REPUBLIQUE DU CAMEROUN

Paix – Travail - Patrie

Ecole Nationale Supérieure des Travaux Publics

DEPARTEMENT DE GENIE CIVIL
DEPARTMENT OF CIVIL ENGINEERING

REPUBLIC OF CAMEROON

Peace – Work - Fatherland

UNIVERSITÀ DEGLI STUDI DI PADOVA

DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

ANALYSIS AND EVALUATION OF THE EFFECTS OF FIRE ON THE STRUCTURAL BEHAVIOR OF WOODEN BUILDINGS: CASE OF OLEMBE SOCIAL HOUSING IN YAOUNDE

A thesis submitted in partial fulfilment of the requirements for the degree Of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Structural Engineering**

Presented by:

DJIALA FOUTSOP Armel

Student number: **16TP21085**

Supervised by:

Prof. Carmelo MAJORANA

Co-supervised by:

Dr. Eng. Guillaume Hervé POH'SIE

Dr. Eng. Cyrille TETOUGUENI

Eng. GIUSEPPE Cardillo

Academic year: 2020/2021

REPUBLIQUE DU CAMEROUN

Paix – Travail - Patrie

Ecole Nationale Supérieure des Travaux Publics

DEPARTEMENT DE GENIE CIVIL
DEPARTMENT OF CIVIL ENGINEERING

REPUBLIC OF CAMEROON

Peace – Work - Fatherland

UNIVERSITÀ DEGLI STUDI DI PADOVA

DEPARTMENT OF CIVIL, ARCHITECTURAL AND ENVIRONMENTAL ENGINEERING

ANALYSIS AND EVALUATION OF THE EFFECTS OF FIRE ON THE STRUCTURAL BEHAVIOR OF WOODEN BUILDINGS: CASE OF OLEMBE SOCIAL HOUSING IN YAOUNDE

A thesis submitted in partial fulfilment of the requirements for the degree
Of Master of Engineering (MEng) in Civil Engineering

Curriculum: **Structural Engineering**

Presented by:

DJIALA FOUTSOP Armel

Student number: **16TP21085**

Supervised by:

Prof. Carmelo MAJORANA

Co-supervised by:

Dr. Eng. Guillaume Hervé POH'SIE

Dr. Eng. Cyrille TETOUGUENI

Eng. GIUSEPPE Cardillo

Academic year: 2020/2021

DEDICATION

I dedicate this work to my father

Djiala Clement

ACKNOWLEDGEMENTS

This work would not have been completed without the combined efforts of individuals who contributed directly and/or indirectly to its realization. I wish to express my sincere thanks and gratitude to:

- The **President of the jury** for the honour of accepting to preside this jury;
- The **Examiner** of this jury for accepting to bring his criticisms and observations to ameliorate this work;
- Professors **NKENG George ELAMBO** and **Carmelo MAJORANA** for all their academic and administrative support during these five years spent at ENSTP in the MEng program in partnership with University of Padua in Italy;
- The vice-director of ENSTP, Dr**. BWEMBA Charles** for his perpetual help and advices during our sojourn in this school;
- Prof. **MBESSA Michel**, the Head of Department of Civil Engineering for his tutoring and valuable advices;
- My supervisors Prof. Eng. **Carmelo MAJORANA**, Eng. **Giuseppe CARDILLO**, Dr Eng.**Guillaume Hervé POH'SIE** and Dr Eng, **Cyrille TETOUGUENI** for all the guidance, the advices and the patience provided to me during this work;
- **All the teaching staff of ENSTP** and **University of Padua** for their good quality teaching and the motivation they developed in us to continue our studies;
- Eng. **NKAMGING SIMO ABIMAEL JORDAN** for his availability and advice
- My father M. **DJIALA Clement** and my mother Mme **KENNE Therese** for their encouragement, devotion and financial support during all these years;
- My brothers and sisters **JOSILINE**, **ORNELA**, **BARDOLE**, **NELSON**, **CHRISTIAN**, **DADORESSE,** for their infinite love and to be able to count on them at all times and in all situations. I love you so much
- My family members for their unfailing support
- All my **classmates,** my **seniors,** my **juniors,** my **neighbours** and all **my friends** who were a source of motivation and tenacity.
- **EBA ONDOUA DIANE SALOME** for having shared our lives for these 5 years. Rest in peace engineer
- Also, I really thank those who directly or indirectly contributed and supported me in the accomplishment of this work;

LIST OF ABBREVIATIONS AND SYMBOLS

List of abbreviations

CLT	Cross Laminated Timber
CCA	Chromated Copper Arsenate
E	Integrity criterion
I	Insulation criterion
R	Mechanical resistance criterion
EN	European Norm
ELU	Etat Limite Ultime
ELS	Etat Limite de Service
GluLam	Glued Laminated Timber
ISO	International Standard Organisation
LSL	Laminated Strand Lumber
LVL	Laminated Veneer Lumber
NLT	Nailed Laminated Timber
OSB	Oriented Strand Board
PSL	Parallel Strand Lumber
SAP	Structural Analysis Program
SLS	Serviceability Limit State
ULS	Ultimate Limit State
INERIS	Institut National de l'Environnement Industriel et des Risques
min	Minutes
N	Newton
m	meter
°C	Celsius degree

°F	Fahrenheit degree
g	gram
Pa	Pascal

List of symbols

A_r	is the area of the residual cross-section, in m^2
b	width
d_0	depth of the zero-strength layer
d_{char}	Charring depth
d_{ef}	Effective charring depth
E_d	Design effect of actions
$E_{d,fi}$	Design modulus of elasticity in fire; design effect of actions for the fire situation
$E_{fi,d,t}$	is the design value of the relevant effects of actions in the fire situation at time t
$f_{c,0,d}$:	Design compressive strength along the grain
$f_{c,90,k}$	Characteristic compressive strength perpendicular to grain
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete
f_k	Characteristic strength
$f_{m,k}$	Characteristic bending strength
$f_{t,0,d}$	Design tensile strength along the grain
$f_{t,0,k}$	Characteristic tensile strength along the grain
$f_{t,90,d}$	Design tensile strength perpendicular to the grain
$f_{v,0,d}$	Design panel shear strength
$f_{v,d}$	Design shear strength
$f_{v,k}$	Characteristic shear strength
G_k	Permanent load

Q_k	Live load
h	Height or depth
h_p	is the panel thickness, in millimetres;
I_{Gz} or I_{Gy}	Moment of inertia
$k_{c,z}$ or $k_{c,y}$	Buckling coefficient
k_{crit}	Factor used for lateral buckling
k_{crit}	Factor used for lateral buckling
k_{def}	Deformation factor
k_{flux}	is a coefficient taking into account increased heat flux through the fastener;
k_h	Depth factor
k_j	Joint coefficient
k_{mod}	Modification factor for duration of load and moisture content
$k_{mod,fi}$	Modification factor for fire
k_{sys}:	System strength factor
k_v	Reduction factor for notched beams
l_{ef}	Buckling length
P	is the perimeter of the fire exposed residual cross-section, in metres;
$R_{fi,d,t}$	is the design value of the resistance of the member in the fire situation at time t
t	is the time [min].
$t_{d,fi}$	is the fire resistance period of the unprotected connection given in table 2.11
t_f	is the failure time, in minutes;
$t_{fi,d}$	is the design value of the fire resistance
$t_{fi,requ}$	is the required fire resistance time
w	Bending modulus
w_{creep}	Creep deflection
w_{fin}	Final deflection
w_{inst}	Instantaneous deflection

$w_{net,fin}$	Net final deflection
β_0	is the design charring rate for one-dimensional charring under standard fire exposure, in mm/min.
γ_M	Partial factor for material properties, also accounting for model uncertainties and dimensional variations
$\Theta_{cr,d}$	is the design value of the critical material temperature
Θ_d	is the design value of material temperature
θg	is the gas temperature [°C];
$\lambda_{rel,y}$	Relative slenderness ratio corresponding to bending about the y-axis
$\lambda_{rel,z}$	Relative slenderness ratio corresponding to bending about the z-axis
λ_y	Slenderness ratio corresponding to bending about the y-axis
λ_z	Slenderness ratio corresponding to bending about the z-axis
ρ_k	Characteristic density
$\sigma_{c,0,d}$	Design compressive stress along the grain
$\sigma_{m,d}$	Design bending stress about the principal axis
$\sigma_{t,0,d}$	Design tensile stress along the grain
$\psi_{1,1}$	Combination factor for frequent value of a variable action
$\psi_{2,1}$	Combination factor for quasi-permanent value of a variable action
ψ_{fi}	Combination factor for frequent values of variable actions in the fire situation

ABSTRACT

The main objective of this work was to analyse and evaluate the effect of fire on the structural behaviour of wooden structures. To achieve this objective, the methodology consisted of a static analysis of the structural elements at the ULS, as well as a verification at the SLS. The different mechanical properties of the different elements under different stresses were then analysed under fire load to assess their response in fire condition. At last, the analysis and interpretation of the different results was done. The case study considered was that of a residential building for social housing, ground floor + 2 storeys. The analysis of wooden structures under fire conditions for different stresses gave variable time resistance depending on the wood species chosen, the existence or not of a partial (in the case of the joist) or total (in the case of the column) fire protection and the choice of the method among the 2 analytical methods prescribed by the EC5 : part 1-2 (that of the reduced cross-section applicable to both hardwood and softwood and that of the reduced properties applicable only to softwood). The analysis of the unprotected joist for different species showed that Moabi (hardwood) performed better than Fir (softwood) with a time resistance of 97 minutes against 57 in bending, 101 minutes against 61 in shear and 52 minutes against 06 for deflection respectively in an analysis with the reduced cross-section method common to both species. Then, the analysis of the Fir joist protected by Moabi panels (with a fire time protection of 26 minutes) increased the resistance times of the joist in bending, shear and deflection respectively from 57 to 81 minutes, 61 to 87 minutes and 06 to 24 minutes for the analysis by the reduced cross-section method, and from 60 to 83 minutes, 65 to 90 minutes and 12 to 25 minutes for the analysis by the reduced properties method. Analysing the Fir column subjected to compression in the unprotected state and in the protected state by a firewall (of EI 18), the compression resistance time of the column will increase from 23 minutes to 41 for the effective cross-section method and from 26 minutes to 44 for the reduced property method. The previous results showed that the reduced cross-section method was much more conservative than the reduced properties method, this mainly due to the coefficient characterizing the depth of the zero-strength layer d_0 (taken at 7 mm by Eurocode 5) in evaluation of the cross section. Thus, a reduction of this coefficient from 7 to 5 mm made it possible to reduce the difference in the evaluation of the fire resistance time between the 2 methods from 05 to 0% for bending, 6.15 to 0% for shear, 11.54 to 0% for column compression and 50 to 16.67% for beam deflection.

Keywords: wood, fire, reduced cross-section method, reduced properties method

RESUME

L'objectif principal de ce travail était d'analyser et d'évaluer l'effet du feu sur le comportement structurel des structures en bois. Pour atteindre cet objectif, la méthodologie a consisté en une analyse statique des éléments structuraux à l'ELU, ainsi qu'une vérification à l'ELS. Les différentes propriétés mécaniques des différents éléments sous sollicitations différentes ont ensuite été analysé sous charge d'incendie pour évaluer leurs comportements au feu. Enfin, l'analyse et l'interprétation des différents résultats a été effectué. Le cas d'étude considéré était celui d'un bâtiment à usage d'habitation pour logement sociaux rez-de-chaussée + 2 étages. Ainsi, l'analyse des structures en bois en situation d'incendie pour différente sollicitation a donné un temps de résistance variable en fonction notamment de l'essence de bois choisi, de l'existence ou non d'une protection partielle (cas de la solive) ou totale (cas du poteau) et enfin de la méthode choisie parmi les 2 méthodes analytiques prescrite par l'EC5 : partie 1-2 (celle de la section efficace applicable autant aux feuillis qu'aux résineux, et celle des propriétés réduites applicable uniquement aux résineux). L'analyse de la solive non protégée a montré que le Moabi (feuillis) était plus performant que le Sapin (résineux) avec notamment des temps de résistance respectifs de 97 min contre 57 en flexion, 101 min contre 61 en cisaillement et 52 min contre 06 pour la déflexion dans le cadre d'une analyse avec la méthode de la section efficace commune aux 2 espèces. Ensuite, l'analyse de la solive en Sapin protégée par des panneaux de Moabi (de temps de protection de 26 min) a permis d'accroitre les temps de résistance de la solive en flexion, cisaillement et déflexion respectivement de 57 à 81 min, 61 à 87 min et 06 à 24 min pour l'analyse par la méthode de la section efficace et de 60 à 83 min, 65 90 min et 12 à 25 min pour l'analyse par la méthode des propriétés réduites. L'analyse du poteau en Sapin à la compression en état non protégé et protégé par un mur pare feu (EI 18) en Moabi a permis d'évoluer respectivement le temps de résistance à la compression du poteau de 23 min à 41 avec la méthode de la section efficace et de 26 min à 44 avec la méthode des propriétés réduites. Les précédant résultats ont montré que la méthode de la section efficace était beaucoup plus conservative que la méthode des propriétés réduites, ceci dû principalement au coefficient caractérisant la profondeur de la couche pyrolysée/ de résistance-zéro d_0 (pris à 7 mm par l'EC 5 : 1-2) dans l'évaluation de la section efficace. Ainsi, une réduction de ce coefficient de 7 à 5 mm a permis de réduire la différence d'estimation de temps de résistance entre les 2 méthodes de 05 à 0% pour la flexion, 6,15 à 0% pour le cisaillement, 11,54 à 0% pour la compression du poteau, et 50 à 16,67 % pour la déflexion de la poutre

Mots clés : bois, feu, méthode de la section efficace, méthode des propriétés réduites

LIST OF FIGURES

LIST OF TABLES

TABLE OF CONTENTS

GENERAL INTRODUCTION

The world is faced with an upsurge in fires due to various causes, notably technical causes (electricity, machinery, mixture of products, etc.), human causes (negligence, carelessness or malice) and natural causes (lightning, etc.). In the Cameroonian context, although the environment does not allow for precise statistics, it has been observed that a large proportion of fires in the construction sector are related to electrical failure. Furthermore, the absence of fire defence mechanisms, emergency evacuation systems and even rapid access routes contribute to the prolongation of the exposure time of goods and people to fire, thus leading to more harmful economic and social consequences, notably a great risk to national and world heritage, as well as loss of human life.

All building materials can be damaged by prolonged exposure to fire: steel warps, concrete spalls and wood burns. The problem of the choice of building material and its impact on the structural behaviour of structures in the event of fire, in order to be able to evaluate the life of the structure in the event of fire, is not to be neglected, because any time saving in a fire situation, even by the choice of building materials due to their fire resistance, is an important plus for the safety of goods and people in the event of fire. Today's modern wood-frame and mass timber buildings have a proven fire safety record. Efficient design and the use of advanced technologies in fire protection in wooden structures provide added assurance and help save lives. With this in mind, some countries, such as France, Switzerland, Austria, Denmark, Germany, Japan, China, Norway and many others, are encouraging the use of wood among existing construction materials (concrete, wood, steel, etc.), which approximatively transmits heat 10 times more slowly than concrete and 250 times less quickly than steel.

Given the diversity of wood species and the different properties they confer, the main objective is to analyse and evaluate the structural behaviour of wooden element in fire situations. To achieve this objective, the work will be structured around three chapters, the first of which will present the state of the art on wood, its structure, durability and properties, the second will define the design procedures for wooden structures, as well as the analytical methods for analysing these structures under fire loads, and the last chapter will be devoted to the presentation and interpretation of the results obtained.

CHAPTER 1: LITERATURE REVIEW

Introduction

Wood is an organic material, produced by a large number of woody plants and quite variable in properties. It's one of the most used natural building materials in the world. A number of valuable properties such as small bulk density compared to its relatively high strength, low heat conductivity, amenability to mechanical working, etc, makes wood a famous building material. Wood is a material described as environmentally friendly due to its relatively small carbon footprint. However, its use requires a perfect knowledge of its structure, composition, characteristics, typologies and defects, in order to confer a durability. In this Chapter, the presentation of the wood and the above-mentioned aspects, but also, common wooden structures and the effect of fire on them.

1.1. General Knowledge on Wood

Here, the presentation of different aspects of wood such as its structure, properties, species, common uses, defects and their treatment and prevention will be done.

1.1.1. Structure of wood

An understanding of the essence of this material precedes the mastery of the use of any material existing in nature. This undeniably passes by mastery of its composition as well on the physical plan as on the chemical plan; because, the comprehension of its composition informs at best on its assets as well as its defects and especially how to try to mitigate these defects and pathologies.

1.1.1.1. Physical structure

The physical structure of wood consists of a cross-section, perpendicular to the trunk or branch. Depending on the representation of the cross-section, wood can be known by two names, including "goal wood" or "standing wood" and "wire wood" or "threaded wood". The detailed representation of the wood structure is studied at two scales, one macroscopic and the other microscopic (Britannica.com, 2022)

a. Macrostructure

Examination of a stump or the transverse section (cross section) of a tree trunk shows three parts: pith, wood, and bark. Between the wood and bark is the cambium, although this thin layer of tissue is indistinguishable with the naked eye or a hand lens. Pith is normally small and located at the center of the transverse section. Wood is marked by the presence of concentric layers known as growth rings or annual rings. In temperate regions, one growth ring is normally produced during each season of growth, but false rings may also be present, and in some cases, certain rings may be locally discontinuous. In tropical regions, growth rings are formed in response to wet and dry periods or other, incompletely understood factors. For these reasons, the term growth ring is preferred over annual ring. Barring the above deviations, however, the number of growth rings, as counted in a transverse section near the ground, can be used to find the age of a tree (George T. 1991).

Parts of a tree trunk can be observed in the figure 1.1

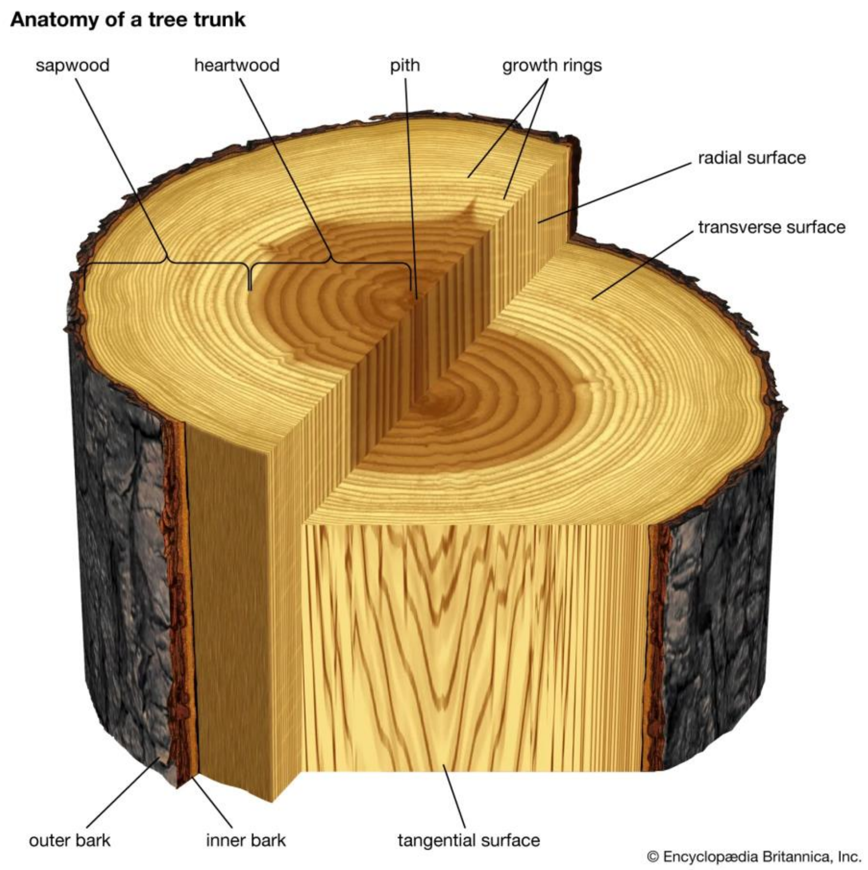


Figure 1.1. Transverse slice of tree trunk (Encyclopædia Britannica, Inc)

Bark surrounds the central cylinder of wood. It is differentiated into inner bark, which is relatively light-coloured and conducts synthesized food from the leaves downward, and outer bark, which is dark-coloured and dry, with an insulating function.

i. Earlywood, latewood, and pores

Growth rings are visible because of macroscopic differences in structure between earlywood and latewood—i.e., wood produced in the spring and later in a season of growth. The two kinds of wood may differ in density, color, or other characteristics. In coniferous species, latewood is darker in color and has a greater density. In the wood of broad-leaved species, the presence of pores is a characteristic macroscopic feature of growth rings. An example of a transverse section of eastern white pine is shown at Figure 1.2

Figure 1.2. Transverse section of eastern white pine (Courtesy of Michael Clayton, University of Wisconsin)

According to the relative size and distribution of pores, woods of broad-leaved species are further classified into ring-porous and diffuse-porous types. In ring-porous woods, such as oak and chestnut, the pores of earlywood are large compared with those of latewood. In diffuse-porous woods, such as basswood and poplar, all pores are about the same size and evenly distribute. An example of a transverse section of northern red oak is shown at Figure 1.3

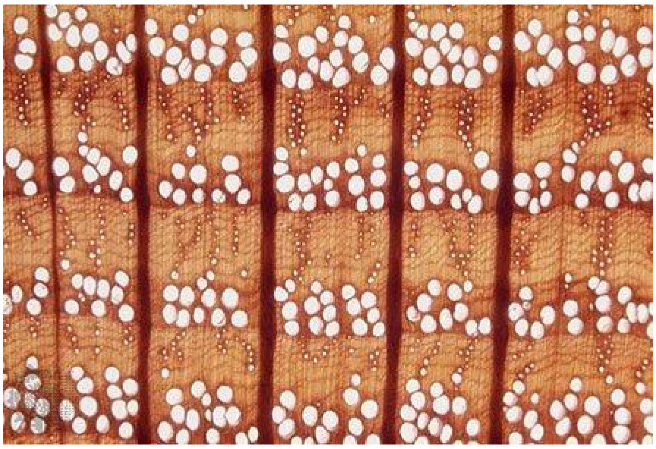


Figure 1.3. Transverse section of northern red oak (Courtesy of Michael Clayton, University of Wisconsin)

ii. Heartwood and sapwood

In many tree species, the central part of the transverse section of trunk is darker in color than the peripheral wood. This inner part is called heartwood, and the surrounding zone sapwood. Sapwood comprises the newer growth rings and participates in the life processes of a tree. As the diameter of the tree increases with growth, the older, inner layers no longer take part in the transport and storage of water and nutrients and become heartwood. After a certain age, heartwood exists in all species, even though there may be no color change.

iii. Rays and resin canals

A transverse section of trunk also shows linear features called rays radiating from pith to bark and ranging in width from very distinct, as in oak, to indistinct to the naked eye, as in pine and poplar. Certain softwoods, such as pine, spruce, larch, and Douglas fir, possess resin canals. In a transverse section examined with the naked eye or a hand lens, resin canals appear as small dark or whitish dots.

iv. Radial and tangential sections

Sections of the trunk made perpendicular to the cross-section show a different picture of the macroscopic features. Radial sections, i.e. longitudinal sections through the pit, are characterised by the parallel arrangement of growth rings and the appearance of rays in the form of striations called speckles (in species with clearly visible rays, such as oak). In tangential sections, i.e. longitudinal sections cut tangentially to the rings, the arrangement of the growth rings takes the form of a series of arcs or parabolas.

b. Microstructure

The microscope reveals that wood is composed of minute units called cells. According to estimates, 1 cubic meter (about 35 cubic feet) of spruce wood contains 350 billion at 500 billion cells. The basic cell types are called tracheids, vessel members, fibers, and parenchyma. Softwoods are made of tracheids and parenchyma, and hardwoods of vessel members, fibers, and parenchyma. A few hardwood species contain tracheids, but such instances are rare. Tracheids are considered a primitive cell type that gave rise, through evolution, to both vessel members and fibers (George T. 1991). The Figure 1.4 shows the type of cells present in hardwood and softwood.

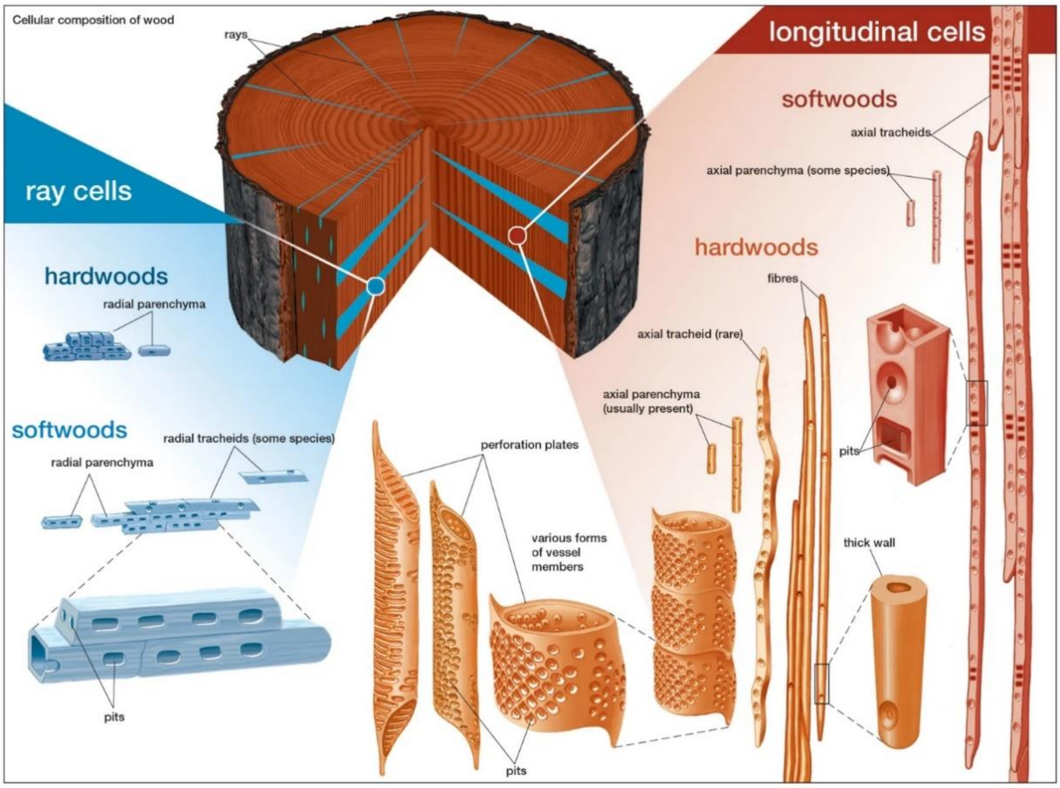


Figure 1.4. Types of cells present in hardwoods and softwoods (Encyclopædia Britannica, Inc.)

The wood of softwood species is composed predominantly of tracheids. These cells are mainly longitudinal, or axial: their long axis runs parallel to the axis of the trunk (vertical in the standing tree). Axial parenchyma is present in certain softwood species, but radial parenchyma is always present and constitutes the rays, sometimes together with radial tracheids.

In hardwoods the proportion of constituent cell types, vessel members, fibers, and parenchyma, depends mainly on species. Vessel members and fibers are always present and axially oriented;

axial parenchyma is seldom absent. Rays in hardwoods are made entirely of radial parenchyma cells.

All the above cells are tubelike. Tracheids and fibers have closed ends. Vessel members have ends wholly or partly open; in wood tissue, vessel members are connected end to end to form vertical pipelike stacks (vessels) of indeterminate length. The characteristic pores visible in the transverse section of hardwoods are actually vessel members. Axial tracheids in softwood species and vessel members in hardwood species are the principal water-conducting cells. Although fibers in hardwood trees may also participate in conduction, their main function is to provide mechanical support.

Parenchyma cells are bricklike in shape and very small. They are mainly concerned with the storage of food and its transport (horizontally in the case of radial parenchyma). Radial tracheids somewhat resemble parenchyma in shape and length, although their shape can be more irregular.

Almost all wood cells, even in living trees, are dead, that is, devoid of protoplasm and nucleus. The exceptions are a few layers of young cells produced during current growth by the cambium and by parenchyma cells located in sapwood. Cambium derives by differentiation of cells of the apical meristem, generative tissue that comprises the growing tips (stem, branches, and roots) of the plant and is responsible for primary growth, or growth in length. Cambium is considered to be lateral meristem; by producing new wood and bark, it carries out secondary growth, or growth in diameter. Microscopic observation of thin transverse sections shows the cambium to be a one-cell-wide layer of dividing initials and of a small but varying number of undifferentiated derivative cells, which together form the cambial zone. Further division and differentiation of the derivative cells gives rise to wood and bark.

Observed microscopically, the cells of wood appear to be composed of cell wall and cell cavity. Gaps of various shapes, called pits, are often seen in great numbers in the cell walls. Pits serve as passages of communication between neighboring cells and come in pairs, one in each of the adjoining cell walls, separated by a membrane. Other microscopic features are tyloses, plugs comprising various plant materials that obstruct the vessel members of hardwoods and that form mainly when sapwood is transformed to heartwood.

1.1.1.2. Chemical structure

The chemical composition of wood varies from species to species, but is approximately 50% carbon, 42% oxygen, 6% hydrogen, 1% nitrogen, and 1% other elements (mainly calcium, potassium, sodium, magnesium, iron, and manganese) by weight. Wood also contains sulfur, chlorine, silicon, phosphorus, and other elements in small quantity.

Aside from water, wood has three main components. Cellulose, a crystalline polymer derived from glucose, constitutes about 41–43%. Next in abundance is hemicellulose, which is around 20% in deciduous trees but near 30% in conifers. It is mainly five-carbon sugars that are linked in an irregular manner, in contrast to the cellulose. Lignin is the third component at around 27% in coniferous wood vs. 23% in deciduous trees. Lignin confers the hydrophobic properties reflecting the fact that it is based on aromatic rings. These three components are interwoven, and direct covalent linkages exist between the lignin and the hemicellulose. A major focus of the paper industry is the separation of the lignin from the cellulose, from which paper is made.

In chemical terms, the difference between hardwood and softwood is reflected in the composition of the constituent lignin. Hardwood lignin is primarily derived from sinapyl alcohol and coniferyl alcohol. Softwood lignin is mainly derived from coniferyl alcohol.

Chemical structure of lignin is shows by Figure 1.5

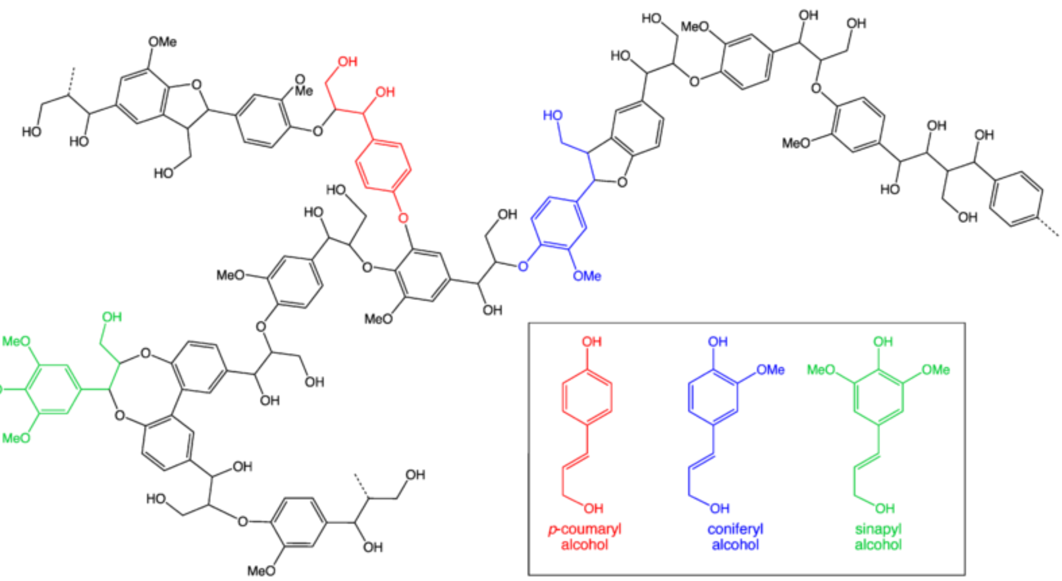


Figure 1.5. Chemical structure of lignin, which makes up about 25% of wood dry matter and is responsible for many of its properties (Encyclopædia Britannica, Inc.)

1.1.2. Characteristics of wood

The strength and quality of a timber construction is governed by its design, implementation and also, the performance of the materials that make up the timber. This performance depends on the different properties of wood.

1.1.2.1. Physical characteristics

a. Sensory characteristics

The color, luster, smell, taste, texture, grain, figure, weight and hardness of the wood are the sensory characteristics. These additional macroscopic characteristics are useful in describing a piece of wood for identification and other purposes.

Wood has a wide range of colors including yellow, green, red, brown, black and almost pure white woods. However, most woods have shades of white and brown. Variations can occur in the same piece of wood, depending on the color differences between heartwood, sapwood, early wood, late wood, rays and resin channels. The natural color can be altered by prolonged exposure to the atmosphere and by bleaching or staining. Some woods such as black locust, and many tropical species fluoresce.

The natural gloss is characteristic of certain wood species such as spruce, ash, lime, poplar and is most prominent on radial surfaces.

Volatile substances contained in wood are responsible for its smell and taste. Although difficult to describe, they are useful distinguishing characteristics in some cases.

The term texture describes the degree of uniformity in the appearance of the wood surface, usually transverse. Grain is often used as a synonym for texture, as in coarse, fine, or even texture or grain, and also to indicate the direction of the wood elements, whether straight, spiral or wavy for example.

As sensory characteristics, weight and hardness are included in a diagnostic rather than technical sense: weight is judged by simply lifting the hand and hardness by pressing with the thumbnail. The weight of common woods in temperate climates varies between 300 and 900 kg per cubic meter in the dry state, but there are lighter and heavier woods in the tropics, ranging from 80 to 1300 kg per cubic meter for balsa and lignum vitae, respectively

b. Density and specific gravity

Density is the weight or mass of a unit volume of wood, or the ratio of mass to volume of wood, while specific gravity is the ratio of wood density to water density. In the metric system of measurement, density and specific gravity are numerically identical; for example, the average density of Douglas fir wood is 0.45 grams per cc, and its specific gravity is 0.45, because 1 cc of water weighs 1 gram.

Due to its hygroscopy, the determination of the density of wood is more difficult than for other materials, because of the strong influence of the moisture content on its weight and volume. In order to obtain comparable figures, weight and volume are determined at specific moisture contents. The standards are oven-dry weight (virtually zero moisture content) and oven-dry volume or green volume (green referring to a moisture content above the fiber saturation point, which on average is about 30 per cent). Other expressions of density, such as those based on air dry weight and volume or green weight and volume, have some practical importance, such as for shipping wood, but are not accurate.

The dry mass of wood in a given volume is determined by the density, which is obtained by dividing the air-dry weight by the volume, air dry or green. Kiln dry volume is difficult to determine, at least by immersion in water, due to the hygroscopic nature of wood. Kiln-dried samples are first immersed in hot molten paraffin to form a thin protective layer, before being immersed in water. For small wood samples, mercury is sometimes used instead of water; a special apparatus (Breuil volumeter) is available. For regularly shaped specimens, the volume can be calculated on the basis of their dimensions. In addition, radiation methods are used to measure density directly.

The density of a wood sample can be assessed visually by observing the width (thickness) of the growth rings and the proportion of late wood. In general, late wood, due to its thicker cell walls and smaller cell cavities, is denser than early wood, and with increasing ring width, its proportion decreases in softwoods and increases in hardwoods with porous rings. Therefore, wider rings indicate lower density in softwoods and higher density in hardwoods with ring porosity. In diffusely porous hardwoods, the final wood is not clearly distinct, and ring width is not an indication of density.

The density of temperate woods varies from about 0.3 to 0.9 gram per cc, but the range worldwide is approximately from 0.2 to 1.2 grams per cc. Differences among species or samples of the same species are due to varying proportions of wood substance and void volume and to content of extractives. The density of wood substance is about 1.5 grams per cc, and practically no differences in this value exist among species.

c. Hygroscopy

Due to its cellular structure, wood has the ability to absorb various liquids and gases. Although it can absorb other liquids and gases, water is the most important. Thus, wood can absorb water in its liquid form if they are in contact, or as vapor from the surrounding atmosphere. Because of its hygroscopicity, wood, whether it is part of the living tree or a material, always contains moisture. (The terms water and moisture are used here without distinction.) Moisture affects all the properties of wood, but it should be noted that only the moisture contained in the cell walls is important; moisture in the cell cavities only adds weight.

The moisture content of a wood sample is calculated on the basis of its actual weight and its oven weight. It can also be determined directly using portable electrical moisture meters, which measure the change in the electrical properties of wood as the moisture content changes.

For example, the amount of moisture contained in the cell walls varies from about 20 to 40 percent, but for practical purposes it is assumed to be 30 percent (expressed as a percentage of the oven-dry weight of the wood). The fiber saturation point is the theoretical point at which the cell walls are completely saturated and the cell cavities are empty. Beyond this point, moisture enters the cavities and, when they are completely filled, the maximum moisture content that the wood can hold is reached. This maximum, which depends mainly on the density, can be very high. For example, a very light wood, such as balsa, can contain up to 800% moisture, pine 250% and beech 120%.

As for the moisture content of wood in living trees, it varies from about 30 to 300% depending on the species, the position of the wood in the tree and the season of the year. When green wood is exposed to the atmosphere, its moisture content gradually decreases. The moisture in the cell cavities is lost first. Over time, the moisture content drops to levels ranging (for temperate and sheltered locations) from about 6 to 25 percent (average 12 to 15 percent). Local conditions of temperature and relative air humidity determine the final moisture content.

The species and size of the wood do not have a practical influence on the final moisture content, although refractory species and larger woods take longer to reach it. It is important to note, however, that due to hygroscopy, the moisture content of air-dry wood does not remain unchanged even when the wood is stored under cover. On the contrary, it is subject to continuous variation, within certain limits, due to changes in temperature and relative air humidity.

Hygroscopy is therefore of paramount importance for wood because the moisture content in wood affects all the properties of the wood. For example, moisture content can increase the weight by 100% or more, with consequent effects on transport costs. Variation in moisture content causes wood to shrink or swell, changing its dimensions. Resistance to rot and insects is greatly affected. The working, gluing and finishing of wood as well as its mechanical, thermal and acoustic properties are all influenced by moisture content. Processing operations, such as drying, preservative treatment and pulping, are also affected.

d. Shrinkage and swelling

Wood undergoes dimensional changes when its moisture fluctuates below the fiber saturation point. Loss of moisture results in shrinkage, and gain in swelling. It is characteristic that these dimensional changes are anisotropic, different in axial, radial, and tangential directions. Average values for shrinkage are roughly 0.4 percent, 4 percent, and 8 percent, respectively. Shrinkage in volume averages 12 percent, but large variations are exhibited among species. These values refer to changes from green to oven-dry condition and are expressed in percentage of green dimensions. The differential shrinkage and swelling in different growth directions is attributed mainly to cell wall structure. The difference between axial and the two lateral (radial and tangential) directions can be explained on the basis of respective orientation of microfibrils in the layers of the secondary cell wall, but the reasons for the differences between radial and tangential directions are not well understood.Figure 1.6 showed the distortions in sawn wood due to shrinkage and swelling

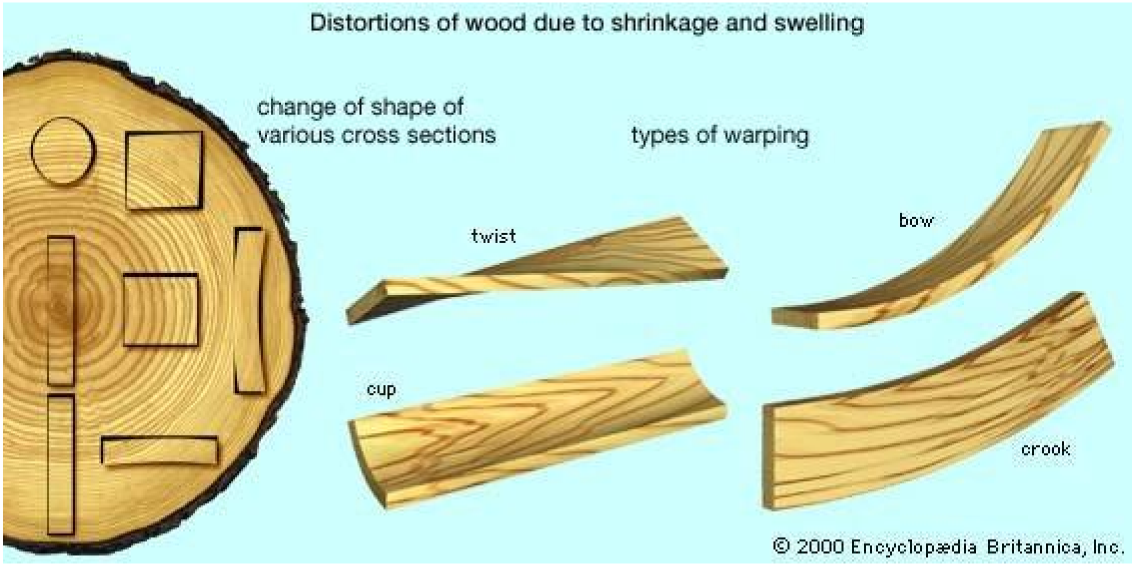


Figure 1.6. Distortions in sawn wood due to shrinkage and swelling (Encyclopædia Britannica, Inc.)

Distortions in sawn wood due to shrinkage and swelling. At left are shown the initial (dark outlines) and final shapes that may result from differences in radial and tangential shrinkage, depending on the original position of the wood in the tree trunk. Various kinds of warping, shown at right, may result from differential shrinkage and swelling or from differences in the distribution of moisture content in the wood.

In general, the factors that affect shrinkage and swelling are moisture content, density, content of extractives, mechanical stresses, and abnormalities in wood structure. The amount of shrinkage or swelling that occurs is approximately proportional to the change in moisture content. The higher the density of wood, the greater is its shrinkage and swelling, because denser (heavier) woods contain more moisture in their cell walls. For example, at the same moisture content, say, 15 percent, 1 cubic metre of a wood having a density of 0.8 gram per cc contains 120 kg of water, whereas the same volume of a wood having a density of 0.4 gram per cc contains only 60 kg of water. Extractives reduce shrinkage and swelling because they occupy spaces within cell walls that otherwise could be taken by water. Mechanical stresses (compression or tension) may cause permanent deformation of wood cells, which in turn affects shrinkage and swelling. Finally, abnormal structure results in greater shrinkage longitudinally but less in radial and tangential directions; change in volume remains about the same.

Dimensional changes in wood caused by shrinkage and swelling can result in opening or tightening of joints, change of cross-sectional shape, warping, checking (formation of cracks), case-hardening (release of stresses in resawing or other machining, with consequent warping) and collapse (distortion of cells, causing a corrugated appearance of the surface of lumber). Thus, the fact that wood shrinks and swells constitutes a great obstacle to its utilization.

1.1.2.2. Mechanical characteristics

The use of any material, especially for structural purposes, requires knowledge of its mechanical properties. As for wood in modern construction, it has the particularity of being light for high strength, mainly softwoods which are very practical for light constructions.
Spruce, for example, has a density of between 430 and 470 kg/m3 when air-dried (at a moisture content of 15%). This is five times (5 times) less than concrete and seventeen times (17 times) less than steel. Hardwoods such as oak are denser and have fewer uses in carpentry today, which was not the case in the Middle Ages when they were preferred to softwoods. Oak wood, for example, has a density of between 610 and 980 kg/m3.
The wood is very resistant to compression and tension in the direction of the fibers, and quite resistant to transverse bending (especially in the case of glued laminated timber). However, although it does not break, it deforms if the cross-section of the parts subject to bending (crossbeams, beams) is insufficient. In compression, the problem of buckling, due to the flexibility of the wood, has to be solved by a relatively small height-to-width ratio. However, the compressive strength of wood is high. For the same strength, wood requires a larger cross-section than steel or concrete.

1.1.2.3. Thermal properties

Although temperature has an impact on the dimensions of wood through expansion and contraction, this impact is considered small compared to the shrinkage and swelling caused by changes in moisture content. In most cases, these temperature-related expansions and contractions are negligible and of no practical importance. Only temperatures below 0°C (32°F) are likely to cause surface cracks; in living trees, uneven contraction of the outer and inner layers can lead to frost cracks.

Due to its cellular structure, which traps air in small volumes, wood is a poor conductor of heat (high thermal insulation capacity) compared to materials such as metals, marble, glass and concrete. Thermal conductivity is highest in the axial and radial direction and increases with density and moisture content, so light and dry woods are better insulators.

When exposed to sufficiently high temperatures, wood burns: it is an excellent fuel. This property makes wood suitable for heating, but is disadvantageous for its technical use. The maximum calorific value of one kilogram of kiln-dried wood is on average 4,500 kilocalories (with a range of 4,100 to 6,800 kilocalories). In general, softwoods have a higher calorific value than hardwoods. Also, moisture reduces the calorific value; air-dried wood has a calorific value about 15% lower than kiln-dried wood.

Wood must be heated to about 250°C (about 480°F) for a spark or flame to ignite it, but at a temperature of about 500°C (about 930°F), ignition is spontaneous. The flammability of wood can be reduced by chemical treatment.

1.1.3. Wood species

Woods, according to their properties, are classified into two principal categories: hardwoods and softwoods (GUITIYA, 2020).

1.1.3.1. Hardwoods

They are characterized by broad, flat leaves supported by veins. Most often, their leaves are deciduous, i.e. these trees lose their leaves in autumn to better withstand dry conditions. This category of wood does not produce resin. There exist several types, including:

- **Hardwoods**: They are dense and have qualities are strength, elasticity and durability (oak, ash, elm, chestnut, beech, walnut, hornbeam, acacia);

- **White woods**: They are lighter in weight, lightly coloured and less durable than the previous ones. They are classified as soft whitewoods (poplar, aspen, birch, lime) and semi-hardwoods (alder, plane tree, chestnut, maple);
- **Fine woods**, which are at least as hard as those in the first category, but which differ from them in that they are much more resistant to friction (boxwood, cormier, pear);
- **Exotic woods**: They generally have a high density and a bright colour, which makes them suitable for cabinet making. The most used are mahogany, rosewood, ebony, teak, tulipwood, etc. tulipwood, etc. The interlocking texture of the fibres increases the difficulty of shaping.

1.1.3.2. Softwoods or conifers

They are characterized by needle-like leaves, which are persistent in winter. Most conifers have conifers have resin-secreting cells in their bark, wood or leaves and are very hardy and are very strong but light. They are very straight and of very even quality under long lengths. The most commonly used are fir, spruce, pine, larch.

1.1.4. Wood defects

Wood is subject to degradation by bacteria, fungi, insects, marine borers, and climatic, mechanical, chemical, and thermal factors. Degradation can affect wood of living trees, logs, or products, causing changes in appearance, structure, or chemical composition; these changes range from simple discoloration to alterations that render wood useless. Wood is degraded or destroyed under the action of multiple factors such as: natural forces, attack by insects, bacteria, fungi, seasoning and processing, as will be seen below.

1.1.4.1. Defects caused by natural forces

- **Knots:** Knots are the most common defects caused due to natural forces. During the growth of a tree, branches close to the ground or lower branches die. Bases of those branches remain in the tree as it grows. These bases may create imperfection known as knots. They decrease the strength of the wood and thus lower its value for structural uses.
- **Twists:** Twist in timber rotates the ends of the timber in opposite directions. The main reason behind this defect is twisting of the trees by the strong wind. Also, trees that are covered by taller trees are believed to bend in search of sunlight.

- **Spiral grain:** It is caused by the twisting of the tree during its growth.
- **Burls:** Also called excrescences. They are formed when a tree receives an injury in its young age. Due to this injury, irregular projections then appear on the body of the tree.
- **Resin canals:** Some trees which are exposed to high winds, develop internal splits. These splits fill with resin or gum to make the wood resinous.
- **Shakes:** Shakes are timber defects that occur around the annual ring or growth ring of a timber. In other words, cracks or splits in the woods are called shakes. They are caused by shrinkage, old age, cold weather, rapid drying, etc. There many categories: Heart/Star shakes, Ring shakes, frost shakes, radial shakes.

These defects can be observed in the figure 1.7.

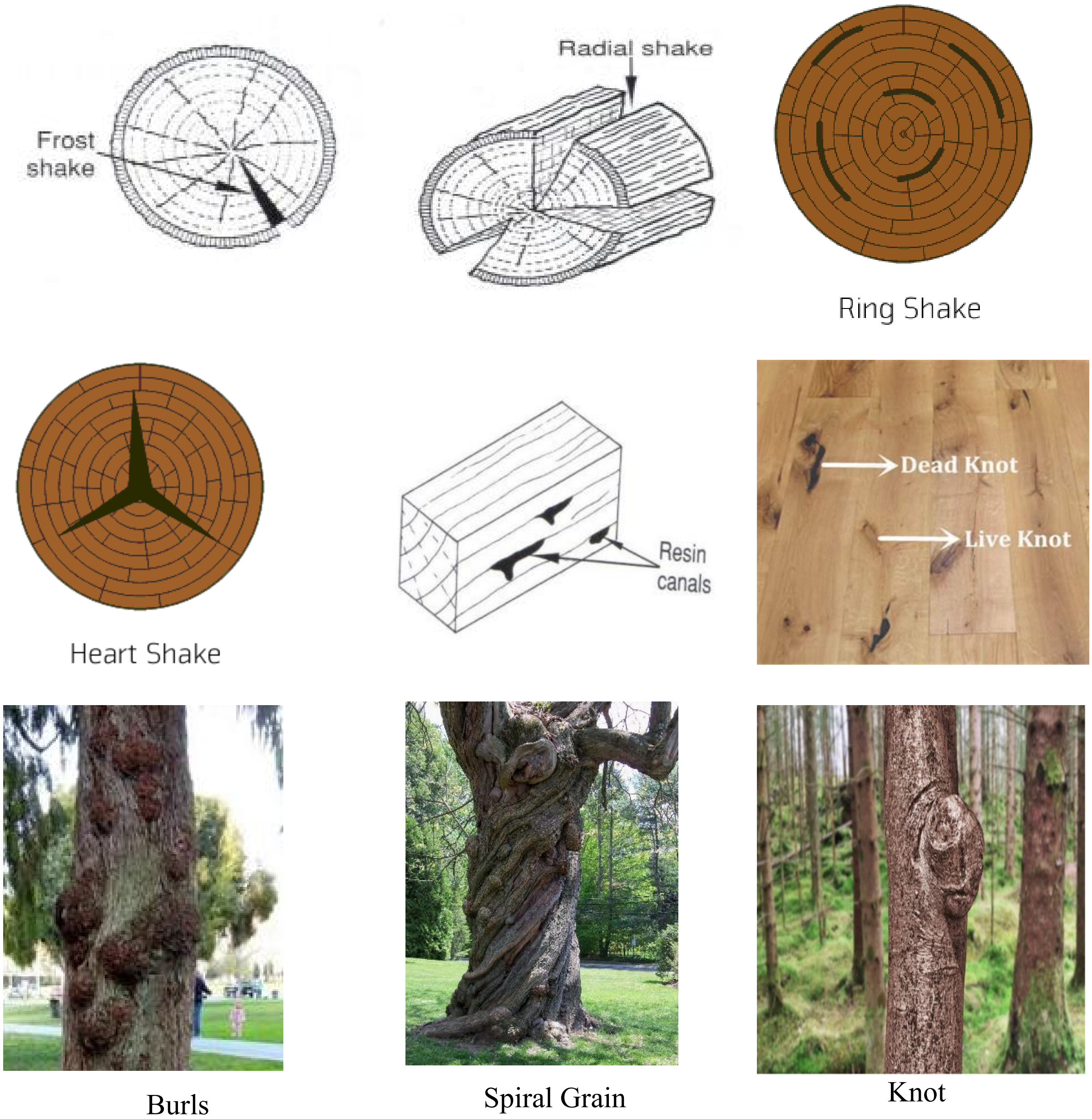


Figure 1.7. Natural defects in wood (greyandsanders.com, 2022)

1.1.4.2. Defects caused by fungi, insects and marine borers

a. Fungi

Fungi that attack wood are responsible for discoloration or decay. Decay fungi are, by far, the most important cause of wood loss. Decay is not an innate property of wood, however; it takes place only if the conditions of exposure namely: moisture, air, and temperature, are suitable for growth and activity of fungi. They cause several defects, some of which will be presented below.

- **Blue stain** (sap stain) of pines is the most common and serious consequence of attack by stain fungi. The sapwood becomes bluish or blackish, usually in wedge-shaped patches.
- **Dry rot** is caused by a type of fungi that eats wood and make food by converting timber into dry powder form. This occurs mainly when there is no ventilation of air or if the wood improperly seasoned.
- **Wet rot** is caused by fungi that decompose the timber and convert it into a grayish-brown powder form. Wet rot causing fungi grows mainly when there are alternate dry and wet conditions of timber.
- **Heart rot** occurs in trees when fungi attack the heartwood through its newly formed branch. This type of fungi makes the tree hollow by consuming heartwood.
- **White rot** which is caused by some types of fungi, that attack lignin of timber and leaves cellulose compounds; hence the wood will turn into white color,
-

Dry Rot

Wet Rot

White Rot

Heart Rot

Blue stain/Sap Stain

Figure 1.8. Defects in wood caused by fungi (theconstructor.org,2022)

b. Insects

Insects too can attack the wood of living trees, logs, or products. Once trees are fell, the region between wood and bark (rich in nutrients) is especially vulnerable to insect attack, and for this reason, prompt debarking is a protective measure. Insects bore holes and tunnels, and some reduce the interior of wood to dust, leaving only a thin outer layer. Conditions of exposure are the same as for fungi, i.e suitable temperature, moisture, and air. Infested wood can be rendered free of insects at temperatures of 50–60 °C, by introducing insecticides, exposure to toxic gases or surface coatings of paint or varnish. Defects caused by termites and beetles are seen in the figure 1.9.

Figure 1.9. Termites at left and beetles at right in wood (vulcantermite.com,2022)

c. Marine borers

Marine borers (certain species of mollusks and crustaceans) attack wooden structures in seawater (wharf pilings, boats, and other submerged wood) or immediate environment, and cause severe damage. The wood attacked by marine borers is of less strength and discoloured. All wood species are vulnerable, but toxic extractives (as in certain tropical woods) provide some temporary protection. Preservative treatment imparts considerable resistance to these organisms

Figure 1.10. Marine borers in wood (theconstructor.org,2022)

1.1.4.3. Defects caused by seasoning

Seasoning is the process of removing moisture from green wood to improve its serviceability, by exposure to air or to an artificial heat. Faulty method of seasoning causes serious defects in woods. During seasoning of timber, exterior or surface layer of the timber dries before the interior surface. So, stress is developed due to the difference in shrinkage. Some of the defects resulting from improper seasoning are as follows:

- **Bow**: Curvature formed in direction of the length of the timber, caused by poor stacking
- **Cup**: Curvature formed in the transverse direction of the timber. It is caused by unequal amounts of shrinkage along the growth rings.
- **Check or surface splits:** Check is a kind of crack that separates fibers, lies along the grain, and is Caused by rapid drying out on the surface of the wood.
- **End split:** It is a special type of check that extends from one end to another. Occurs at the transverse exposed end of the board. Caused by rapid drying out from the sun, and is prevented by painting the ends of the timber with bituminous paint (waterproof).

- **Honey combing:** Stress is developed in the heartwood during the drying process or seasoning. For these stresses, cracks are created in the form of honeycomb texture.
- **Twisting/warping**: When the ends of the board are twisted in opposite directions. Caused by shrinkage along spiral or interlocking grain.
- **Springing**: When the face of the board remains flat and the edge bends inwards to form a curve. Caused by shrinkage longitudinally along irregular grain.

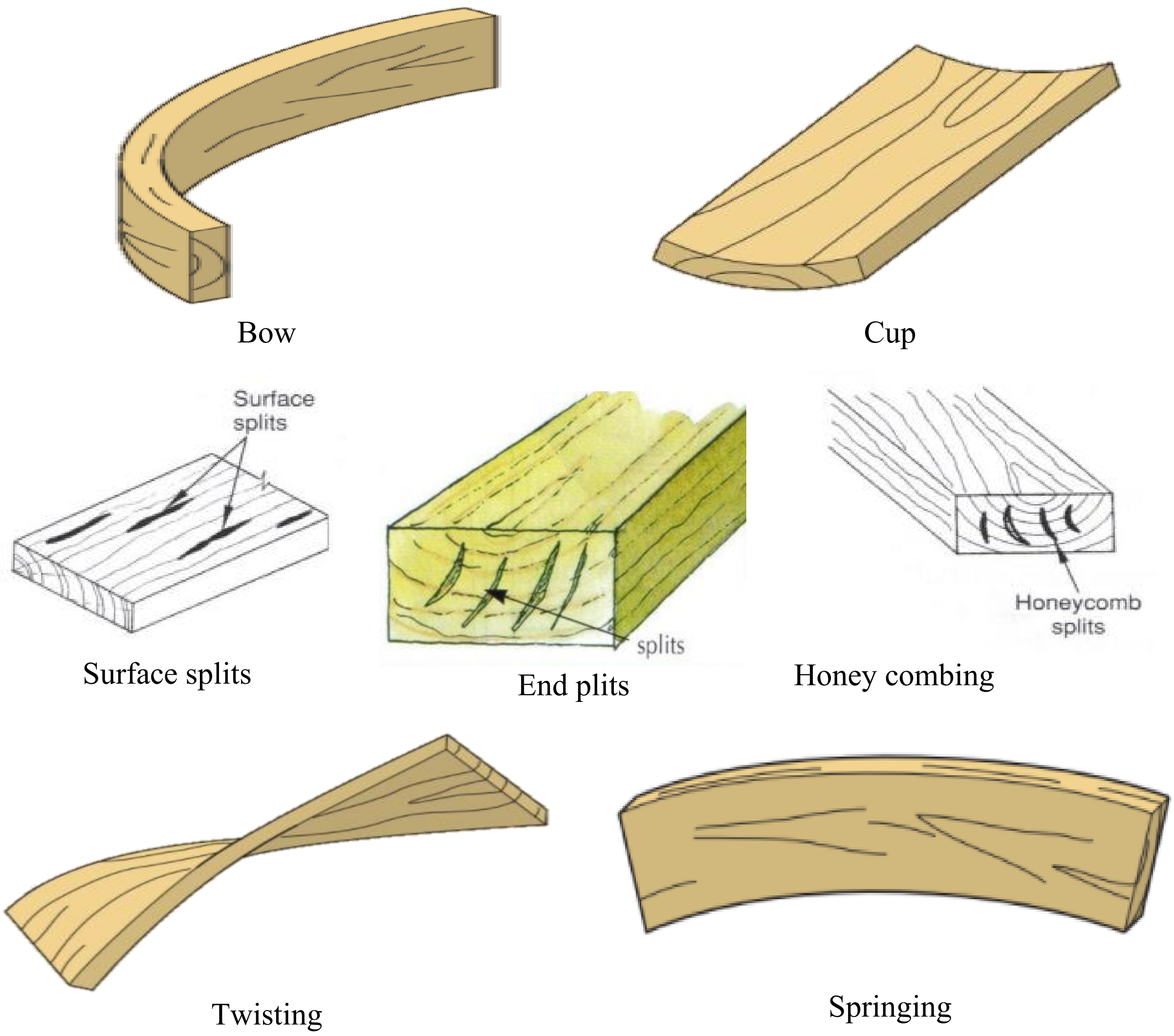


Figure 1.11. Seasoning defects in wood (mfhwoodworking.wordpress.com,2022)

1.1.4.4. Defects caused by Conversion

Conversion is the process of turning wood logs into timber or lumber that is ready to use. During this process, lots of defects can occur to the timber, and some of them will be discussed below.

- **Boxed Heart:** This term is applied to the timber, which is sawn in a way that the pith or the center heart falls entirely within the surface throughout its length.
- **Machine Notches**: Defective holding and pulling causes this defect.

- **Imperfect Grain**: It is a mismatch in grain alignment that occurs during the conversion of timber. It happens if there is improper use of different cutting saws and diagonal grains will appear (diagonal grain defect), or if any of the tools or other heavy things fall accidentally on the finished surface of timber it will cause small depression, which is called torn grain.

Figure 1.12. Diagonal grain (theconstructor.org,2022)

Figure 1.13. Torn grain (theconstructor.org,2022)

1.1.5. Prevention and treatment of defects

As has been seen above, there are different factors that damage wood. But these defects can be treated so as to avoid or cure these defects. The performance of the processes depends on the impregnability of the wood, the technique, the material and possibly the product used. There are different types of treatment: chemical, thermal or a combination of both as described in EN 351-1 and EN 460, which can be grouped into preventive and curative treatments.

1.1.5.1. Preventive Treatments

These are treatments that are carried out before the wood is used. We are going to discuss in the following lines, mainly two types of preventive treatments: drying or seasoning and chemical treatment.

a. Drying or Seasoning

It is the process of removing moisture from green wood to improve its serviceability. Lumber and other wood products usually contain considerable moisture after their production, and drying is essential to prepare them for further use. Proper drying reduces the magnitude of dimensional changes due to shrinkage, protects wood from microorganisms, reduces weight

and transportation costs, better prepares wood for most finishing and other preservation methods, and increases its strength. Drying is done in yards in the open air or in closed kilns.
Air drying can be accelerated by means of fans, sometimes in combination with low-temperature heating. When this technique is used, the piled lumber is placed in sheds. In the case of beech, walnut, and some other woods, steaming is employed before air drying. This practice reduces drying time by increasing the rate of drying and at the same time darkens the wood, making it more desirable for use in furniture.

Kiln drying is conducted in a closed chamber, under artificially induced and controlled conditions of temperature, relative humidity, and air circulation. This method permits much faster reduction of moisture content to levels that are independent of weather conditions.

In addition, wood can be dried by special methods that include solar drying (use of greenhouse-type dryers or those equipped with solar collectors), high-temperature drying, dehumidification kiln drying (in which the evaporated wood moisture is condensed and the latent heat recaptured and used for additional evaporation), and boiling in oils (a combination of drying and preservation, usually with creosote). Some other methods, such as drying with applied chemicals (salt seasoning), organic vapours (e.g., xylene), a solvent (specifically acetone), high-frequency electricity, centrifuging, infrared radiation, a vacuum, and microwaves, are inhibitively expensive and therefore are not commercially applicable (*Encyclopedia Britannica*).

b. Chemical Treatments

Wood can be protected from the action of destructive agents such as fungi, insects, and marine organisms, by impregnation with toxic chemicals. Preservatives used against such organisms are of three groups: oils, oil-soluble chemicals, and water-soluble chemicals. The solvents enable the preservative to be transported into the wood before evaporating. A combination of these active ingredients may be necessary to cover a wider field of action (*Encyclopedia Britannica*).
The effectiveness of preservative treatment depends on the chemical formulation selected, the method of application, the proportion of sapwood to heartwood, the moisture content of the wood, the amount of preservative retained, the depth of chemical penetration, and the distribution of the chemical in the wood (*G. Thomasson & al, 2015*),
The product is applied using different techniques: Brushing (brush); Spraying (nozzle); Soaking (tank); Vacuum/pressure (autoclave) (*GUITIYA, 2020*),

Commonly used preservatives are described below:

- **Creosote**. An oily liquid produced when coal is heated in the absence of air; it is the by-product of making coke from bituminous coal for the steel industry. It is very effective for treatment of railroad ties, large timbers, fence posts, poles, and pilings, and can extend their useful life several fold. Creosote-treated wood, however, resists painting and gluing and can exude the preservative, which is a pollutant. It is a "restricted-use" chemical i.e only people trained or licensed, may purchase or use this preservative fold (*G. Thomasson & al, 2015*).
- The main representative of oil-soluble preservatives is **Pentachlorophenol (Penta)**. It is produced by chlorinating phenol under tightly controlled conditions that limit formation of other products. It is insoluble in water, so it generally is dissolved in petroleum or other organic solvents that will penetrate wood. Penta is used to treat poles, cross-arms, lumber, timber, and fence posts. Its main advantage over creosote is that the preserved wood is kept clean, and can be painted or glued. It is not recommended for use in marine installations or close to plants, and it may not be used inside buildings except when the treated wood is sealed to limit volatilization. The compound is polluting, and its use is banned in several countries (*G. Thomasson & al, 2015*).
- **Inorganic Arsenicals.** These are water-soluble preservatives composed of various inorganic chemicals such as copper, chromium, arsenic, and mercury. The most commonly used compounds are Chromated Copper Arsenate (CCA), ammoniacal copper zinc arsenate, and alkaline, copper-based systems. These preservatives are water-soluble but, when applied to wood, they become fixed in the wood in an insoluble form, and they leach from the wood under damp conditions. The copper provides protection against attack by fungi, and the arsenic prevents insect attack. These preservatives no longer are used for residential applications, but can be used for farm, highway, and marine applications. Alkaline copper compounds use either amines or ammonia-based copper plus a secondary fungicide. Like CCA, these systems are water based, but they are less strongly fixed to the wood following treatment. Their primary benefits are the absence of chromium or arsenic. Amine copper azole and alkaline copper azole are the two most commonly used compounds. Both tend to be corrosive to steel (*G. Thomasson & al, 2015*).
- Wood can be made resistant to fire with **chemical retardants**. Fire retardants are water-soluble and not toxic. They contain silica and other chemical compounds and act by

creating a barrier (charred wood or foam) to the spread of flame or by generating non-combustible gases (*Encyclopedia Britannica*).

c. Fumigation

Fumigation is a treatment of wood with toxic gases such as methyl bromide, hydrogen cyanide, hydrogen phosphide, ethylene oxide, carbon dioxide, sulphuryl fluoride, against insects present in debarked wood. In some European countries, this operation must be carried out by a company approved by authority (usually a ministry) (*NIMP, 2008*).

d. Thermal Treatment

The principle of this treatment consists of placing the wood in a chamber under a controlled atmosphere (after prior drying), with inert gases (mainly nitrogen) and no oxygen to prevent the material from burning. The temperature is then gradually increased to a temperature between 180°C and 250°C (*GUITIYA, 2020*).

1.1.5.2. Curative Treatments

The International Standards for Phytosanitary Measures (ISPM/NIMP) is established by the International Plant Protection Convention (IPPC), which is part of the Food and Agriculture Organization (FAO) of the United Nations.
The Principal Curative treatment consists of heating the wood to a minimum core temperature of 56°C for at least 30 minutes. These conditions are lethal to insects in all their forms (eggs, larvae, pupae, imagos). This is a curative treatment with no guarantee of success over time (not preventive) (*GUITIYA, 2020*).

Having presented some of the characteristics, properties, defects and treatments of wood, we shall in the lines that follow, discuss about wood in construction.

1.2. Wood structure

In addition to wood classes, this section will discuss about timber products used for construction and present some examples of timber structures from around the world.

1.2.1. Different classes of wood

1.2.1.1. Use classes

EN 335 distinguishes between 5 classes of use.

The biological classes of use define the environment in which the wood is used.

They are differentiated according to the standard:

- **Use class 1:** Situation in which the wood or wood-based product is sheltered, fully protected from the weather and not exposed to moisture;
- **Use class 2:** Situation in which the wood or wood-based product is sheltered and fully protected from the weather, but where high ambient humidity may lead to occasional but not persistent wetting;
- **Use class 3:** Situation where the wood or wood-based product is neither sheltered nor in contact with the ground. It is either continuously exposed to the weather, or protected from the weather but subject to frequent wetting;
- **Use class 4**: Situation in which the wood or wood-based product is in contact with the ground or fresh water and is thus permanently exposed to moisture;
- **Use class 5**: Situation in which the wood or wood-based product is permanently exposed to salt water.

1.2.1.2. Load-duration classes

The loads on timber buildings will be described according to BS-EN1995-1-1, 2004, in its clause 2.3.1 given in the table 1.1, with some examples in table 1.2.

Table 1.1. Load duration classes (Eurocode 5 part-1-1, 2004)

N°	Load-duration class	Order of accumulated duration of characteristic load
1	Permanent	More than 10 years
2	Long-term	6 months – 10 years

3	Medium-term	1 week – 6 months
4	Short-term	Less than one week
5	Instantaneous	

Table 1.2. Example of load-duration assignment (Eurocode 5 part-1-1, 2004)

Load-duration class	Examples of loading
Permanent	Self-weight
Long-term	Storage
Medium-term	Imposed floor load, snow
Short-term	Snow, wind
Instantaneous	Wind, accidental load

1.2.1.3. Service classes

Structures shall be assigned to one of the service classes given below:

- **Service class 1** is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year;
  It's important to note that in service class 1, the average moisture content in most softwoods will not exceed 12 %.
- **Service class 2** is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.
  In service class 2, the average moisture content in most softwoods will not exceed 20 %;
- **Service class 3** is characterised by climatic conditions leading to higher moisture contents than in service class 2;

The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions

1.2.2. Timber products used for construction

Timber products offer high performance and dimensionally stable options for any building project, whether large or small, residential or commercial, offering incredible design versatility for architects. Sawn timber and engineering wood products (EWP) are the common timber products. Different species of timber are sawn in specific dimensions for the former while the latter are made from solid and composite wood-based products such as dimension lumbers, boards, and panels (Milner, 2007).

1.2.2.1. Cross laminated timber (CLT)

This is an engineering wood panel made up of layers of dimension lumber oriented at right angles to one another and glued in three (3), five (5) or seven (7) layers to form structural panels with exceptional strength, rigid enough and dimensionally stable as shown in figure 1.14 (https://continuingeducation.bnpmedia.com/)

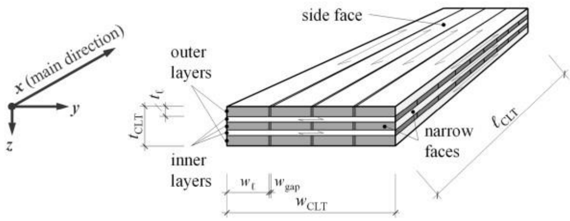


Figure 1.14. Cross laminated timber (Sandhaas & Dietsch, 2018)

Having a thickness in the range 50 – 500 mm and thanks to their structural properties and dimensional stability, this product is suitable for floor, wall and roof elements of up to 13.5 m in length, providing additional aesthetic attributes (Structutal Timber Association, 2005).

1.2.2.2. Oriented strand board (OSB)

This is an engineered wood-base panel material in which long strands of wood are bonded together with a synthetic resin adhesive, as illustrated in figure 1.15. OSB is usually composed of three (3) layers. The strands of the outer two (2) layers oriented in a particular direction, more often than not in the long direction of the panel. The timber used in OSB manufacture include both softwood and hardwood (europanels.org,2022).

Figure 1.15. Oriented strand board (naturallywood.com, 2022).

1.2.2.3. Nailed laminated timber (NLT)

This is a panel created by fastening individual lumber, stacked on edge, into one structural element with nails, as indicated in figure 1.16. In addition to being used in floors, decks and roofs, NLT panels are used for timber elevator and stair shafts. These panels offer a consistent and attractive appearance for decorative and exposed applications. Sheathing can be added to one top side to provide a structural diaphragm and allows the product to be used as a wall panel (naturallywood.com, 2022).

Figure 1.16. Nailed laminated timber (naturallywood.com, 2022).

1.2.2.4. Glued Laminated Timber (GluLam)

Defined as a structural timber member composed of at least two (2) essentially parallel laminations. These laminations may comprise of one (1) or two (2) boards side by side having finished thicknesses from 6 mm up to 45 mm (BS EN 14080, 2013), as shown in figure 1.17. These are typically used to fabricate curved and long beams limited only by methods of transport. GluLam is allocated to specific strength classes defined in BS EN 14080 (2013) and it can be used in horizontal applications as a beam, or vertically as a column (naturallywood.com, 2022).

Figure 1.17. Glued laminated timber (naturallywood.com, 2022).

1.2.2.5. Laminated Veneer Lumber (LVL)

This product consists of reconstituted dimensional timber that is commonly twice the strength of dimensional timber of the same species manufactured from rotary peeled veneers of spruce, pine or douglas fir as shown in figure 1.18. Commonly the veneer grain is oriented in a single direction but cross-grained sections are also manufactured to offer tailored mechanical properties. Lengths of short veneer are jointed end-to-end with a scarf joint allowing limitless dimensional lengths (Ramage et al., 2017). LVL columns, beams and lintels are often chosen to replace dimension lumber or GluLam as columns, beams and headers (naturallywood.com, 2022).

Figure 1.18. Laminated veneer lumber (naturallywood.com,2022).

1.2.2.6. Laminated Strand Lumber (LSL)

Laminated strand lumber is a wood-based composite. It is manufactured from water resistant adhesive and poplar wood strands measuring 0.8 mm in thickness, 25 mm width and 300 mm in length. As the wood strands are encapsulated in adhesive and due to the homogenous structure of the composite, LSL is weather-resistant under most conditions (though exposure to direct weathering should be avoided). Two (2) types of LSL can be distinguished: boards where the strands are all aligned in the direction of the major axis of the product and boards where a portion of the strands are aligned in the direction of the minor axis of the product as shown in figure 1.19. The former is suitable for use as beams, rafters, sills and columns, and the latter for use as walls, floors and ceilings (europeanwood.org,2022).

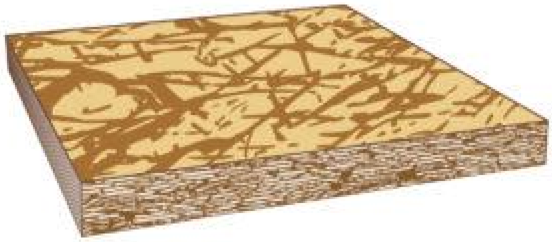


Figure 1.19. Laminated strand lumber (naturallywood.com, 2022).

1.2.2.7. Parallel strand lumber (PSL)

Parallel strand lumber is a product manufactured from strips of veneer measuring approximately 3 mm in thickness and 15 mm in width as illustrated in figure 1.20 Phenolic resin is used to bond the individual veneer strips. These can be up to 2.6 m long, before the strips are bundled together with their individual ends offset and with fibres oriented primarily parallel to the major axis of the beam. In a continuous press the veneer strips are pressed to form an endless beam. Douglas fir and southern yellow pine are the most commonly used wood species (europeanwood.org,2022). It is well suited for use as beams and columns in post and beam construction, and for beams, headers and lintels in light framing.

Figure 1.20. Parallel strand lumber (https://www.dataholz.eu,2022).

1.2.3. Wooden Buildings

Wood is one of the oldest building materials used in construction. However, the rise of concrete and steel, due to man's ability to master their potential, has largely contributed to the demotion of the use of wood, especially in the construction of tall buildings; the myth of tall wooden buildings remains in the mind of man, especially in Africa; yet there are many tall wooden buildings in the world, including :

- **Voll Arkitekter's Mjøstårne in Brumunddal, Norway**

The 85-metre-high tower shown in Figure 1.21 has claimed the title of the world's tallest timber building after its completion in March 2019 (constructionreviewonline.com,2022). Mjøstårnet was built entirely of cross-laminated timber, from the large-scale interior trusses to its lifts. The 18-storey building is made of CLT and glulam, which are strong enough to withstand heavy loads.

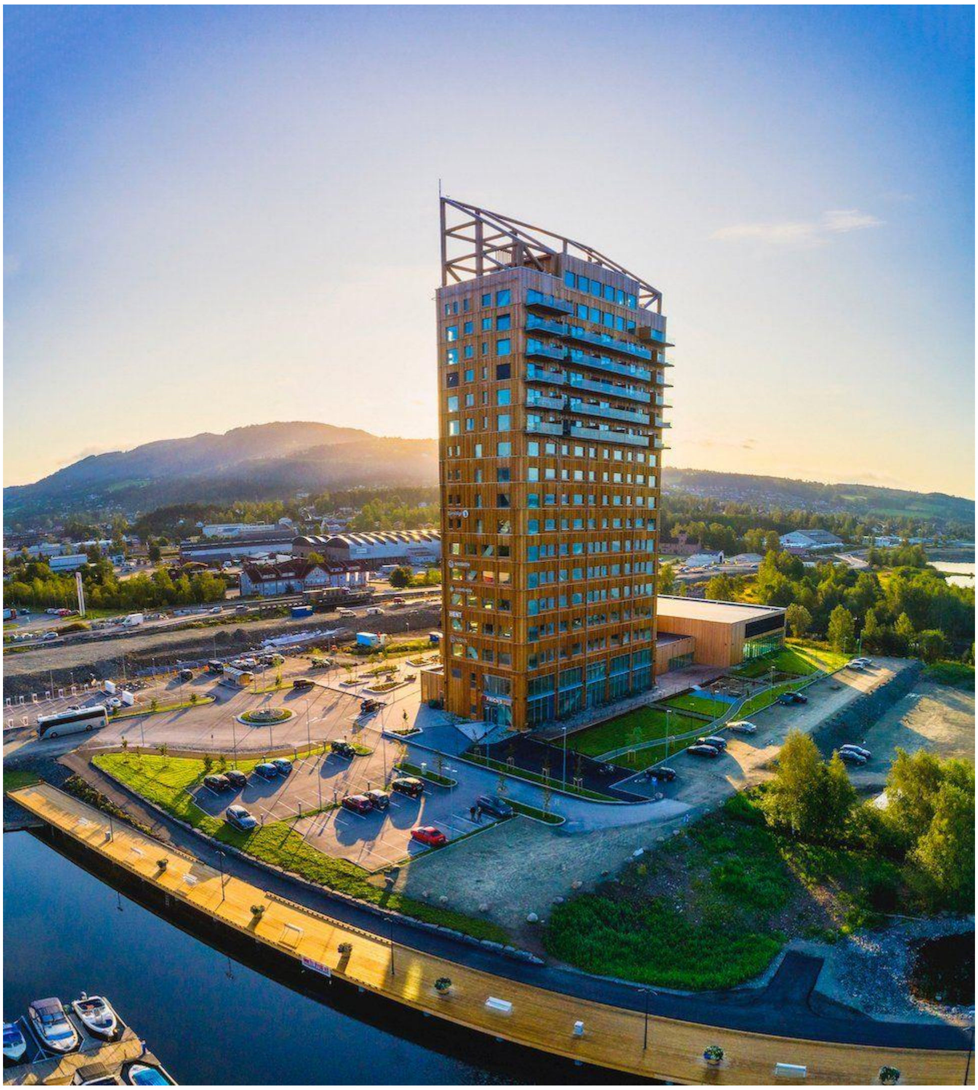


Figure 1.21. Voll arkitekter's mjøstårne (constructionreviewonline.com,2022)

- **Tall Wood Residence in Vancouver, Canada**

As shown in Figure 1.22, the Tall Wood Residence is a 53-metre high, 18-storey building housing a total of 404 students as the Brock Common Student Residence at UBC (the University of British Columbia). However, prefabricated steel elements and a concrete core were used to assist the solid wood structure (constructionreviewonline.com, 2022). It is currently the 2nd tallest wood building in the world.

Figure 1.22. Tall wood residence, vancouver, Canada (naturallywood.com,2022)

- **The Tree in Bergen, Norvège**

Located on Dansgardsveien, 99, Asrtad, 5058 Bergen, Hordaland, Norway, The Tree or Treet (showed in Figure 1.23) stands 49 meters tall and one of the tallest timber buildings in the world. The building was constructed in modules, the modules stacked together on site. Its load-bearing framework structure comprises glulam truss work, then a prefabricated module with a platform at the top made of a fortified concrete deck. The purpose of the concrete slabs was to give the structure the required weight. The building of 14-story building has a total of 62 apartments (constructionreviewonline.com,2022).

Figure 1.23. The Tree, Bergen, Norvège (teknos.com,2022)

1.3. Structural fire analysis of wooden structure

1.3.1. Fire scenario considered

In accordance with Eurocode 5: part 1-2, for applying the prescriptive approach, the nominal fire will be considered to generate thermal action. EN 1991-1-2 (2002) proposes three different nominal temperature-time curves which are:

1.3.1.1. Standard temperature–time curve

The standard temperature-time curve is the one that has been historically used, and it is still used today, in standard fire tests to rate structural elements. It is used to represent a fully developed fire in a compartment. It is often referred to as the ISO curve because the expression was taken from the ISO 834 standard. This standard curve is given by equation (1.1).

$$\boldsymbol{\theta_g(t)} = 20 + 345 * \log10\,(8t + 1) \qquad \textbf{(1.1)}$$

Where:

θg is the gas temperature [°C]; and

t is the time [min].

1.3.1.2. The external time-temperature curve

The external time-temperature curve is used for the outside surface of separating external walls of a building which are exposed to a fire that develops outside the building or to the flames coming through the windows of a compartment situated below or adjacent to the external wall. The external curve is given by equation (1.2).

$$\boldsymbol{\theta_g(t)} = 20 + 660 * (1 - 0.687e{-}0.32t - 0.313e{-}3.8t) \qquad \textbf{(1.2)}$$

1.3.1.3. The hydrocarbon time-temperature curve

The hydrocarbon time-temperature curve is used for representing the effects of a hydrocarbon type fire. In situations where petrochemicals or plastics form a significant part of the overall fire load, the temperature rise is very rapid due to the much higher calorific values of these materials. Therefore, for such situations, an alternative temperature–time curve has been developed of the form of equation (1.3).

$$\boldsymbol{\theta_g(t)} = 20 + 1080 * (1 - 0.325e{-}0.167t - 0.675e{-}2.5t) \qquad \textbf{(1.3)}$$

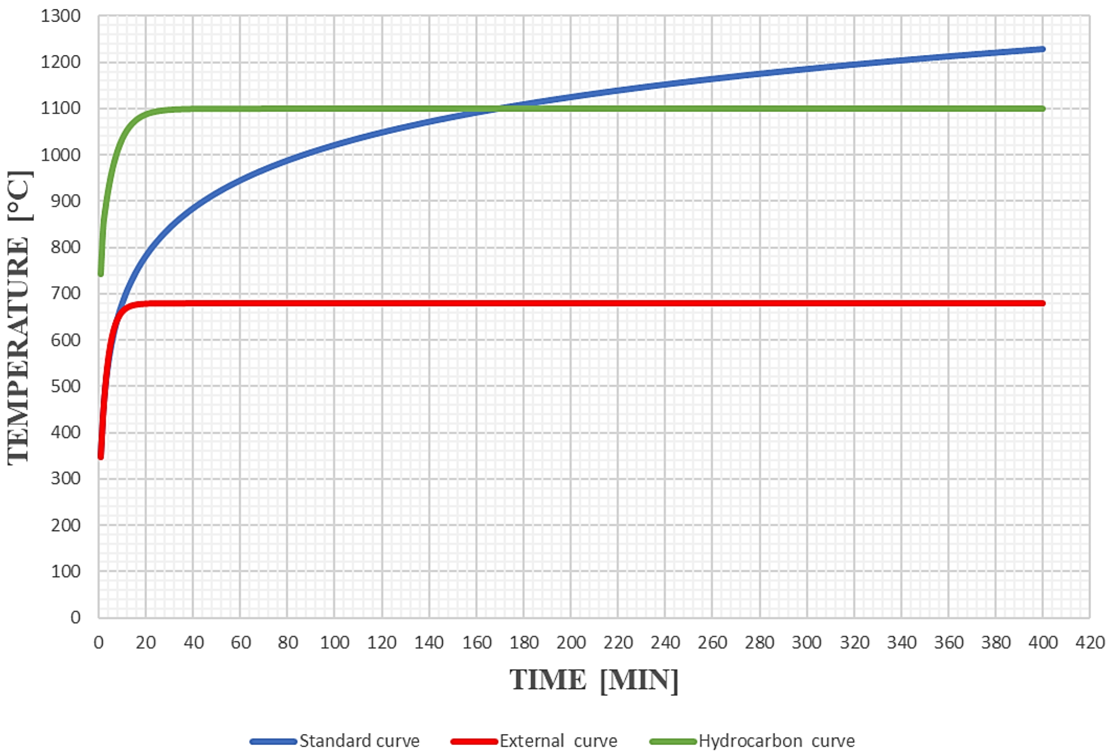


Figure 1.24. Time-Temperature curve

1.3.2. Action on structure in fire situation

The effect of actions $E_{d,fi}$ may be obtained from the analysis for normal temperature as given in equation (1.4)

$$E_{d,fi} = \eta_{fi} \, Ed \qquad \textbf{(1.4)}$$

Where:

Ed is the design effect of actions for normal temperature design for the fundamental combination of actions, see EN 1990, 2002;

η_{fi} is the reduction factor for the design load in the fire situation.

The reduction factor η_{fi} for load combination (6.10) in EN 1990:2002 should be taken as given by equation (1.5)

$$\eta_{fi} = \frac{\sum \gamma_{GA,j} G_{k,j} + A_d + \Psi_{1,1} Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}}{\sum \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i}} \quad \textbf{(1.5)}$$

where:

$Q_{k,1}$ is the characteristic value of the leading variable action;

G_k is the characteristic value of the permanent action;

γ_G is the partial factor for permanent actions;

$\gamma_{Q,i}$ is the partial factor for variable action I;

$\psi_{1,1}$ Combination factor for frequent value of a variable action;

$\psi_{2,i}$ Combination factor for quasi-permanent value of a variable action.

According to Eurocode 5-part 1-2 section 2.5.3 (3), the effect of the actions (e.g., internal forces and moments) relative to the initial support and boundary conditions can be deduced from the overall analysis of the structure in normal temperature design according to equation (1.6):

$$\mathbf{E_{fi,d} = [0,6]\ E_d} \quad \textbf{(1.6)}$$

1.3.3. Verification condition of wooden structure in fire situation

In situation of fire, the verification of fire resistance of different element should be made in function of time, strength, or temperature like define respectively in equation (1.7), (1.8) and (1.9).

- in the time domain $\mathbf{t_{fi,d} \geq t_{fi,requ}}$ **(1.7)**
- or in the strength domain $\mathbf{R_{fi,d,t} \geq E_{fi,d,t}}$ **(1.8)**
- or in the temperature domain $\mathbf{\Theta_d \leq \Theta_{cr,d}}$ **(1.9)**

Where:

$t_{fi,d}$ is the design value of the fire resistance

$t_{fi,requ}$ is the required fire resistance time

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

$E_{fi,d,t}$ is the design value of the relevant effects of actions in the fire situation at time t

Θ_d is the design value of material temperature

$\Theta_{cr,d}$ is the design value of the critical material temperature

1.3.4. Design values of material properties and resistances

For verification of mechanical resistance, the design values of strength and stiffness properties shall be determined by equation (1.10) and (1.11).

$$f_{d,fi} = k_{mod,fi}\frac{f_{20}}{\gamma_{M,fi}} \quad \textbf{(1.10)}$$

$$S_{d,fi} = k_{mod,fi}\frac{S_{20}}{\gamma_{M,fi}} \quad \textbf{(1.11)}$$

$$f_{20} = k_{fi}f_k \quad \textbf{(1.12)}$$

$$f_{20} = k_{fi}f_k \quad \textbf{(1.13)}$$

Where:

$f_{d,fi}$ is the design strength in fire;

$S_{d,fi}$ is the design stiffness property (modulus of elasticity $E_{d,fi}$ or shear modulus $G_{d,fi}$) in fire;

f_{20} is the 20 % fractile of a strength property at normal temperature;

S_{20} is the 20 % fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature;

$k_{mod,fi}$ is the modification factor for fire (Table 1.4);

$\gamma_{M,fi}$ is the partial safety factor for timber in fire;

k_{fi} is given in table 1.3.

Table 1.3. Values of k_{fi} (EN 1995-1-2:2004)

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

Note:

The design of load-bearing timber elements considers the load duration classes and the design of load-bearing timber elements takes into account the load duration class and the class of the environment in which the structure will be located, modifying the design values of the element by strength reduction coefficients $kmod$ (modification factor for duration of load and moisture content), and the coefficient defining the creep effects in timber k_{def}. These coefficients are presented in Tables 1.4 and 1.5

.

Table 1.4. Value of K_{mod} (EN 1995-1-2:2004)

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instanta-neous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Glued laminated timber	EN 14080	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
LVL	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90
Plywood	EN 636						
	Part 1, Part 2, Part 3	1	0,60	0,70	0,80	0,90	1,10
	Part 2, Part 3	2	0,60	0,70	0,80	0,90	1,10
	Part 3	3	0,50	0,55	0,65	0,70	0,90
OSB	EN 300						
	OSB/2	1	0,30	0,45	0,65	0,85	1,10
	OSB/3, OSB/4	1	0,40	0,50	0,70	0,90	1,10
	OSB/3, OSB/4	2	0,30	0,40	0,55	0,70	0,90
Particle-board	EN 312						
	Part 4, Part 5	1	0,30	0,45	0,65	0,85	1,10
	Part 5	2	0,20	0,30	0,45	0,60	0,80
	Part 6, Part 7	1	0,40	0,50	0,70	0,90	1,10
	Part 7	2	0,30	0,40	0,55	0,70	0,90
Fibreboard, hard	EN 622-2						
	HB.LA, HB.HLA 1 or 2	1	0,30	0,45	0,65	0,85	1,10
	HB.HLA1 or 2	2	0,20	0,30	0,45	0,60	0,80
Fibreboard, medium	EN 622-3						
	MBH.LA1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	1	0,20	0,40	0,60	0,80	1,10
	MBH.HLS1 or 2	2	–	–	–	0,45	0,80
Fibreboard, MDF	EN 622-5						
	MDF.LA, MDF.HLS	1	0,20	0,40	0,60	0,80	1,10
	MDF.HLS	2	–	–	–	0,45	0,80

Table 1.5. Values of K_{def} for timber and wood-based materials (EN 1995-1-2:2004)

Material	Standard	Service class 1	Service class 2	Service class 3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636			
	Part 1	0,80	–	–
	Part 2	0,80	1,00	–
	Part 3	0,80	1,00	2,50
OSB	EN 300			
	OSB/2	2,25	–	–
	OSB/3, OSB/4	1,50	2,25	–
Particleboard	EN 312			
	Part 4	2,25	–	–
	Part 5	2,25	3,00	–
	Part 6	1,50	–	–
	Part 7	1,50	2,25	–
Fibreboard, hard	EN 622-2			
	HB.LA	2,25	–	–
	HB.HLA1, HB.HLA2	2,25	3,00	–
Fibreboard, medium	EN 622-3			
	MBH.LA1, MBH.LA2	3,00	–	–
	MBH.HLS1, MBH.HLS2	3,00	4,00	–
Fibreboard, MDF	EN 622-5			
	MDF.LA	2,25	–	–
	MDF.HLS	2,25	3,00	–

1.3.5. Design procedures for mechanical resistance

The Eurocode prescribes 2 analytical methods for the determination of the loss of resistance of a timber structure in a fire situation at specific times. These methods are:

- Reduced cross-section method (used for both softwood and hardwood in respect to Eurocode 5);
- Reduced properties method (used only for softwood).

These 2 methods are based on the charring depth approach d_{char} present in equation (1.14) :

$$d_{char} = \beta_0 . t \qquad \textbf{(1.14)}$$

where d_{char} Charring depth

β_0 Charring rates (Table 1.6)

t Time

Table 1.6. Design charring rates β_0 and β_n of timber, LVL, wood panelling and wood-based panels (EN 1995-1-2:2004)

	β_0 (mm/min)	β_n (mm/min)
a) Softwood and beech Glued laminated timber with a characteristic density of ≥ 290 kg/m3 Solid timber with a characteristic density of ≥ 290 kg/m3	0,65 0,65	0,7 0,8
b) Hardwood Solid or glued laminated hardwood with a characteristic density of 290 kg/m^3 Solid or glued laminated hardwood with a characteristic density of ≥ 450 kg/m^3	0,65 0,50	0,7 0,55
c) LVL with a characteristic density of ≥ 480 kg/m^3	0,65	0,7
d) Panels Wood panelling Plywood Wood-based panels other than plywood	0,9[a] 1,0[a] 0,9[a]	– – –
[a] The values apply to a characteristic density of 450 kg/m^3 and a panel thickness of 20 mm; see 3.4.2(9) for other thicknesses and densities		

1.3.6. Time resistance of unprotected wood connections in fire situation

The fire resistance of unprotected wood-to-wood connections where spacings, edge and end distances and side member dimensions comply with the minimum requirements given in EN 1995-1-1 section 8, may be taken from table 1.7.

Table 1.7. Fire resistances of unprotected connections with side members of wood (EN 1995-1-2:2004)

	Time of fire resistance $t_{d,fi}$ min	**Provisions[a]**
Nails	15	$d \geq 2{,}8$ mm
Screws	15	$d \geq 3{,}5$ mm
Bolts	15	$t_1 \geq 45$ mm
Dowels	20	$t_1 \geq 45$ mm
Connectors according to EN 912	15	$t_1 \geq 45$ mm
[a] d is the diameter of the fastener and t_1 is the thickness of the		

For connections with dowels, nails or screws with non-projecting heads, fire resistance periods $t_{d,fi}$ greater than those given in table 1.7, but not exceeding 30 minutes, may be achieved by increasing the following dimensions by a_{fi} :

− the thickness of side members,

− the width of the side members,

− the end and edge distance to fasteners.

with:

$$a_{fi} = \beta_n . k_{flux}(t_{req} - t_{d,fi}) \qquad \textbf{(1.15)}$$

where:

β_n	is the charring rate according to table 1.6;
k_{flux}	is a coefficient taking into account increased heat flux through the fastener;
t_{req}	is the required standard fire resistance period;
$t_{d,fi}$	is the fire resistance period of the unprotected connection given in table 1.7.

Note: The factor k_{flux} should be taken as k_{flux} = 1,5

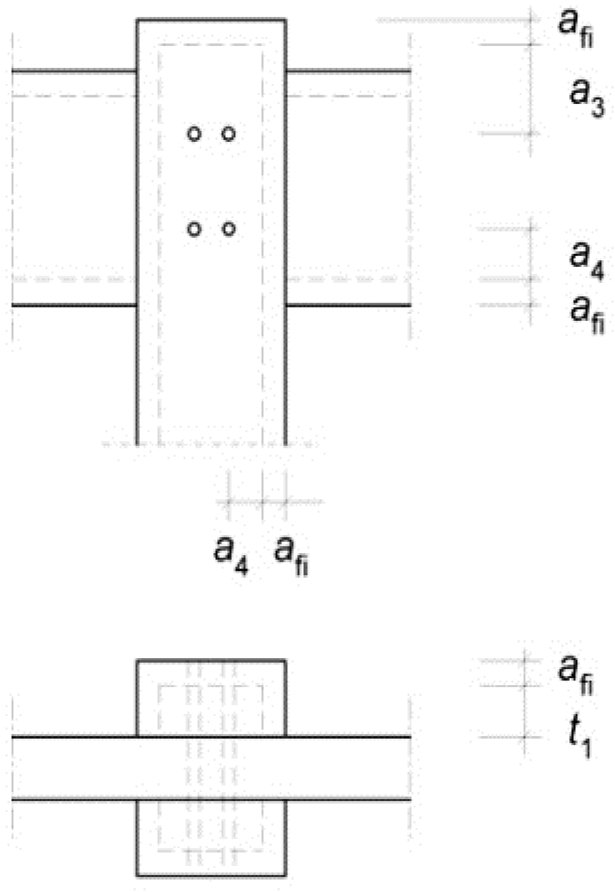


Figure 1.25. Extra thickness and extra end and edge distances of connections (EN 1995-1-2, 2004)

1.3.7. Flowchart of design procedure of wooden structure under fire design

The design procedure for timber structures under fire load as prescribed by Eurocode 5 Part 1.2 is shown in Figure 1.26

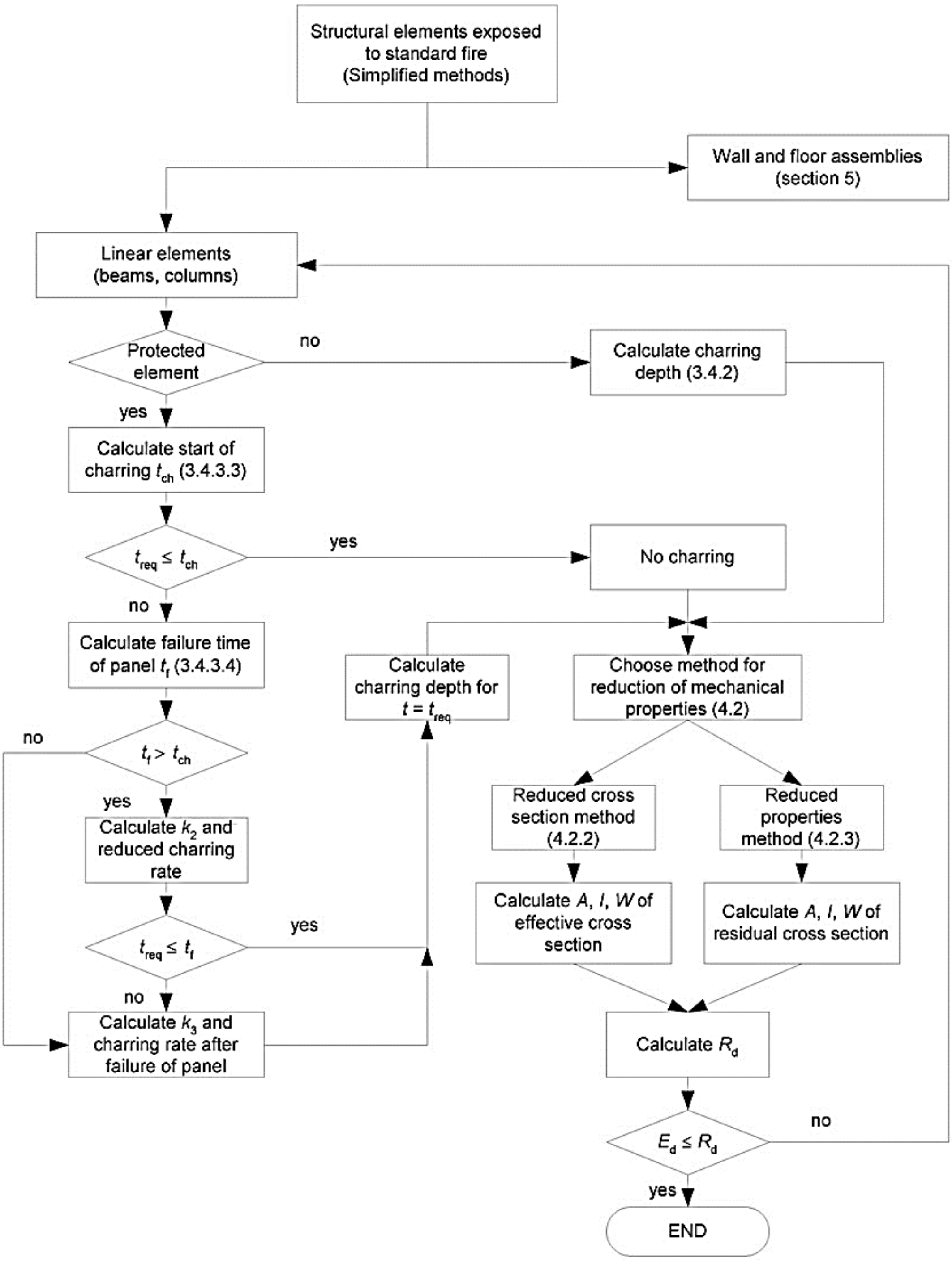


Figure 1.26. Flow chart for the design procedure of structural members (EN 1995-1-2, 2004)

1.4. Damaged structures due to fire

In French-speaking Africa, Cameroon situated on the Gulf of Guinea has a great amount of companies that shine in the national triangle producing various products that are useful to the population. However, for some time now, events have been taking place which are likely to weaken the economic fabric of Cameroon. After the fires in markets and houses, today it is some huge Cameroonian industries that are suffering the damage of flames.

- In 2019, a massive explosion rocked one of the giant storage tanks at the National Refining Company (SONARA) in Limbola, Limbe, Southwest region of Cameroon, on Friday 31 May, 2019, (Figure 1.26). “Flames consumed four of the 13 production units and partially blew up three others, stopping all the SONARA refining process” the Minister of Water Resources and Energy, Gaston Eloundou Essomba said (Cameroon Tribune, 2019). On the 3rd of June 2019, one of the tanks of the refinery, which is used to store crude oil, was licked by fire. This company, which is almost entirely state owned apart from a 4% stake held by Total, has a capacity of 2.1 million tonnes of crude a year. It serves the whole country, so the delayed caused by the fire caused important damages to the country. It had a significant impact on the revenue collected by the General Tax Directorate (DGI) of the

Figure 1.27. Damaged SONARA structure (crtv.cm, 2022)

- On the 1st of January 2018, a huge fire broke out at the Cameroonian Fermentation company (FERMENCAM), leader of Cameroon's sachet whiskey market, burning its main warehouses located in the quarter named Bonaberi of the city of Douala which is the economic capital of Cameroon. The numerous flammable products in the warehouses of this firm have caused the spread of the fire to eight neighbouring houses and a school. The disaster having occurred on the new-year's day in the absence of employees which were at home, only important property damages were sustained as shown in figure 1.27.

Figure 1.28. Damaged structure of Fermencam warehouses (www.cameroun24.net)

- On Thursday, 3rd of November 2018, a fire broke out around 11:40 p.m., in the premises of Annex 2 of the cosmetics company Biopharma in Douala the economical capital of Cameroon (Cameroon Tribune, 2018). The employees who were on duty at that time immediately alerted the neighbouring companies to try to fight the flames. As the flames became increasingly violent, the elements of the Ter brigade from Logbaba were the first to arrive at the site with their riot control trucks to rescue the company. The elements of the city's fire fighters also arrived at the site and literally put themselves in the thick of the action. For several hours, the battle against the flames was fierce until the early hours of the morning and it was at 7:00 a.m. that the fire was completely brought under control. This caused serious damaged to the structure of the building (Figure 1.28).

Figure 1.29. Damaged structure of Biopharma's warehouse (cameroon-tribune.cm, 2022)

- A fire ravaged part of the Cameroonian National Assembly in Yaounde on Thursday night, 17 November 2017. The flames ravaged the offices on 4 levels. After more than 2 hours of exercise, the fire brigade managed to control the flames avoiding that it spreads in the hemicycle where the deputies sit during the sessions. The fire caused considerable material damage, but there were no casualties. The fire broke out in the evening inside the administrative part of the Assembly, on the back side of the building (figure 1.29).

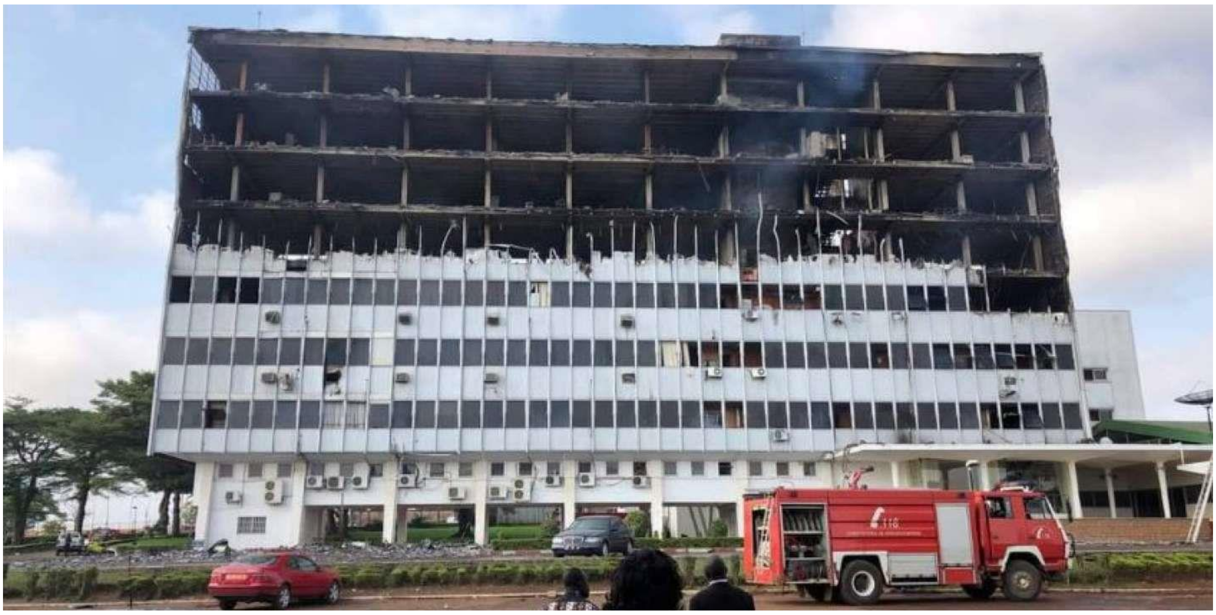


Figure 1.30. Damaged structure of Cameroonian National Assembly (lemonde.fr,2022)

1.5. Fire protective measures for wooden buildings

1.5.1. Active fire protection

The main active fire protection measure is an automatic sprinkler system. The biggest question for designers and code writers is the effectiveness of the sprinkler system in tall buildings. If sprinklers can be guaranteed to work, the additional requirements for fire safety and fire resistance are minimal, but if the sprinklers do not work for any reason, timber buildings are in a special category, needing more careful attention.

1.5.2. Passive fire protection

The fire resistance of structures can be increased by increasing the size of the structural elements (oversizing the structure), by wrapping the element with low thermal inertia insulation or by protecting the entire assembly or structure with an insulating membrane.

The burning of wood produces a charred layer on the surface of the item, which insulates the unburned wood from the heat of the flames. This considerably reduces the rate of charring, which remains relatively constant throughout the fire. In addition, wood has a very low thermal conductivity, which means that the interior of the wood is little affected while the outer surfaces are burning. In fact, there is a fairly good correlation between the load bearing capacity of a piece of wood and its reduced cross-section.

The fire resistance of a timber structure can be improved by the addition of thermal protection on the sides likely to be subject to the action of fire. This protection can be mechanically fixed by nailing or screwing (wood, wood-based panels, plasterboard) or by embedding (sprayed plaster on mesh).

Fireproofing processes can also be used to delay ignition and limit the development of the fire. Fireproofing modifies the reaction to fire of materials and delays their combustion by absorbing heat. The following two processes are commonly used:

- Incorporation of diluted salt fibres which penetrate the wood by soaking, spraying or impregnating;
- Application of paints, coatings and varnishes which either form an insulating and protective envelope by simple application, or form a barrier several centimetres thick between the flames and the wood by swelling (intumescence) under the action of heat (from 180 °C).

Regarding the latter, fireproof paints and varnishes are available in liquid form. They can be applied to raw wood (pallets, beams) or wood already treated against insects or mould.

Over time, the wood absorbs and eliminates the action of the fireproofing products. Fireproofing is never a completely permanent action. It must be renewed regularly.

However, this process is not always satisfactory, especially for parquet floors or ceilings, as fireproof products cannot withstand abrasion, impact or the presence of water. In addition, fireproofing improves the reaction to fire by delaying the ignition of the wood, but it does not directly contribute to its resistance.

Conclusion

A general presentation of wood, including its structure, qualities, types, flaws, and various treatments, as well as its usage in construction, wooden construction, wooden protective measure, fire damaged structure and general wood fire analysis approach were the goal of this chapter's. Wood was discovered to be a biodegradable anisotropic material with properties that vary from direction to direction, as well as physical (density, water saturation, deformability), mechanical (mechanical strength, hardness, elasticity), and chemical (corrosion, durability) characteristics that allow it to be used as a variety of construction materials. Additionally, there are many different uses for wood because it is widely employed in construction, the paper industry, and carpentry. Generally speaking, if the study of durability is taken into account, wood can last for centuries. The high density, chemical composition, and cutting period related to the species used are taken into account, as well as the dimensioning of the structure and its structural behaviour to fire, which must be defined by a well-structured methodology. In fact, durability is influenced by environmental parameters and is conferred when the atmosphere is kept dry, at less than 20 percent.

CHAPTER 2: METHODOLOGY

Introduction

The methodology is a part allowing to establish the procedure of the research in order to attain the fixed objectives. In other words, it will be question of describing the different constitutive elements of our research. Thus, for this work, after the design and verification of our structure at the ultimate limit state and the serviceability limit state, the study of the structural behaviour when the structure is exposed to fire will be done for the load-bearing elements..

2.1. General site recognition

The recognition of the site will be based on documentary researched on the study site. It allows the knowledge of the physical parameters like the geographical location, the climate, and the hydrology and on the other hand the socio-economic parameters.

2.2. Site visit

The site visit is the phase that consists in going down to the study site in order to discover it. The site visit will be done in two phases, the first phase for direct observation of the site and the second for surveys.

2.3. Data collection

The data collected are the different plans and the data concerning the different properties of the materials used on the study.

2.3.1. Architectural data

Architectural information refers to information about the entire structure. This comprises the building's footprint and floor heights, as well as 2D and 3D plans created with AUTOCAD, ArchiCAD, and REVIT drawing software.

2.3.2. Characteristics of materials

These are the data that characterise the materials that have been used for the implementation of the structure. The knowledge of the material properties will allow to obtain the forces and moments of resistance of the elements.

The material properties will be divided into two parts, one for the wooden elements, and the other for the concrete used as a foundation.

2.4. Codes and loads conditions determination method

The codes used and the different types of loads acting on timber buildings will be presented in this section. This study will focus on three main types of actions acting on timber buildings. These actions are permanent, variable and accidental loads.

2.4.1. Codes

The building will be described according to the BS-EN1991-1-1 norm, clause 6.3.1 as indicated in annex 3. Specifications will also be given concerning the regularity or non-regularity of the building. It'll be design and analyse according to Eurocode 1: part 1-1, Eurocode 1: part 1-2, Eurocode 5: part 1-1 and Eurocode 5: part 1-2, and Eurocode 2: part 1-1, Eurocode 7: part 1-1 specifically for infrastructure part of building.

2.4.2. Determination of load on buildings method

2.4.2.1. Permanent loads

Also known as static or dead loads, these are actions that act on the whole nominal life of the structure with a negligible variation of their intensity in time. These include the self-weight of the structural elements, and the self-weight of the non-structural elements present in the nominal life of the structure, but which do not take part in the load bearing mechanism.

2.4.2.2. Imposed loads

These are actions on structures for which their variation in magnitude with time is not negligible. These actions include imposed loads and wind loads.

Based on the building category and provisions of BS-EN1991-1-1 clause 6.3.1, the loads assigned to different parts of the building will be as indicated in the table 2.4

Table 2.1. Imposed loads of floors, balconies and stairs in buildings (BS-EN1991-1-1, 2002)

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

2.4.2.3. Accidental loads

These loads are usually of short duration but of significant magnitude, that are unlikely to occur on a given structure during the design working life. An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken. These loads include fire, explosions, impact from vehicles etc. In this work, the accidental load that will be taking into consideration is the fire load.

2.4.3. Limit states

A structure is designed according to the corresponding limit states in such a way to sustain all actions acting upon it during its intended life. This implies it will be designed having adequate structural stability (ultimate limit states) and remain fit for the use it is required (serviceability limit states).

2.4.3.1. Ultimate limit states

According to Eurocode 1990 (2002), ultimate limit states (ULS) correspond to the loss of structural capacity of the whole structure or one of its fundamental elements (for example structural collapse). It concerns the safety of people and/or the safety of the structure. The loss of structural capacity includes:

- Excessive displacements or deformations;
- Loss of equilibrium of the whole structure or one of its fundamental parts;

- Reaching of the maximum strength capacity of the entire structure;
- Reaching of the maximum strength capacity of parts of structures, joints, foundations;
- Reaching of failure mechanisms in the soils.

2.4.3.2. Serviceability limit states

Also, according to the Eurocode 1990 (2002), serviceability limit state (SLS) is the inability of the structure to meet the specify service requirement. This includes mainly:

- Deformations and displacements that affect the appearance or effective use of the structure (including the operation of machinery or services) or damage finishes or non-structural elements;
- Vibrations which affect the comfort of people, damage the structure or the equipment it supports, or limit its efficiency;
- Damage (including cracks) that may affect the appearance, durability or function of the structure;
- Visible damage caused by fatigue and other time-related effects.

2.4.4. Load combinations

A combination of actions defines a set of values used for the verification of the structural reliability for a limit state under the simultaneous influence of different actions. For the verification of the structure at ultimate limit state (ULS), the load combinations used were given by equation (2.1) and (2.2).

ULS1: $$\gamma_{G,1}\, G_1 + \gamma_{G,2}\, G2 + \gamma_{Q,m}\, Q_m + \gamma_{Q,w}\Psi_{0,w}\, Q_w \quad \textbf{(2.1)}$$

ULS2: $$\gamma_{G,1}\, G1 + \gamma_{G,2}\, G2 + \gamma_{Q,w}\, Q_w + \gamma_{Q,m}\Psi_{0,m}\, Q_m \quad \textbf{(2.2)}$$

Where :

Q_m is the maintenance load on the roof;

Q_w are the wind loads acting on the roof, windward and leeward side;

G_1 are the self-weight of the structural components;

G_2 is the self-weight of the aluminium roof;

$\gamma_{i,j}$ is the safety factor for permanent and variable loads and its values are obtained from annex 4;

$\Psi_{i,j}$ are the combination coefficients and their values are given in annex 4.

For the verification of the structure at serviceability limit state (SLS), the load combination used is given in equation (2.3), (2.4) and (2.5).

SLS1(characteristic combination): $G_1 + G_2 + Q_m + \Psi_{0,w} Q_w$ **(2.3)**

SLS2 (frequent combination): $G_1 + G_2 + \Psi_{1,w} Q_w + \Psi_{2,m} Q_m$ **(2.4)**

SLS3 (quasi-permanent combination) : $G_1 + G_2 + \Psi_{2,m} Q_m + \Psi_{2,w} Q_w$ **(2.5)**

2.5. Structural design method of wooden structures

In this section, the design methods of the structure in a pre-fire situation and in a fire situation.

2.5.1. Design in normal temperature

2.5.1.1. Ultimate Limit State Design

At the ultimate limit states, the design of the elements consists of verifying the resistance of the element to the different stresses, namely bending, compression, shear and tension.

a. Checking an element subject to tension

- The tensile stress design will be assessed by the equation (2.6)

$$\sigma_{t,0,d} = \frac{N}{A} \tag{2.6}$$

Where

N: the tensile force in Newton (N);

A: the cross-section area in mm^2;

$\sigma_{c,0,d}$: Design tensile stress along the grain (MPa).

- The tensile strength design will be assessed by the equations (2.7)

$$f_{t,0,d} = f_{t,0,k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_h \quad \textbf{(2.7)}$$

$$f_{t,0,k} = 0{,}6 f_{m,k} \quad \textbf{(2.8)}$$

Where

$f_{t,0,k}$: Characteristic tensile strength along the grain (MPa);

$f_{m,k}$: Characteristic bending strength (MPa);

$f_{t,0,d}$: Design tensile strength along the grain (MPa);

k_{mod}: Modification factor for duration of load and moisture content;

γ_m: is the partial coefficient for the material;

k_h: Depth factor it's given by:

- For solid timber — if h ≥ 150 mm, $K_h = 1$;
  if h < 150 mm, K_h= min (1,3; $(150/h)^{0,2}$);
- For Glued laminated timber — if h ≥ 600 mm, $K_h = 1$;
  if h < 150 mm, K_h= min (1,1; $(600/h)^{0,1}$).

The equation (2.9) should be checked.

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} \leq 1 \quad \textbf{(2.9)}$$

b. Checking an element subject to compression

- The Compressive stress design will be assessed by the equation (2.10).

$$\sigma_{c,0,d} = \frac{N}{A} \quad \textbf{(2.10)}$$

Where

N: the tensile force in Newton (N);

A: the cross-section area in mm^2;

$\sigma_{c,0,d}$: Design tensile stress along the grain (MPa).

- The compressive strength design will be assessed by the equation (2.11)

$$f_{c,0,d} = f_{c,0,k} \cdot \frac{k_{mod}}{\gamma_M} \tag{2.11}$$

Where

$f_{c,0,k}$: Characteristic compressive strength along the grain (MPa);

$f_{c,0,d}$: Design compressive strength along the grain (MPa);

K_{mod}: Modification factor for duration of load and moisture content;

γ_m: is the partial coefficient for the material.

The equation (2.12) should be checked.

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} \leq 1 \tag{2.12}$$

$k_{c,z}$ or $k_{c,y}$ is a buckling coefficient given by equation (2.13)

$$k_{c,z} = \frac{1}{k_z + \sqrt{{k_y}^2 + {\lambda_{rel,z}}^2}} \tag{2.13}$$

$$k_c = 0,5[1 + \beta_c(\lambda_{rel,z} - 0,3) + {\lambda_{rel,z}}^2 \tag{2.14}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \qquad \textbf{(2.15)}$$

$$\lambda_z = \frac{l_{ef}}{\sqrt{\frac{I_{Gz}}{A}}} \qquad \textbf{(2.16)}$$

c. Checking an element subject to bending

- The bending stress design will be assessed by the equations (2.17)

$$\sigma_{m,d} = \frac{M}{W} \qquad \textbf{(2.17)}$$

$$W = \frac{I_{G,x}}{V} \qquad \textbf{(2.18)}$$

Where

M: the bending moment (kN.m);
W: the bending modulus in mm^3;
$I_{g,x}$: the moment inertia in mm^4;
V: the position of neutral axis in mm;
$f_{m,d}$: Design bending strength (MPa).

- The strength design will be assessed by the equation (2.19)

$$f_{m,d} = f_{m,k} \cdot \frac{k_{mod}}{\gamma_M} \cdot K_{sys} K_h \qquad \textbf{(2.19)}$$

Where

f_{mk}: Characteristic compressive strength along the grain (MPa);
$f_{m,d}$: Design compressive strength along the grain (MPa);
k_{mod}: Modification factor for duration of load and moisture content;

k_h: Depth factor;

k_{sys}: System strength factor;

γ_m: is the partial coefficient for the material.

The equation (2.20) should be checked.

$$\frac{\sigma_{m,d}}{k_{crit} \cdot f_{m,d}} \leq 1 \tag{2.20}$$

k_{crit} is factor used for lateral buckling.

d. Checking an element subject to shear force

- The shear stress design will be assessed by the equation (2.21)

$$\tau_{v,d} = \frac{k_f T_x}{b * h} \tag{2.21}$$

Where

T_x: the shear force in Newton (N);

A= b*h: the cross-section area in mm^2;

τ_{vd}: Design shear stress along the grain (MPa);

K_h: Shape coefficient of section (3\2 for rectangular cross-section and 4\3 for circular cross-section).

- The shear strength design will be assessed by the equation (2.22)

$$f_{v,d} = f_{v,k} \cdot \frac{k_{mod}}{\gamma_M} \tag{2.22}$$

$$f_{v,k} = \min(3{,}8\,;0{,}2(f_{m,k})^{0{,}8} \tag{2.23}$$

Where

$f_{v,k}$: Characteristic shear strength (MPa);

$f_{m,k}$: Characteristic bending strength (MPa);

$f_{v,d}$: Design shear strength (MPa);

k_{mod}: Modification factor for duration of load and moisture content;

γ_m is the partial coefficient for the material.

The equation (2.24) should be checked.

$$\frac{\tau_{v,d}}{f_{v,d}} \leq 1 \tag{2.24}$$

2.5.1.2. Serviceability Limit State Verification

This will have to do with a verification of deflection with respect to the corresponding load acting on the building. Limiting deflection values will be determined by taking into consideration the different deflection components that will be calculated and compared with the values provided in table 2.8. The various deflection values are summarized in the figure 2.1.

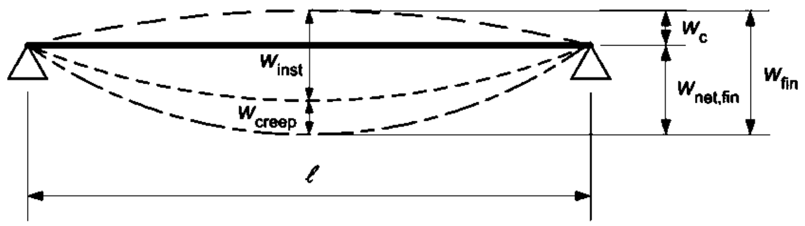


Figure 2.1. Components of deflection at serviceability limit state (EN 1995-1-2:2004)

where:

w_c is the prechamber (if applied);

w_{inst} is the instantaneous deflection;

w_{creep} is the creep deflection;

w_{fin} is the final deflection;

$w_{net,fin}$ is the net final deflection.

Thus, the net deflection below a straight line between the supports, $w_{net,fin}$ is obtained from equation (2.25) or (2.26):

$$w_{net,fin} = w_{inst} + w_{creep} - w_c \tag{2.25}$$

$$w_{net,fin} = w_{fin} - w_c \tag{2.26}$$

The different last deflexion module (**W**i) is defined by equation (2.27)

$$W_i = \frac{5.q_i.L^4}{384.E_{0,mean}.I_{G,x}} \quad \textbf{(2.27)}$$

Where

Wi is the deflection i of element;

L is the span of element;

$E_{o,mean}$ is the mean young modulus of material;

Ig,x is the moment of inertia of element.

Table 2.2. Examples of limiting values for deflections of beams (Eurocode 5: part 1-1_2004)

	W_{inst}	$W_{net,fin}$	W_{fin}
Beams on two supports	l/300 to l/500	l/250 to l/350	l/150 to l/300
Cantilevering beams	l/150 to l/250	l/125 to l/175	l/75 to l/150

2.5.2. Design in fire condition

2.5.2.1. Analytical design method in fire scenario

In the analytical approach, there are two main design methods, namely the Reduced cross-section method and Reduced properties method

a. Reduced cross section method

In the reduced cross-section section method, the fire resistance time depends on the resistance capacity of the residual uncharred section. The effective cross-section should therefore be calculated by reducing the initial cross-section by a conventional charring depth using equation 2.28. Figure 2.2 showed different cross-section of element

$$d_{ef} = d_{char} + k_0 d_0 \qquad \textbf{(2.28)}$$

Where

d_{char} Charring depth;

d_{ef} Effective charring depth;

d_0 =7 mm is a depth of the zero-strength layer;

k_0 is given by annex 5;

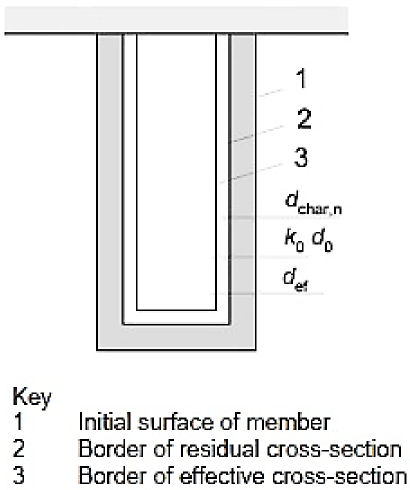


Figure 2.2. Illustration of residual cross-section and effective cross-section (EN 1995-1-2:2004)

b. Reduced properties method (only applied for softwood)

The bending, compression and tensile strength can be calculated by the reduced property method using the residual cross-section as defined in equation (1.14) if corner rounding is not taken into account, or using the residual cross-section as defined in equation (2.29) if corner rounding is taken into account

$$\mathbf{d_{char} = \beta_n . t} \qquad \mathbf{(2.29)}$$

with β_n Charring rates

The modification factor for fire $k_{mod,fi}$ are given by equations (2.30), (2.31) and (2.32).

- For bending strength:

$$k_{mod,fi} = 1{,}0 - \frac{1}{200}\frac{P}{A_r} \qquad \mathbf{(2.30)}$$

- For compressive strength:

$$k_{mod,fi} = 1{,}0 - \frac{1}{125}\frac{P}{A_r} \qquad \mathbf{(2.31)}$$

- For tensile strength and modulus of elasticity:

$$k_{mod,fi} = 1{,}0 - \frac{1}{330}\frac{P}{A_r} \qquad \mathbf{(2.32)}$$

where:

P is the perimeter of the fire exposed residual cross-section, in metres;

A_r is the area of the residual cross-section, in m^2.

c. Protection panels (wood-based panels)

The failure time of claddings made of wood-based panels should be taken as define by equation (2.33)

$$t_f = \frac{h_p}{\beta_0} - 4 \qquad \mathbf{(2.33)}$$

where:

t_f is the failure time, in minutes [min];

h_p is the panel thickness, in millimetres [mm];

β_0 is the design charring rate for one-dimensional charring under standard fire exposure, in mm/min

Design charring rates for wood-based panels in accordance with EN 309, EN 313-1, EN 300 and EN 316, and wood panelling are given in Table 1.6. The values apply to a characteristic density of **450 kg/m³** and a panel thickness of **20 mm**.

For other characteristic densities ρ_k and panel thicknesses hp smaller than 20 mm, the charring rate should be calculated as define by equations (2.34)

$$\boldsymbol{\beta_{0,\rho,t}} = \beta_0 . k_\rho . k_h \qquad \textbf{(2.34)}$$

with

$$k_\rho = \sqrt{\frac{450}{\rho_k}} \qquad \textbf{(2.35)}$$

$$k_h = \sqrt{\frac{20}{h_p}} \qquad \textbf{(2.36)}$$

where:

ρ_k is the characteristic density, in kg/m^3;

h_p is the panel thickness, in millimetres.

d. Walls and floors

The spacing of wall studs and floor joists should not be greater than 625 mm. For walls, individual panels should have a minimum thickness given by equation (2.37)

$$\mathbf{t_{p,min} = max\ (l_p/70\ ;\ 8)} \qquad \textbf{(2.37)}$$

where:

$t_{p,min}$ is the minimum thickness of panel in millimetres;

l_p is the span of the panel (spacing between timber frame members or battens) in millimetres.

Wood-based panels in constructions with a single layer on each side should have a characteristic density of at least 350 kg/m.

The failure times of floors exposed to fire from below near the joints are taken as in equation (2.38)

$$t_f = k_j \frac{h_p}{\beta_0} \tag{2.38}$$

where k_j is joint coefficient are given in annex 6

2.5.3. Design foundation method

The length L of the pile is determined by the soil stress determined by geotechnical tests and the desired cross-section of the element. From this soil stress, the cross-section of the base (Figure 2.3) is determined by Equation (2.39) and that of the piling by Equation (2.40). The cross-section of their reinforcement when determined by equations (2.41) and (2.42) respectively and must be within the limits presented in Table 2.3.

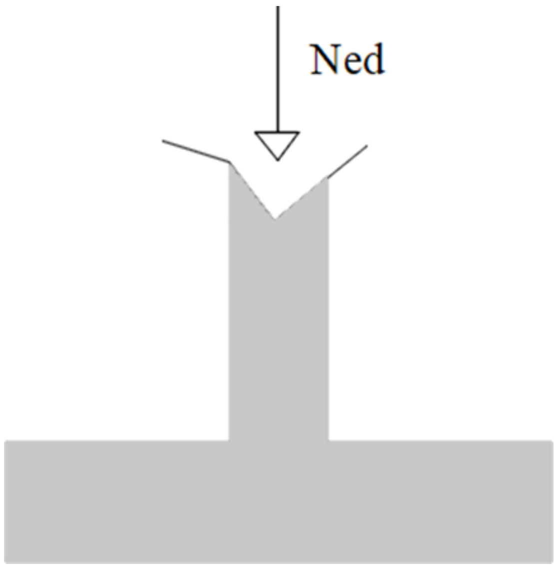


Figure 2.3. Representation of the base

$$S \geq \frac{N_{ed}}{\sigma_{soil}} \quad \textbf{(2.39)}$$

$$A_C = 0{,}9\,\frac{N_{ed}}{f_{cd}} \quad \textbf{(2.40)}$$

$$A_S = 0{,}1\,\frac{N_{ed}}{f_{yd}} \quad \textbf{(2.41)}$$

$$A_S = \frac{(A-a).N_{ed}}{8d.f_{yd}} \quad \textbf{(2.42)}$$

Where

N_{ed}: The axial force in N,

S: Surface of the footing;

A_c: Area of the pile;

As: reinforcement cross-section;

A: The dimension of one side of the footing in mm;

a: The dimension of one side of the column in mm;

d: The useful height of the footing in mm;

Table 2.3. Minimum values for reinforcement cross-sections in the pile

$Ac \leq 0{,}5$ m²	$As \geq 0{,}005$ m²
0,5 m² $< Ac \leq 1$ m²	$As \geq 25$ m²
$Ac > 1$ m²	$As \geq 0{,}0025$ m²

2.6. Modelling and analysis procedure

The modelling will be done on SAP2000; thus, the first load-bearing elements will be modelled and loaded according to the conditions defined in parts 2.4.2 and 2.4.3 of this part.

The obtained solicitations will then be used in the Excel software for the analysis in normal temperature (determination of the cross-section verifying the resistance conditions), and the analysis in fire situation according to the procedures described in part 2.5 of the present part.

Conclusion

This chapter presents the methods and procedures used to arrive at the results of the analysis of the influence of fire on the structural behaviour of timber structures. It follows that an analysis of the structure (design) will be carried out in static condition at normal temperature; thereafter the performance of the studied structural elements will be evaluated under fire conditions and thus the verification failure time for the different loads will be recorded. This work will be done with a variation of the 2 analytical methods prescribed by Eurocode 5: part 1-2, and a variation of the wood species to better analyse and evaluate the behaviour of our structure under fire load. All this will of course be preceded by determination of the actions either manually in Excel, or automatically in SAP 2000.

CHAPTER 3: PRESENTATION AND INTERPRETATION OF RESULTS

Introduction

The third chapter presents the results of the procedure described in chapter two. Thus, it consists of presenting the different results of the research on the site where the case study is located, the results of the static verification of the different load-bearing elements of the structure (vertical and horizontal).in normal temperature.

Then, the results of the fire analysis (analysis by element allowed by Eurocode 1: part 1-2 and Eurocode 5: part 1-2) of the different load-bearing elements (vertical and horizontal one) according to the different analytical methods (effective cross-section method and the reduced strength and stiffness method in respect of Eurocode 5: part 1-2) used for the species considered will also be presented in situation of nominal fire (normalised fire: ISO 834 fire, in respect of prescriptive approach)

Finally, a case study on structural elements under fire protection cladding will be carried out to evaluate the performance gain acquired by the protection in fire situations.

3.1. General presentation of the site

Here is a general overview of the Yaounde site, including its geographic position, geology, relief, climate, hydrology, population, and socioeconomic activity.

3.1.1. Geographical data

Yaounde serves as both the national capital (political capital) of Cameroon and regional capital of Centre region. It is 726 metres above sea level and situated at 3.85° North latitude and 11.52° East longitude. The distance to the Atlantic Ocean is 300 kilometres. Yaounde is located in the centre section of the country and has a total area of 180 km^2 (18000 ha). Figure 3.1 show a view of geographical location of site

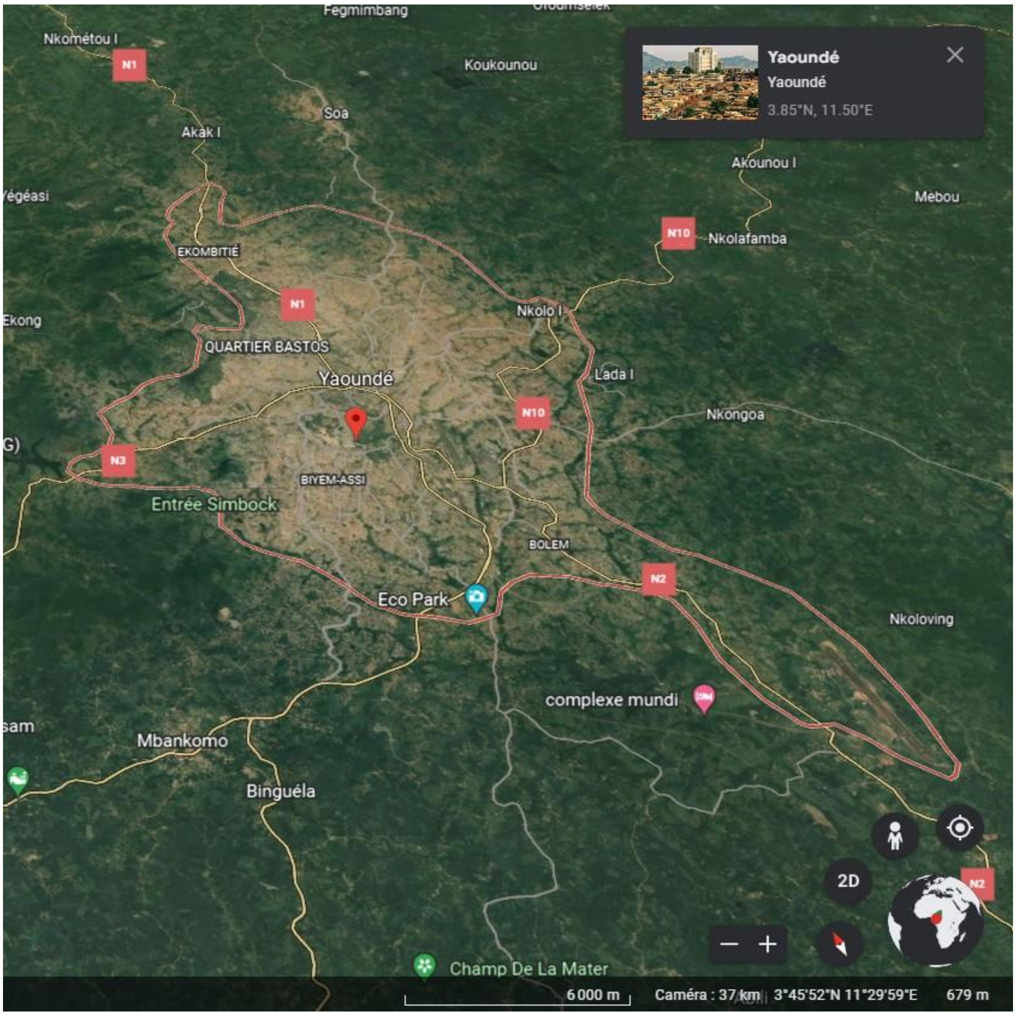


Figure 3.1. Location of Yaounde (Google Earth)

3.1.2. Geology and relief

Gneiss makes up the majority of Yaounde's bedrock. Although this rock isn't porous or soluble, its discontinuities (faults, diaclases) give the formation fissure permeability. The bedrock's continuous aquifers, which are roughly exploitable and overlaid or isolated, highly water-bearing fissures or fracture aquifers, are what define the hydrogeology. When it comes to relief, the land gently escarpments from the southwest coastal plain before connecting to the Adamawa Plateau via depressions and granite massifs. The field has gently sloping hills covered in forest, the tallest of which have barren, rocky tops.

3.1.3. Climate

Yaoundé has a humid and dry tropical climate, with record high temperatures of 36°C, an average of 23.8°C and a record low temperature of 14°C. Mainly due to the altitude, temperatures are not as high as one would expect for a city located near the equator. The city of Yaounde has a long rainy season covering a nine-month period between March and November. However, there is a noticeable decrease

However, there is a noticeable decrease in rainfall during the rainy season, observed during the months of July and August, giving the city the appearance of having two rainy seasons and two dry seasons. The average rainfall is 1650 mm per year and the average humidity is 80%.

3.1.4. Hydrology

The hydrographic network of Yaounde is very dense and composed of permanent rivers such as the Mfoundi River which crosses the city from North to South, some creeks and lakes. Yaounde is part of the western sector of the southern Cameroonian plateau. The area is characterised by

The area is characterised by gentle chains of rolling hills and numerous valleys and wetlands: this varied physical landscape allows for a combination of streams, hydromorphic soils and a large variety of plants and wildlife

3.1.5. Population and economic activities

4.3 million people are projected to live in Yaounde as of 2022 (worldpopulationreview.com,2022). Yaounde is a cosmopolitan metropolis with a fairly large population that hails from several other parts of the nation (East, West, North-West, Sud-west, far North etc.). The diplomatic services and the civil service's administrative framework make up the majority of Yaounde’s economy. Yaounde is safer and has a higher level of living than the rest of Cameroon as a result of these factors. Although Yaounde is a secondary city, there are a few industries there, including breweries, sawmills, carpentry, tobacco, paper mills, machinery, and building supplies.

3.2. Presentation of the project

The architectural data, material properties and different design will be made in this part

3.2.1. Architectural data

The project is that of a ground floor + 2 storeys, for social housing use, consisting of 4 flats (composed of a living room + 3 bedrooms, kitchen and toilets) per level, making a total of 12 flats. The building is made of wood and has a height of 9 m for a floor area of 520.50 m^2 for a perimeter of 193.97 m. The different plane of the structure are showed in Figure 3.2, 3.3, 3.4 and 3.34.

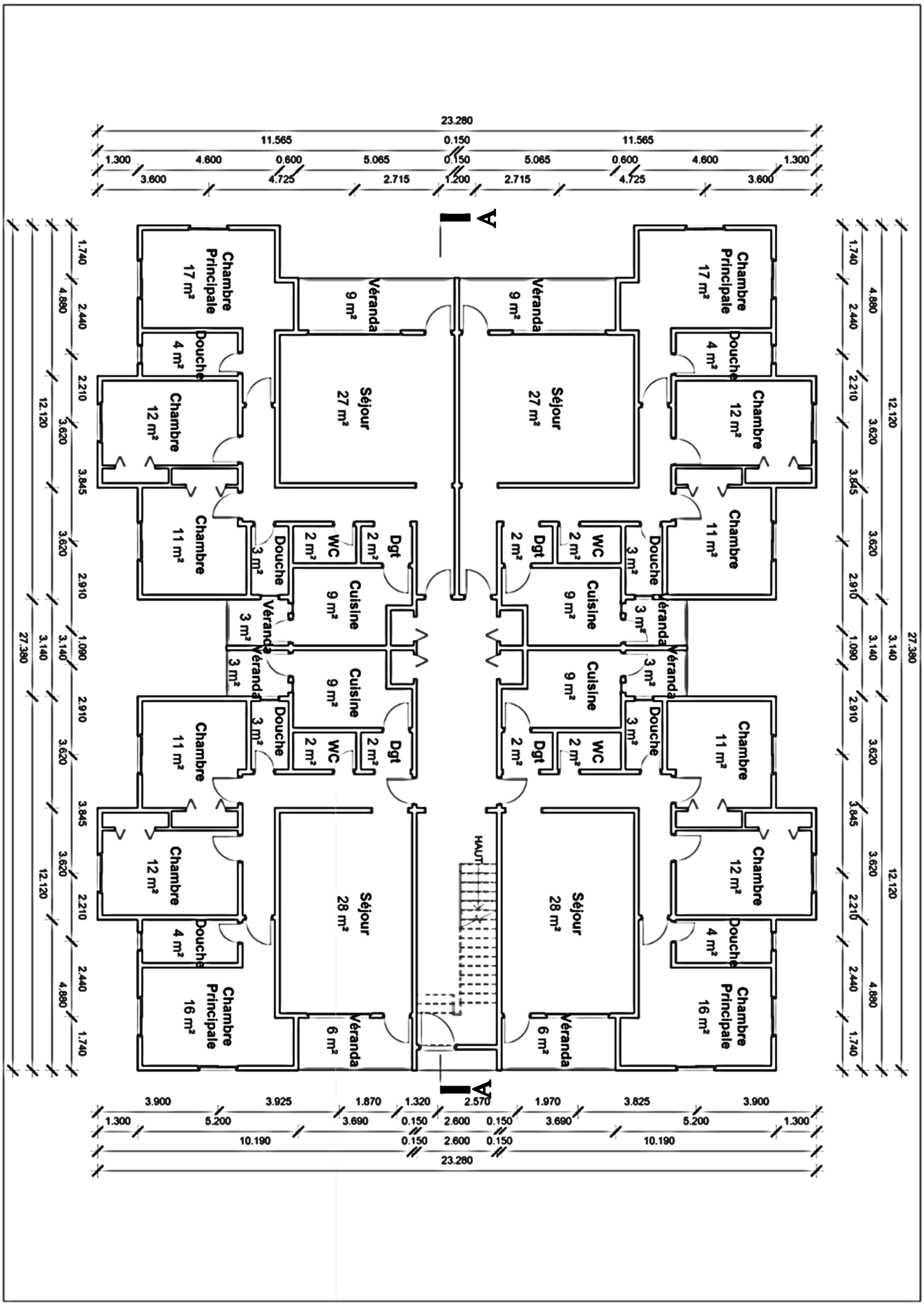


Figure 3.2. Presentation of the floor plane

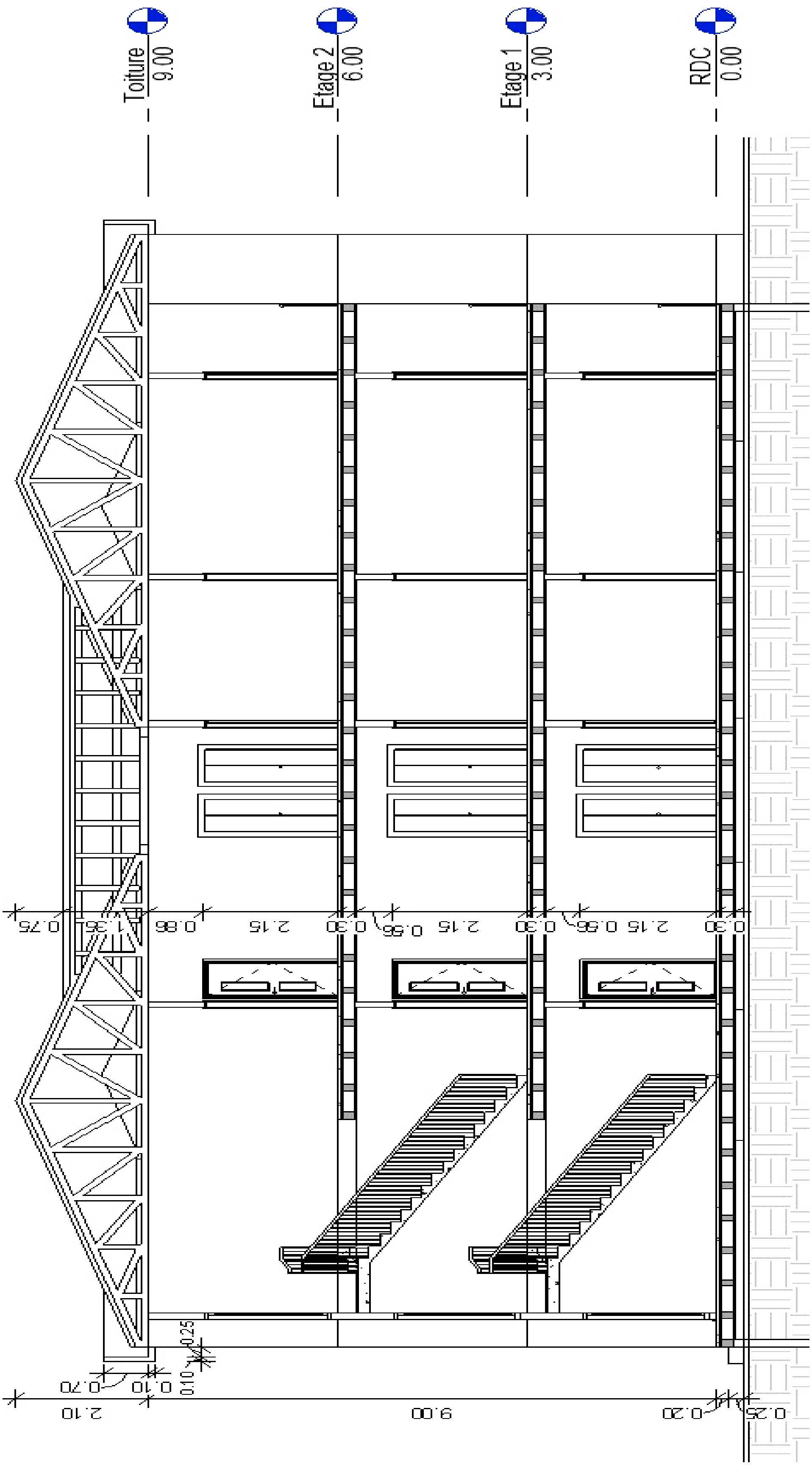


Figure 3.3. Presentation of the section A-A.

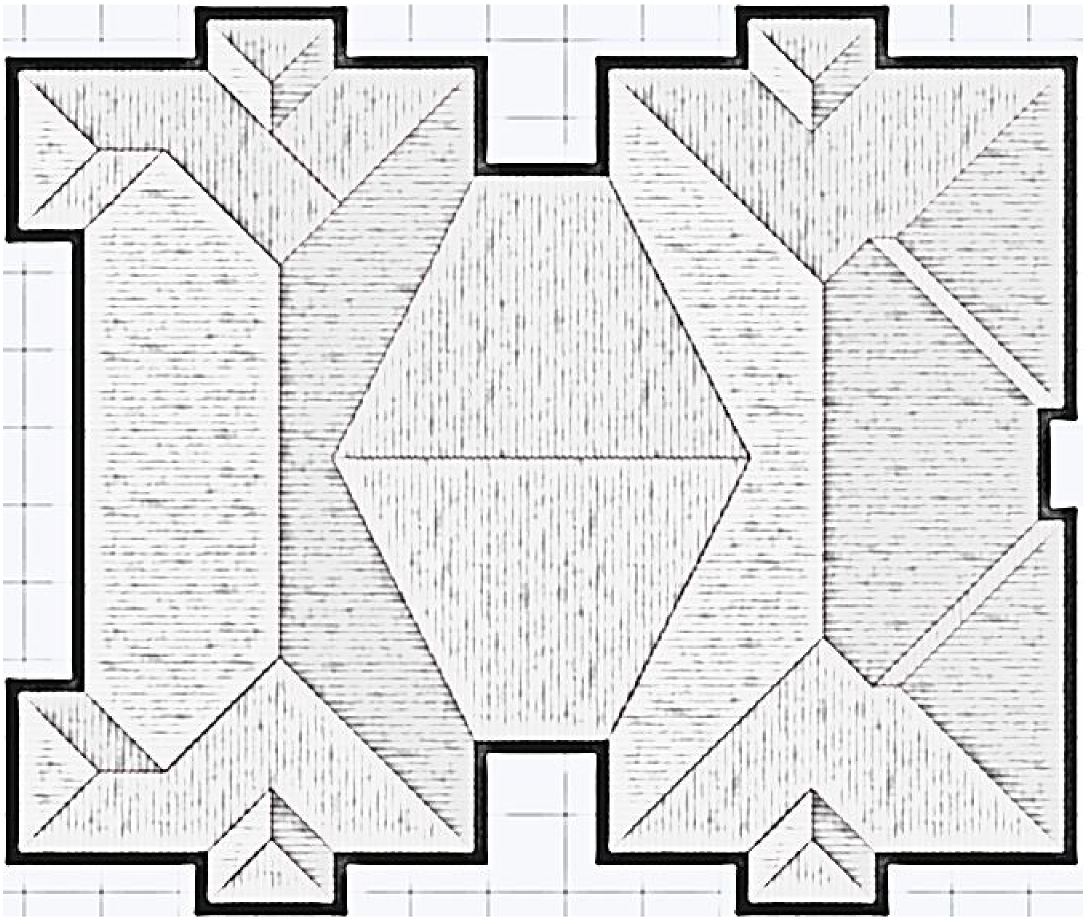


Figure 3.4. Illustration of 2D view of roof

3.2.2. Presentation of characteristics of the materials

The characteristics of the timber used in the design of the structure, as well as those of the concrete and steel used for the foundations will be presented in this section

3.2.2.1. Mechanical properties of wood

After investigation of the different wood species found in Cameroon and elsewhere, two species were selected for the study: Moabi (hardwood type) and Fir (softwood type).

a. Characteristic strength

The different characteristics values of different wood species given by the technical data sheets are given in Table 3.1

Table 3.1. Summary table showing the characteristic mechanical properties of the various species of wood

Property	SPECIES:		Unit	definition
	Moabi	Fir		
ρ	0,87	0,45	$g\backslash cm^3$	**Density weight**
γ	8,53	4,41	$kN\backslash m^3$	**Density force**
f_{mk}	143	86	MPa	**Characteristic value of bending strength**
$f_{c,0,k}$	74	46	MPa	**Characteristic value of compressive strength parallel to grain**
$f_{t,0,k}$	85,80	51,60	MPa	**Characteristic value of tensile strength parallel to grain**
$f_{v,k}$	10,60	7,06	MPa	**Characteristic value of shear strength**
$E_{0,mean}$	21040	12200	MPa	**mean characteristic value of Modulus of elasticity parallel to grain**
$E_{0,05}$	14096,8	8174	MPa	**5-percentile characteristic value of modulus of elasticity parallel to grain**

b. Design strength

Assumptions considered for the assessment of design values:

- Class of service: Class 1
- Type of material: Solid timber

These assumptions allow the determination of the coefficients show in Table 3.2

Table 3.2. Structural coefficient for design

γ_M	k_{mod} (class 1)	k_h	k_{sys}	k_{def}	Ψ_2	k_f	k_V
1,3	0,8	1,0	1,1	0,6	0,3	1,5	1

The Table 3.3 gives the different design values that will be used for the verification of the different sections of the different dimensioned elements

Table 3.3. Summary table showing the design mechanical properties of the various species of wood

Property	SPECIES:		Unit	Definition
	Moabi	Fir		
f_{md}	96,8	58,2	MPa	**Design value of bending strength**
$f_{c,0,d}$	45,54	28,31	MPa	**Design value of compressive strength parallelel to grain**
$f_{t,0,d}$	52,80	31,75	MPa	**Design value of tensile strength parallelel to grain**
$f_{v,d}$	6,52	4,34	MPa	**Design value of shear strength**

3.2.2.2. Mechanical properties of concrete

The characteristics of the concrete used in the foundation are presented in Table 3.4.

Table 3.4. Concrete characteristics

Property	Value	Unit	Definition
Class	C25/30	-	Concrete class
f_{ck}	25	N/mm^2	Characteristic compressive strength at 28 days
f_{cm}	33	N/mm^2	Mean value of concrete cylinder compressive strength
γ_c	1.5	-	Partial safety factor for concrete
f_{cm}	14.16	N/mm^2	Design value of compressive strength
f_{ctm}	2.56	N/mm^2	Mean value of axial tensile strength of concrete
f_{ctd}	1.2	N/mm^2	Design resistance in traction
E_{cm}	31476	N/mm^2	Secant modulus of elasticity
ν	0.5	-	Poisson ratio
G	13115	N/mm^2	Shear modulus
γ	25	kN/m^3	Specific weight of concrete

3.2.2.3. Mechanical properties of steel

The characteristics of the steel used in the foundation are presented in Table 3.5.

Table 3.5. Steel characteristics

Property	Value	Unit	Definition
Class	B400B		Steel class

f_{yk}	400	N/mm^2	Characteristic yield stress
γ_s	1.15	-	Partial safety factor for steel
γ	78.5	kN/m^3	Specific weight of steel
ν	0.3	-	Poisson ratio

3.2.3. Presentation of the determination of load

The different actions that will apply to the floors of the structure are presented in this part.

3.2.3.1. Permanent load

a. Structural load

This part will be presented on a case-by-case basis depending on the element analysed.

b. Non-structural load

The different non-structural permanent loads are presented in Table 3.6.

Table 3.6. Load applied on different floor

Non-Structural Dead Load G_2 (kN\m^2)			
Designation	***Density (kN\m^3)***	***Thickness (cm)***	***G (kN\m^2)***
Tiles (\cm)	0,2	1	0,2
Chape (\cm)	0,2	5	1
Wall			0,4
Total load G2 (kN\m^2)			1,6
Total load G2 (kN\m)			**0,8**

3.2.3.2. Live load

The live load $\mathbf{Q_K}$ considered for the structure is of modulus **2 kN\m^2**.

3.3. Presentation of the results of analysis

For the analysis of the structure, one horizontal and one vertical element will be analysed, which will be among the most loaded elements of the structure; this will be done in turn in a pre-fire and fire situation. Thus, the horizontal element analysed will be the joist and the vertical element will be the column. The analysis in a fire situation will be done either by using the reduced cross section method (recommended for both softwoods and hardwoods), or the reduced property method (recommended only for softwoods), or both depending on the wood species considered.

3.3.1. Result of analysis of the horizontal load-bearing element: case of the joist

Here, it is important to first analyse and assess the joist's structural behaviour in the absence of fire, then take into account the fire situation, and finally to assess the effect of a wood protective coating on the joist's structural behaviour.

3.3.1.1. Result of analysis of joist in normal temperature (without fire)

a. Joist location

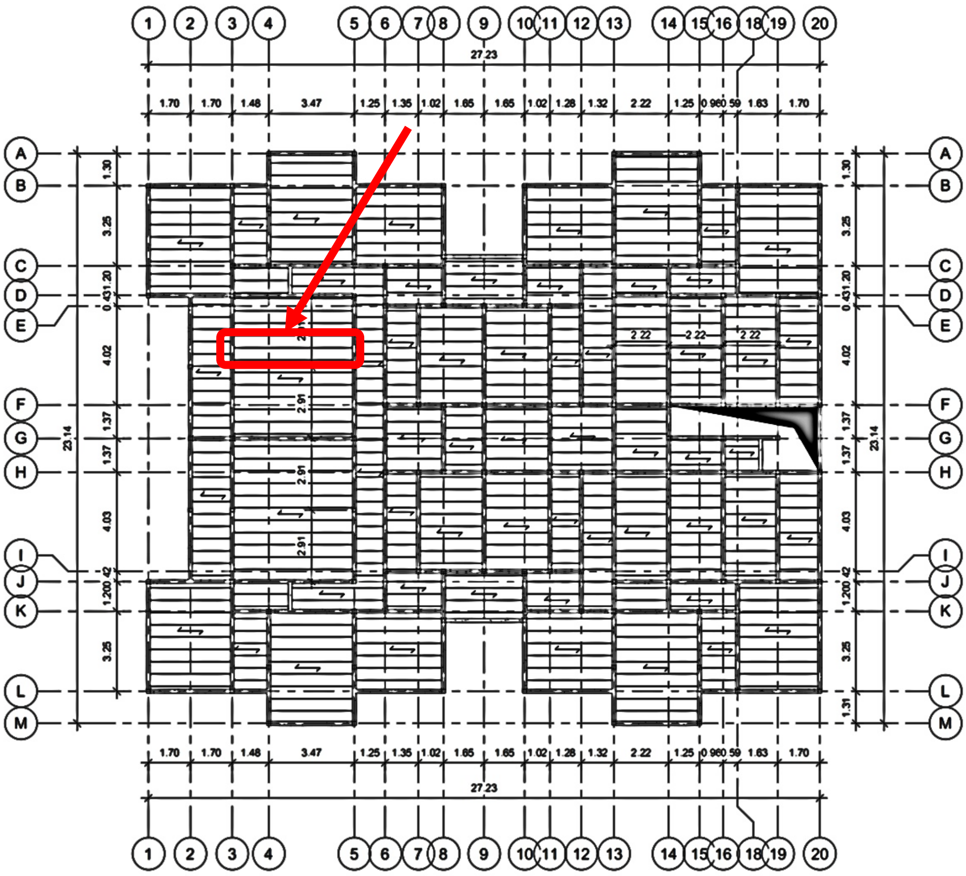


Figure 3.5. Presentation of the joist considered

b. Geometrical characteristics of the joist

The geometrical characteristics of the joist are presented in table 3.7.

Table 3.7. Geometrical characteristics of the joist in normal temperature

Properties	Value	Unit	Definition
b	0,12	**m**	**Width**
h	0,2	**m**	**Height**
L	4,95	**m**	**Span**
$I_{g,x}$	80000000	**mm^4**	**Moment of inertia**
V	100	**mm**	**Neutral axis distance**
W	800000	**mm^3**	**Section modulus**
δ	0,5	**m**	**Spacing**

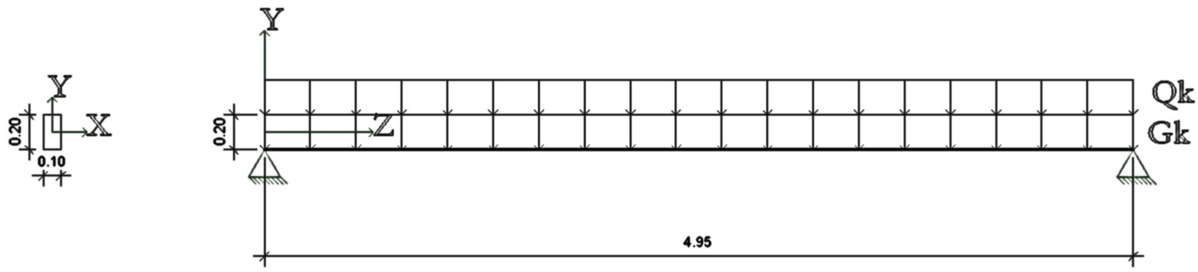


Figure 3.6. Geometrical characteristic of the joist

c. Total load applied to the joist depending on the type of wood considered

The Table 3.8 shows the different combined loads in ULS applied to the joist

Table 3.8. ULS loads applied to the joist depending on the type of wood considered

Load	Moabi	Fir	Unit
Joist load G_{k1}	0,205	0,106	**kN\m**
Panels Load G_{k1}	0,085	0,044	**kN\m**
Total load G_{k2}	0,8	0,8	**kN\m**
Permanent Load on Joist ΣG_{ki}	1,090	0,950	**kN\m**
Live load Q_k (kN\m)	1	1	**kN\m**
Final joist load: $1.3G_k+1.5Q_k$	2,917	2,735	**kN\m**

d. Result of the verification of the joist

The joist has been checked for bending, shear, and deflection, which have been presented here.

i. Moment verification for the joist

The bending moments within the joist for both wood species are show in Figure 3.7.

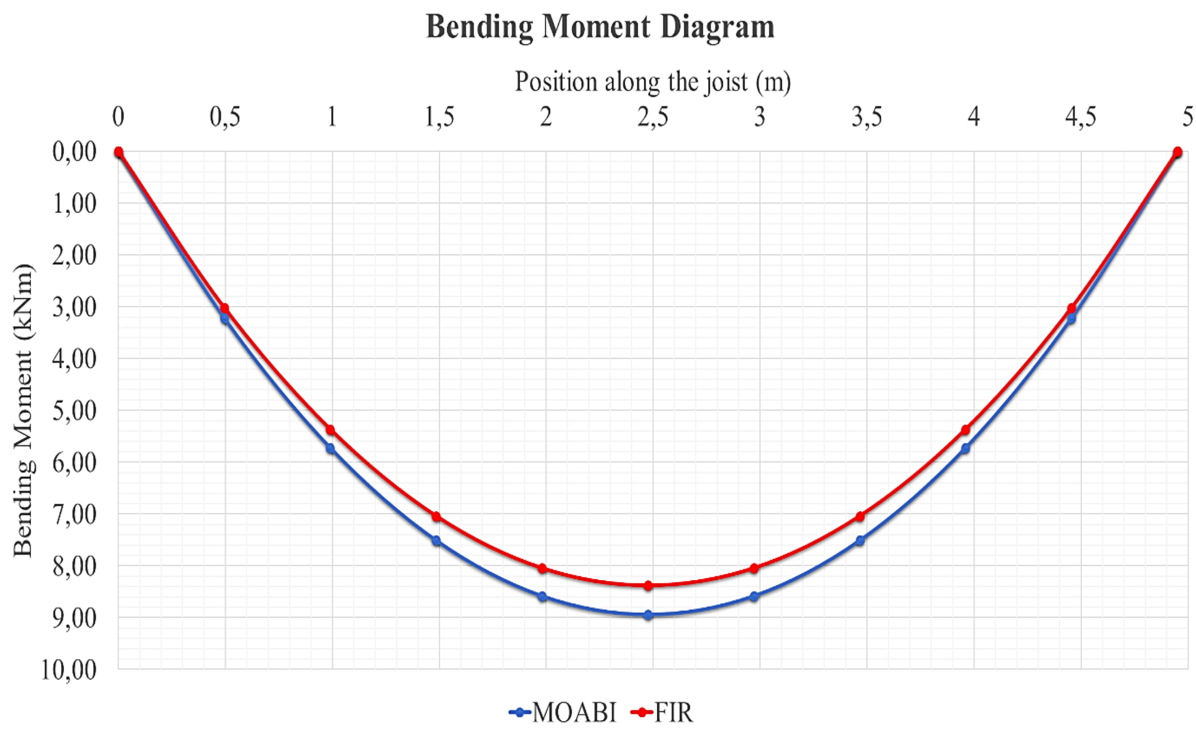


Figure 3.7. Bending Moment Diagram of the joist

The previous bending moments obtained from the joist analysis were checked against the design values and the result is presented in Table 3.9

Table 3.9. Moment verification for the joist

	Moabi	Fir	Unit
Bending Moment Mx	8,93	8,38	**kN.m**
Bending stress $\sigma_{m,d}$	11,17	10,47	**MPa**
Bending strenght $f_{m,d}$	96,8	58,2	**MPa**
Critical bending stress $\sigma_{m,crit}$	159,93	92,74	**MPa**

Relative slenderness for bending $\lambda_{rel,m}$	0,90	0,91	-
Lateral buckling factor K_{crit}	0,89	0,87	-
$K_{crit}.f_{m,d}$	86,15	50,63	-
Condition of verification	$\frac{\sigma_{m,d}}{k_{crit}.f_{m,d}} \leq 1$		
Evaluation of result	**0,13**	**0,21**	-
	Verified	**Verified**	-

ii. Shear verification for the joist

The shear forces within the joist for both wood species are show in Figure 3.8.

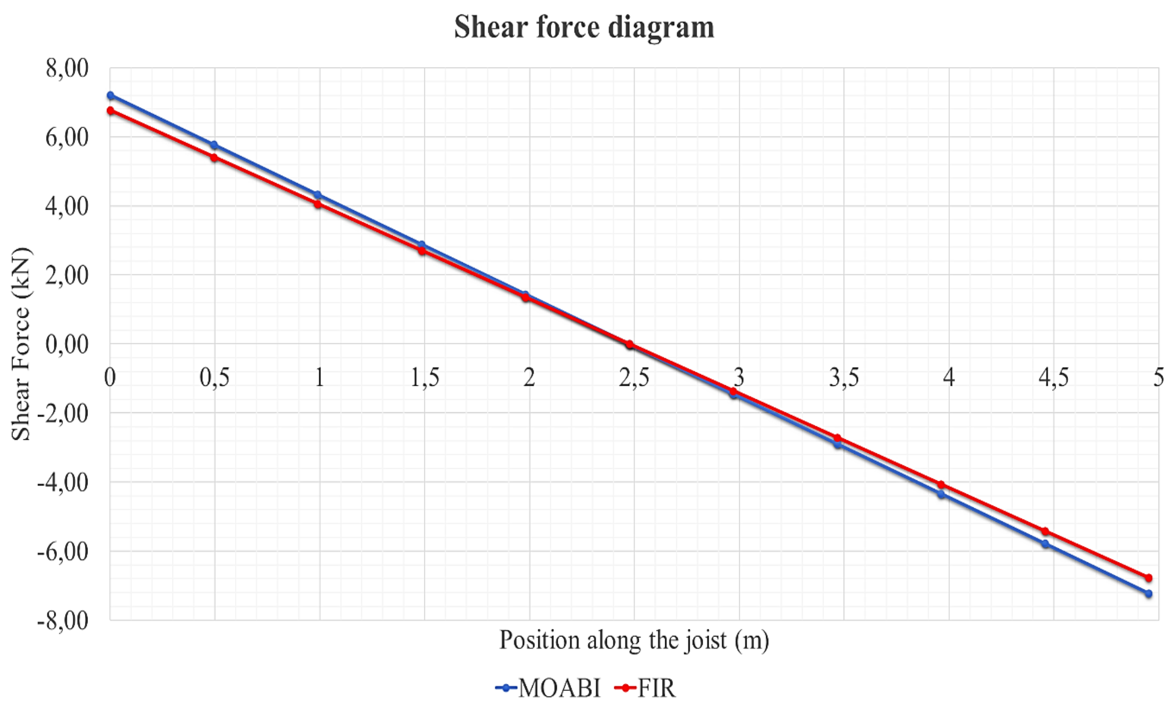


Figure 3.8. Shear force Diagram of joist

The shear forces obtained from the joist analysis were checked against the design values and the result is presented in Table 3.10.

Table 3.10. Shear verification for the joist

SPECIES:	Moabi	Fir	Unit
Shear force T_y	7,22	6,77	**kN**
Shear stress τ_{ed}	0,45	0,42	**MPa**
Shear strenght τ_{Rd}	6,52	4,34	**MPa**
Condition of verification	$\frac{\tau_{ed}}{\tau_{Rd}} \leq 1$		
Evaluation of result	**0,07**	**0,10**	**-**
	Verified	**Verified**	**-**

iii. Deflexion verification (SLS verification)

- Computation of the joist's limits deflections allowed (Table 3.11):

Table 3.11. Joist's limits deflections

$W_{inst,lim}$ ***(Q)***	16,5	mm
$W_{net,fin,lim}$	24,75	mm

- Load combination used for deflection assessment (Table 3.12)

Table 3.12. SLS loads applied to the joist depending on the type of wood considered

Load	SPECIES:		Unit
	Moabi	**Fir**	
q_{inst} **(Q)**	1	1	**kN\m**
q_{inst} **(G+Q)**	2,090	1,950	**kN\m**
q_{creep} **[$K_{def}(G+\Psi_2 Q)$]**	0,834	0,750	**kN\m**

- Evaluation of the deflection in respect to the limit deflection

Table 3.13 gives the verification of the deflections for each wood species in relation to the limit values prescribed by the Eurocode calculated in Table 3.11. The condition of verification is given by:

$$\frac{W}{W_{lim}} \leq 1$$

Table 3.13. Deflection verification

SPECIES:		**Moabi**	**Fir**	
Creep deflection W_{creep}	W_{creep} *(G, Q)*	3,87	6,01	**mm**
Instantaneous deflection W_{inst}	W_{inst} *(G, Q)*	9,71	15,62	**mm**
Instantaneous deflection W_{inst} (Q)	W_{inst} *(Q)*	4,64	8,01	**mm**
	$W_{inst} \backslash W_{inst,lim}$	**0,28**	**0,49**	**-**
Observation		**Verified**	**Verified**	
Final net deflection $W_{net,fin}$	$W_{net,fin}$	13,58	21,63	**mm**
	$W_{net,fin} \backslash W_{net,fin,lim}$	**0,55**	**0,87**	**-**
Observation		**Verified**	**Verified**	

3.3.1.2. Result of analysis and evaluation of the joist in fire situation

Assumptions considered for the assessment of fire design:

- Eurocode 5-part 1-2 section 2.5.3 (3): The effect of the actions (e.g., internal forces and moments) relative to the initial support and boundary conditions can be deduced from the overall analysis of the structure in normal temperature design according to equation (2.37)

$$\mathbf{E_{fi,d} = [0,6]\ Ed}$$

- Class of service: Class 1
- Type of material: Solid timber
- The connection between the joists through the wooden floor will serve as a provision against lateral buckling of the joists

a. Case of the unprotected joist in fire exposition situation (3 exposed sides)

This section is reserved for the analysis of a joist exposed on 3 of its 4 faces to the fire, the 4th face being protected by the wood floor. This analysis will be carried out successively with the reduced cross-section method for Moabi and Fir, and the reduced property method only for Fir which is a softwood as prescribed by the Eurocode 5

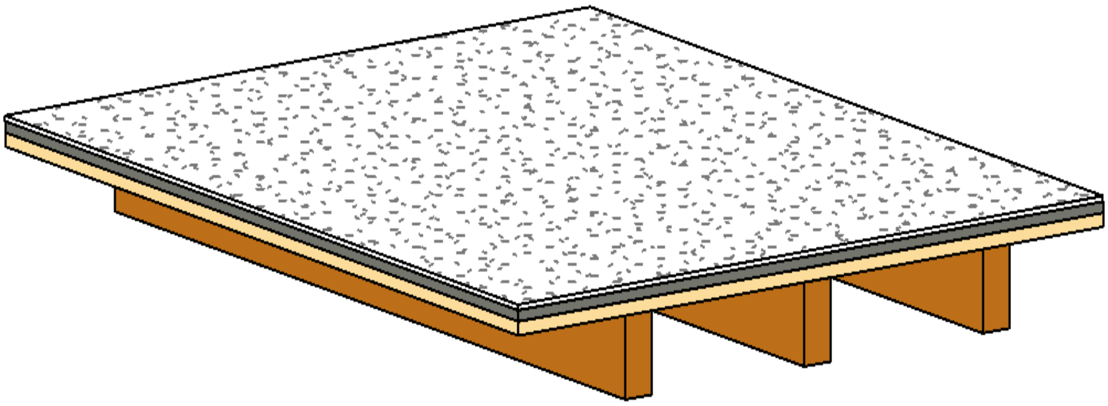


Figure 3.9. View of the unprotected joist (3 faces exposed)

i. Result of the reduced cross-section method

The Table 3.14 gives the different coefficients needed to evaluate the structural behaviour of the joist in a fire situation with the reduced cross-section method

Table 3.14. Structural coefficient used for design with reduced cross-section method

$k_{mod,fi}$ =	1,0		$\gamma_{M,fi}$ =	1,0	k_f =	1,5	k_{fi} =	1,25
$t_{fi,req}$ max(mm)=	**105**			**k_0 =**	1		**d_0 (mm)=**	7
Design charring rate of softwood Fir β_0(mm\min):							0,8	
Design charring rate of hardwood Moabi β_0(mm\min):							0,5	

- **Geometrical characteristics evolution of the unprotected 3-sides joist**

The evolution of the geometric characteristics of the joist over the time of exposure to fire by the reduced cross-section method for each of the wood species studied here are presented in Table 3.15 for Moabi and Table 3.16 for Fir.

Table 3.15. Evolution of geometrical characteristics of the unprotected Moabi-joist over time

Geometrical characteristics							
Time (min)	**d_{char} (mm)**	**d_{ef} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,X}$ (mm^4)**	**V (mm)**	**W (mm^3)**
0	0,0	0,0	120	200	80000000,00	100,00	800000,00
5	2,5	9,5	101	190,5	58186879,59	95,25	610885,88
10	5,0	12,0	96	188,0	53157376,00	94,00	565504,00

15	7,5	14,5	91	185,5	48405185,43	92,75	521888,79
20	10,0	17,0	86	183,0	43920823,50	91,50	480009,00
25	12,5	19,5	81	180,5	39694962,09	90,25	439833,38
30	15,0	22,0	76	178,0	35718429,33	89,00	401330,67
35	17,5	24,5	71	175,5	31982209,59	87,75	364469,63
40	20,0	27,0	66	173,0	28477443,50	86,50	329219,00
45	22,5	29,5	61	170,5	25195427,93	85,25	295547,54
50	25,0	32,0	56	168,0	22127616,00	84,00	263424,00
55	27,5	34,5	51	165,5	19265617,09	82,75	232817,13
60	30,0	37,0	46	163,0	16601196,83	81,50	203695,67
65	32,5	39,5	41	160,5	14126277,09	80,25	176028,38
70	35,0	42,0	36	158,0	11832936,00	79,00	149784,00
75	37,5	44,5	31	155,5	9713407,93	77,75	124931,29
80	40,0	47,0	26	153,0	7760083,50	76,50	101439,00
85	42,5	49,5	21	150,5	5965509,59	75,25	79275,88
90	45,0	52,0	16	148,0	4322389,33	74,00	58410,67
95	47,5	54,5	11	145,5	2823582,09	72,75	38812,13
100	50,0	57,0	6	143,0	1462103,50	71,50	20449,00
105	52,5	59,5	1	140,5	231125,43	70,25	3290,04

Table 3.16. Evolution of geometrical characteristics of the unprotected Fir-joist over time

Geometrical characteristics							
Time (min)	**d_{char} (mm)**	**d_{ef} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,X}$ (mm^4)**	**V (mm)**	**W (mm^3)**
0	0,0	0,0	120	200	80000000,00	100,00	800000,00
5	4,0	11,0	98	189,0	55135363,50	94,50	583443,00
10	8,0	15,0	90	185,0	47487187,50	92,50	513375,00
15	12,0	19,0	82	181,0	40519896,83	90,50	447733,67
20	16,0	23,0	74	177,0	34195603,50	88,50	386391,00
25	20,0	27,0	66	173,0	28477443,50	86,50	329219,00
30	24,0	31,0	58	169,0	23329576,83	84,50	276089,67
35	28,0	35,0	50	165,0	18717187,50	82,50	226875,00

40	32,0	39,0	42	161,0	14606483,50	80,50	181447,00
45	36,0	43,0	34	157,0	10964696,83	78,50	139677,67
50	40,0	47,0	26	153,0	7760083,50	76,50	101439,00
55	44,0	51,0	18	149,0	4961923,50	74,50	66603,00
60	48,0	55,0	10	145,0	2540520,83	72,50	35041,67
65	52,0	59,0	2	141,0	467203,50	70,50	6627,00

- **Bending behaviour of the unprotect joist**

The evolution of the bending stress and the bending strength of the joist by the reduced cross-section method for Moabi and Fir are presented in Figure 3.10 and Figure 3.9 respectively.

It was found that the joist bending resistance for Moabi species was no longer verified after **97 minutes (R97)** of exposure to fire, compared to **57 minutes (R57)** for Fir; this makes Moabi a more fire-resistant species than Fir in the unprotected joist bending analysis with reduced cross-section method.

It should be noted that the bending strength design resistance are constant with the reduced cross-section method due to the fact that the modification coefficient (k_{mod}) does not vary; it is constant: 1.0.

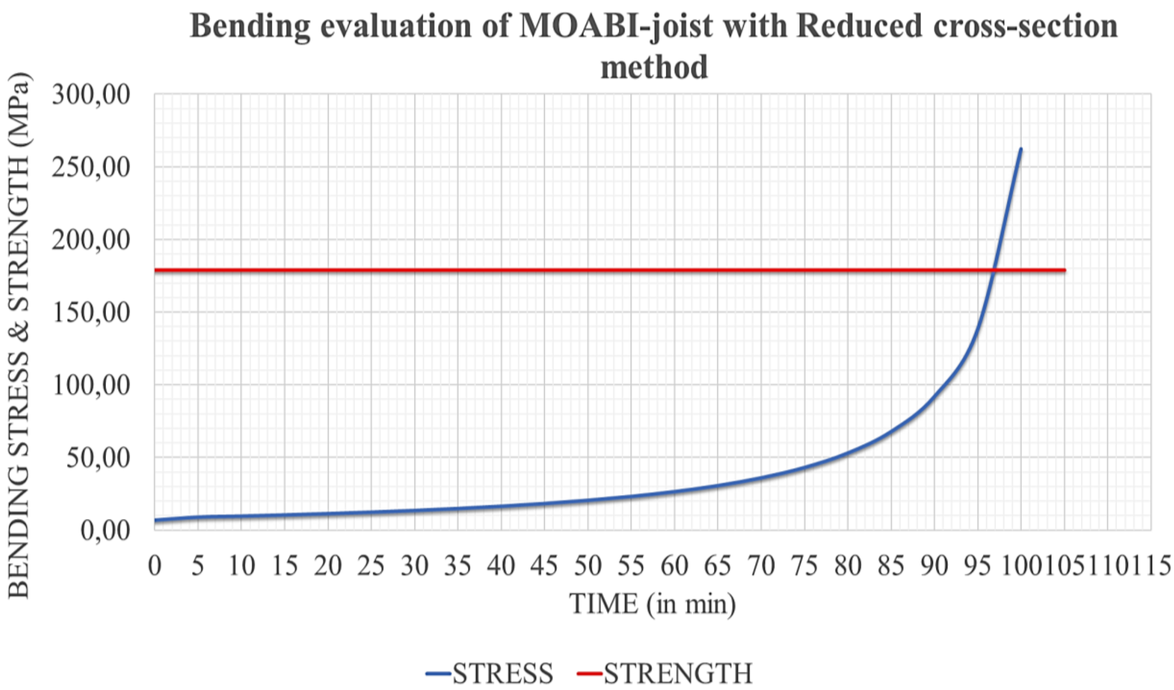


Figure 3.10. Evaluating curve of the bending behaviour of the unprotected Moabi joist over time

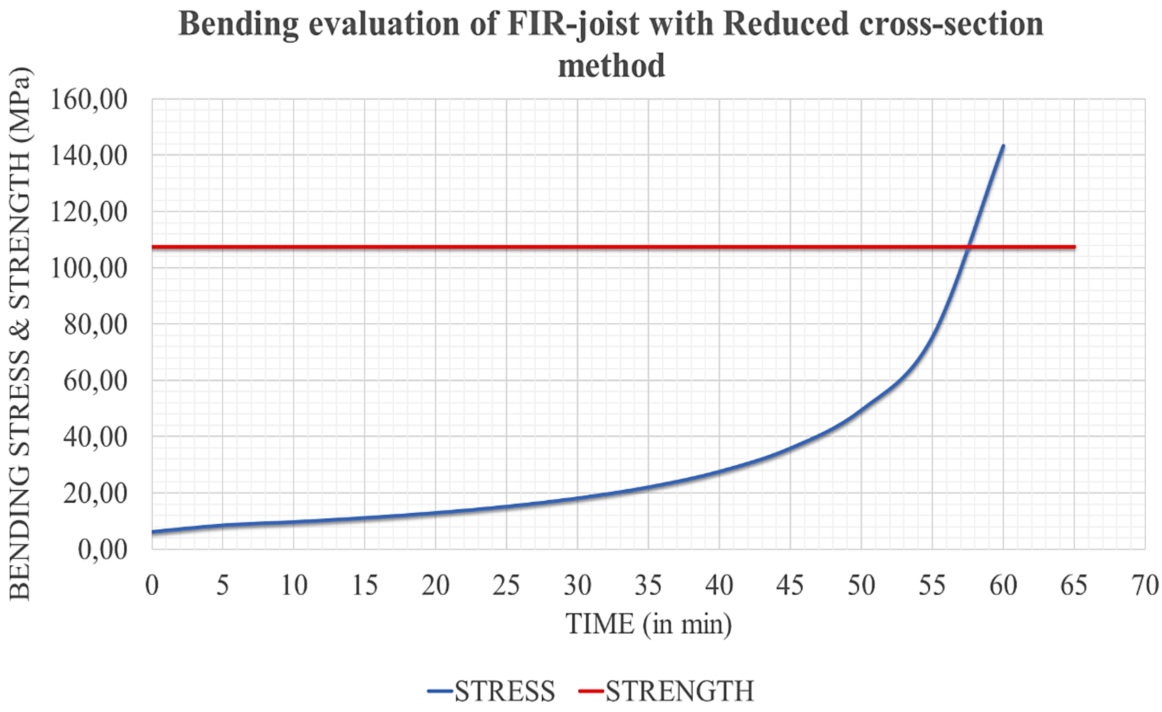


Figure 3.11. Evaluating curve of the bending behaviour of the unprotected Fir joist over time

- **Shear behaviour of the unprotect joist**

The evolution of the shear stress and the shear strength of the joist by the reduced cross-section method for Moabi and Fir are presented in Figure 3.12 and Figure 3.13 respectively.

It was found that the joist shear resistance for Moabi species was no longer verified after **101 minutes (R101)** of exposure to fire, compared to **61 minutes (R61)** for Fir; this makes Moabi a more fire-resistant species than Fir in the unprotected joist shear analysis with reduced cross-section method.

It should be noted that like the bending strength design, the shear strength design is constant with the reduced cross-section method due to the fact that the modification coefficient (k_{mod}) does not vary; it is constant: 1.0.

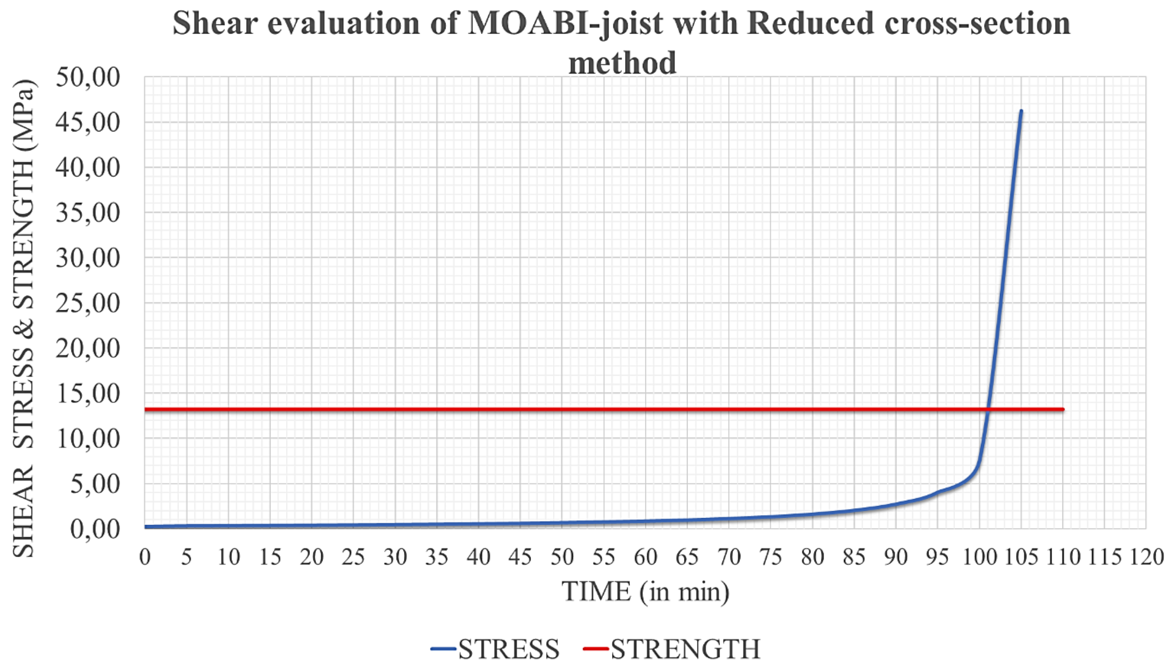


Figure 3.12. Evaluating curve of the shear behaviour of the unprotected Moabi joist over time

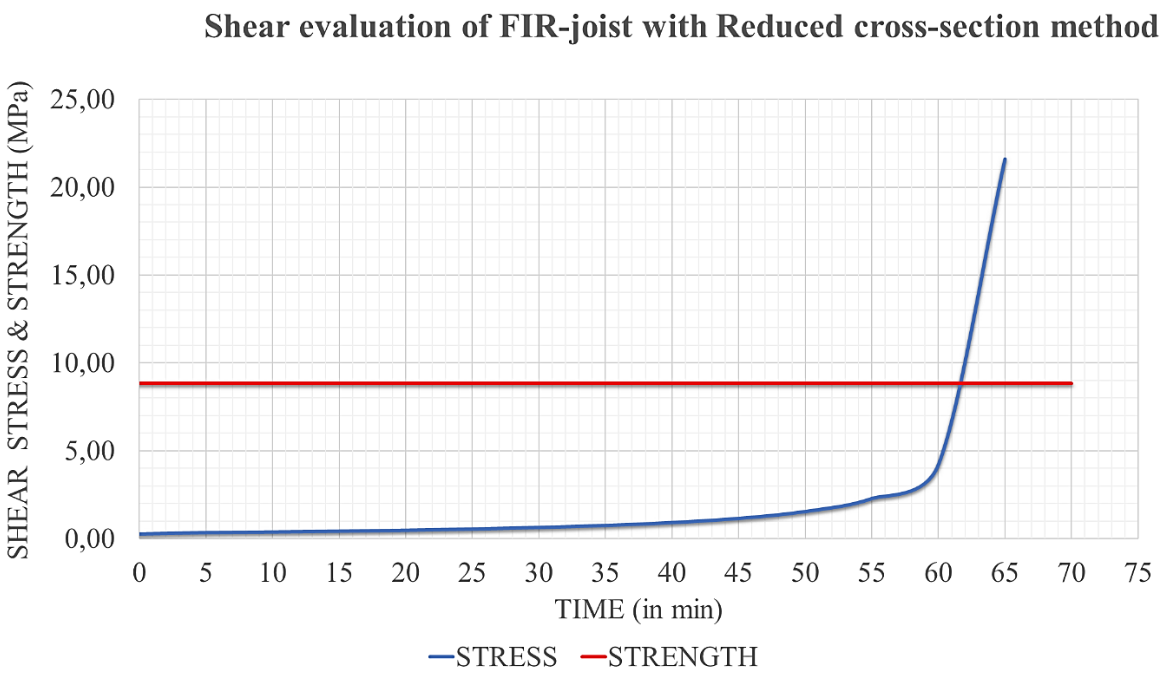


Figure 3.13. Evaluating curve of the shear behaviour of the unprotected Fir joist over time

- **Deflexion behaviour of an unprotect joist under fire condition**

The evolution of the deflection of the joist during the time of exposure to fire compared to the limit deflection prescribed by Eurocode 5: part 1.1 with the reduced cross-section method for Moabi and Fir are presented in Figure 3.14 and Figure 3.15 respectively.

It was found that the joist deflexion for Moabi specie was no longer verified after **52 minutes (R52)** of exposure to fire, compared to **06 minutes (R06)** for Fir; this makes Moabi a more fire-resistant species than Fir in the unprotected joist bending analysis with reduced cross-section method.

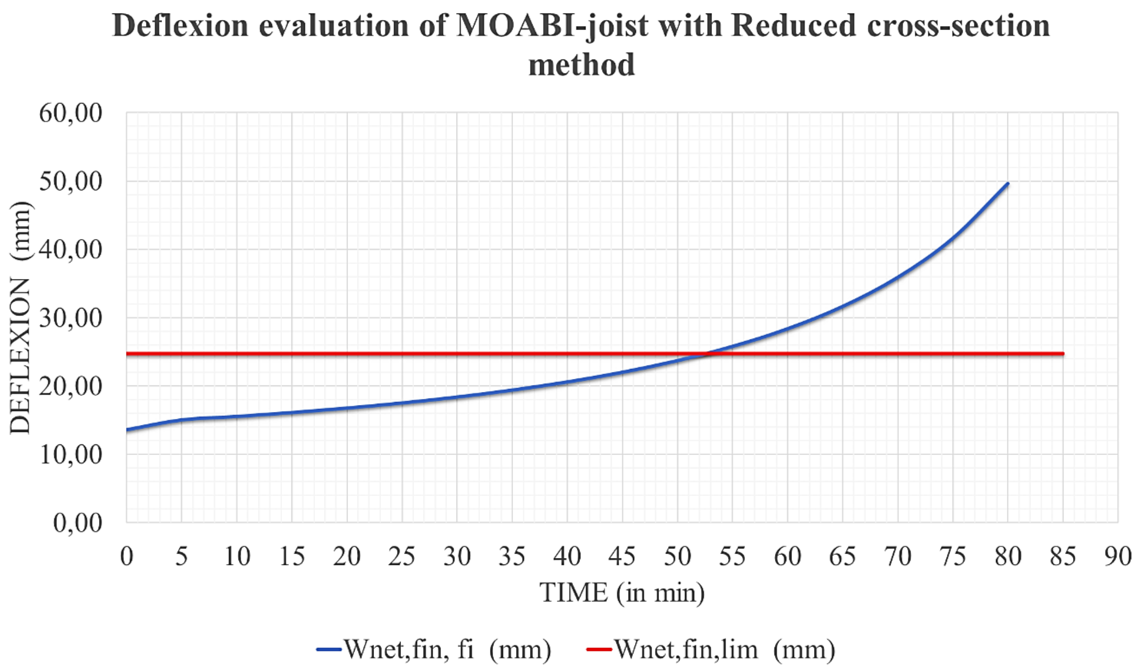


Figure 3.14. Evaluating curve of the deflexion behaviour of the unprotected Moabi joist over time

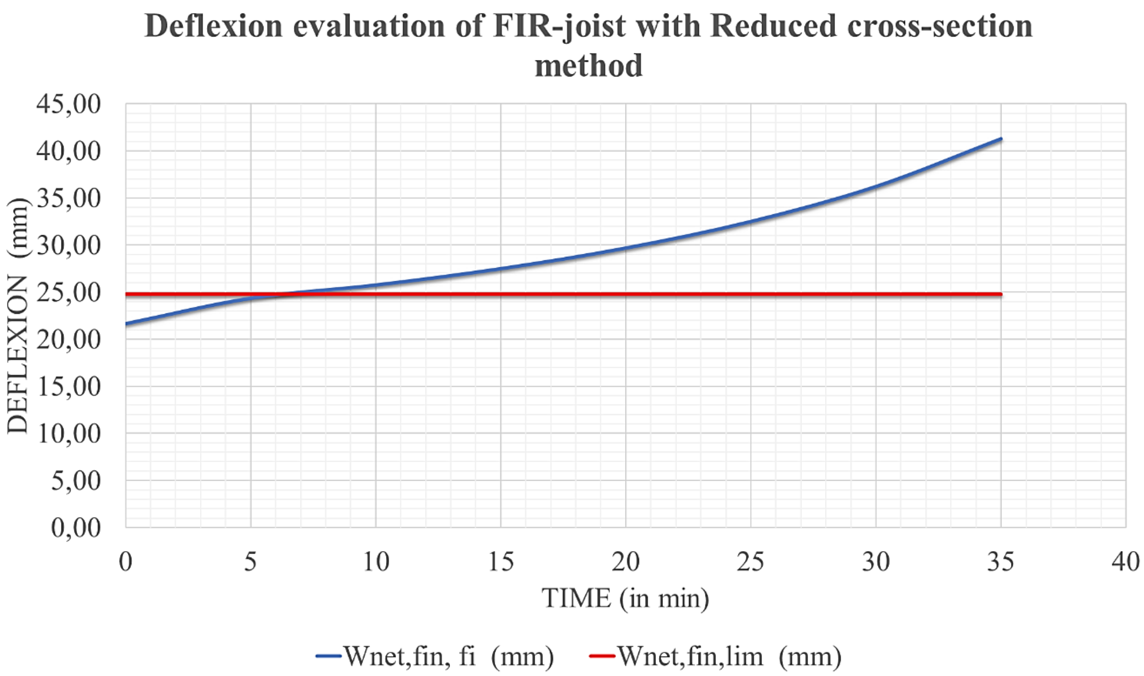


Figure 3.15. Evaluating curve of the deflexion behaviour of the unprotected Fir joist over time

ii. Result of the reduced properties method

The Table 3.17 gives the different coefficients needed to evaluate the structural behaviour of the joist in a fire situation with the reduced properties method

Table 3.17. Structural coefficient used for design with reduced properties method

$k_{mod,fi}$ (t=0) =	1,0	**$\gamma_{M,fi}$ =**	1,0	**k_f =**	1,5	**k_{fi} =**	1,25
$t_{fi,req}$ max(mm)=	**105**		**k_0 =**	0		**d_0 (mm)=**	0
Design charring rate of softwood Fir β_0(mm\min):						0,8	

- **Geometrical characteristics of the unprotected 3-sides joist, $K_{mod,fi}$ and $K_{mod,fi,E}$**

The evolution of the geometric characteristics of the joist over the time of exposure to fire by the reduced properties method for Fir species is presented in Table 3.18.

Table 3.18. Evolution of geometrical characteristics of the unprotected Fir-joist, $K_{mod,fi}$ and $K_{mod,E,fi}$

Geometrical characteristics								
Temps (min)	**d_{char} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,x}$ (mm^4)**	**V (mm)**	**W (mm^3)**	**$K_{mod,fi}$**	**$K_{mod,E,fi}$**
0	0,0	120	200	80000000,00	100,00	800000,00	1,00	1,00
5	4,0	112	196,0	70275669,33	98,00	717098,67	0,86	0,91
10	8,0	104	192,0	61341696,00	96,00	638976,00	0,85	0,91
15	12,0	96	188,0	53157376,00	94,00	565504,00	0,84	0,90
20	16,0	88	184,0	45683029,33	92,00	496554,67	0,83	0,90
25	20,0	80	180,0	38880000,00	90,00	432000,00	0,82	0,89
30	24,0	72	176,0	32710656,00	88,00	371712,00	0,80	0,88
35	28,0	64	172,0	27138389,33	86,00	315562,67	0,79	0,87
40	32,0	56	168,0	22127616,00	84,00	263424,00	0,76	0,86
45	36,0	48	164,0	17643776,00	82,00	215168,00	0,73	0,84
50	40,0	40	160,0	13653333,33	80,00	170666,67	0,69	0,81
55	44,0	32	156,0	10123776,00	78,00	129792,00	0,62	0,77
60	48,0	24	152,0	7023616,00	76,00	92416,00	0,52	0,71

65	52,0	16	148,0	4322389,33	74,00	58410,67	0,31	0,58
70	56,0	8	144,0	1990656,00	72,00	27648,00	-0,32	0,20

- **Bending behaviour of the unprotect Fir joist**

The evolution of the bending stress and the bending strength of the joist by the reduced properties method for Fir are presented in Figure 3.16.

It was found that the bending resistance for Fir-joist was no longer verified after **60 minutes (R60)** of exposure to fire with this method.

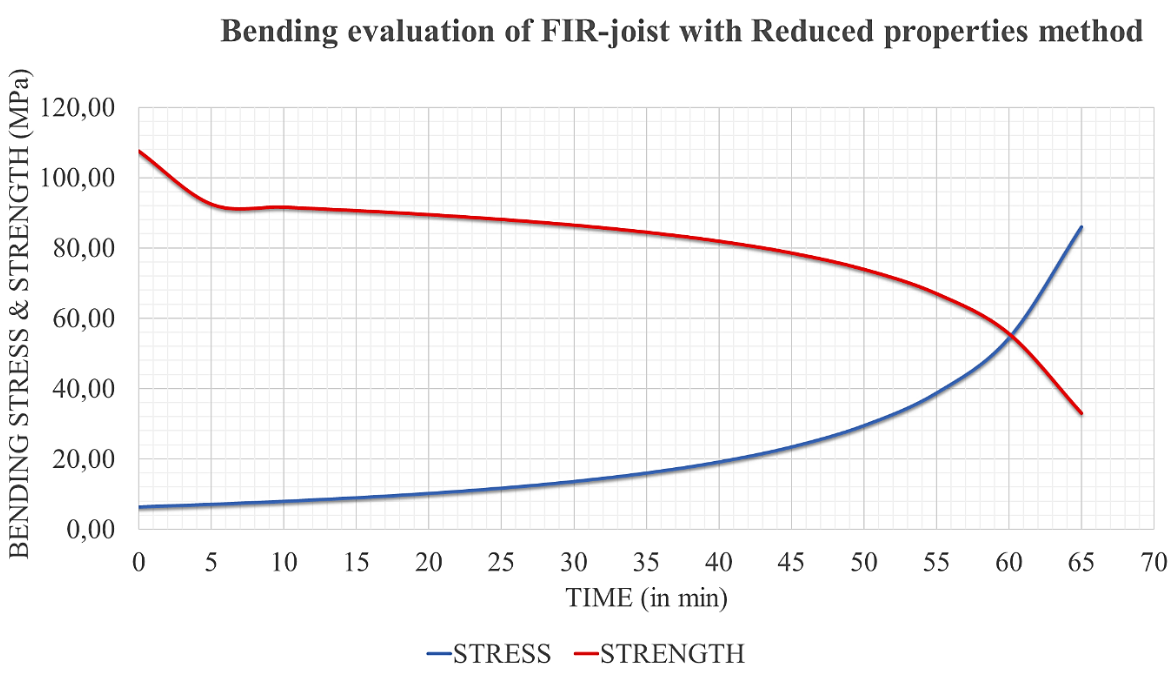


Figure 3.16. Evaluating curve of the bending behaviour of the unprotected Fir joist over time

- **Shear verification of the unprotect Fir joist**

The evolution of the shear stress and the shear strength of the joist by the reduced properties method for Fir are presented in Figure 3.17.

It was found that the shear resistance for Fir-joist was no longer verified after 65 minutes (R65) of exposure to fire with this method.

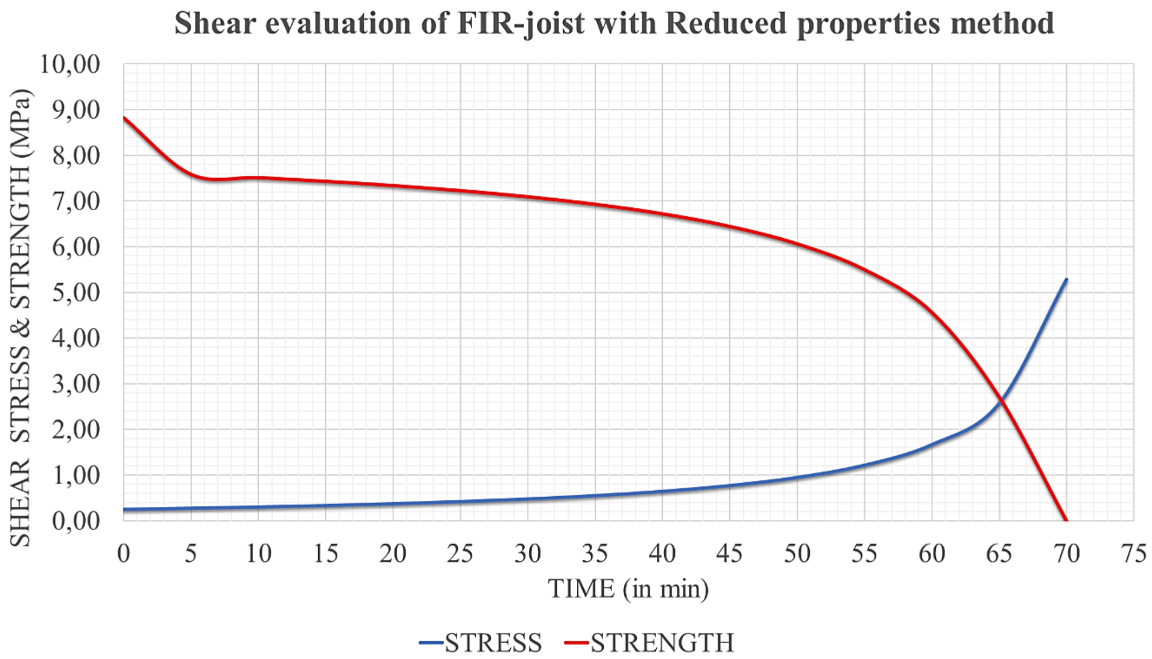


Figure 3.17. Evaluating curve of the shear behaviour of the unprotected Fir joist over time

- **Deflexion verification of the unprotect joist**

The evolution of the deflection of the Fir joist during the time of exposure to fire compared to the limit deflection prescribed by Eurocode 5: part 1.1 with the reduced properties method is presented in Figure 3.18.

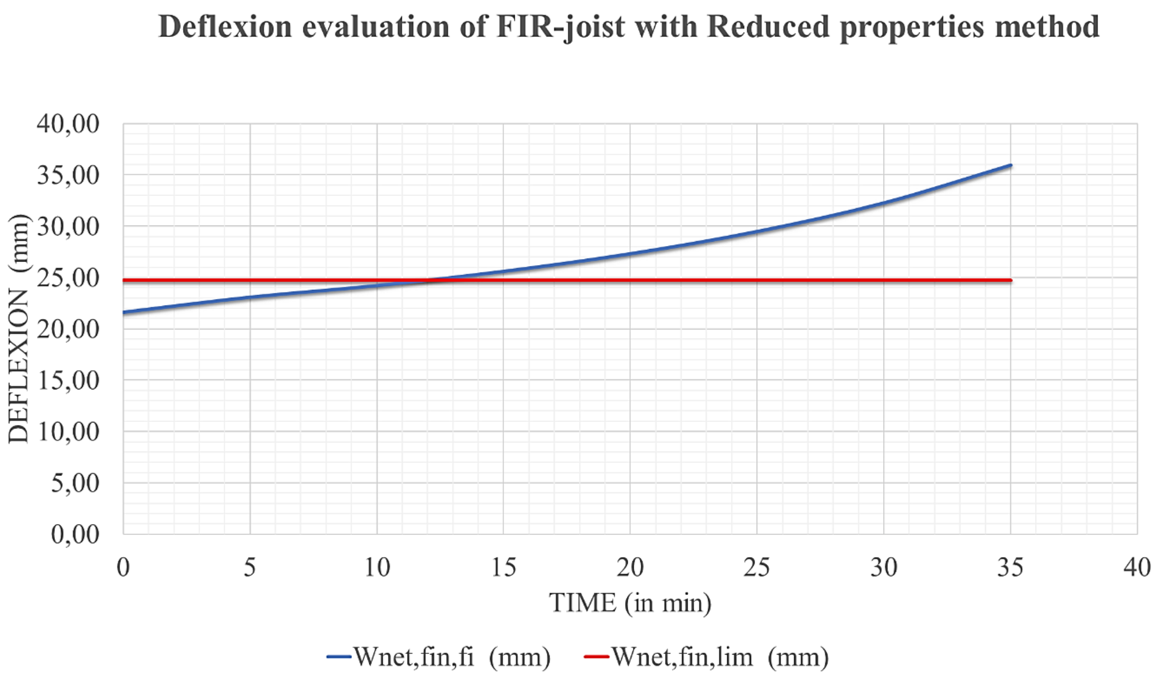


Figure 3.18. Evaluating curve of the deflexion behaviour of the unprotected Fir joist over time

It was found that the joist deflexion for Fir-joist was no longer verified after **12 minutes (R12)** of exposure to fire with this method.

iii. Synthesis of the analysis without fire protection elements on joist (3 exposed sides)

The purpose of this part is to gather all the results previously obtained (those of the fire analysis of a joist exposed on 3 faces) in the form of a histogram in order to better perceive the different parameters that could influence the fire resistance of wooden structures with the particular case of the joist studied here

- **Influence of the wood species on the structural response of the joist**

Figures 3.18, 3.19 and 3.20 highlight, for the particular case studied in this work, the influence of the choice of wood species on the resistance capacity of wooden structures in a fire situation by comparing the fire resistance times of Moabi and Fir through the joist studied here.

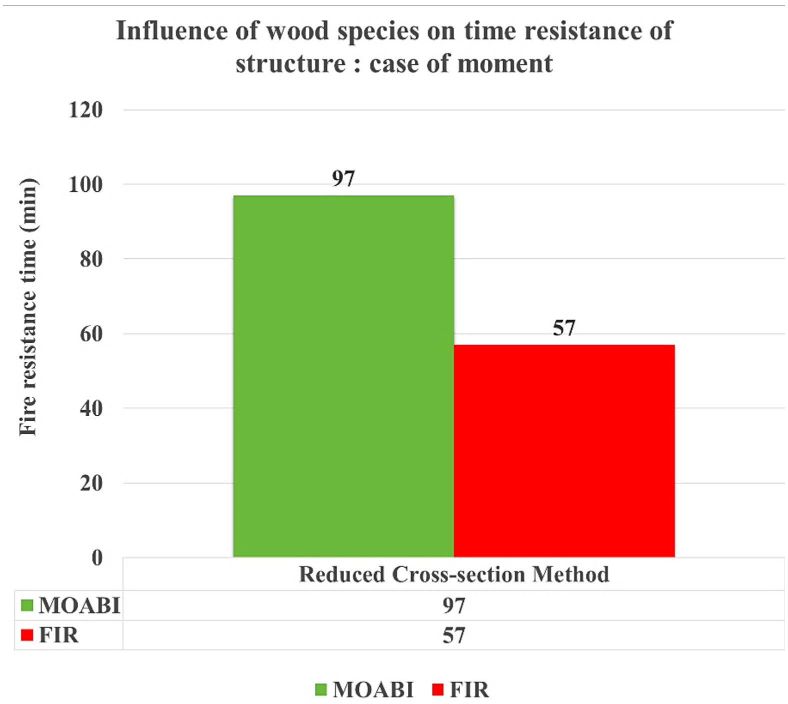


Figure 3.19. Result of the reduced cross-section method applied to the moment

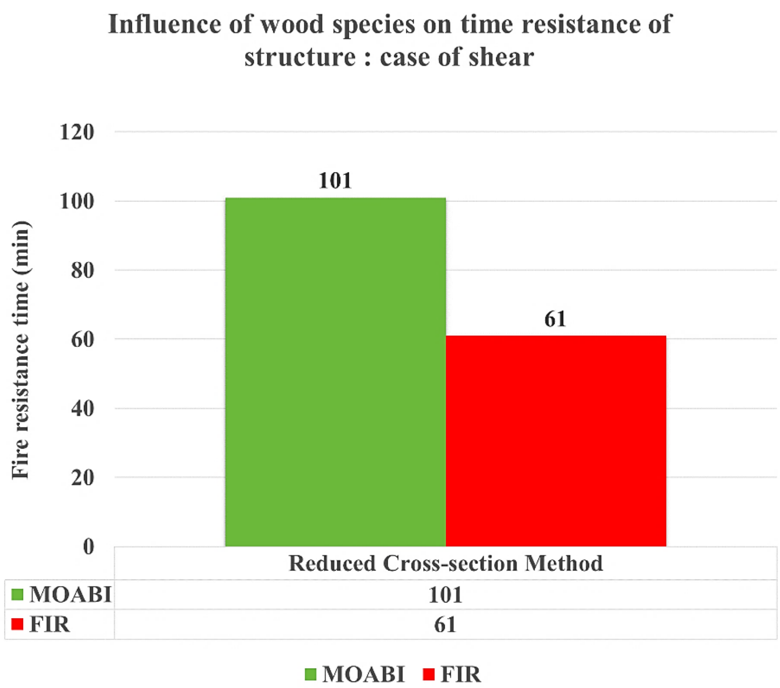


Figure 3.20. Result of the reduced cross-section method applied to the shear

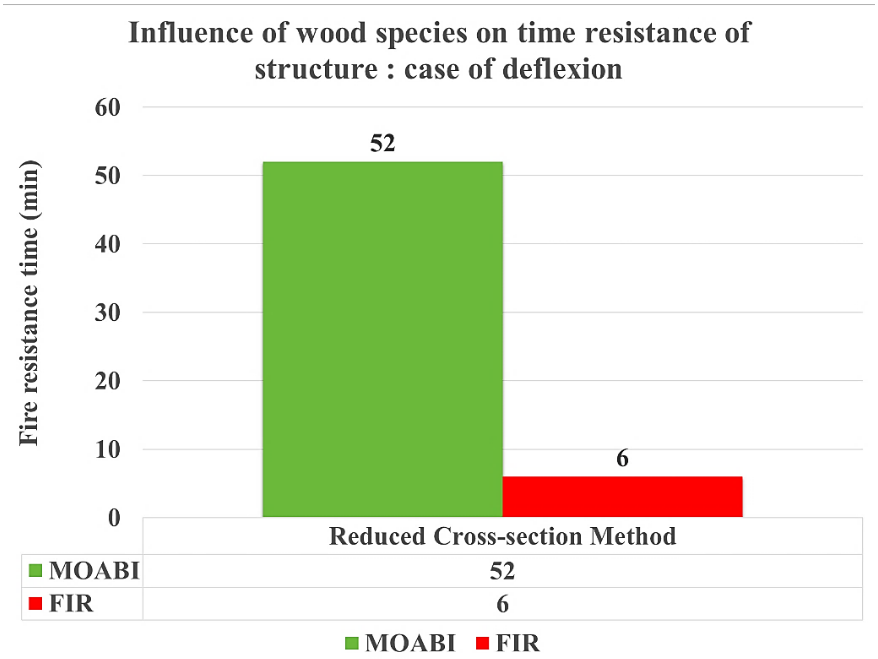


Figure 3.21. Result of the reduced cross-section method applied to the deflection

The analysis of figures 3.19, 3.20 and 3.21 shows that Moabi is better than Fir in all respects; Indeed, within the limits of this case study, and as presented in figure 3.21, Moabi performs **1,70** times better than Fir in joist bending strength (**97 minutes against 57**), **1,66** times better in joist shear strength (**101 minutes against 61**), and **8,67** times better in joist deflection check (**52 minutes against 06**).

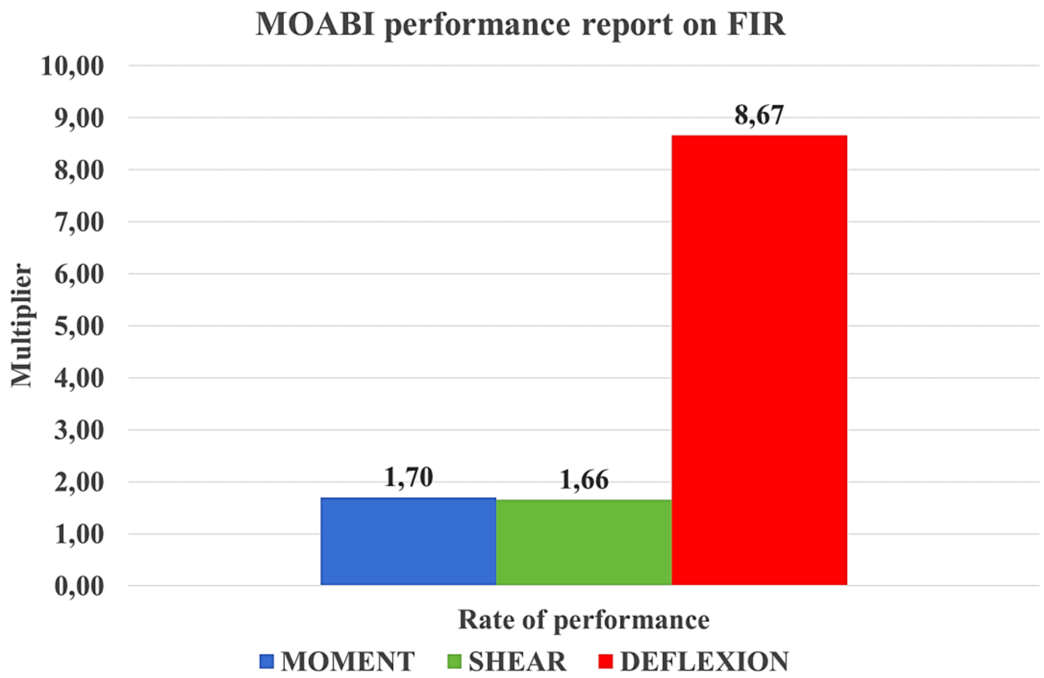


Figure 3.22. Performance ratio of the Moabi compared to the Fir based on the effective cross-section method

- **Influence of the choice of design method on structural behaviour response of the Fir joist**

The influence of the choice of the method studied here (reduced cross-section and reduced properties) on the behaviour of the structure exposed to fire will be discussed in section 3.2.5 of this part.

b. Case of fire protection elements using to protect joist (2 protected faces on 3)

This section is reserved for the analysis of the joist protected on 2 of its 3 previous exposed faces to the fire. This analysis will be carried out successively with the reduced cross-section method and the reduced property method only for one species (Fir in this case) and protection will be provided by the species that gives the greatest time of protection against fire between Moabi and Fir.

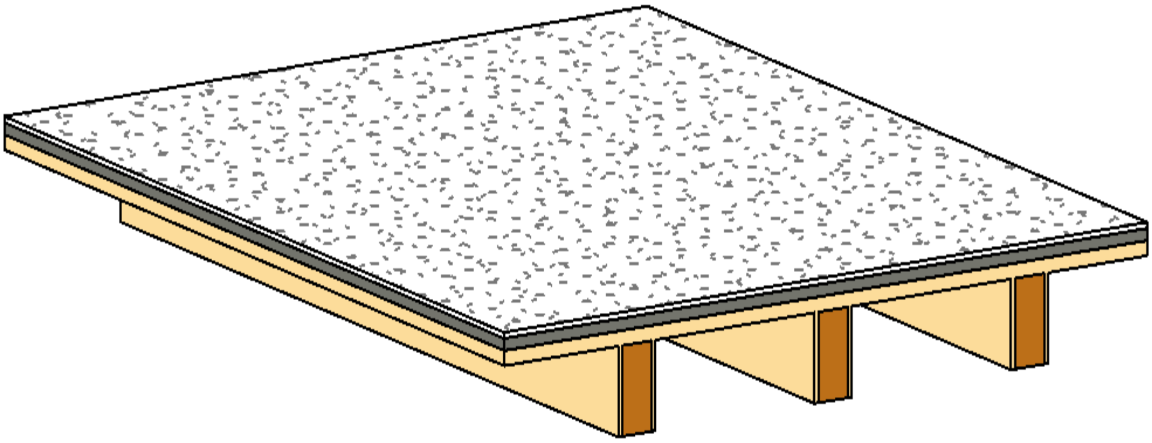


Figure 3.23. View of protected joist (2 protected faces of the 3 unprotected)

i. Result of choice of species of wood for protection

The Table 3.19 presents the analysis of the protection time provided by each of the 2 species in order to know which one will be selected to protect our fir joist against fire; the result thus obtained is illustrated by figure 3.24.

Table 3.19. Choice of wood species for protection

Species	**Maobi**	**Fir**
Thickness (mm)	20	20
Density (kg\m^3)	870 > **450 kg\m^3**	450 = **450 kg\m^3**
Charring rate β_0 (mm\min)	0,65	0,9
Protection time t_f (min)	**26**	**18**

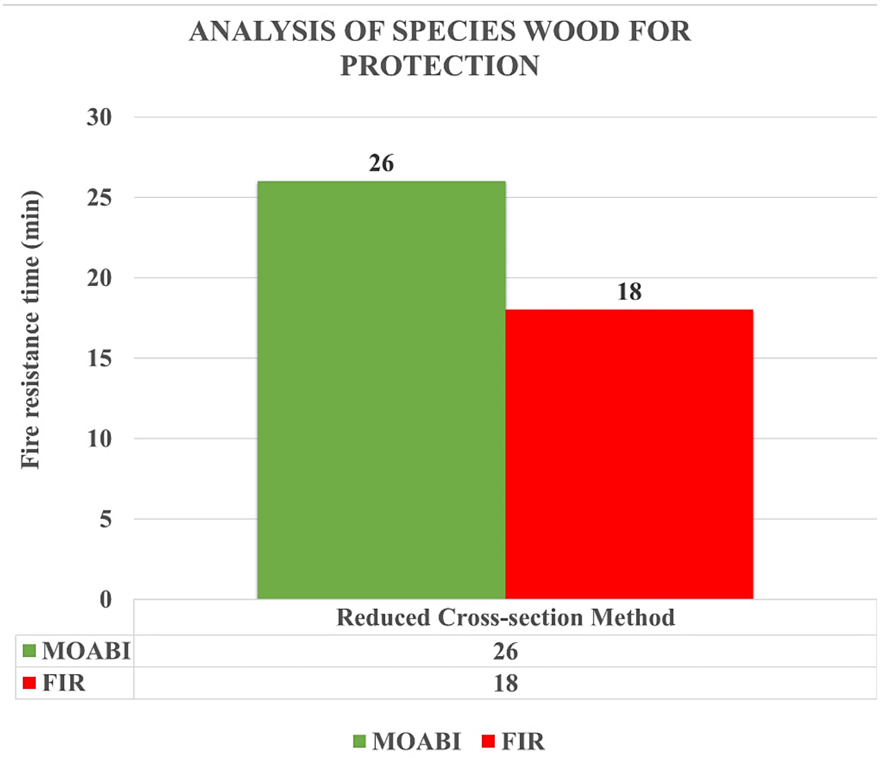


Figure 3.24. Time diagram of analysis of species wood for protection

It emerges from this analysis that Moabi will be used as a protection species for the joist to the detriment of Fir, as it has a protection time of 26 minutes compared to 18 minutes for Fir

ii. Result of the reduced cross-section method

The Table 3.20 gives the different coefficients needed to evaluate the structural behaviour of the partial protected Fir-joist in a fire situation with the reduced cross-section method.

Table 3.20. Structural coefficient used for design with reduced cross-section method

$k_{mod,fi}$ =	1,0		$\gamma_{M,fi}$ =	1,0	k_f =	1,5	k_{fi} =	1,25
$t_{fi,req,max}$ (mm)=	105			k_0 =	1		d_0 (mm)=	7
Design charring rate of softwood Fir β_0(mm\mim):							0,8	

- **Geometrical characteristics of the partial fire protect joist**

The evolution of the geometric characteristics of the partial fire protected Fir-joist over the time of exposure to fire by the reduced cross-section method is showed Table 3.21.

Table 3.21. Evolution of geometrical characteristics of partial protected Fir-joist with time fire exposure

Geometrical characteristics							
Time (min)	**d_{ef-b} (mm)**	**d_{ef-h} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,X}$ (mm^4)**	**V (mm)**	**W (mm^3)**
0	0,0	0,0	120	200	80000000,00	100,00	800000,00
5	0,0	11,0	120	189,0	67512690,00	94,50	714420,00
10	0,0	15,0	120	185,0	63316250,00	92,50	684500,00
15	0,0	19,0	120	181,0	59297410,00	90,50	655220,00
20	0,0	23,0	120	177,0	55452330,00	88,50	626580,00
25	0,0	27,0	120	173,0	51777170,00	86,50	598580,00
30	10,2	31,0	99,6	169,0	40062514,70	84,50	474112,60
35	14,2	35,0	91,6	165,0	34289887,50	82,50	415635,00
40	18,2	39,0	83,6	161,0	29073857,63	80,50	361165,93
45	22,2	43,0	75,6	157,0	24380325,90	78,50	310577,40
50	26,2	47,0	67,6	153,0	20176217,10	76,50	263741,40
55	30,2	51,0	59,6	149,0	16429480,03	74,50	220529,93
60	34,2	55,0	51,6	145,0	13109087,50	72,50	180815,00
65	38,2	59,0	43,6	141,0	10185036,30	70,50	144468,60
70	42,2	63,0	35,6	137,0	7628347,23	68,50	111362,73
75	46,2	67,0	27,6	133,0	5411065,10	66,50	81369,40
80	50,2	71,0	19,6	129,0	3506258,70	64,50	54360,60
85	54,2	75,0	11,6	125,0	1888020,83	62,50	30208,33
90	58,2	79,0	3,6	121,0	531468,30	60,50	8784,60

- **Bending behaviour of the partial fire protected Fir-joist**

The evolution of the bending stress and the bending strength of the partial protected Fir-joist by the reduced cross-section method is presented in Figure 3.25.

It was found that the partial protected Fir-joist bending resistance was no longer verified after **81 minutes (R81)** of exposure to fire.

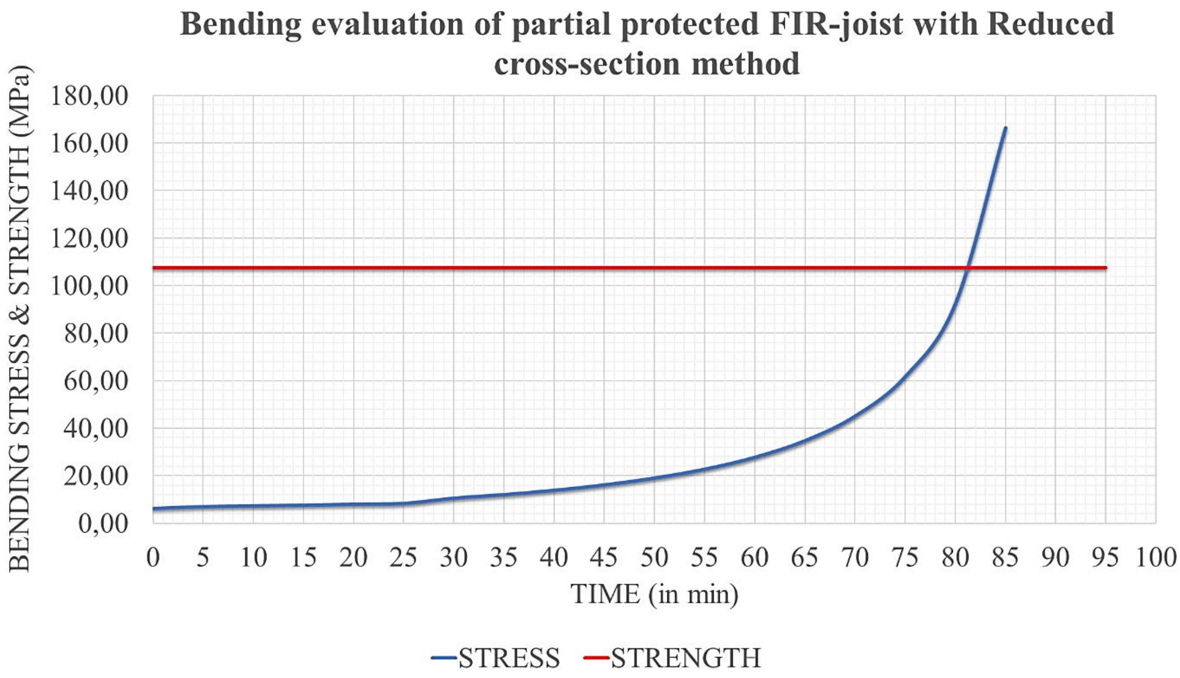


Figure 3.25. Evaluating curve of the bending behaviour of partial protected Fir joist over time

It should be noted that the bending strength design resistance are constant with the reduced cross-section method due to the fact that the modification coefficient (k_{mod}) does not vary; it is constant: 1.0.

- **Shear behaviour of the partial fire protected Fir-joist**

The evolution of the shear stress and the shear strength of the partial protected Fir-joist by the reduced cross-section method is presented in Figure 3.26.

It was found that the partial protected Fir-joist bending resistance was no longer verified after **87 minutes (R87)** of exposure to fire.

It should be noted that like the bending strength design, the shear strength design is constant with the reduced cross-section method due to the fact that the modification coefficient (kmod) does not vary; it is constant: 1.0

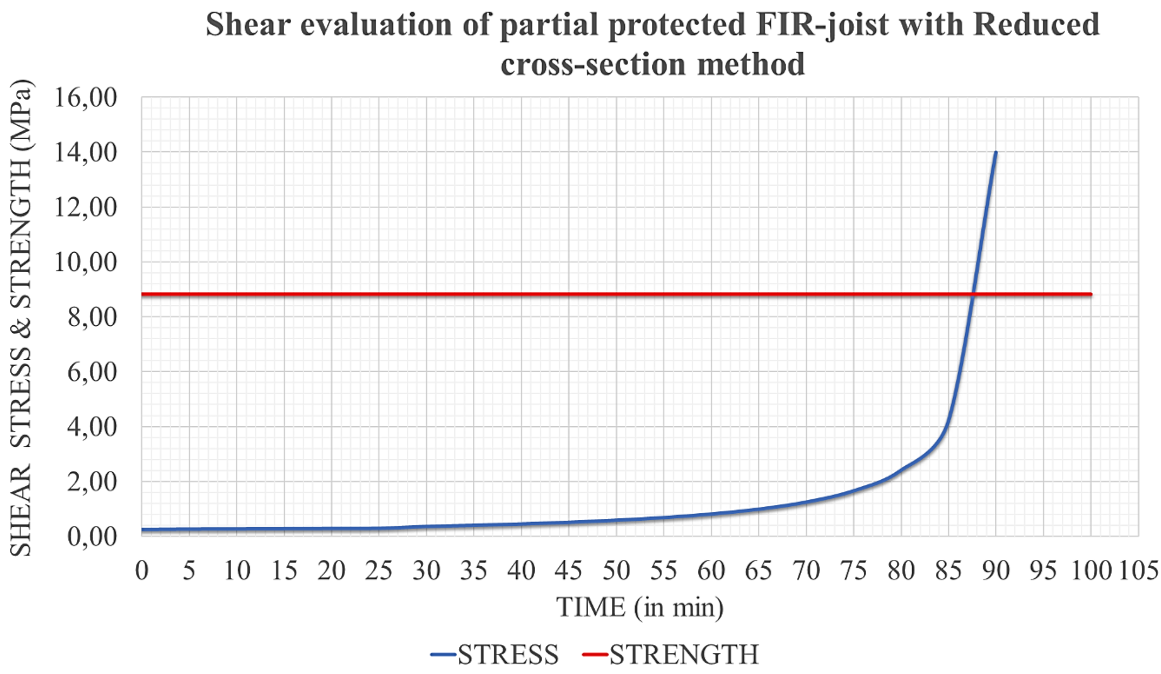


Figure 3.26. Evaluating curve of the shear behaviour of partial protected Fir joist over time

- **Deflexion behaviour of a partial fire protected Fir-joist**

The evolution of the deflection of the partial protected Fir-joist during the time of exposure to fire compared to the limit deflection prescribed by Eurocode 5: part 1.1 with the reduced cross-section method is presented in Figure 3.27.

It was found that the partial protected Fir-joist deflexion check was no longer verified after **24 minutes** of exposure to fire.

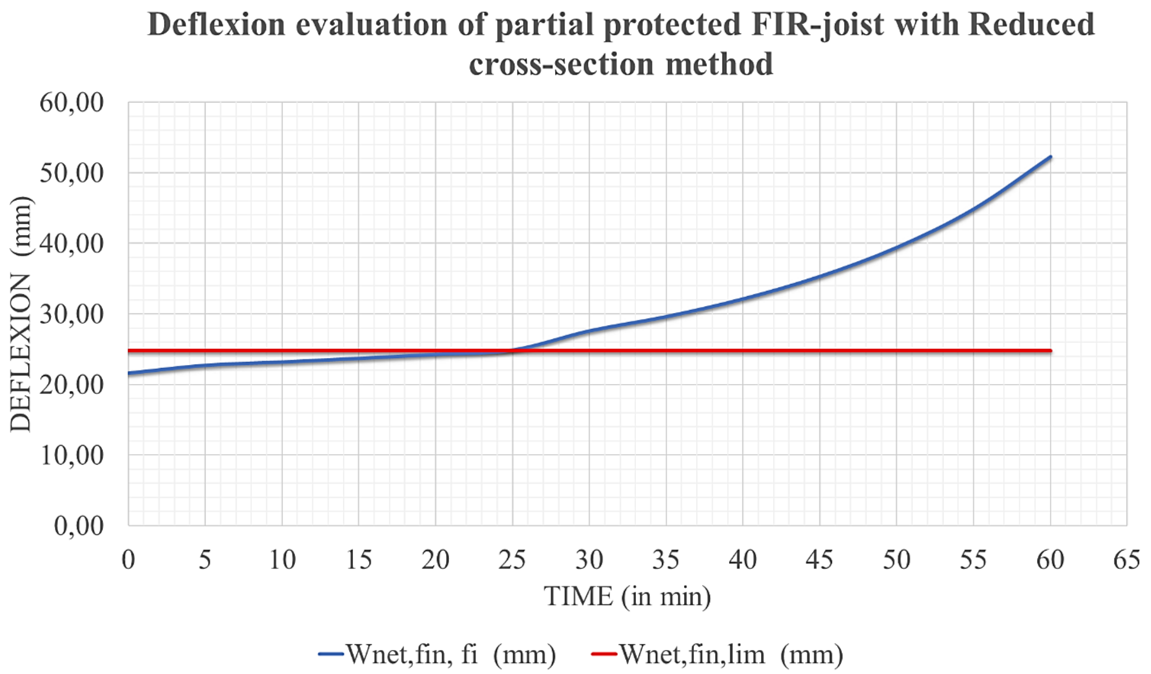


Figure 3.27. Evaluating curve of the deflexion behaviour of partial protected Fir joist over time

iii. Result of the reduced properties method

The Table 3.22 gives the different coefficients needed to evaluate the structural behaviour of the partial protected Fir-joist in a fire situation with the reduced properties method.

Table 3.22. Structural coefficient used for design with reduced properties method

$k_{mod,fi}$ (t=0) =	1,0		**$\gamma_{M,fi}$ =**	1,0	**k_f =**	1,5	**k_{fi} =**	1,25
$t_{fi,req}$ (mm)=	**105**			**k_0 =**	0		**d_0 (mm)=**	0
Design charring rate of softwood Fir β_0(mm\mim):							0,8	

- **Geometrical characteristics of the partial fire protect Fir-joist and $K_{mod,fi}$ & $K_{mod,fi,E}$**

The evolution of the geometric characteristics of the partial fire protected Fir-joist over the time of exposure to fire by the reduced properties method is showed Table 3.23.

Table 3.23. Evolution of geometrical characteristics of partial protected joist and $K_{mod,fi}$ & $K_{mod,fi,E}$ with time fire exposure

Geometrical characteristics									
Time (min)	**d_{char-b} (mm)**	**d_{char-h} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,x}$ (mm^4)**	**V (mm)**	**W (mm^3)**	**$K_{mod,fi}$**	**$K_{mod,E,fi}$**
0	0,2	0,0	120	200	80000000,00	100,00	800000,00	1,00	1,00
5	0,0	4,0	120	196,0	75295360,00	98,00	768320,00	0,87	0,92
10	0,0	8,0	120	192,0	70778880,00	96,00	737280,00	0,86	0,92
15	0,0	12,0	120	188,0	66446720,00	94,00	706880,00	0,86	0,92
20	0,0	16,0	120	184,0	62295040,00	92,00	677120,00	0,86	0,92
25	0,0	20,0	120	180,0	58320000,00	90,00	648000,00	0,86	0,92
30	3,2	24,0	113,6	176,0	51610146,13	88,00	586478,93	0,86	0,91
35	7,2	28,0	105,6	172,0	44778342,40	86,00	520678,40	0,85	0,91
40	11,2	32,0	97,6	168,0	38565273,60	84,00	459110,40	0,84	0,90
45	15,2	36,0	89,6	164,0	32935048,53	82,00	401646,93	0,83	0,90
50	19,2	40,0	81,6	160,0	27852800,00	80,00	348160,00	0,81	0,89
55	23,2	44,0	73,6	156,0	23284684,80	78,00	298521,60	0,80	0,88
60	27,2	48,0	65,6	152,0	19197883,73	76,00	252603,73	0,78	0,87
65	31,2	52,0	57,6	148,0	15560601,60	74,00	210278,40	0,76	0,85
70	35,2	56,0	49,6	144,0	12342067,20	72,00	171417,60	0,73	0,84
75	39,2	60,0	41,6	140,0	9512533,33	70,00	135893,33	0,69	0,81
80	43,2	64,0	33,6	136,0	7043276,80	68,00	103577,60	0,63	0,78
85	47,2	68,0	25,6	132,0	4906598,40	66,00	74342,40	0,53	0,72
90	51,2	72,0	17,6	128,0	3075822,93	64,00	48059,73	0,35	0,61
95	55,2	76,0	9,6	124,0	1525299,20	62,00	24601,60	-0,12	0,32
100	59,2	80,0	1,6	120,0	230400,00	60,00	3840,00	-5,33	-2,84

- **Bending behaviour of the partial fire protected Fir-joist**

The evolution of the bending stress and the bending strength of the partial protected Fir-joist by the reduced property method is presented in Figure 3.28.

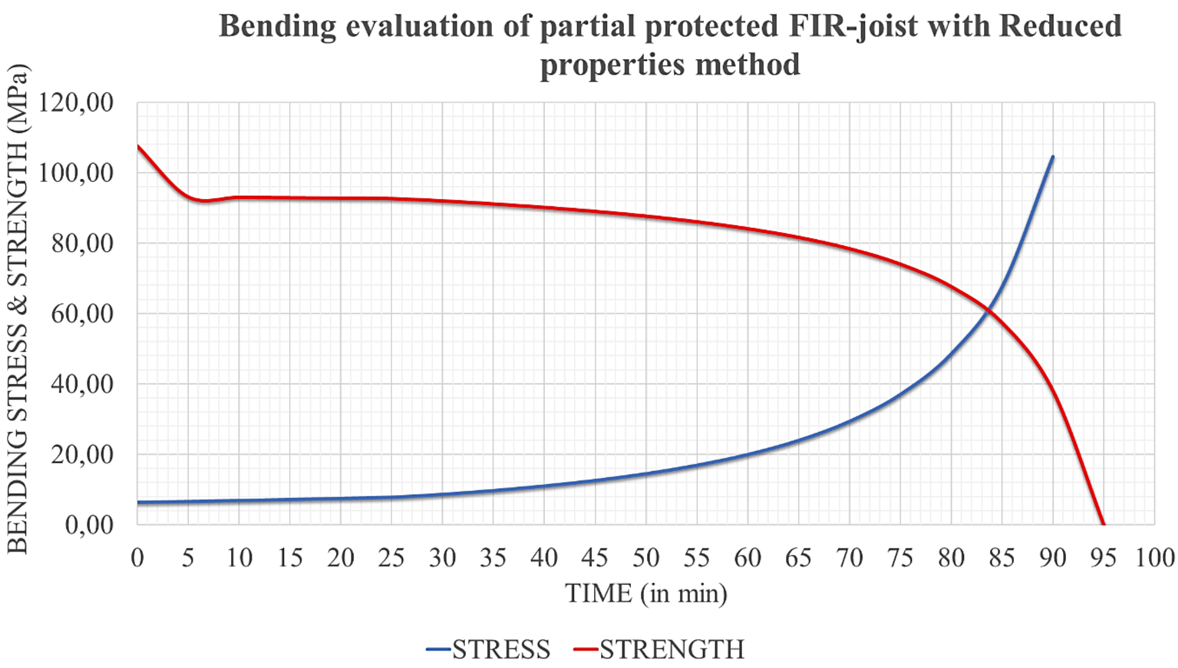


Figure 3.28. Evaluating curve of the bending behaviour of partial protected Fir joist over time

It was found that the partial protected Fir-joist bending resistance was no longer verified after **83 minutes (R83)** of exposure to fire.

- **Shear behaviour of a partial fire protected Fir-joist under**

The evolution of the shear stress and the shear strength of the partial protected Fir-joist by the reduced property method is presented in Figure 3.29.

It was found that the partial protected Fir-joist bending resistance was no longer verified after **90 minutes (R90)** of exposure to fire.

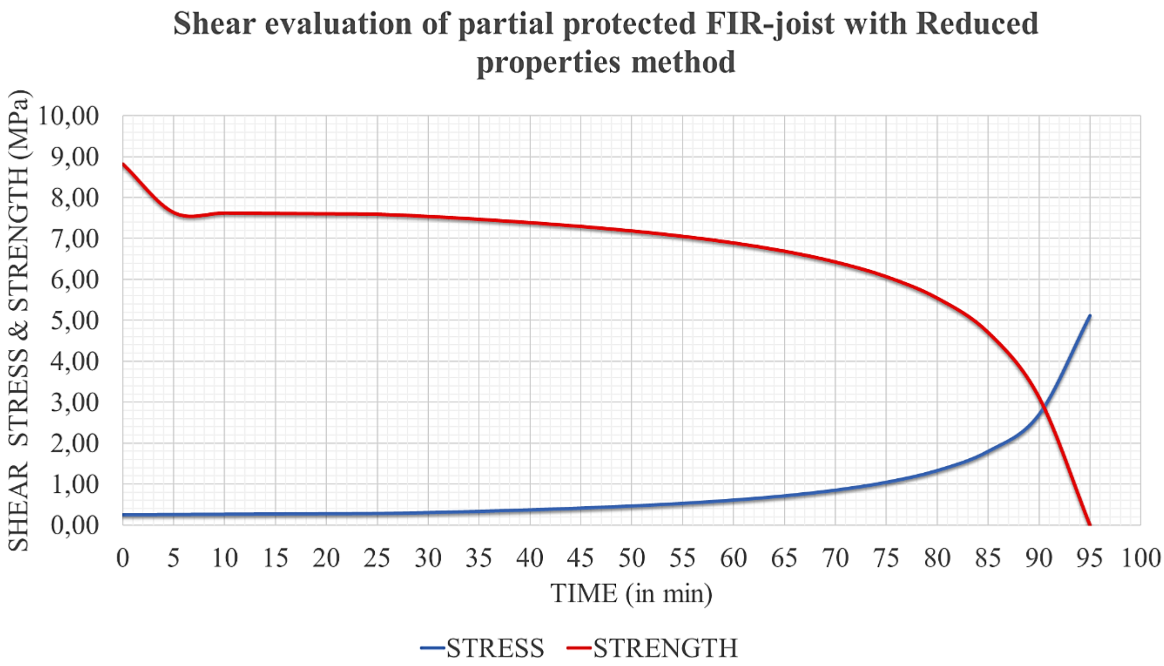


Figure 3.29. Evaluating curve of the shear behaviour of partial protected Fir joist over time

- **Deflexion behaviour of a partial fire protected Fir-joist**

The evolution of the deflection of the partial protected Fir-joist during the time of exposure to fire compared to the limit deflection prescribed by Eurocode 5: part 1.1 with the reduced properties method is presented in Figure 3.30.

It was found that the partial protected Fir-joist deflexion check was no longer verified after **25 minutes** of exposure to fire.

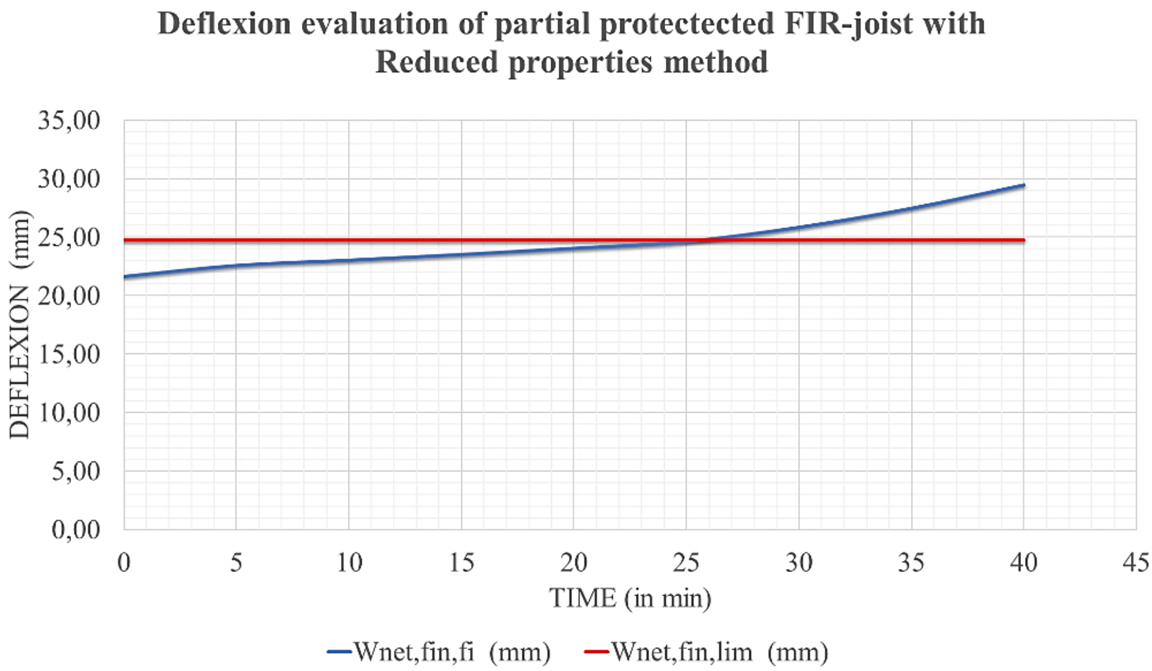


Figure 3.30. Evaluating curve of the deflexion behaviour of partial protected Fir joist over time

iv. Synthesis of the analysis of the effect of the fire protection elements on Fir-joist fire resistance (2 protected faces on 3)

The purpose of this part is to gather all the results previously obtained (those of the fire analysis of a partial protected joist on 2 sides over 3) in the form of a histogram in order to better perceive the different parameters that could influence the fire resistance of wooden structures with the particular case of the joist studied here.

Thus, Figure 3.31 and 3.30 show for the particular case studied in this work, the evolution of the fire resistance time of the fir joist under the effect of the partial protection by means of the reduced cross-section method and the reduced properties method respectively; It appears that by the reduced cross-section method the fire resistance times went from **57 minutes to 81** for bending, **61 minutes to 87** for shear and **06 minutes to 24** for deflection. The reduced properties method shows that the joist resistance increased from **60 minutes to 83** for bending, from **65 minutes to 90** for shear and from **12 minutes to 25** for deflection.

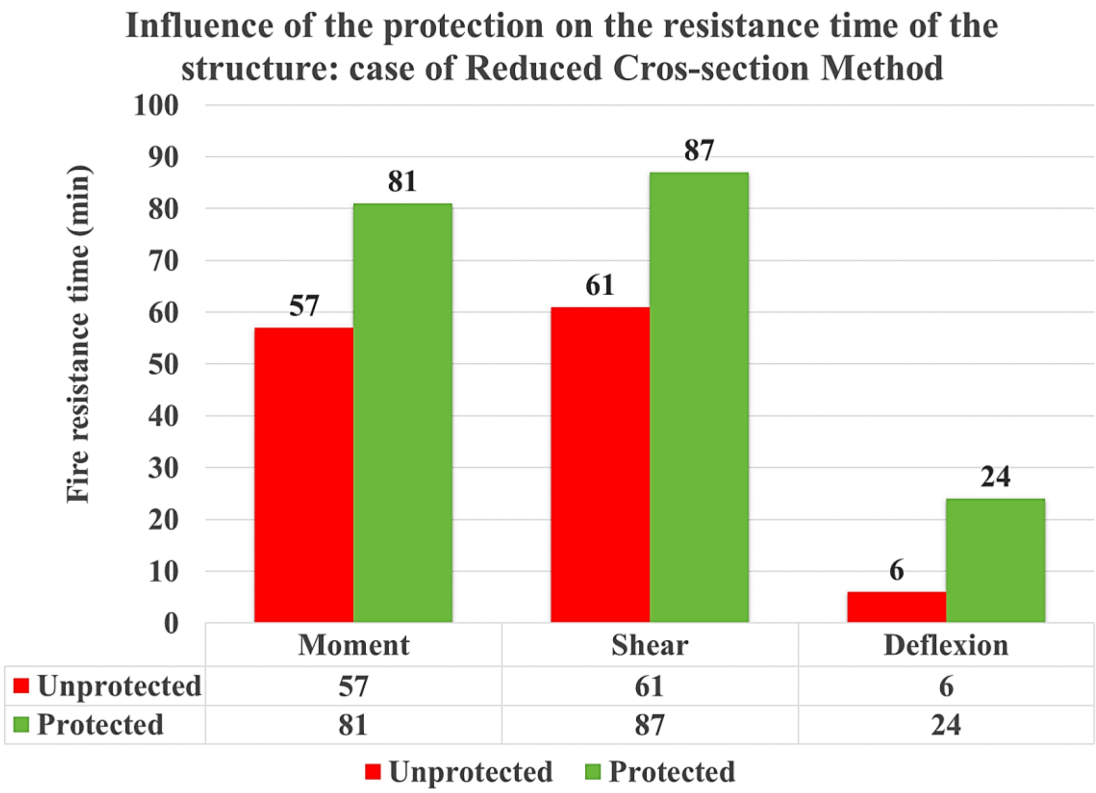


Figure 3.31. Influence of the protection on the resistance time of the structure: case of Reduced Cros-section Method

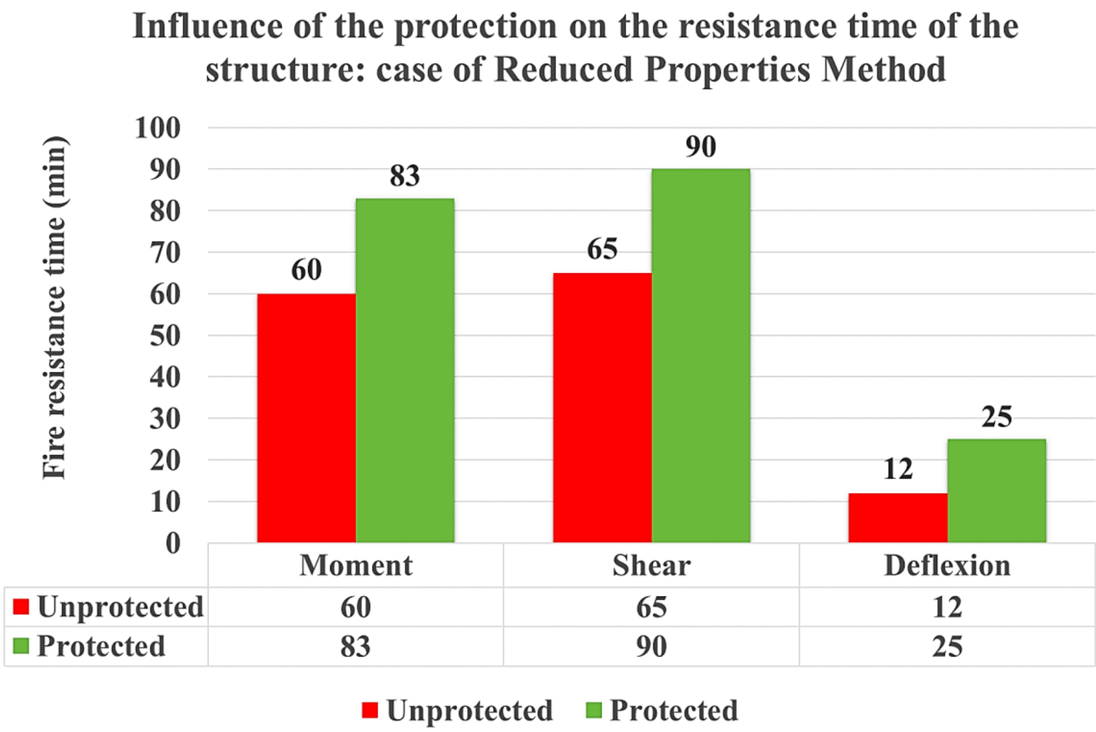


Figure 3.32. Influence of the protection on the resistance time of the structure: case of Reduced Properties Method

v. Optimisation of the choice of protection time in partial protections (Case of deflection)

The analysis of the time of protection of the protection element in a partial protection of the Fir-joist compared to the time of failure after protection has generated the curve showed in Figure 3.33.

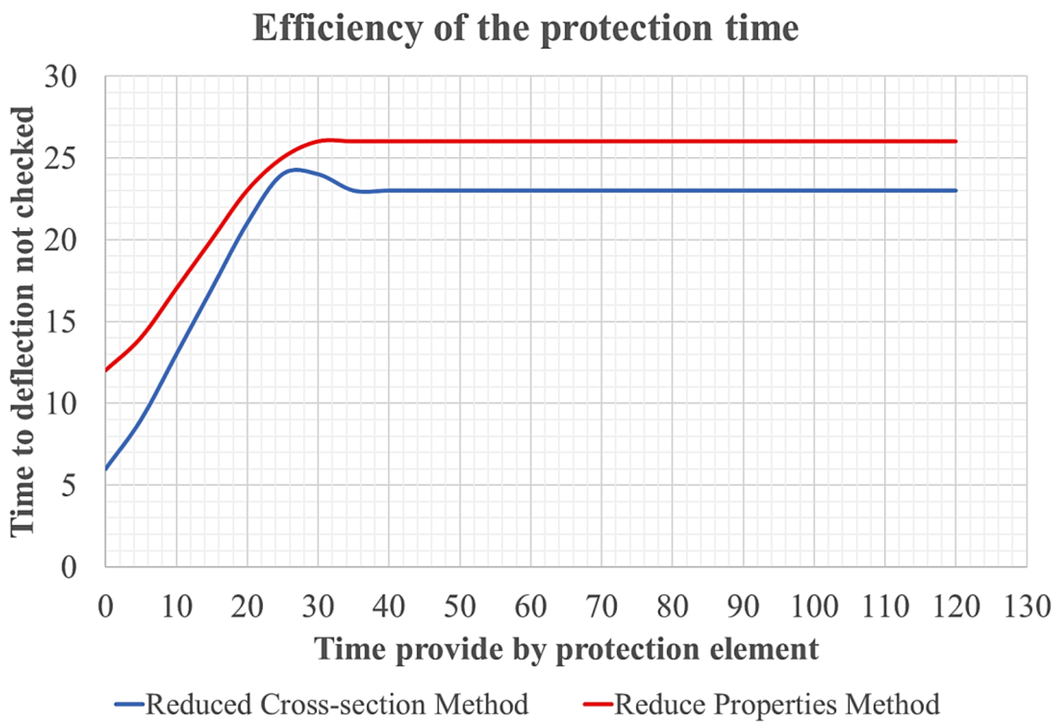


Figure 3.33. Efficiency of the protection time on joist partially protected

The analysis of result in Figure 3.33. shows in the particular case studied in this work that for a partial protection, whatever the protection time of the protection element, beyond **30 minutes** of protection capacity of the protection element, the deflection of the joist will not be verified anymore from the **25th minute** of exposure to fire. This analysis is valid for both methods.

These data made it possible to study the profitability of different protection times for partial protection of the Fir-joist and thus determine the most profitable protection time as show in Figure 3.34. This shows that the most cost-effective protection time for the partial protection of the fir joist is around **20 minutes** regardless of the evaluation method used.

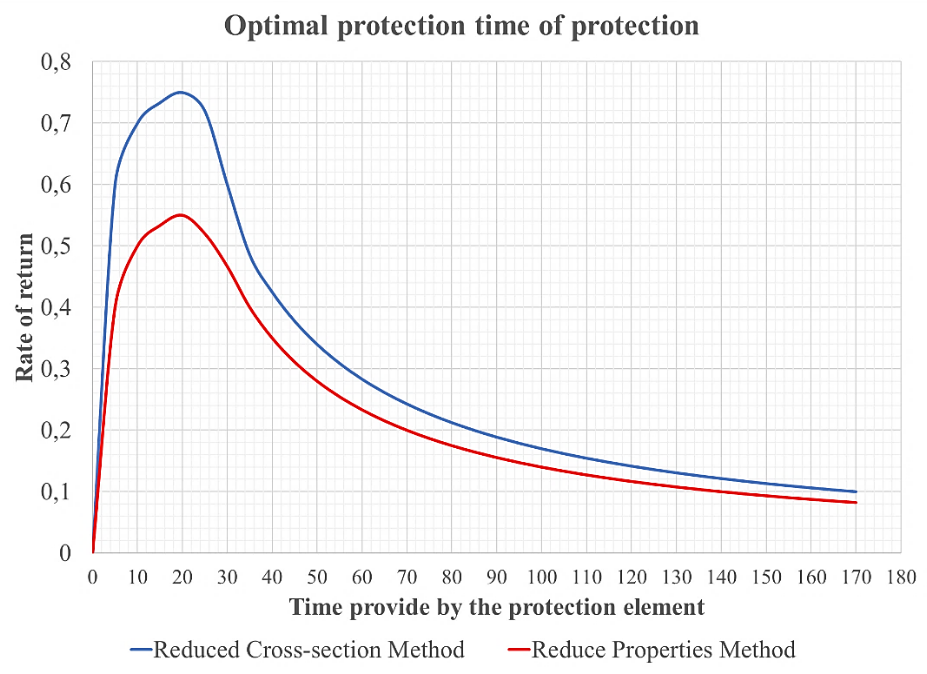


Figure 3.34. Optimal time of protection

3.3.2. Result of analysis of a vertical load-bearing element: case of column (Fir column)

Here, it is important to first analyse and assess the columns structural behaviour in the absence of fire, then take into account the fire situation, and finally to assess the effect of fire wall on the column's structural behaviour.

3.3.2.1. Result of analysis of column in normal temperature (without fire)

Before analysing the pole in a fire situation, it is important first to check it at normal temperature before assessing the impact of the fire on it.

a. Geometrical characteristics of the column

The geometrical characteristics of the joist are presented in table 3.24.

Table 3.24. Geometrical characteristic of column in normal temperature

Properties	Values	Unit	Parameters
b	0,16	**m**	**Width**
h	0,16	**m**	**Height**
L	3	**m**	**Span**

$I_{g,x}$	54613333,33	mm^4	**Moment of inertia**
V	75	mm	**Neutral axis distance**
L_f	3000	-	**Buckling length**
$W = I_{g,x} \backslash V$	562500	mm^3	**Section modulus**
λ_z	64,95	-	**Slenderness ratio**
λ_{rel}	1,55	-	**Relative slenderness ratio**
$K_{c,z}$	0,36	-	**Buckling coefficient**

b. Result of the verification of column subject to the axial load obtain from SAP2000 analysis

i. Load applied on the column

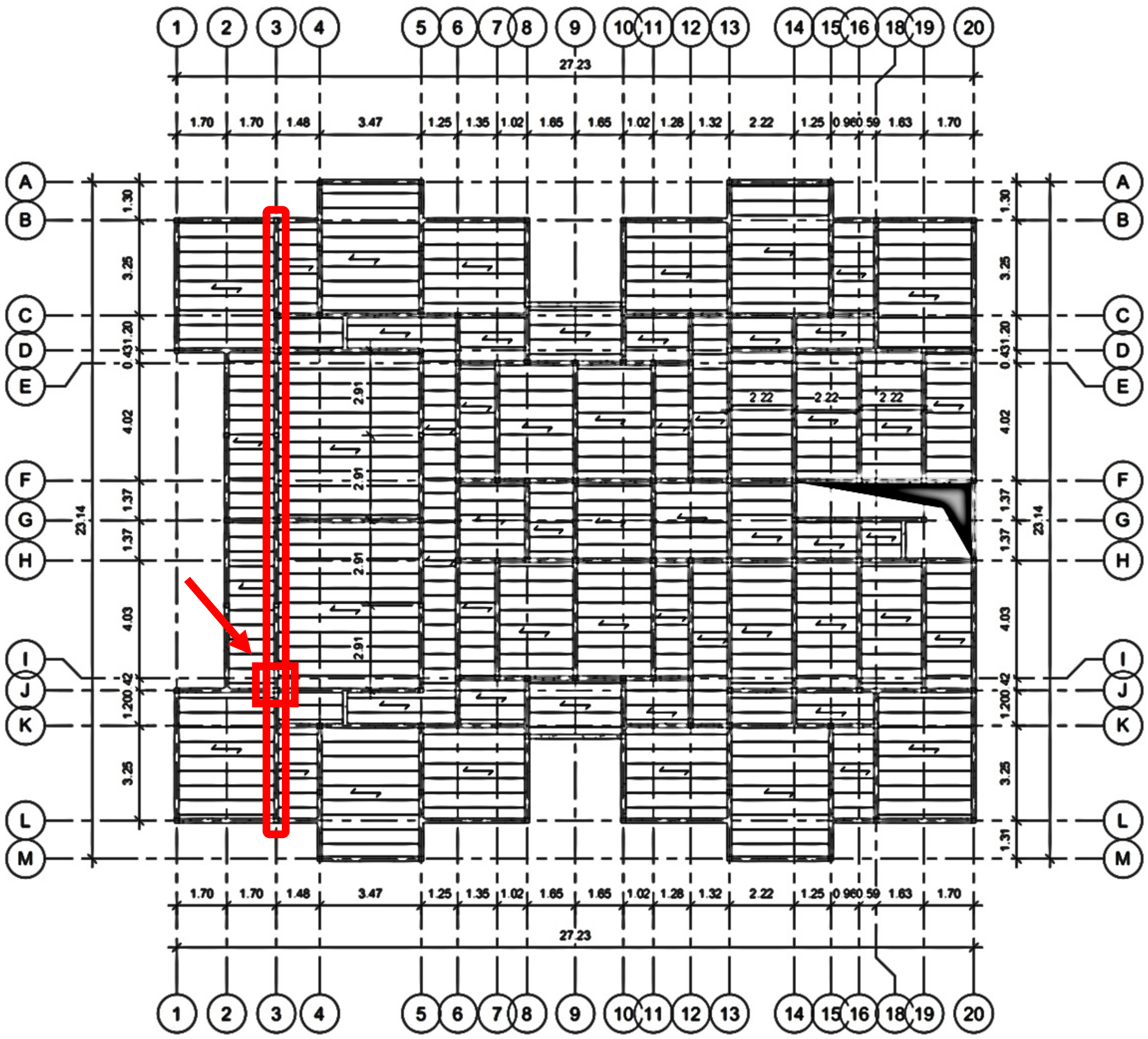


Figure 3.35. Floor direction: 1 ways slab

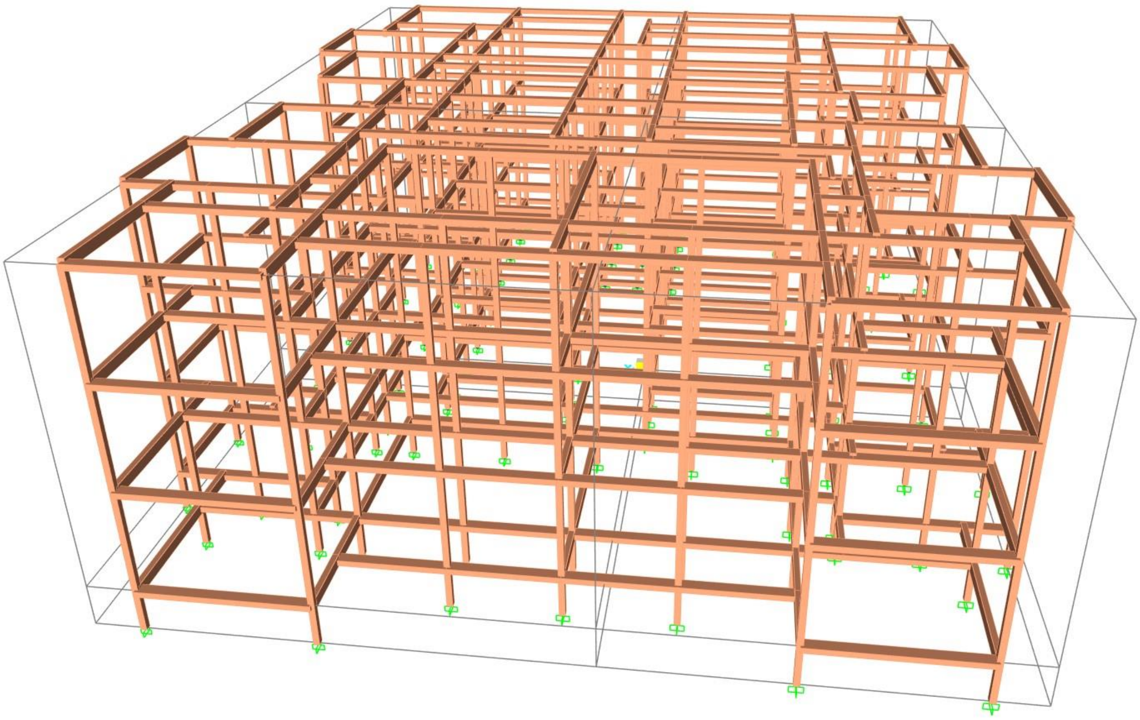


Figure 3.36. SAP 2000 view numerical-model of the structure

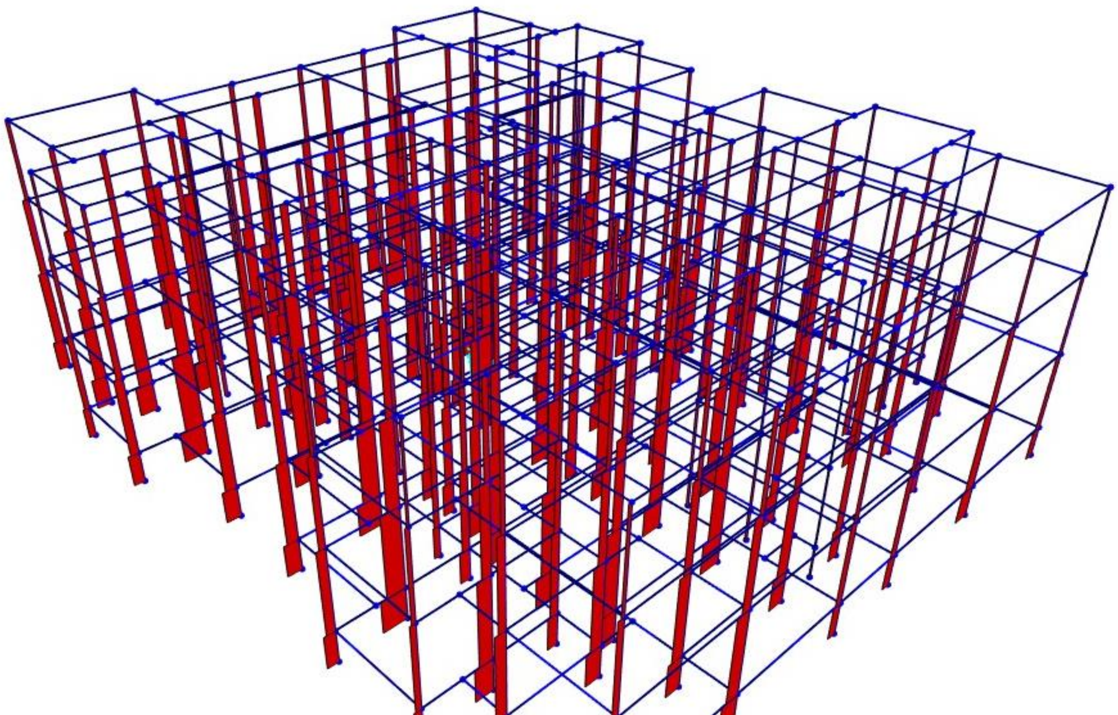


Figure 3.37. Numerical-view of the axial load in different column

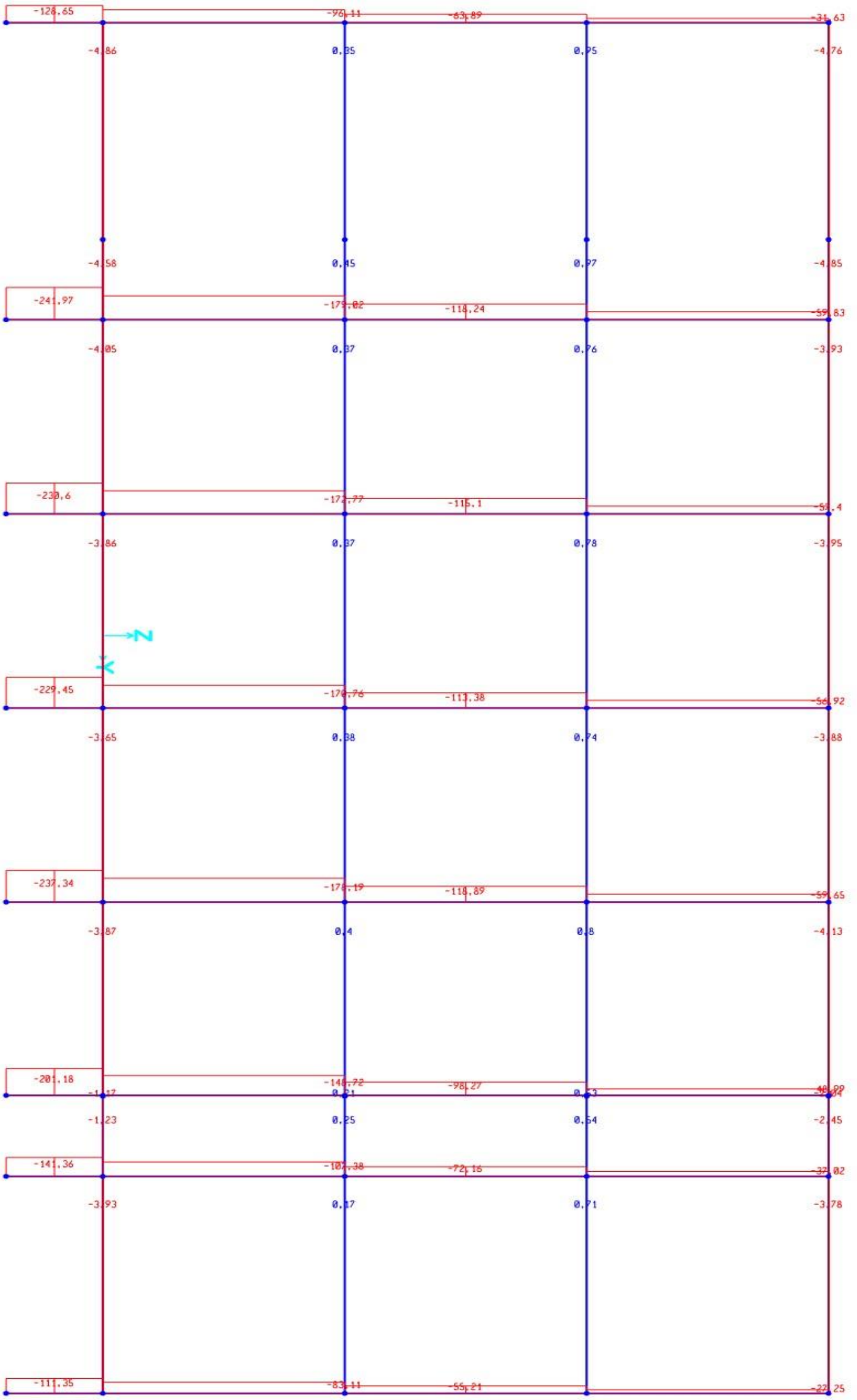


Figure 3.38. 2D view of the frame of the most loaded column

ii. Verification of column

The compressive forces obtained from the numerical analysis were checked against the design values and the result is presented in Table 3.25.

Table 3.25. Compressive load and buckling verification

Parameters	Values	Unit
Compressive force N_{ed}	242	**kN**
Compressive stress design $\sigma_{c,o,d}$	9,45	**MPa**
Compressive strenght design $f_{c,o,d}$	28,31	**MPa**
$K_{c,z}.f_{c,o,d}$	10,13	**kN**
Condition of verification	$\frac{\sigma_{c,0,d}}{k_{c,z}.f_{c,0,d}} \leq 1$	
Evaluation of result	**0,93**	
	Verified	

3.3.2.2. Analysis and evaluation of column in fire condition

Assumptions considered for the assessment of fire design:

- Eurocode 5-part 1-2 section 2.5.3 (3): The effect of the actions (e.g., internal forces and moments) relative to the initial support and boundary conditions can be deduced from the overall analysis of the structure in normal temperature design according to equation 2.37

  $E_{fi,d}$ = [0,6] Ed
- Class of service: Class 1
- Type of material: Solid timber

a. Case of unprotected column in fire exposition situation (4 exposed faces)

This section is reserved for the analysis of a column totally exposed to the fire. This analysis will be carried out successively with the reduced cross-section method and reduced property method as prescribed by the Eurocode 5; So, the column species choose is Fir which is a softwood and then be evaluated for both method

i. Result of the reduced cross-section method

The Table 3.26 gives the different coefficients needed to evaluate the structural behaviour of the column in a fire condition with the reduced cross-section method

Table 3.26. Structural coefficient used for design with reduced cross-section method

$k_{mod,fi}$ =	1,0	$\gamma_{M,fi}$ =	1,0	β_c =	0,2		k_{fi} =	1,25
$t_{fi,req}$ max(mm)=	**90**			**k_0 =**	1		**d_0 (mm)=**	7
Design charring rate of softwood Fir β_0(mm\min):							0,8	

- **Geometrical characteristics evolution of the unprotected Fir-column**

The evolution of the geometric characteristics of the Fir-column over the time of exposure to fire by the reduced cross-section method are presented in Table 3.27.

Table 3.27. Evolution of geometrical characteristics of the unprotected Fir-column over time

Geometrical characteristics							
Time (min)	**d_{char} (mm)**	**d_{ef} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,X}$ (mm^4)**	**V (mm)**	**W (mm^3)**
0	0,0	0,0	160	160	54613333,33	80,00	682666,67
5	4,0	11,0	138	138,0	30222828,00	69,00	438012,00
10	8,0	15,0	130	130,0	23800833,33	65,00	366166,67
15	12,0	19,0	122	122,0	18461121,33	61,00	302641,33
20	16,0	23,0	114	114,0	14074668,00	57,00	246924,00
25	20,0	27,0	106	106,0	10520641,33	53,00	198502,67
30	24,0	31,0	98	98,0	7686401,33	49,00	156865,33
35	28,0	35,0	90	90,0	5467500,00	45,00	121500,00
40	32,0	39,0	82	82,0	3767681,33	41,00	91894,67
45	36,0	43,0	74	74,0	2498881,33	37,00	67537,33
50	40,0	47,0	66	66,0	1581228,00	33,00	47916,00
55	44,0	51,0	58	58,0	943041,33	29,00	32518,67
60	48,0	55,0	50	50,0	520833,33	25,00	20833,33
65	52,0	59,0	42	42,0	259308,00	21,00	12348,00

70	56,0	63,0	34	34,0	111361,33	17,00	6550,67
75	60,0	67,0	26	26,0	38081,33	13,00	2929,33
80	64,0	71,0	18	18,0	8748,00	9,00	972,00
85	68,0	75,0	10	10,0	833,33	5,00	166,67
90	72,0	79,0	2	2,0	1,33	1,00	1,33

- **Compressive behaviour of the unprotect Fir-column**

The table 3.28 showed the variations of buckling coefficient of exposed Fir-column with fire time exposition.

Table 3.28. Evolution of buckling coefficient with the time fire exposition

Design modulus of elasticity in fire (MPa) $E_{fi,d}$ =			10217,5	
Temps (min)	**λ_z**	**λ_{rel}**	**K_z**	**$K_{c,z}$**
0	64,95	1,39	1,57	0,43
5	75,31	1,61	1,92	0,34
10	79,94	1,71	2,10	0,30
15	85,18	1,82	2,31	0,27
20	91,16	1,95	2,56	0,24
25	98,04	2,09	2,87	0,21
30	106,04	2,26	3,26	0,18
35	115,47	2,47	3,76	0,15
40	126,74	2,71	4,40	0,13
45	140,44	3,00	5,27	0,10
50	157,46	3,36	6,46	0,08
55	179,18	3,83	8,18	0,06
60	207,85	4,44	10,77	0,05
65	247,44	5,28	14,96	0,03
70	305,66	6,53	22,43	0,02
75	399,70	8,54	37,76	0,01
80	577,35	12,33	77,73	0,01
85	1039,23	22,20	249,01	0,00
90	5196,15	110,98	6169,67	0,00

The evolution of the compressive stress and the compressive strength of the exposed Fir-column by the reduced cross-section method is presented in Figure 3.39.

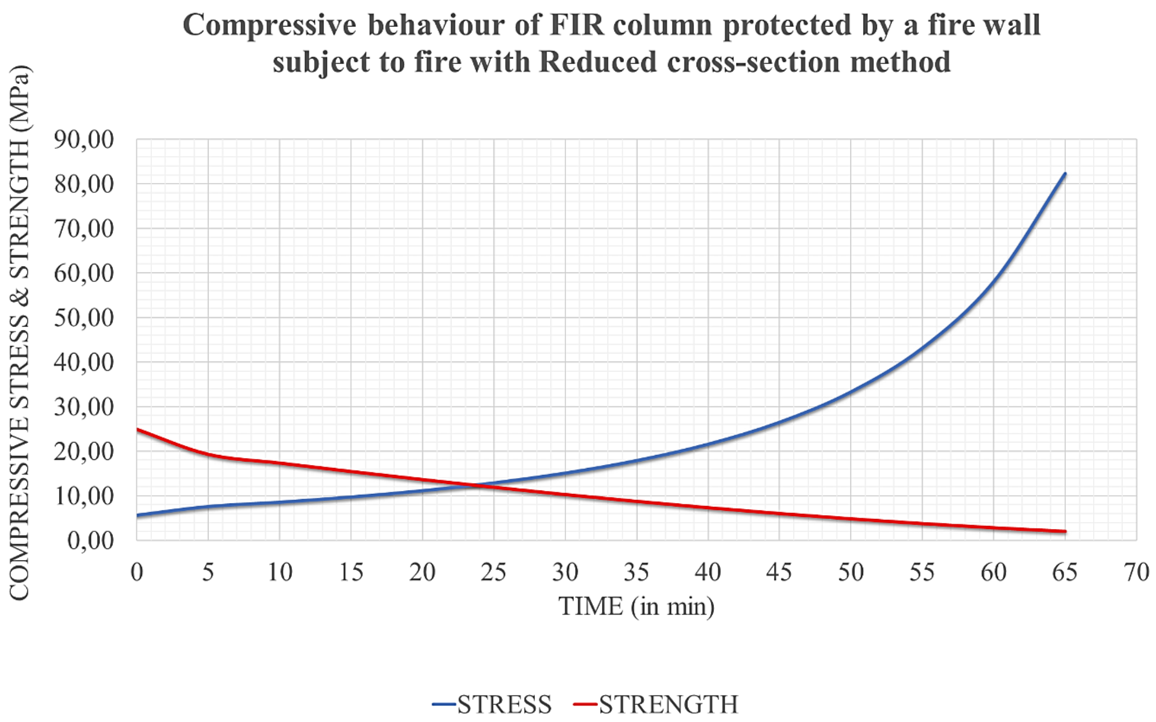


Figure 3.39. Evaluating curve of the compressive behaviour of the Fir column over time

It was found that the exposed Fir-column compressive resistance was no longer verified after **23 minutes (R23)** of exposure to fire with reduced cross-section method.

ii. Result of the reduced properties method

The Table 3.29 gives the different coefficients needed to evaluate the structural behaviour of the exposed Fir-column in a fire condition with the reduced properties method

Table 3.29. Structural coefficient used for design with reduced properties method

$k_{mod,fi}$ (t=0)=	1,0	**$\gamma_{M,fi}$ =**	1,0	**βc =**	0,2	**k_{fi} =**	1,25
$t_{fi,req}$ max(mm)=	**90**		**k_0 =**	0		**d_0 (mm)=**	0
Design charring rate of softwood Fir β_0(mm\min):						0,8	

- **Geometrical characteristics evolution of the unprotected Fir-column**

The evolution of the geometric characteristics of the Fir-column over the time of exposure to fire by the reduced properties method are presented in Table 3.30.

Table 3.30. Evolution of geometrical characteristics of the unprotected Fir-column and $K_{mod,fi}$ with time fire exposure

Geometrical characteristics							
Time (min)	**d_{char} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,x}$ (mm^4)**	**V (mm)**	**W (mm^3)**	**$K_{mod,fi}$**
0	**0,0**	**160**	**160**	**54613333,33**	**80,00**	**682666,67**	**1,00**
5	**4,0**	**152**	**152,0**	**44482901,33**	**76,00**	**585301,33**	**0,79**
10	**8,0**	**144**	**144,0**	**35831808,00**	**72,00**	**497664,00**	**0,78**
15	**12,0**	**136**	**136,0**	**28508501,33**	**68,00**	**419242,67**	**0,76**
20	**16,0**	**128**	**128,0**	**22369621,33**	**64,00**	**349525,33**	**0,75**
25	**20,0**	**120**	**120,0**	**17280000,00**	**60,00**	**288000,00**	**0,73**
30	**24,0**	**112**	**112,0**	**13112661,33**	**56,00**	**234154,67**	**0,71**
35	**28,0**	**104**	**104,0**	**9748821,33**	**52,00**	**187477,33**	**0,69**
40	**32,0**	**96**	**96,0**	**7077888,00**	**48,00**	**147456,00**	**0,67**
45	**36,0**	**88**	**88,0**	**4997461,33**	**44,00**	**113578,67**	**0,64**
50	**40,0**	**80**	**80,0**	**3413333,33**	**40,00**	**85333,33**	**0,60**
55	**44,0**	**72**	**72,0**	**2239488,00**	**36,00**	**62208,00**	**0,56**
60	**48,0**	**64**	**64,0**	**1398101,33**	**32,00**	**43690,67**	**0,50**
65	**52,0**	**56**	**56,0**	**819541,33**	**28,00**	**29269,33**	**0,43**
70	**56,0**	**48**	**48,0**	**442368,00**	**24,00**	**18432,00**	**0,33**
75	**60,0**	**40**	**40,0**	**213333,33**	**20,00**	**10666,67**	**0,20**
80	**64,0**	**32**	**32,0**	**87381,33**	**16,00**	**5461,33**	**0,00**
85	68,0	24	24,0	27648,00	12,00	2304,00	-0,33
90	72,0	16	16,0	5461,33	8,00	682,67	-1,00
95	76,0	8	8,0	341,33	4,00	85,33	-3,00

- **Compressive behaviour of the unprotect Fir-column**

The Table 3.31 showed the variations of buckling coefficient of exposed Fir-column with fire time exposition.

Table 3.31. Evolution of buckling coefficient with the time fire exposition

Design modulus of elasticity in fire $E_{fi,d}$ =			10217,5	
Temps (min)	λ_z	λ_{rel}	K_z	$K_{c,z}$
0	64,95	1,39	1,57	0,43
5	68,37	1,46	1,68	0,40
10	72,17	1,54	1,81	0,36
15	76,41	1,63	1,96	0,33
20	81,19	1,73	2,15	0,29
25	86,60	1,85	2,37	0,26
30	92,79	1,98	2,63	0,23
35	99,93	2,13	2,96	0,20
40	108,25	2,31	3,37	0,17
45	118,09	2,52	3,90	0,15
50	129,90	2,77	4,60	0,12
55	144,34	3,08	5,53	0,10
60	162,38	3,47	6,83	0,08
65	185,58	3,96	8,72	0,06
70	216,51	4,62	11,62	0,04
75	259,81	5,55	16,42	0,03
80	324,76	6,94	25,22	0,02
85	433,01	9,25	44,16	0,01
90	649,52	13,87	98,08	0,01
95	1299,04	27,74	388,13	0,00

The evolution of the compressive stress and the compressive strength of the exposed Fir-column by the reduced cross-section method is presented in Figure 3.40.

It was found that the exposed Fir-column compressive resistance was no longer verified after **26 minutes (R26)** of exposure to fire with reduced properties method.

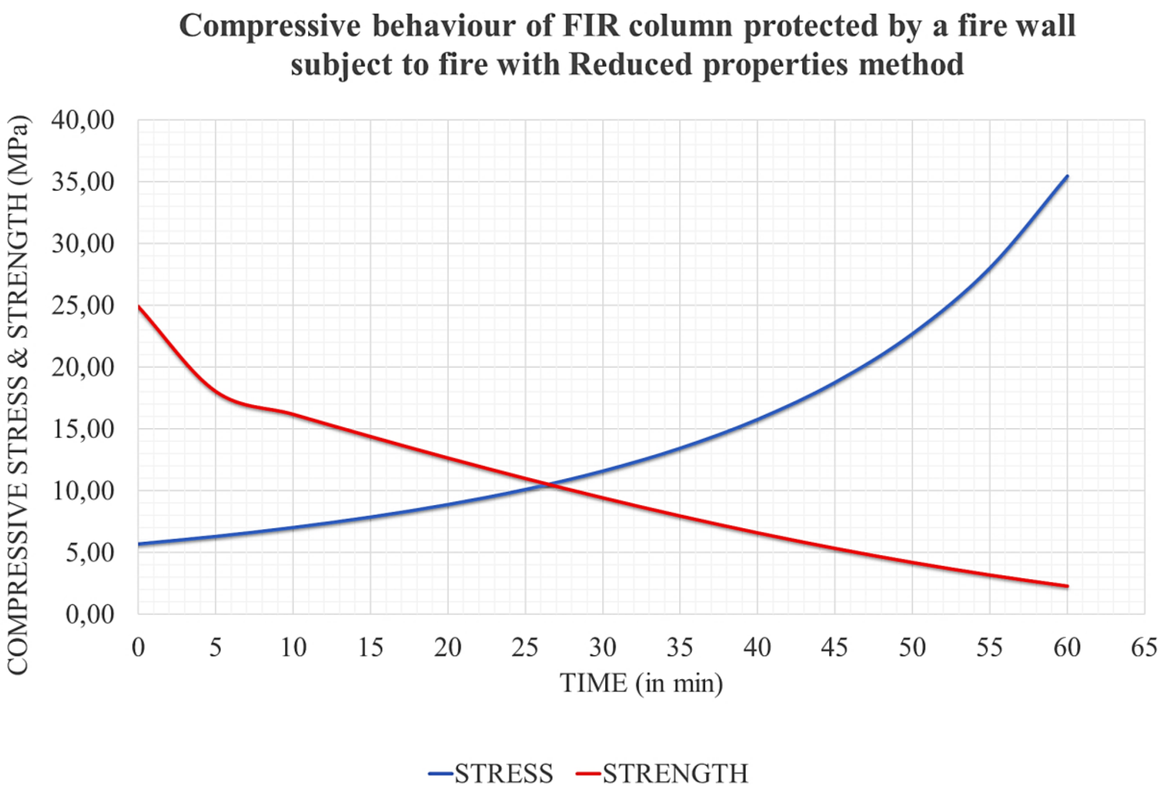


Figure 3.40. Evaluating curve of the compressive behaviour of the Fir column over time

b. Case of a fire wall (EI) protecting the column in fire condition

This section is reserved for the analysis of a Fir-column protected by an insulation and integrity (EI) fire wall. This analysis will be carried out successively with the reduced cross-section method and the reduced property method and the fire wall will be made of Moabi.

i. Design of fire wall

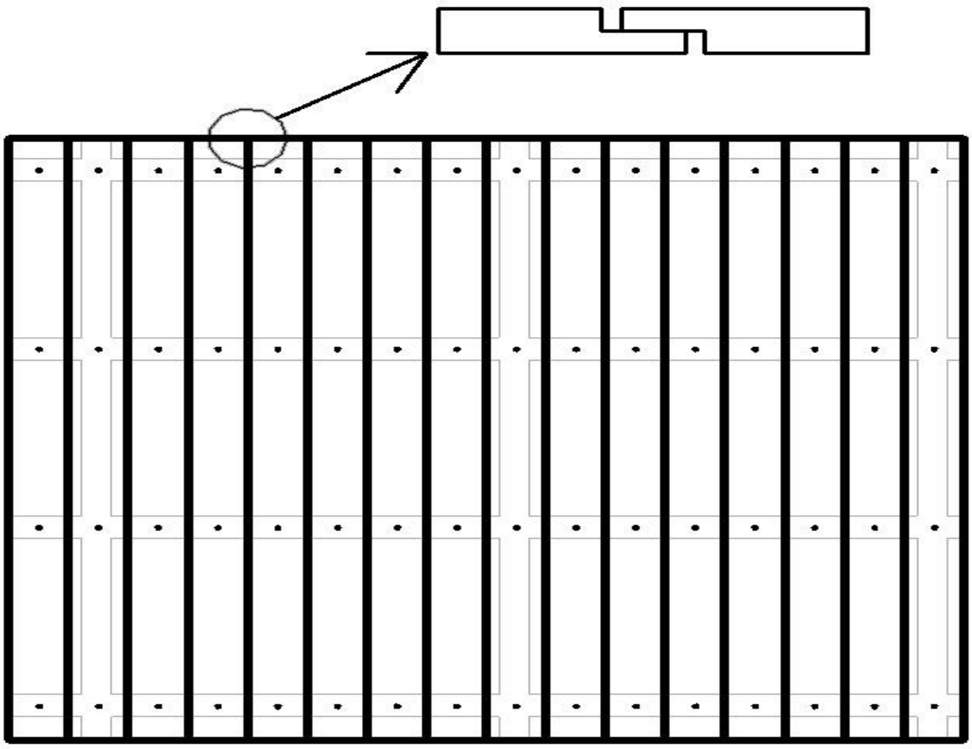


Figure 3.41. Fire wall (EI)

The geometry of the fire wall (width, height and thickness) and the fire resistance time in terms of integrity (E) and insulation (I) must be determined and show by Table 3.32

Table 3.32. Fire wall properties

Properties	Values	Unit	Parameters
h	800	**mm**	**Span of panel**
$t_{p,min}$	11,43 mm >8 mm ok	**mm**	**Minimum thickness**
t_p	40 mm	**mm**	**Final thickness**
ξ	0,3	**-**	**Reduction coefficient**
β_0	0,65	**-**	**Design charring rate of Moabi pannel**
t_{pr}	**18**	**min**	**Time integrity and insulation (EI) resist**

So, the wall will provide **18 min (EI) protection** to the column

ii. Result of the reduced cross-section method

The Table 3.33 gives the different coefficients needed to evaluate the structural behaviour of the total protected Fir-column by fire wall in a fire condition with the reduced cross-section method.

Table 3.33. Structural coefficient used for design with reduced cross-section method

$k_{mod,fi}$ =	1,0	**$\gamma_{M,fi}$ =**	1,0	**β_c =**	0,2		**k_{fi} =**	1,25
$t_{fi,req}$ max(mm)=	110			**k_0 =**	0		**d_0 (mm)=**	0
Design charring rate of softwood Fir β_0(mm\min):							0,8	

- **Geometrical characteristics of the Fir-column protected by the fire wall under fire condition**

The evolution of the geometric characteristics of the total protected Fir-column over the time of exposure to fire by the reduced cross-section method is showed Table 3.34.

Table 3.34. Evolution of geometrical characteristics of a fire wall protected column with time fire exposure

Section properties							
Time (min)	**d_{char} (mm)**	**d_{ef} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,x}$ (mm^4)**	**V (mm)**	**W (mm^3)**
0	0,0	0,0	160,0	160	54613333,33	80,00	682666,67
5	0,0	0,0	160,0	160	54613333,33	80,00	682666,67
10	0,0	0,0	160,0	160	54613333,33	80,00	682666,67
15	0,0	0,0	160,0	160	54613333,33	80,00	682666,67
20	1,6	8,6	142,8	142,8	34652261,55	71,40	485325,79
25	5,6	12,6	134,8	134,8	27515557,89	67,40	408242,70
30	9,6	16,6	126,8	126,8	21542483,46	63,40	339786,81
35	13,6	20,6	118,8	118,8	16599099,05	59,40	279446,11
40	17,6	24,6	110,8	110,8	12559657,47	55,40	226708,62
45	21,6	28,6	102,8	102,8	9306603,52	51,40	181062,33
50	25,6	32,6	94,8	94,8	6730574,00	47,40	141995,23
55	29,6	36,6	86,8	86,8	4730397,70	43,40	108995,34

60	33,6	40,6	78,8	78,8	3213095,43	39,40	81550,65
65	37,6	44,6	70,8	70,8	2093879,98	35,40	59149,15
70	41,6	48,6	62,8	62,8	1296156,16	31,40	41278,86
75	45,6	52,6	54,8	54,8	751520,77	27,40	27427,77
80	49,6	56,6	46,8	46,8	399762,60	23,40	17083,87
85	53,6	60,6	38,8	38,8	188862,47	19,40	9735,18
90	57,6	64,6	30,8	30,8	74993,15	15,40	4869,69
95	61,6	68,6	22,8	22,8	22519,47	11,40	1975,39
100	65,6	72,6	14,8	14,8	3998,21	7,40	540,30
105	69,6	76,6	6,8	6,8	178,18	3,40	52,41
110	73,6	80,6	0,0	-1,2	0,00	-0,60	0,00

- **Compressive behaviour of the protect Fir-column under fire condition**

The table 3.33 showed the variations of buckling coefficient of protected Fir column with fire time exposition.

Table 3.35. Evolution of buckling coefficient with the time fire exposition using reduced cross-section method

Design modulus of elasticity in fire $E_{fi,d}$ =			10217,5	
Temps (min)	**λ_z**	**λ_{rel}**	**K_z**	**$K_{c,z}$**
0	**64,95**	**1,39**	**1,57**	**0,43**
5	**64,95**	**1,39**	**1,57**	**0,43**
10	**64,95**	**1,39**	**1,57**	**0,43**
15	**64,95**	**1,39**	**1,57**	**0,43**
20	**72,78**	**1,55**	**1,83**	**0,36**
25	**77,09**	**1,65**	**1,99**	**0,32**
30	**81,96**	**1,75**	**2,18**	**0,29**
35	**87,48**	**1,87**	**2,40**	**0,26**
40	**93,79**	**2,00**	**2,68**	**0,22**
45	**101,09**	**2,16**	**3,02**	**0,20**
50	**109,62**	**2,34**	**3,45**	**0,17**
55	**119,73**	**2,56**	**4,00**	**0,14**
60	**131,88**	**2,82**	**4,72**	**0,12**

65	**146,78**	**3,13**	**5,70**	**0,10**
70	**165,48**	**3,53**	**7,07**	**0,08**
75	**189,64**	**4,05**	**9,08**	**0,06**
80	**222,06**	**4,74**	**12,19**	**0,04**
85	**267,84**	**5,72**	**17,40**	**0,03**
90	**337,41**	**7,21**	**27,16**	**0,02**
95	**455,80**	**9,73**	**48,83**	**0,01**
100	702,18	15,00	114,43	0,00
105	1528,28	32,64	536,44	0,00

The evolution of the compressive stress and the compressive strength of the protected Fir-column by the reduced cross-section method is presented in Figure 3.42

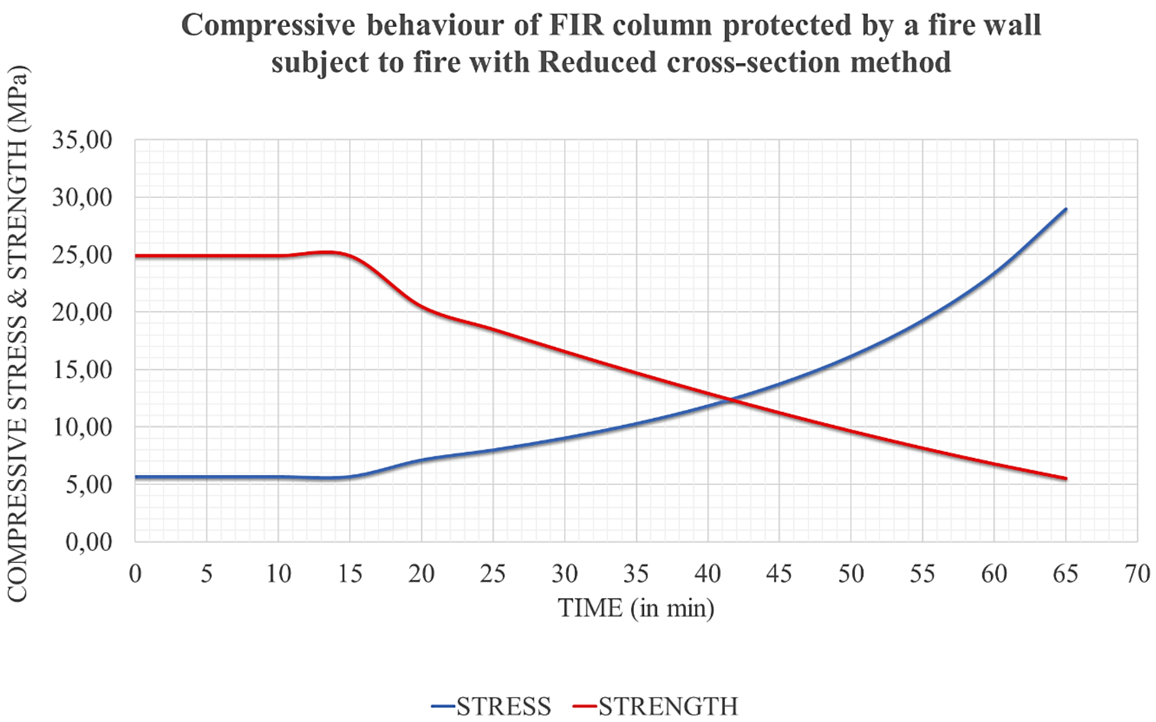


Figure 3.42. Evaluating curve of the compressive behaviour of a fire wall protected the Fir column

It was found that the protected Fir-column compressive resistance was no longer verified after **41 minutes (R41)** of exposure to fire with reduced cross-section method.

iii. Result of the reduced properties method

The Table 3.36 gives the different coefficients needed to evaluate the structural behaviour of the protected Fir-column in a fire condition with the reduced properties method

Table 3.36. Structural coefficient used for design with reduced properties method

$k_{mod,fi}$ (t=0)=	1,0	**$\gamma_{M,fi}$ =**	1,0	**β_c =**	0,2	**k_{fi} =**	1,25
$t_{fi,req}$ max(mm)=	120		**k_0 =**	0		**d_0 (mm)=**	0
Design charring rate of softwood Fir β_0(mm\mim):						0,8	

- **Geometrical characteristics of the Fir-column protected by the fire wall**

The evolution of the geometric characteristics of the protected Fir-column over the time of exposure to fire by the reduced properties method are presented in Table 3.37.

Table 3.37. Evolution of geometrical characteristics of a fire wall protected column and $K_{mod,fi}$ with time fire exposure

Geometrical characteristics							
Time (min)	**d_{char} (mm)**	**b (mm)**	**h (mm)**	**$I_{g,x}$ (mm^4)**	**V (mm)**	**W (mm^3)**	**$K_{mod,fi}$**
0	**0,0**	**160,0**	**160,0**	**54613333,33**	**80,00**	**682666,67**	**1,00**
5	**0,0**	**160,0**	**160,0**	**54613333,33**	**80,00**	**682666,67**	**1,00**
10	**0,0**	**160,0**	**160,0**	**54613333,33**	**80,00**	**682666,67**	**1,00**
15	**0,0**	**160,0**	**160,0**	**54613333,33**	**80,00**	**682666,67**	**1,00**
20	**1,6**	**156,8**	**156,8**	**50373599,78**	**78,40**	**642520,41**	**0,80**
25	**5,6**	**148,8**	**148,8**	**40853613,77**	**74,40**	**549107,71**	**0,78**
30	**9,6**	**140,8**	**140,8**	**32751362,59**	**70,40**	**465218,22**	**0,77**
35	**13,6**	**132,8**	**132,8**	**25918571,04**	**66,40**	**390339,93**	**0,76**
40	**17,6**	**124,8**	**124,8**	**20215155,92**	**62,40**	**323960,83**	**0,74**
45	**21,6**	**116,8**	**116,8**	**15509226,02**	**58,40**	**265568,94**	**0,73**
50	**25,6**	**108,8**	**108,8**	**11677082,15**	**54,40**	**214652,25**	**0,71**
55	**29,6**	**100,8**	**100,8**	**8603217,10**	**50,40**	**170698,75**	**0,68**
60	**33,6**	**92,8**	**92,8**	**6180315,68**	**46,40**	**133196,46**	**0,66**

65	**37,6**	**84,8**	**84,8**	**4309254,69**	**42,40**	**101633,37**	**0,62**
70	**41,6**	**76,8**	**76,8**	**2899102,92**	**38,40**	**75497,47**	**0,58**
75	**45,6**	**68,8**	**68,8**	**1867121,19**	**34,40**	**54276,78**	**0,53**
80	**49,6**	**60,8**	**60,8**	**1138762,27**	**30,40**	**37459,29**	**0,47**
85	**53,6**	**52,8**	**52,8**	**647670,99**	**26,40**	**24532,99**	**0,39**
90	**57,6**	**44,8**	**44,8**	**335684,13**	**22,40**	**14985,90**	**0,29**
95	**61,6**	**36,8**	**36,8**	**152830,50**	**18,40**	**8306,01**	**0,13**
100	65,6	28,8	28,8	57330,89	14,40	3981,31	-0,11
105	69,6	20,8	20,8	15598,11	10,40	1499,82	-0,54
110	73,6	12,8	12,8	2236,96	6,40	349,53	-1,50
115	77,6	4,8	4,8	44,24	2,40	18,43	-5,67
120	81,6	0,0	-3,2	54613333,33	80,00	682666,67	1,00

- **Compressive behaviour of the protect Fir-column under fire condition**

The table 3.38 showed the variations of buckling coefficient of protected Fir column with fire time exposition.

Table 3.38. Evolution of buckling coefficient with the time fire exposition

Design modulus of elasticity in fire $E_{fi,d}$ (MPa)=			10217,5	
Temps (min)	**λ_z**	**λ_{rel}**	**K_z**	**$K_{c,z}$**
0	**64,95**	**1,39**	**1,57**	**0,43**
5	**64,95**	**1,39**	**1,57**	**0,43**
10	**64,95**	**1,39**	**1,57**	**0,43**
15	**64,95**	**1,39**	**1,57**	**0,43**
20	**66,28**	**1,42**	**1,61**	**0,42**
25	**69,84**	**1,49**	**1,73**	**0,38**
30	**73,81**	**1,58**	**1,87**	**0,35**
35	**78,26**	**1,67**	**2,03**	**0,31**
40	**83,27**	**1,78**	**2,23**	**0,28**
45	**88,98**	**1,90**	**2,47**	**0,25**
50	**95,52**	**2,04**	**2,75**	**0,22**
55	**103,10**	**2,20**	**3,11**	**0,19**
60	**111,99**	**2,39**	**3,57**	**0,16**

65	**122,55**	**2,62**	**4,16**	**0,14**
70	**135,32**	**2,89**	**4,94**	**0,11**
75	**151,05**	**3,23**	**6,00**	**0,09**
80	**170,93**	**3,65**	**7,50**	**0,07**
85	**196,82**	**4,20**	**9,73**	**0,05**
90	**231,97**	**4,95**	**13,24**	**0,04**
95	**282,40**	**6,03**	**19,26**	**0,03**
100	**360,84**	**7,71**	**30,94**	**0,02**
105	**499,63**	**10,67**	**58,47**	**0,01**
110	811,90	17,34	152,55	0,00
115	2165,06	46,24	1074,21	0,00

The evolution of the compressive stress and the compressive strength of the protected Fir-column by the reduced properties method is presented in Figure 3.43

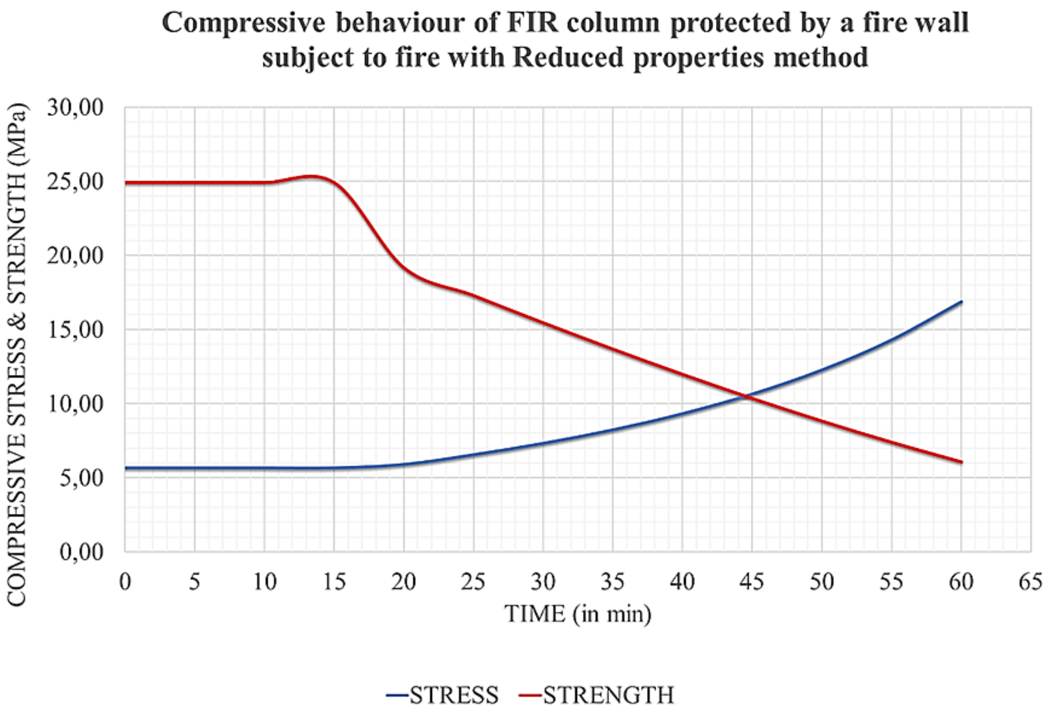
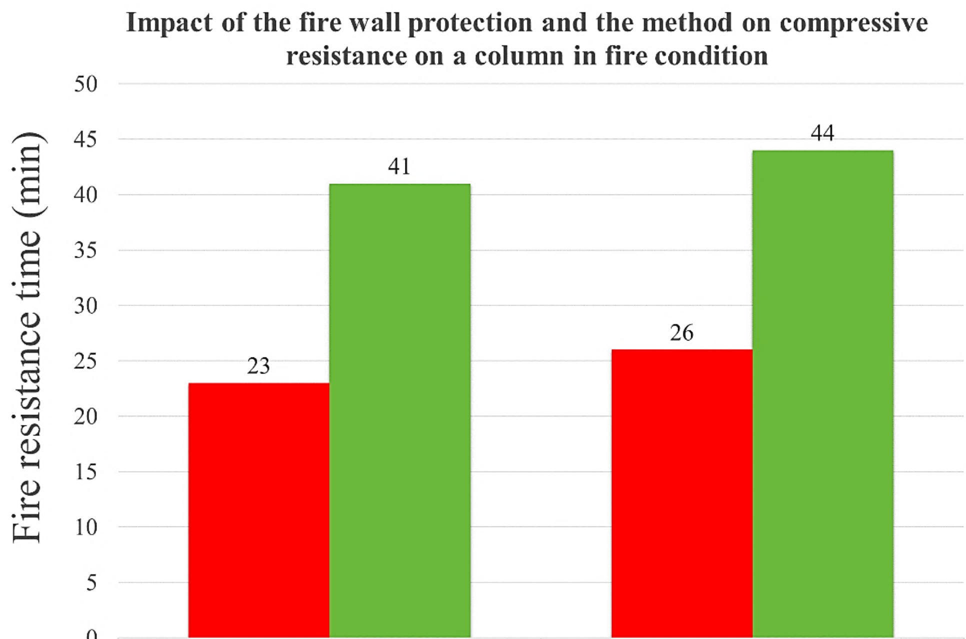


Figure 3.43. Evaluating curve of the compressive behaviour of a fire wall protected the Fir column with time fire exposure

It was found that the protected Fir-column compressive resistance was no longer verified after **44 minutes (R44)** of exposure to fire with reduced properties method.

iv. Synthesis of the analysis with the fire wall protection elements of column

The purpose of this part is to gather all the results previously obtained (those of the fire analysis of a protected and unprotected Fir-column) in the form of a histogram in order to better perceive the impact of fire wall protection on time resistance of the Fir column



	Reduced Cross section Method	Reduce properties Method
Unprotected Column	23	26
Fire wall Protected	41	44

Figure 3.44. Impact of the fire wall protection and the method on compressive resistance on a column in fire condition

The Figure 3.43 show for the particular case studied in this work, the evolution of the fire resistance time of the Fir-column under the effect of the total protection by means of the reduced cross-section method and the reduced properties method respectively; It appears that the fire resistance times in compressive resistance went from **23 minutes to 41** for bending using reduced cross-section method, and **26 minutes to 44** for using reduced properties method

3.3.3. Influence of the choice of design method (reduced cross-section and reduced properties) on structural behaviour of the structure in fire condition

The analysis of the fir joist in its fire resistance in bending, shear and deflection by means of the reduced cross-section and reduced properties method in section 3.2.3 of this work, as well as the analysis of the fir column in its fire resistance in compression for each of the 2 previous methods in section 3.2.4 will have given resistance times presented in Figure 3.44

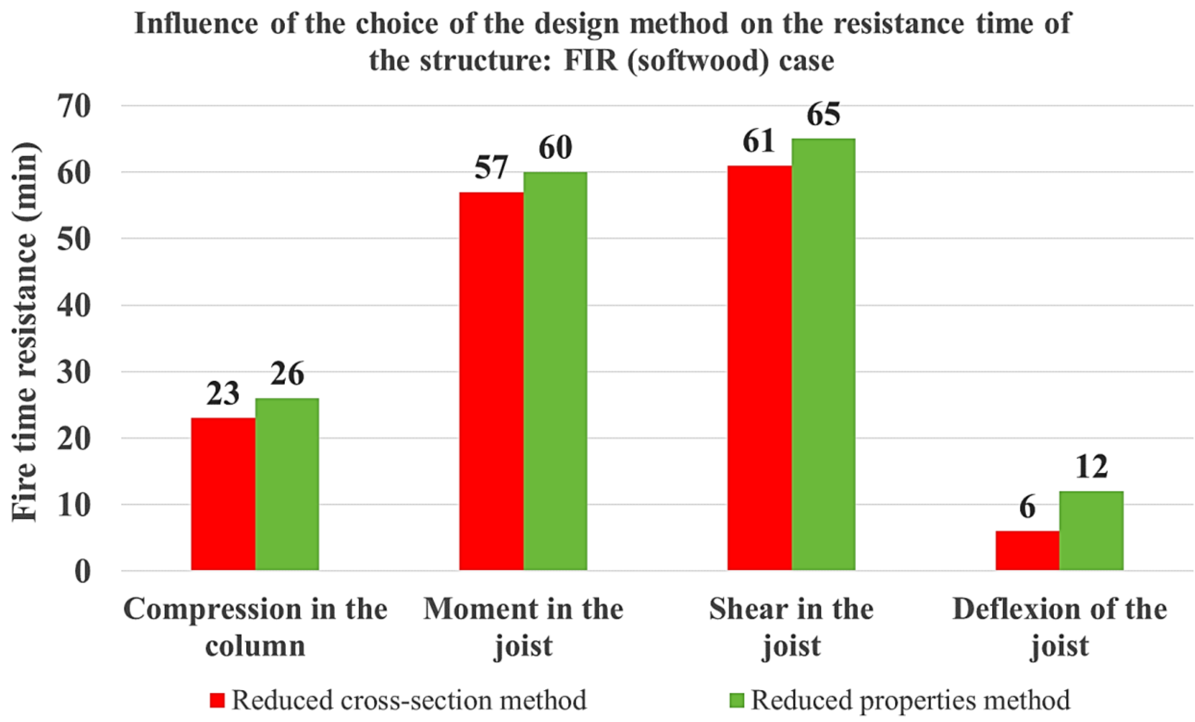


Figure 3.45. Influence of the choice of the design method on the resistance time of the structure: FIR (softwood) case

From the results presented in Figure 3.44, based on the fact that both methods are safe according to Eurocode 5: part 1-2, it is clear that the reduced cross-section method is more conservative (gives lower resistance times) than the reduced properties method with a difference in the order of 11.54% for the compression of the fir column, 05.00% for the bending, 06.15% for the shear and 50.00% for the deflection of the joist;

In order to homogenise the results given by the two methods by reducing the level of conservativity of the reduced cross-section method compared to the reduced properties method, a variation of the coefficient d0 (depth of the zero-strength layer) for a coefficient k =1 (Condition of a required fire resistance time higher than 20 minutes) has been carried out in order to reduce the effect considered by the reduced cross-section method in its estimation of the depth of the zero-strength layer; the results obtained are presented in figures 3.45 and 3.46

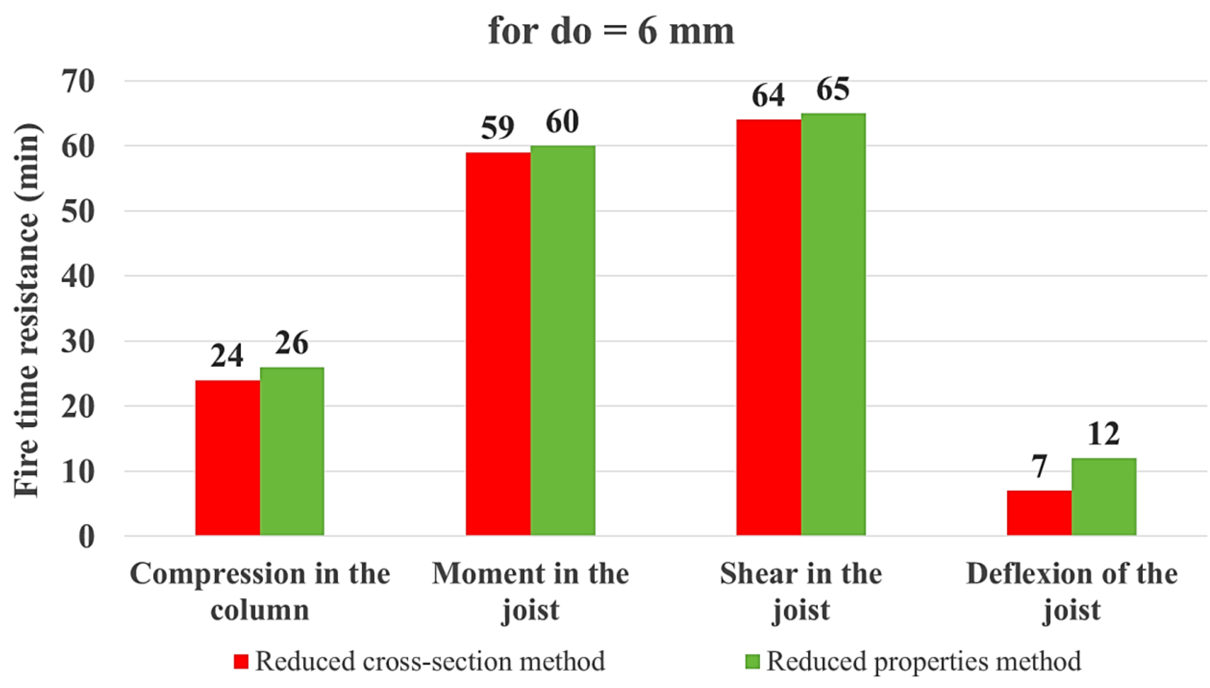


Figure 3.46. Effect of variation of the depth of the zero-strength layer on the time differences given by the two analytical methods with d_0 = 6 mm

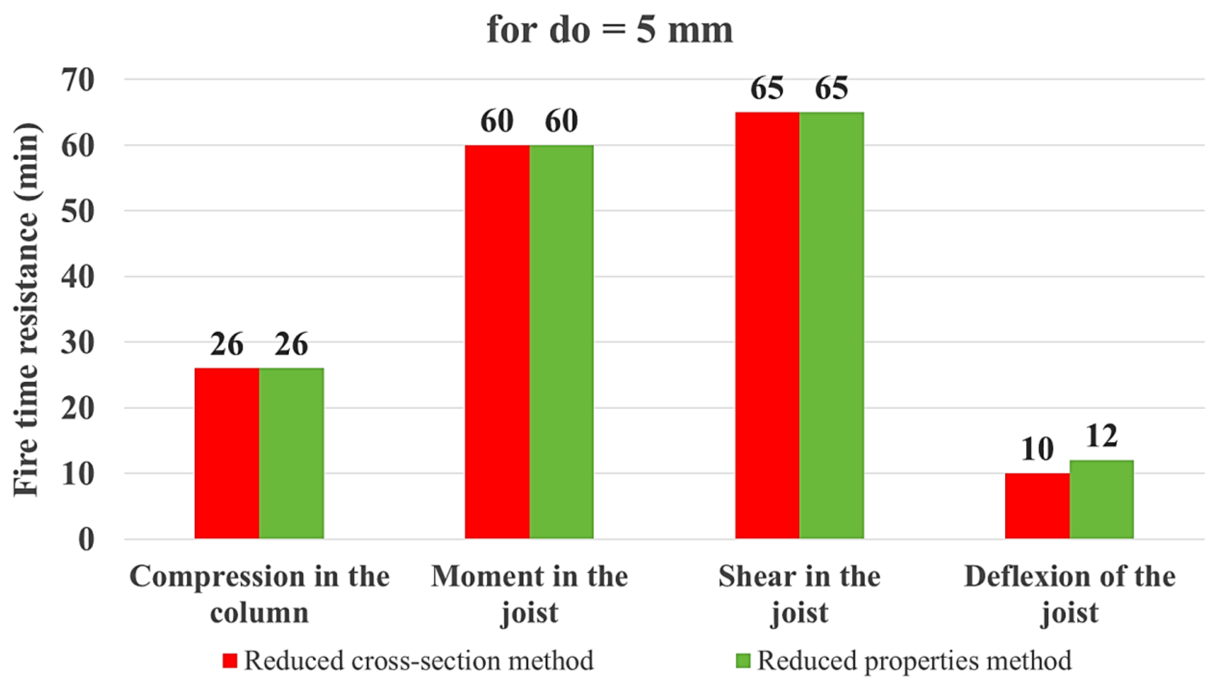


Figure 3.47. Effect of variation of the depth of the zero-strength layer on the time differences given by the two analytical methods with d_0 = 5 mm

The results presented in Figures 3.45 and 3.46 (show within the limits of the case studied here) that, with reference to the reduced properties method, the d_0 coefficient taken at 7 mm as recommended by Eurocode 5: part 1-2 is safer than it needs to be; a d_0 value taken at 5 mm is much better for having much closer estimates of the resistance time of the structure as a function of the analytical method used (reduced cross-section or reduced properties) to analyse the behaviour of the structure in a fire situation.

Thus, apart from the observation made within the limits of this case study on the conservativity of the reduced cross-section method compared to the reduced properties method through the depth of the zero-strength layer d_0 defined at 7 mm by the Eurocode 5: part 1-2, publications have been issued in the context of comparative studies between the results of numerical methods and the analytical one of the reduced cross-section methods, also questioning the value of d0. The following extracts of conclusions have been issued from some of these studies, notably

- «*The load bearing capacities and measured time histories of deflection during heating are compared against predicted responses wherein the experimentally measured char depths are used, along with the Eurocode recommended reduced cross section method and zero-strength layer thickness. The results confirm that the current zero-strength layer value (indeed the zero-strength concept) fails to capture the necessary physics for robust prediction of structural response under nonstandard heating. It is recommended that more detailed thermo-mechanical cross-sectional analyses, which allow the structural implications of real fire exposures to be properly considered, should be developed and that the zero-strength layer concept should be discarded in these situations. Such a novel approach, once developed and suitably validated, could offer more realistic and robust structural fire safety design*» (Lineham & Al, 2016)
- « *In this paper, it is shown that, for the studied cross section, this assumption is often non-conservative. The zero-strength layer depends on the number of sides exposed to fire and, in the case of fire exposure on three sides, on the state of stress (tension or compression) of the exposed side. Since the combination of elevated temperature and compression is most unfavourable in cross sections with such conditions, a larger zero-strength layer should be adopted. It was shown that the depth of the cross section has some effect on the zero-strength layer d0. In general, for lesser depths of cross section, the zero-strength layer is smaller. However, the influence of depth becomes less for larger depths, finally becoming zero. The reason for this is that, for shallow cross-section depths, a larger part of the heat-affected zone (approximately 40 mm for initially*

unprotected members) below the char layer is close to the neutral layer and therefore has less effect on bending resistance. For large depth cross sections, the stress distribution within the heat-affected zone is, at normal temperature, almost uniform, with strength reduction due to elevated temperatures having a greater influence on the bending resistance of the timber member» (Schmid & Al, 2018)

Conclusion

In this chapter, the main objective was to present the results of the static verification of the building, the fire analyses performed and to interpret the results. Firstly, it was proved that the structural elements present in the case study were statically checked according to the Eurocode requirements. Secondly, the fire analysis carried out on the vertical and horizontal load-bearing elements showed that the time of resistance to deflection was the lowest compared to bending and shear in the case of the joist study. It was also observed that the choice of material had a significant impact on the structural behaviour of the wood; The Moabi hardwood has been shown to perform better than the Fir softwood. However, in case of low fire resistance of the element, it is always possible to overcome it by means of protection, especially wood-wood protection in case of this study. Finally, the choice of the method used between Reduced cross-section and Reduced properties method has an impact on the fire resistance of the wood. The difference in resistance time between the two methods is about 4 minutes on average, with the Reduced properties method having a higher time. This highlights the high conservativity of the reduced cross-section method compared to the reduced properties method. The reduction of the d_0 coefficient from 7 to 5 min will still allow the results to be refined and the times given by the two methods to be brought closer together.

GENERAL CONCLUSION

The main objective of this work was to analyse and evaluate the effect of fire on the structural behaviour of wooden structures. To achieve this, a social housing building at ground floor + 2 storeys was use for our case study. In order to achieve this objective, the work was organised around a literature review that provided a better understanding of the wood element, including its structure, composition, physical and mechanical properties, defects and advantages, means of protection, multiple uses ranging from carpentry to wooden construction, as well as a brief presentation of the procedures for analysing wooden structures under fire conditions.

Following the literature review, a methodology was defined, explaining the different stages of the work that will guide the achievement of the objectives. Thus, after modelling the structure in SAP2000, a static analysis of the structure through the analysis of the load-bearing elements at the ultimate limit state (ULS) as well as a verification of the serviceability limit state (SLS) was done, these constitute the design phase. The different mechanical properties of the different elements under different loads were then analysed under fire loading to evaluate their response. Finally, this work was concluded with the analysis and interpretations of the different results obtained.

Thus, the analysis of wooden structures under fire conditions for different stresses give variable time resistance depending on; the wood species chosen, the existence or not of a partial (case of the joist where 2 of the 3 exposed faces were protected), or total (case of the column) fire protection and the choice of the method among the 2 analytical methods prescribed by the EC5 : part 1-2 (that of the reduced cross-section applicable to both hardwood and softwood and that of the reduced properties applicable only to softwood). The analysis of the unprotected joist for different species showed that Moabi (hardwood) performed better than Fir (softwood) with a time resistance of 97 minutes against 57 in bending, 101 minutes against 61 in shear and 52 minutes against 06 for deflection respectively in an analysis with the reduced cross-section method common to both species. Then, the analysis of the Fir joist protected by Moabi panels (with a fire time protection of 26 minutes) increased the resistance times of the joist in bending, shear and deflection respectively from 57 to 81 minutes, 61 to 87 minutes and 06 to 24 minutes for the analysis by the reduced cross-section method, and from 60 to 83 minutes, 65 to 90 minutes and 12 to 25 minutes for the analysis by the reduced properties method. Analysing the Fir column subjected to compression in the unprotected state and in the protected state by a firewall (of EI 18), the compression resistance time of the column increased from 23 minutes to 41 for the effective cross-section method and from 26 minutes to 44 for the reduced property method. In view of the above, it can be concluded that, irrespective of the method chosen, the fire resistance time in deflection is the key criterion for the analysis of the fire resistance time of timber structures to the detriment of bending, shear and compression here studied.

The analysis of the protection elements showed that the effectiveness of the protection is 100% in the case of a total protection, while in the case of a partial protection, it was shown that the increase in the fire resistance time of the element was not necessarily 100% of the protection time (given by the protection element). Thus, for increasing protection time, the fire resistance time of the element increases up to a certain time, considered as the maximum time needed for partial protection, after which any increase in protection time has a reduced impact on the fire resistance time. The case of the deflexion of joist gave the maximum time needed for the protection of about 30 minutes for both methods, while the optimum time was around 20 minutes.

The analysis of impact of the chosen method on the fire resistance time showed that the reduced cross-section method was much more conservative than the reduced properties method. This was mainly due to the coefficient characterising the depth of the zero-strength layer d_0 (taken at 7 mm by Eurocode 5) in evaluation of the cross section. Thus, a reduction of this coefficient from 7 to 5 mm made it possible to reduce the difference in the evaluation of the fire resistance time between the 2 methods from 05% to 0% for bending, 6.15% to 0% for shear, 11.54% to 0% for column compression and 50% to 16.67% for beam deflection.

Complementary to this work, it is suggested to conduct a study on the structural resistance of different types of timber structure connections under fire loading taking into account their level of exposure, and their impact on the overall resistance of timber structures exposed to fire.

It is also suggested to assess the cost - effectiveness of the different means and elements of protection of timber structures against fire.

Given the relative slight difference of the results in terms of time resistance to bending, shear, compression and deflection given by the 2 methods of prescriptive approach studied in this work, it is recommended that in the future a performance-based approach through numerical analysis be carried out to compare with the 2 methods in order to assess the level of conservativity or non-conservativity with respect to the numerically obtained values considered more accurate.

BIBLIOGRAPHY

BOOKS and ARTICLES

CODIFAB. (2014). *Durabilité biologique des menuiseries extérieures en bois* pp. 1-13.

CTFC. (2011) *Forêts et bois des communes forestiers du Cameroun*.

Tsoumis, G. T. (1991). *Science and technology of wood: structure, properties, utilization* (Vol. 115). New York: Van Nostrand Reinhold.

Encyclopædia Britannica. (2008). *The Encyclopædia Britannica ,* Volume 4 - Part 03 of 04

MBESSA, M. (2005). *Dans Traité du Génie Civil "Matériaux de construction"*.

Pilon. A et al (n.d.). *Construction Of A Tall Wood Building Brock Commons Tallwood House*. Naturally Wood Publisher.

G. Thomasson, J. Capizzi, F. Dost, J. Morrell, and D. Miller (2015). *Wood Preservation and Wood Products Treatment, Training Manual.* Oregon State University. 24p.

O. Gagliardini (2020). *Construction Bois selon l'Eurocode* 5. Université de Grenoble Alpes. 48p.

Benoit, Y., Legrand, B., & Tastet, V. (2019). *Calcul des structures en bois.* Paris: Eyrolle

JORDIN, P., ARBOLOR, & Nancy. (1994). *Le bois, matériau d'ingénierie*

Netterer, J., J-L, Sandoz, & Rey, M. (2004). *Construction en bois*.

Ramage, M. H., Burridge, H., Busse-wicher, M., Fereday, G., Reynolds, T., Shah, D. U., … Scherman, O. (2017). *The wood from the trees: The use of timber in construction*. Elsevier. Volume 68. Issued September 2016. P. 333–359.

Porteous, J., & Kermani, A. (2007). *Structural Timber Design to Eurocode 5*. Blackwell Publishing

László, E. (2008). *Analysis of the Load-Slip Behaviour of Dowelled Timber to Timber Joints*. Budapest University of Technology and Economics.

Chávez, R. L. E. (2018). *Design of Wood Connections with dowel type fasteners*. Instituto Politécnico de Leiria

Dolan, J. D. (2005). *Handbook Of Structural Engineering Timber Structures*. CRC Press.

Sandhaas, C., & Dietsch, P. (2018). *Design of Connections in Timber Structures*. Shaker Verlag.

Rochette, P. J. (1964). *Le bois : sa structure, ses propriétés, ses utilisations*. Editions Dunod

UNION NATIONALE FRANCAISE DE CHARPENTE, MENUISERIE, PARQUETS. (1996). *Structure en bois aux etats limites - TOME 1*. PARIS: EYROLLES.

INERIS. (2001). *Développement d'une méthodologie d'évaluation des effets thermiques et toxiques des incendies d'entrepôts (DRA-03) : Comportement de structures soumises à un incendie. Premières réflexions*. Direction des Risques Accidentels.

Normes Internationales pour les Mesures Phytosanitaires-NIMP (2008). *Glossaire Des Termes Phytosanitaires.* 23p.

Afanasyev SV, Korotkov RV, Kuzmin IV, Tripolitsyn AA (2007) *Efficiency study of fire protection compositions based on amidophosphate. Pozharovzryvobezopasnost* 18(3):28–31

Östman B et al (2010) *Fire safety in timber buildings – technical guideline for Europe*. SP Report 2010:19. SP Technical Research Institute of Sweden, Stockholm

König J. *The Reduced Cross-Section Method for Light Timber Frame Construction with Solid Timber Members*. SP Report 2009: 46. SP Technical research Institute of Sweden, Wood Technology, Stockholm. 2009.

Schmid, J. M., König, J. (., & Just, A. P. (2018). *The Reduced Cross-Section Method for the Design of Timber Structures Exposed to Fire*—Background, Limitations and New Developments. Structural Engineering International, 514-522.

Lineham, S. A., Thomson, D., Bartlett, A. I., Bisby, L. A., & Hadden, R. M. (2016). Structural response of fire-exposed cross-laminated timber beams under sustained loads. *Fire Safety Journal, Volume 85*, Pages 23-34.

NORMS

EN1990. *(2002). Basis Of Structural Design*

BS-EN1991-1-1 (2002). *Actions On Structures - Part 1-1: General Actions - Densities, Self-weight, Imposed Loads For Buildings.*

BS-EN1995-1-1 (2004). *Design of timber structures - Part 1-1: General - Common rules and rules for buildings.*

BS-EN1995-1-2 (2004). *Design of timber structures - Part 1-2: General - Structural fire design*

BS-EN1992 (2002): *Design of concrete*

BS-EN1992 (2002)*: Geotechnical design*

THESIS

GUITIYA Moise (2020). *Construction et Durabilité d'un logement en bois : cas d'un bâtiment r+2 dans la ville d'Obala.* Master in Civil Engineering defended at National Advanced School of Public Works Yaoundé.

Isaac NDUMBE JACKAI II (2019). *Structural analysis and design of connections in timber multi–storey buildings.* Master in Civil Engineering defended at National Advanced School of Public Works Yaoundé.

WAMBA KENFACK Anne Sinclaire (2020). I*nfluence of high temperature on the structural behaviour of steel structures and solutions: case of panzani building.* Master in Civil Engineering defended at National Advanced School of Public Works Yaoundé.

WEBOGRAPHY

- https://www.britannica.com/science/wood-plant-tissue/Thermal-properties. Page consulted on March 2022
- https://www.crtv.cm/2019/06/sonara-operations-suspended-following-fire-incident. Page consulted on March 2022
- https://greyandsanders.com/blog/post/features-on-wood. Page consulted on March 2022
- https://theconstructor.org/building/types-of-defects-in-timber/21521. Page consulted on March 2022
  https://fr.wikipedia.org/wiki/Bois_(mat%C3%A9riau_de_construction)#Propri%C3%A9t%C3%A9s_m%C3%A9caniques. Page consulted on April 2022
- https://www.cameroun24.net/blog/?pg=actu&ppg=1&pp=&id=43592. Page consulted on April 2022
- https://www.cameroon-tribune.cm/article.html/21978/fr.html/incendie-biopharma. Page consulted on April 2022
- https://www.naturallywood.com/products. Page consulted on June 2022

- https://fr.constructionreviewonline.com/plus-grands-projets/top-5-des-b%C3%A2timents-en-bois-les-plus-hauts-du-monde. Page consulted on April 2022
- https://continuingeducation.bnpmedia.com/article_print.php?L=312&C=1272#:~:text=Cross%20Laminated%20Timber%20(CLT)%20is,%2C%20dimensional%20stability%2C%20and%20rigidity. Page consulted on April 2022
- https://www.dataholz.eu/en/building-materials/beams-columns/parallel-strand-lumber-psl.html. Page consulted on May 2022
- https://worldpopulationreview.com/world-cities/yaounde-population. Page consulted on May 2022
- https://www.lemonde.fr/afrique/article/2017/11/17/spectaculaire-incendie-a-l-assemblee-nationale-camerounaise_5216423_3212.html. Page consulted on June 2022
- https://mfhwoodworking.wordpress.com/2021/01/13/timber-defects. Page consulted on April 2022
- https://en.wikipedia.org/wiki/Wood. Page consulted on May 2022
- https://worldpopulationreview.com/world-cities/yaounde-population. Page consulted July 2022
- https://www.europeanwood.org.cn. Page consulted on July 2022

ANNEXES

Annex .1 Analysis of most load beams in normal temperature (without fire)

a. Geometrical characteristics of the joist

The geometrical characteristics of the beams is presented in Table A.1

Table A. 1. Geometrical characteristics of the joist in normal temperature

Width b(m)	0,15
Height h(m)	0,30
Span L(m)	3,6
Moment of inertia $I_{g,x}$ (mm^4)	337500000
Neutral axis distance V (mm)	150
Section modulus W (mm^3)	2250000

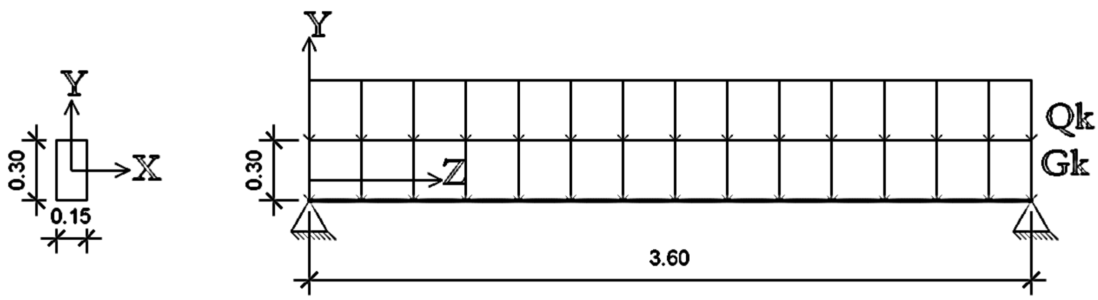


Figure A. 1. Geometrical Characteristic of the beams

b. Total load applied to the joist depending on the type of wood considered

The Table A.2 shows the different combined loads in ULS applied to the joist

Table A. 2. ULS loads applied to the joist depending on the type of wood considered

SPECIES:	Moabi	Fir
Joist load G_{k1} (kN\m)	1,451	0,750
Beams loads G_{k1} (kN\m)	0,384	0,198
Panels Load G_{k1} (kN\m)	0,604	0,313
Total load G_{k2} (kN\m)	5,672	5,672

Permanent Load on Joist ΣG_{ki} (kN\m)	8,111	6,934
Live load Q_k (kN\m)	7,09	7,09
Final joist load: $1.3G_k+1.5Q_k$ (kN\m)	21,179	19,649

c. Joist verification

i. Moment verification for the beam (ULS design)

The bending moments within the beam for both wood species are show in Figure A.2

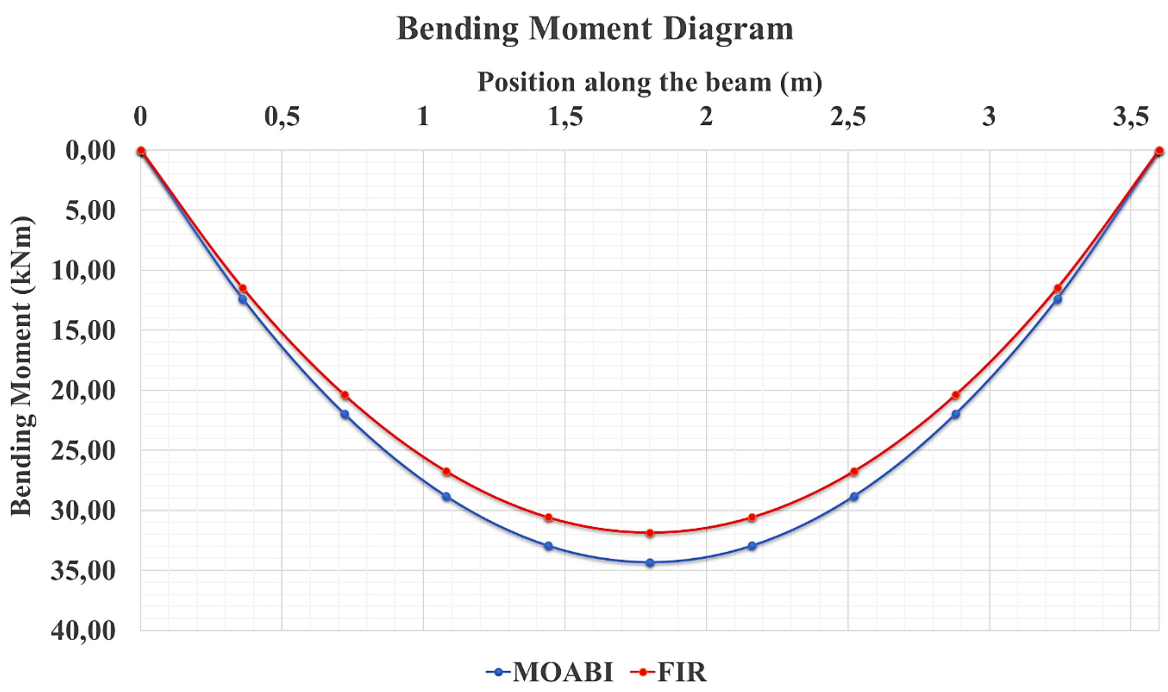


Figure A. 2. Bending Moment Diagram of beam

The previous bending moments obtained from the beams analysis were checked against the design values and the result is presented in Table 3.7

Table A. 3. Moment verification for the beam

SPECIES:	Moabi	Fir
Bending Moment Mx (kN.m)	34,31	31,83
Bending stress $\sigma_{m,d}$ (Mpa)	15,25	14,15
Bending strenght $f_{m,d}$ (Mpa)	96,8	58,2
Critical bending stress $\sigma_{m,crit}$(Mpa)	254,53	147,59

Relative slenderness for bending $\lambda_{rel,m}$	0,75	0,76
Lateral buckling factor K_{crit}	1	0,99
$K_{crit}.f_{m,d}$	96,8	57,62
Calculation of the work rate of joist	**0,16**	**0,26**
	Verified	**Verified**

ii. Shear verification for the beam (ULS design)

The shear forces within the beam for both wood species are show in Figure A.3

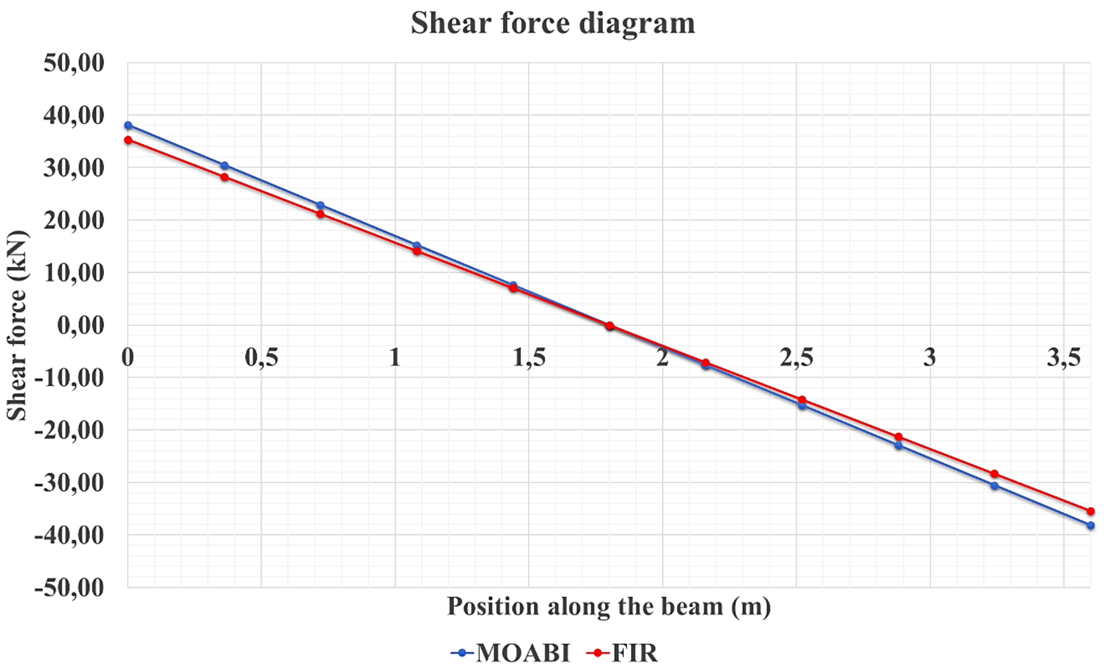


Figure A. 3. Shear force Diagram of beam

The shear forces obtained from the beam analysis were checked against the design values and the result is presented in Table A.4

Table A. 4. Shear verification for the beam

SPECIES:	Moabi	Fir
Shear force T_y (kN)	38,12	35,37
Shear stress τ_{ed} (MPa)	1,27	1,18
Shear strenght τ_{Rd} (MPa)	6,52	4,34
Calculation of the work rate of joist	**0,19**	**0,27**
	Verified	**Verified**

iii. Deflexion verification (SLS verification)

- Computation of the beams limits deflections allowed (Table 3.9):

Table A. 5. Beam limits deflections

$W_{inst,lim}$ (Q)	12	mm
$W_{net,fin,lim}$	18	mm

- Load combination used for deflection assessment (Table A.6)

Table A. 6. SLS loads applied to the joist depending on the type of wood considered

SPECIES:	Moabi	Fir
q_{inst} (Q) (kN\m)	7,09	7,09
q_{inst} (KN\m) G+Q	15,201	14,024
q_{creep}(KN\m) $K_{def}(G+\Psi_2 Q)$	6,143	5,436

- Evaluation of the deflection in respect to the limit deflection

Table A.7. gives the verification of the deflections for each wood species in relation to the limit values prescribed by the Eurocode calculated in Table 3.9

Table A. 7. Deflexions verification

SPECIES:		Moabi	Fir
Creep deflection W_{creep} (mm)	W_{creep} (G, Q) in mm	1,89	2,89
Instantaneous deflection W_{inst} (G, Q)	W_{inst} (G, Q) in mm	4,68	7,45
Instantaneous deflection W_{inst} (Q)	W_{inst} (Q) in mm	2,18	3,77
	$W_{inst}\backslash W_{inst,lim}$ in mm	**0,18**	**0,31**
		Verified	**Verified**
Final net deflection $W_{net,fin}$	$W_{net,fin}$ in mm	6,57	10,34
	$W_{net,fin}\backslash W_{net,fin,lim}$ in mm	**0,37**	**0,57**
		Verified	**Verified**

Note that the fire analysis of the beam follows the same procedure as the joist studied in section 3.2.3.2

Annex .2 Design of foundation in normal temperature

This section deals with the determination of the most stressed pile and footing sections and their reinforcement. Thus, for a compression force N = 242 kN at the base of the piles and a soil stress of 0.3 MPa, the pile and footing sections obtained are presented in Table A.8 and A.9, and their reinforcement in Table A.10

Table A. 8. Design of the pile section

Concrete pile design	
Characteristic compressive cylinder strength of concrete f_{ck} (MPa)	25
Design value of concrete compressive strength f_{cd} (MPa)	14,17
Axial load arriving on the pile N (kN)	242
Cross-sectional area of pile a(mm^2)	15374,12
Square pile side length in (mm)	139,91
Retained side length of pile c (mm)	140,00

Table A. 9. Design of the footing section

Footing design	
Bearing capacity of soil σ_{soil} (MPa)	03
Axial load arriving on the footing N (kN)	242
Cross-sectional area of footing A(mm^2)	806666,67
Square footing side length in (mm)	898,15
Retained side length of footing C (mm)	900,00

Table A. 10. Design of the pile and footing reinforcement section

Pile and footing reinforcement section	
Characteristic yield strength of reinforcement f_{yk} (MPa)	400
Design yield strength of reinforcement f_{yd} (MPa)	347,83
Axial load arriving on the pile N (kN)	242
Retained side length of pile c (mm)	140,00
Retained side length of footing C (mm)	900,00

Height of the footing d (mm)	250
reinforcement section of Pile A_{sp} (mm^2)	69,57
reinforcement section of footing A_{sf} (mm^2)	264,38

The Figure A.4 present the plane of foundation of the structure

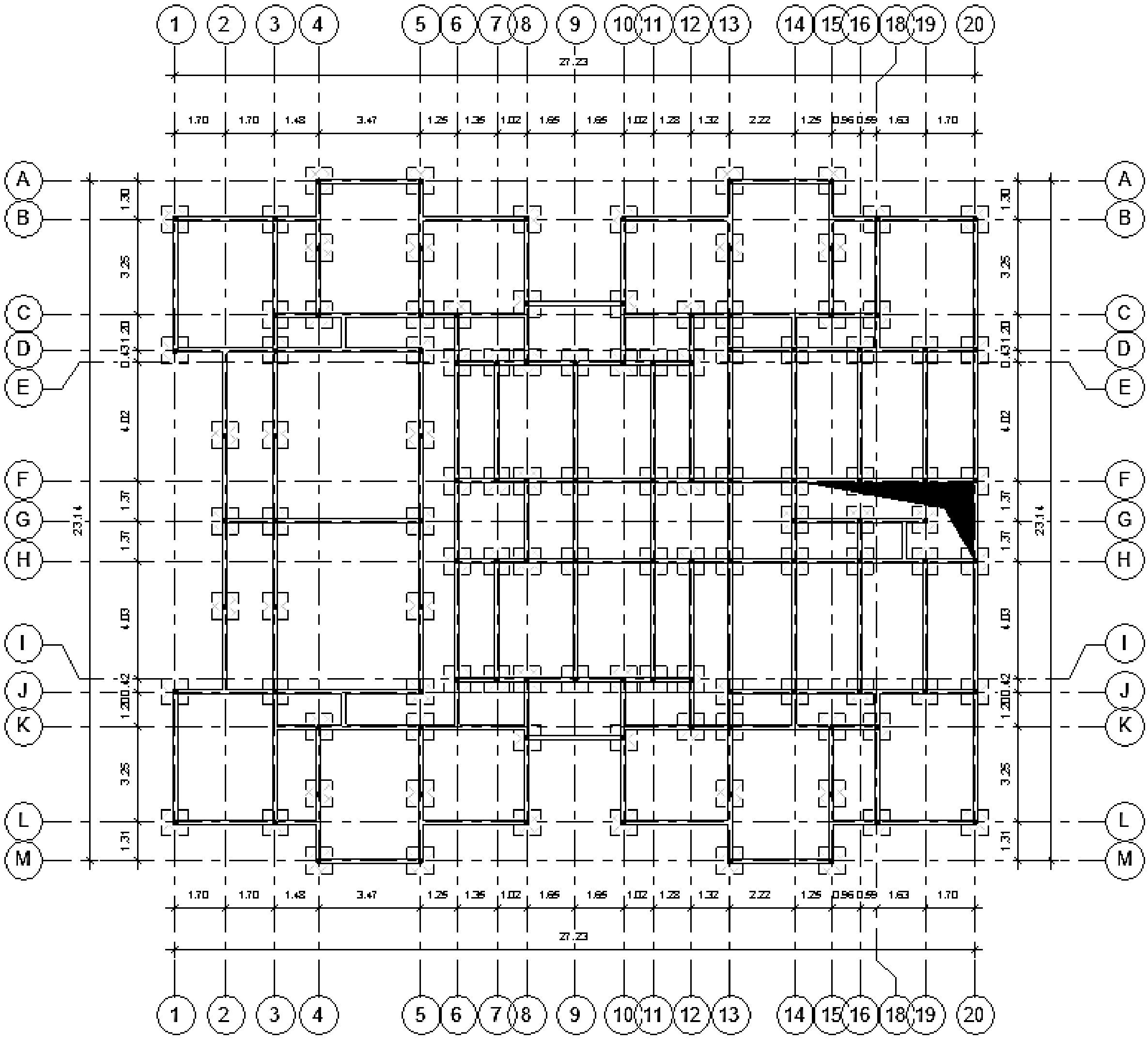


Figure A. 4. Plane of foundation

Annex .3 Building categories of use (BS-EN1991-1-1, 2002)

Category	Specific Use	Example
A	Areas for domestic and residential activities	Rooms in residential buildings and houses; bedrooms and wards in hospitals; bedrooms in hotels and hostels kitchens and toilets.
B	Office areas	
C	Areas where people may congregate (with the exception of areas defined under category A, B, and D[1])	**C1**: Areas with tables, etc. e.g. areas in schools, cafés, restaurants, dining halls, reading rooms, receptions. **C2**: Areas with fixed seats, e.g. areas in churches, theatres or cinemas, conference rooms, lecture halls, assembly halls, waiting rooms, railway waiting rooms. **C3:** Areas without obstacles for moving people, e.g. areas in museums, exhibition rooms, etc. and access areas in public and administration buildings, hotels, hospitals, railway station forecourts. **C4**: Areas with possible physical activities, e.g. dance halls, gymnastic rooms, stages. **C5**: Areas susceptible to large crowds, e.g. in buildings for public events like concert halls, sports halls including stands, terraces and access areas and railway platforms.
D	Shopping areas	**D1:** Areas in general retail shops **D2**: Areas in department stores
[1] Attention is drawn to 6.3.1.1(2), in particular for C4 and C5. See EN 1990 when dynamic effects need to be considered. For Category E, see Table 6.3		
NOTE 1 Depending on their anticipated uses, areas likely to be categorised as C2, C3, C4 may be categorised as C5 by decision of the client and/or National annex. NOTE 2 The National annex may provide sub categories to A, B, C1 to C5, D1 and D2 NOTE 3 See 6.3.2 for storage or industrial activity		

Annex .4 Safety factors for permanent and variable actions

Persistent and transient design situations	Permanent actions		Leading variable action (*)	Accompanying variable actions	
	Unfavourable	Favourable		Main (if any)	Others
(Eq. 6.10)	$\gamma_{Gj,sup}G_{kj,sup}$	$\gamma_{Gj,inf}G_{kj,inf}$	$\gamma_{Q,1}\ Q_{k,1}$		$\gamma_{Q,i}\psi_{0,i}Q_{k,i}$

(*) Variable actions are those considered in Table A1.1

NOTE 1 The γ values may be set by the National annex. The recommended set of values for γ are :

$\gamma_{Gj,sup} = 1,10$

$\gamma_{Gj,inf} = 0,90$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

NOTE 2 In cases where the verification of static equilibrium also involves the resistance of structural members, as an alternative to two separate verifications based on Tables A1.2(A) and A1.2(B), a combined verification, based on Table A1.2(A), may be adopted, if allowed by the National annex, with the following set of recommended values. The recommended values may be altered by the National annex.

$\gamma_{Gj,sup} = 1,35$

$\gamma_{Gj,inf} = 1,15$

$\gamma_{Q,1} = 1,50$ where unfavourable (0 where favourable)

$\gamma_{Q,i} = 1,50$ where unfavourable (0 where favourable)

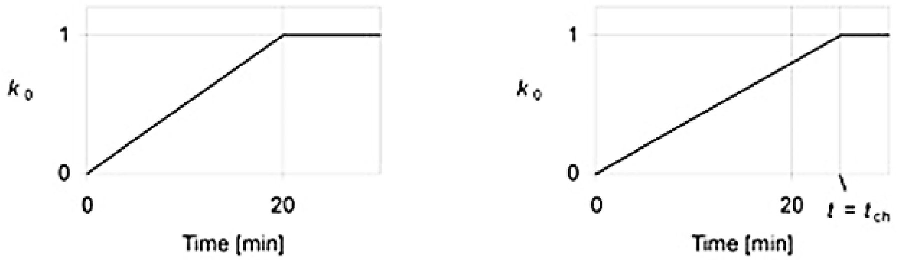
provided that applying $\gamma_{Gj,inf} = 1,00$ both to the favourable part and to the unfavourable part of permanent actions does not give a more unfavourable effect.

Annex .5 Value of k0

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

	k_0
t < 20 minutes	t/20
$t \geq$ 20 minutes	1.0

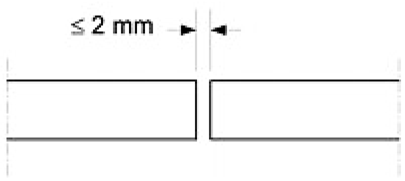
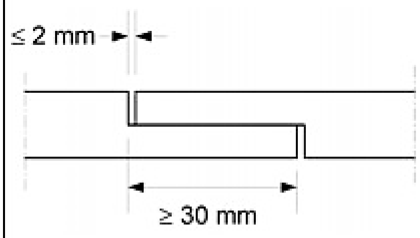
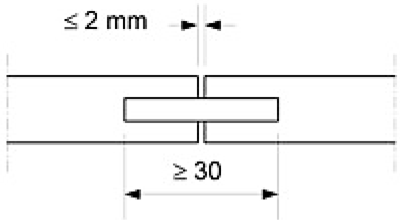
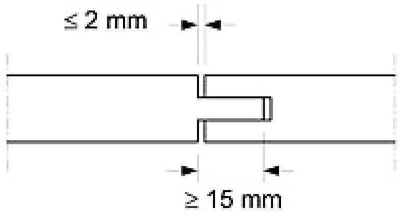
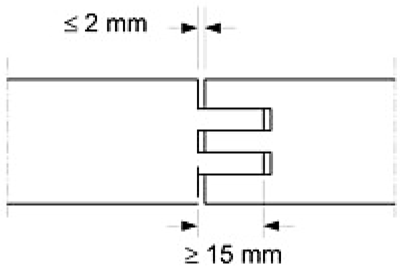
(3) For protected surfaces with t_{ch} > 20 minutes, it should be assumed that k_0 varies linearly from 0 to 1 during the time interval from t = 0 to t = t_{ch}, see figure 4.2b. For protected surfaces with $t_{ch} \leq$ 20 minutes table 4.1 applies.



a) b)

Figure 4.2 — Variation of k_0: a) for unprotected members and protected members where $t_{ch} \leq$ 20 minutes, b) for protected members where t_{ch} > 20 minutes

Annex .5 Joint coefficient kj to account for the effect of joints in wood-based panels which are not backed by battens (EN 1995-1-2:2004)

	Joint type	k_j
a		0,2
b		0,3
c		0,4
d		0,4
e		0,6

Annex .5 3D view of the structure