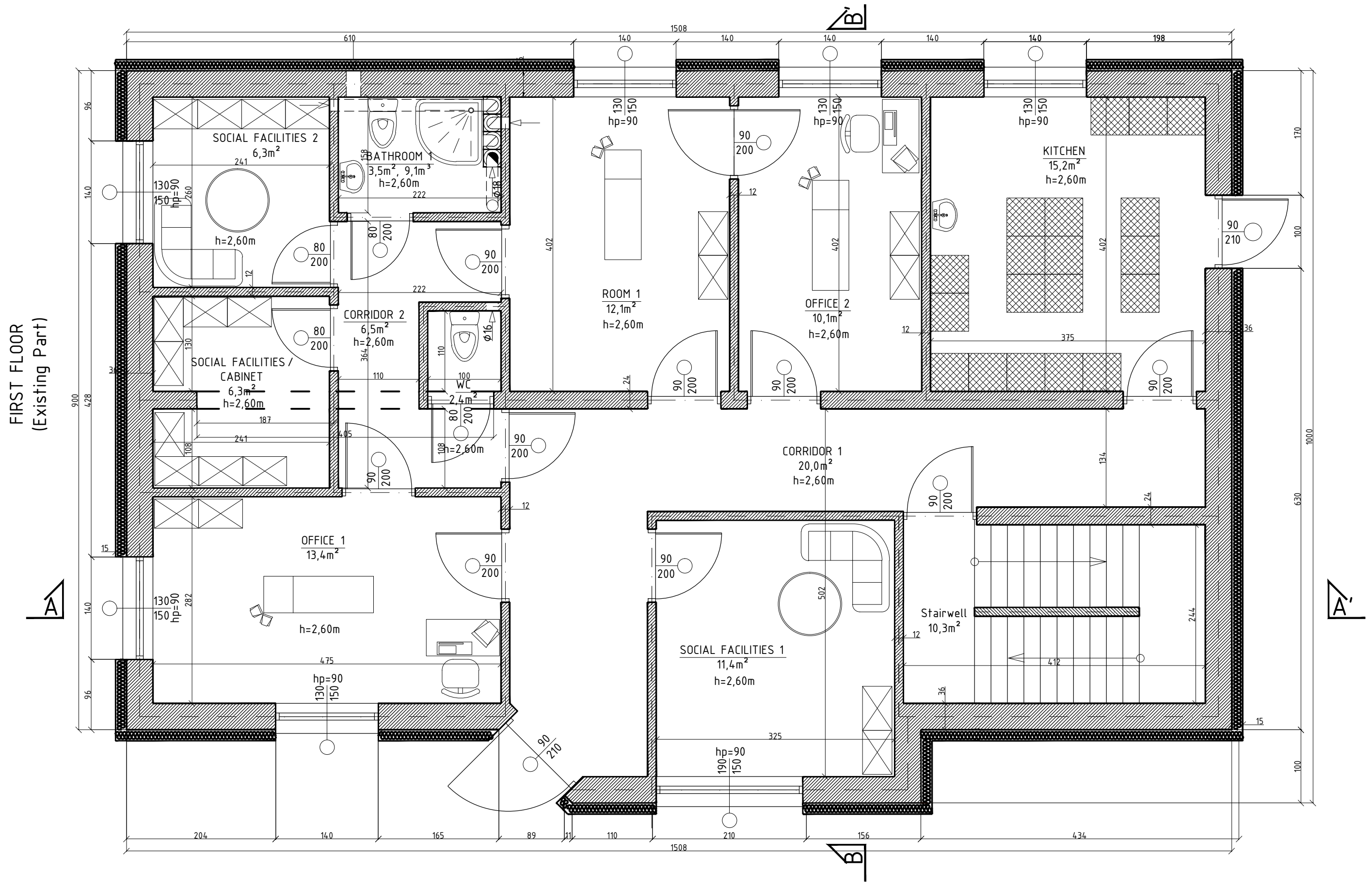
Second Variant, Vertical Extension

Second Variant, Vertical Extension



Second Variant, Vertical Extension

Compartment Areas	
Compartment Name	Area [m²]
Office 1	13,4
Office 2	10,1
Social Facilities 1	11,4
Social Facilities 2	6,3
Social Facilities/Cabinet	6,3
Bathroom 1	3,5
Corridor 1	20,0
Corridor 2	6,5
WC	1,1
Room 1	12,1
Stairwell	10,3
Kitchen	15,2
Σ	116,2



Wroclaw University of Technology

Architectural Plan – First Floor – Second Variant

Student	Bruno Gonçalves	Data:
Super-Visor	Dr. Łukasz Sadowski	Signature:
OBJECT: Office Building ADDRESS: Warsaw	Stage:	Scale: 1/50
		Drawing Number 1

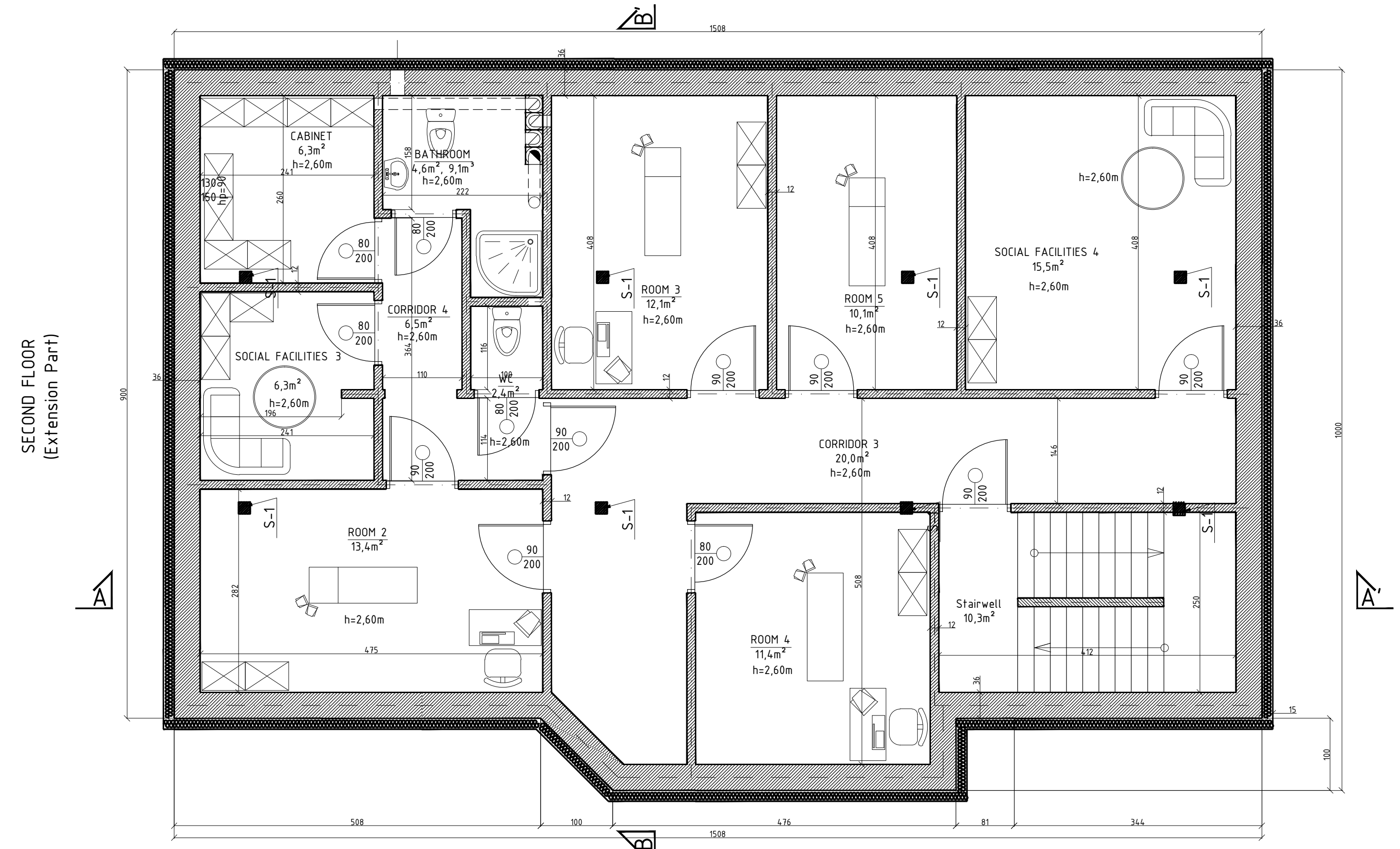
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Compartments Areas

Compartment Name	Area [m²]
Room 2	13,4
Room 3	12,1
Room 4	11,4
Room 5	10,1
Social Facilities 3	6,3
Social Facilities 4	15,5
Cabinet	6,3
Corridor 3	20,0
Corridor 4	6,5
WC	1,1
Bathroom	4,6
Stairwell	10,3
Σ	117,6

USEFULL AREA = 141m²
BUILDING AREA = 141m²

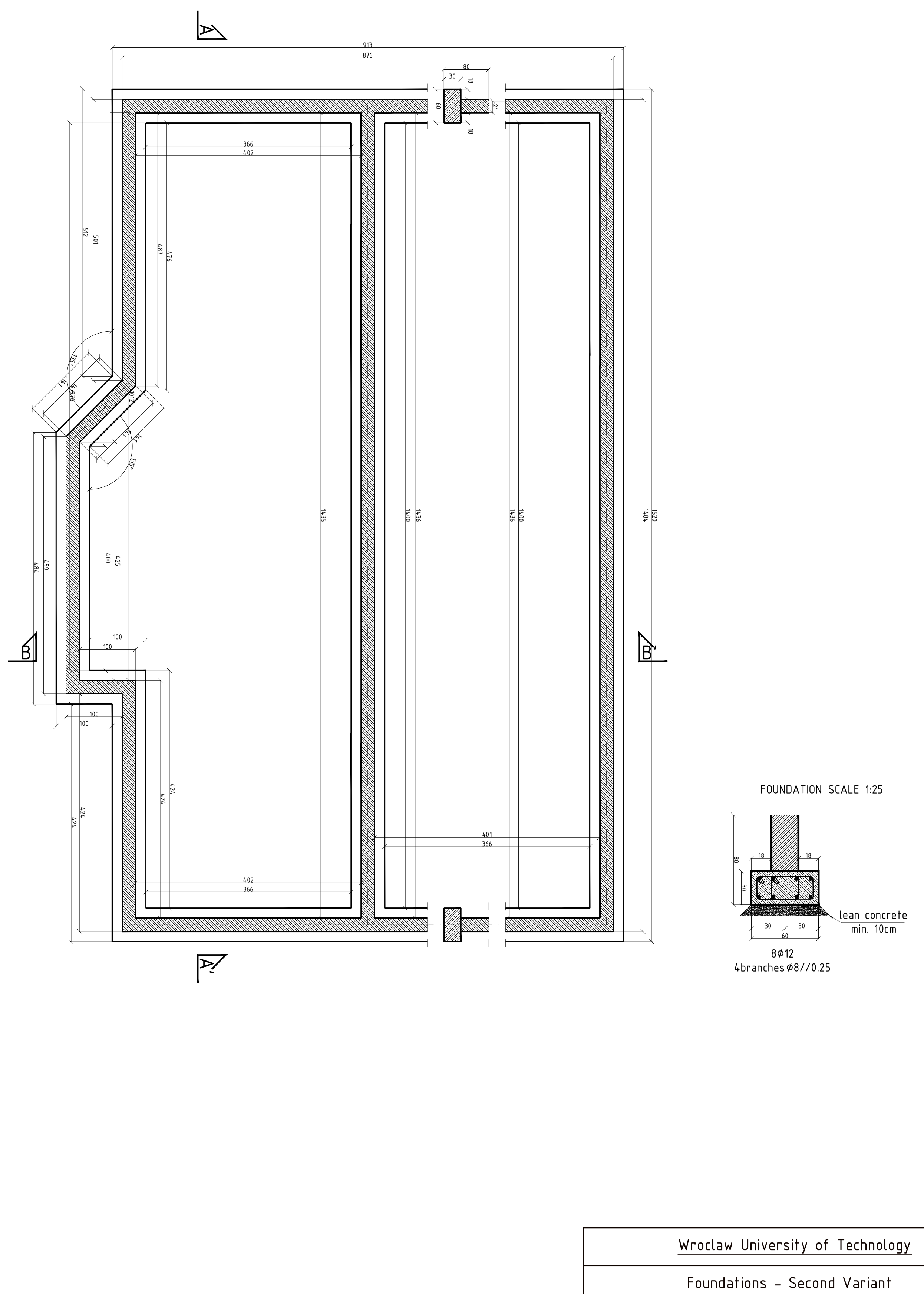
SECOND FLOOR
(Extension Part)



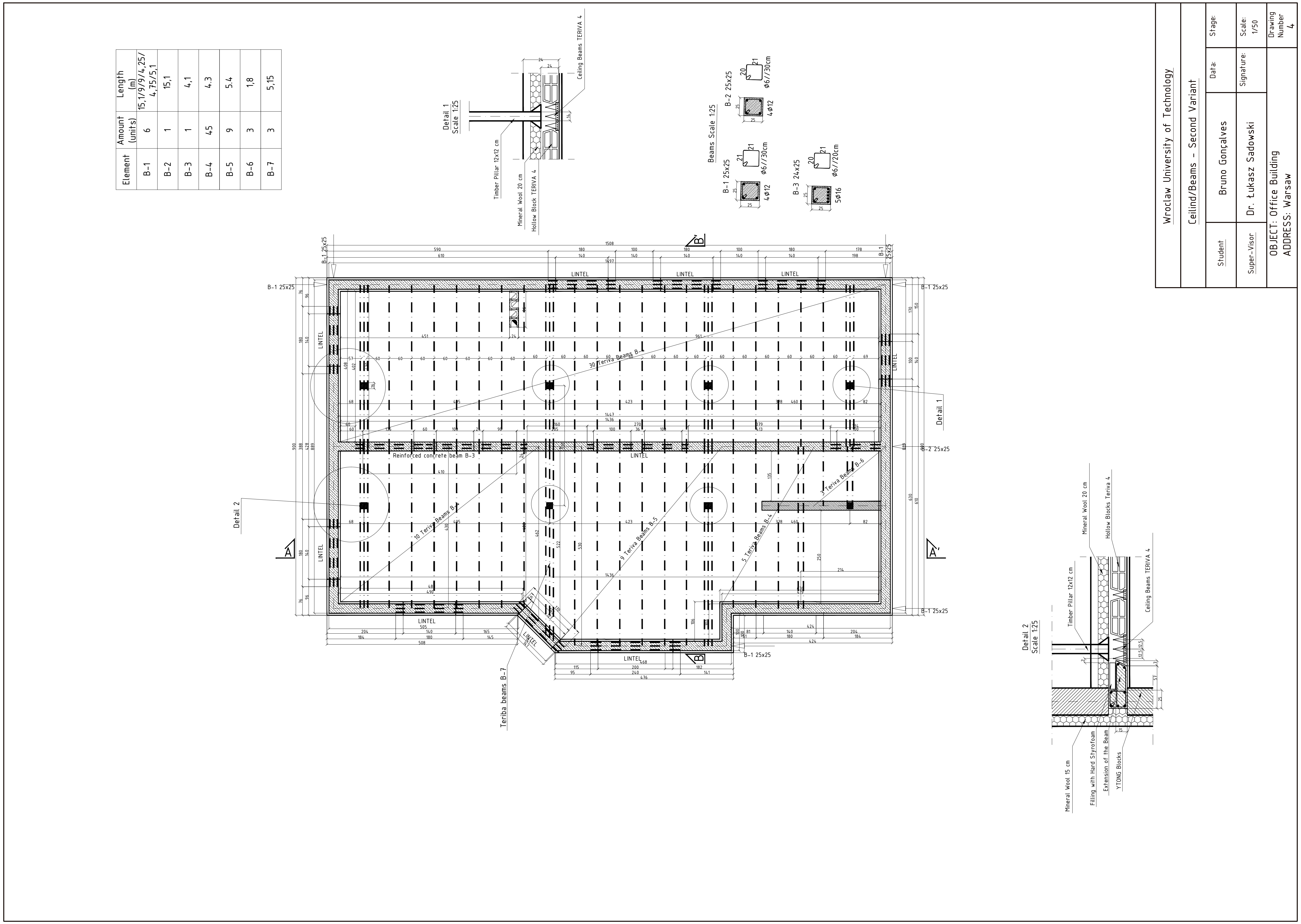
Wroclaw University of Technology		
Architectural Plan – Second Floor – Second Variant		
Student	Bruno Gonçalves	Data:
Super-Visor	Dr. Łukasz Sadowski	Signature:
Stage:	Scale: 1/50	Drawing Number 2
OBJECT: Office Building		
ADDRESS: Warsaw		

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Steel	Function	Flength (m)	Quantite (units)
#12	Longitudinal Reinforcement	12	44
#8	Stirrups	1.2	495



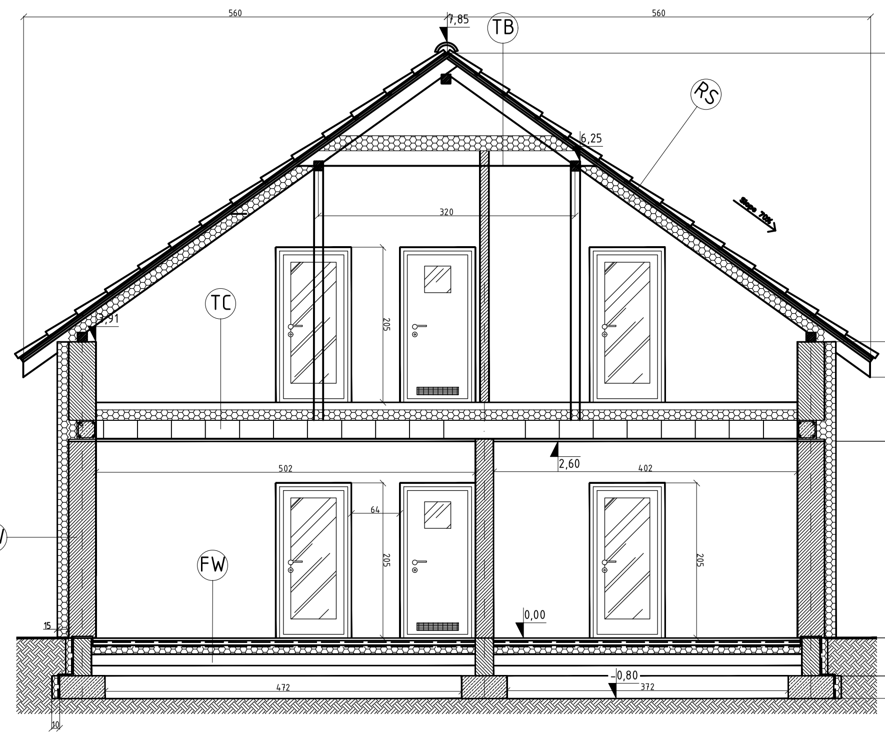
Wroclaw University of Technology	
Foundations - Second Variant	
Student: Bruno Gonçalves	Data: / Stage:
Super-Visor: Dr. Łukasz Sadowski	Signature: / Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw	Drawing Number 3



Element	Amount (units)	Length (m)
B-1	6	15,1/9/9/4,25/4,75/5,1
B-2	1	15,1
B-3	1	4,1
B-4	45	4,3
B-5	9	5,4
B-6	3	1,8
B-7	3	5,15

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

Section B-B



STATEMENT OF LAYERS

FW FOUNDATION BARRIER

1. Ceramic
2. Primer monolithic concrete (3.5cm)
3. Isolation - Mineral wool
4. Lean concrete gravel faded smoothly (10cm)
5, Interior plaster cement-lime
6. Coarse sand or gravel compacted stabilized (15cm)
7. Tamped earth

EW EXTERIOR WALL

1. Interior plaster cement-lime
2. Structural Wall outer monolayer ,
3. Blocks YTONG type
3. Outside mineral plaster on a grid of fiberglass

TC TERIVA CEILLING

1. Self leveling cementious underlayment (3 cm)
2. Protective layer - PCV foli
3. Isolation - mineral wool (35 cm)
4. Hollow Block
5. Reinforced concrete slab+beam
6. Cementious plaster work (1,5 cm)

TB TIMBER BEAM

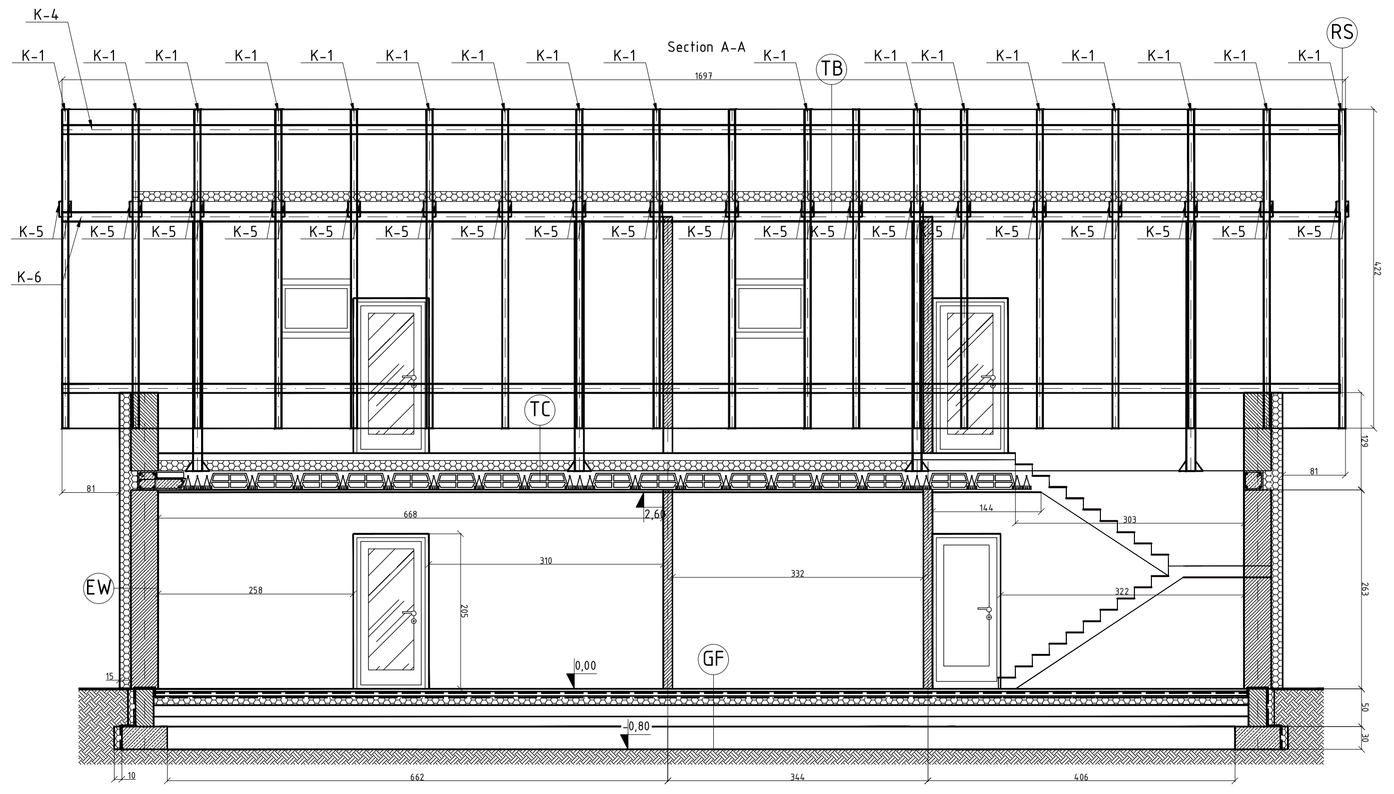
1. Timber Beam (8x20 cm), Class C22
2. Isolation - mineral wool (20 cm)

RS ROOF STRUCTURE

1. Timber Beam 8x20 cm, Class C22
2. Tiles (45 cm)
3. Counter Battens (2x5 cm)
4. Isolation - Mineral Wool (20 cm)

Wroclaw University of Technology			
Section B-B - Second Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 5

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



STATEMENT OF LAYERS

GF GROUND FLOOR

1. Ceramic
2. Primer monolithic concrete 3.5cm
3. Isolation - Mineral wool
4. Lean concrete gravel faded smoothly 10cm
5. Interior plaster cement-lime
6. Coarse sand or gravel compacted stabilized 15cm
7. Tamped earth

EW EXTERIOR WALL

1. Interior plaster cement-lime
2. Structural Wall outer monolayer
3. Blocks YTONG type
4. Outside mineral plaster on a grid of fiberglass
5. Isolation - mineral wool (15 cm)

TC TERIVA CEILLING

1. Self leveling cementious underlayment (3 cm)
2. Protective layer - PCV foli
3. Isolation - mineral wool (20 cm)
4. Hollow Block
5. Reinforced concrete slab+beam
6. Cementious plaster work (1,5 cm)

TB TIMBER BEAM

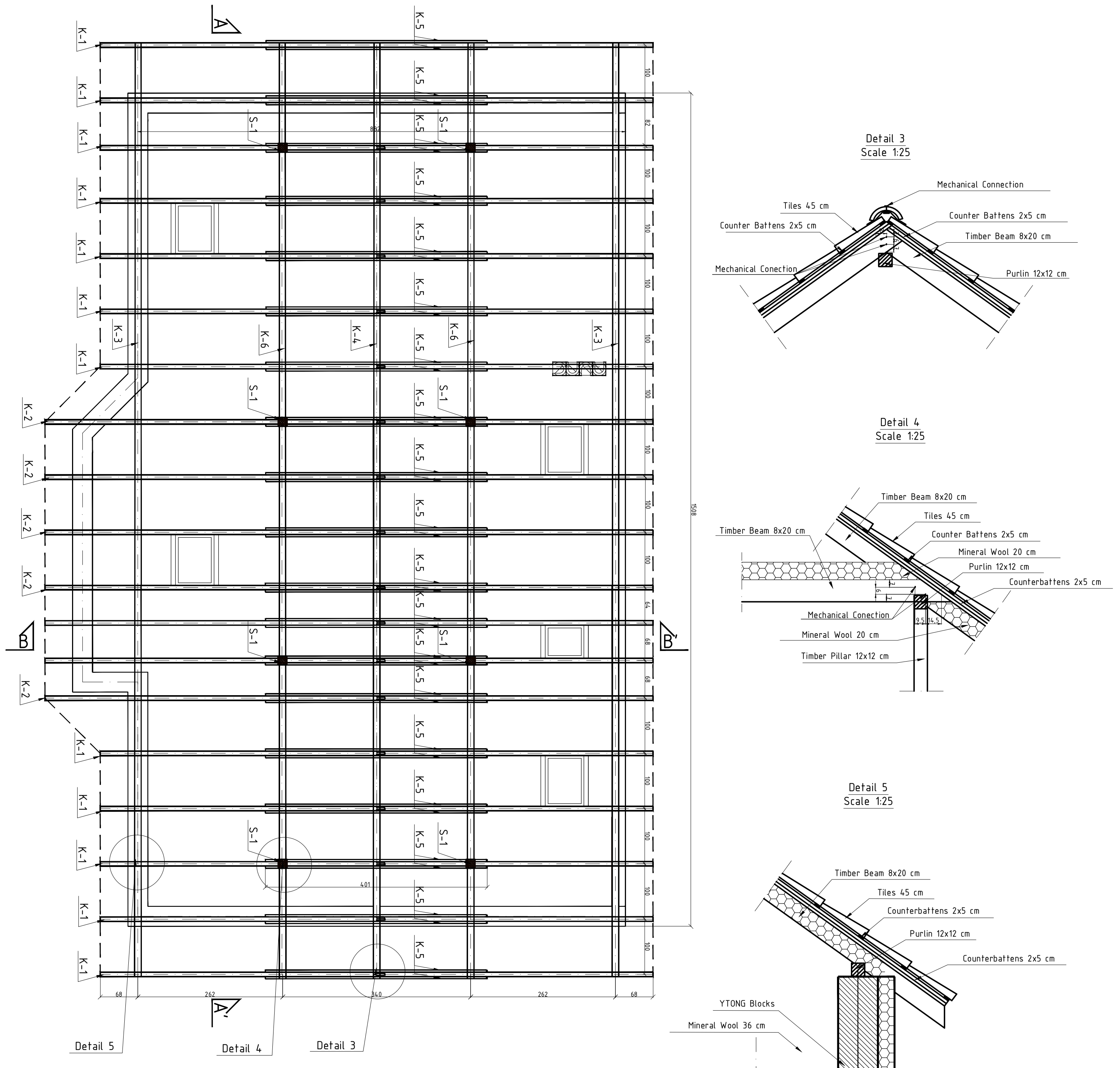
1. Timber Beam (8x20 cm), Class C22
2. Isolation - mineral wool (20 cm)

RS ROOF STRUCTURE

1. Timber Beam 8x20 cm, Class C22
2. Tiles (45 cm)
3. Counter Battens (2x5 cm)
4. Isolation - Mineral Wool (20 cm)

Element	Amount (units)	Length (m)	Cross-section Dimensions (m*m)	Area (m²)	Total Mass 340 Kg/m^3
K-1	31	6,12	0,04*0,2	0,008	516
K-2	7	7,2	0,04*0,2	0,008	137
K-3	2	17,08	0,12*0,12	0,014	163
K-4	1	17,08	0,12*0,12	0,014	82
K-5	38	4	0,04*0,2	0,008	414
K-6	2	17,08	0,12*0,12	0,014	163
S-1	8	3,37	0,16*0,16	0,0256	235
Counter Battens	70	30 uni.-17,08 40 uni.-10	0,02*0,05	0,003	931
				Σ	2641

Wroclaw University of Technology			
Section A-A - Second Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 6



Element	Amount (units)	Length (m)	Cross-section Dimensions (m*m)	Area (m^2)	Total Mass 340 Kg/m^3
K-1	31	6,12	0,04*0,2	0,008	516
K-2	7	7,2	0,04*0,2	0,008	137
K-3	2	17,08	0,12*0,12	0,014	163
K-4	1	17,08	0,12*0,12	0,014	82
K-5	38	4	0,04*0,2	0,008	414
K-6	2	17,08	0,12*0,12	0,014	163
S-1	8	3,37	0,16*0,16	0,0256	235
Counter Battens	70	30 uni.-17,08 40 uni.-10	0,02*0,05	0,003	931
				Σ	2641

ROOF AREA = 135m²
SLOPE OF ROOF = 70% (40 °)

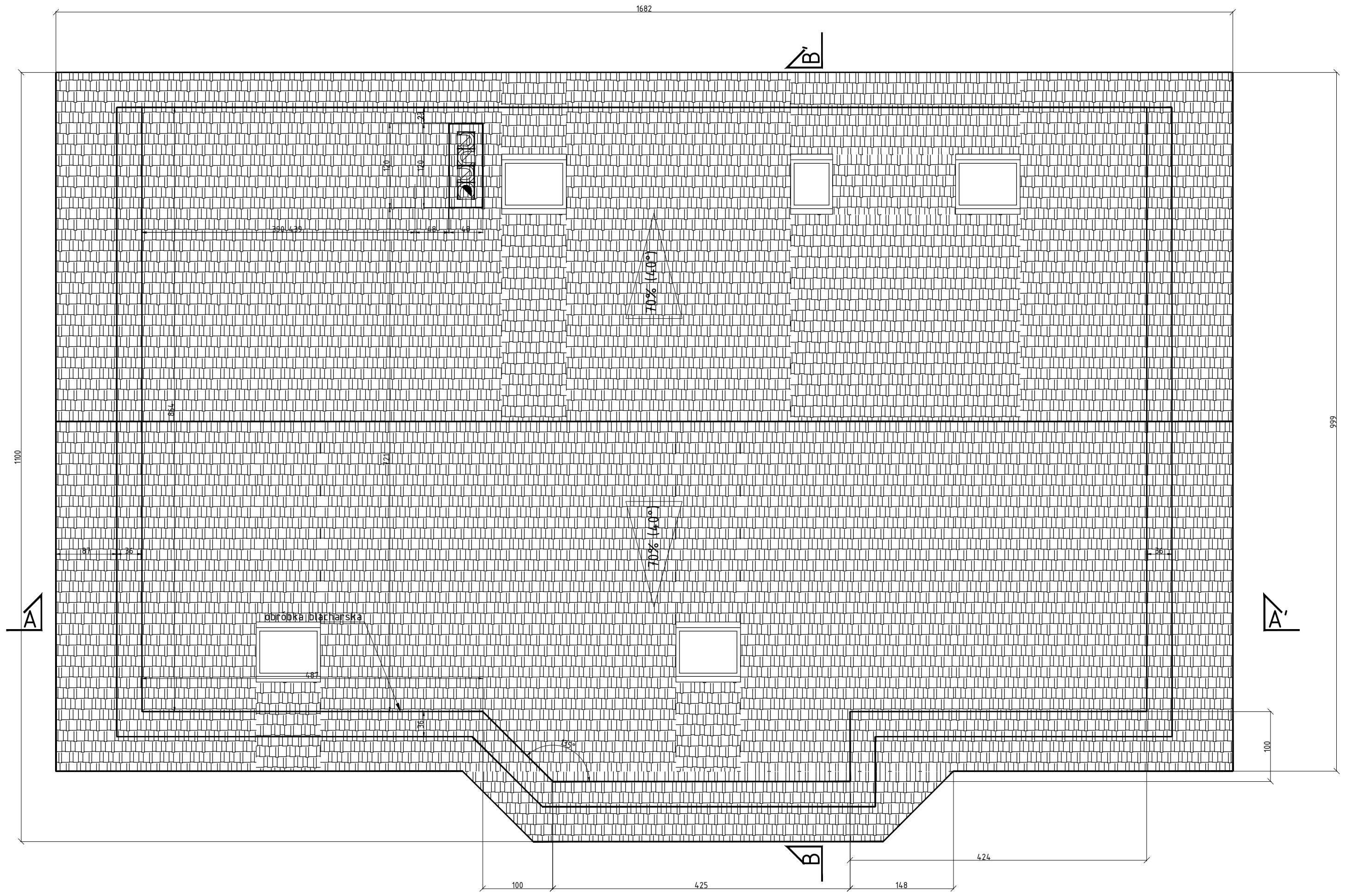
SOFTWOOD ELEMENTS:
- Construction - timber class C22,
- Formwork - timber class C18,

Wroclaw University of Technology			
Roof Structure - Second Variant			
Student	Bruno Gonçalves	Data:	Stage:
Super-Visor	Dr. Łukasz Sadowski	Signature:	Scale: 1/50
OBJECT: Office Building ADDRESS: Warsaw			Drawing Number 7

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

ROOF AREA = $135 m^2$
SLOPE OF ROOF = 70% (40 °)

WOODEN ELEMENTS:
- Construction - timber class C22,
- Formwork - timber class C18,



Wroclaw University of Technology	
Roof Top View - Second Variant	
Student	Bruno Gonçalves
Data:	
Stage:	
Super-Visor	Dr. Łukasz Sadowski
Signature:	
Scale:	1/50
OBJECT: Office Building ADDRESS: Warsaw	
Drawing Number	8

PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT