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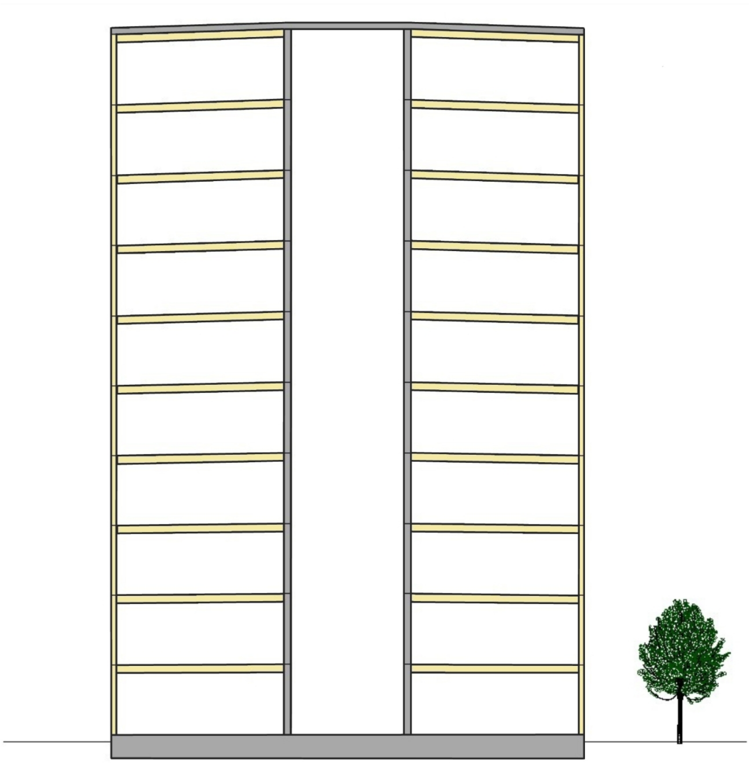

Long-term deformation behaviour of a Timber-Concrete Hybrid Structure

A parametric and analytical study of vertical deformation behaviour and differences between timber and concrete

Master's thesis in Master Programme Structural Engineering and Building Technology

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MASTER'S THESIS 2023

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Cover: Example building with 10 stories showing inclination of slabs from deformation differences between timber and concrete.

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Abstract

The population of Sweden increases every year, requiring more housing and a further developed infrastructure, thus densifying the cities. One solution to increase the amount of housing while preserving green areas within the city is to build taller buildings. The interest of using timber in buildings has increased in recent years, mainly due to its environmental benefits. However, a pure timber high-rise building present limitations and challenges which introduces a solution combining different materials. A building with a mix of different materials is called a hybrid building.

Timber-concrete hybrid structures present challenges. Aspects to consider are different material properties, construction processes, structural systems and their elements as well as joints between elements. The thesis work aimed to analyse vertical deformations in a tall hybrid building of mainly timber, with a concrete core. Deformation differences that arises when timber is connected to concrete have been investigated for time dependent loads and surrounding climate conditions. Possible solutions to minimize the differences have been suggested.

The work of the thesis started with a literature study to acquire understanding of the topic and to establish a basis for the analysis of vertical deformations. Structural components were dimensioned and based on the literature study and material models were created in MATLAB. Lastly, a parametric as well as a general study were performed on a fictitious building. The parametric study consists of six different scenarios and the result is showed with a plot of deformation over time for a bottom column or wall element.

The general analysis shows that differences of deformations between the timber and concrete components will occur along the height of the building. The total number of floors determines how large the differences will be and also where along the height the maximum difference will emerge. From the parametric study, the most efficient parameters to adjust to reduce the differences are the timber strength classes as well as the timber component dimensions.

Keywords: hybrid structure, timber, high-rise building, vertical deformations, long-term behaviour

Långtidsdeformationer i en Hybridbyggnad av Trä och Betong
En parametrisk och analytisk studie av vertikala deformationer och skillnader mellan trä och betong
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Sammanfattning

Befolkningen i Sverige ökar årligen. Det medför behov av fler bostäder och en vidareutvecklad infrastruktur, vilket förtätar städerna. En lösning till att öka mängden bostäder och samtidigt bevara grönområden i städerna är att bygga högre byggnader. Intresset för att använda trämaterial i byggnader har ökat de senaste åren, främst på grund av dess miljöfördelar. Ett höghus i rent trä innebär dock begränsningar och utmaningar som skapar efterfrågan på en lösning där olika material kombineras. En byggnad av olika material kallas hybridbyggnad.

Hybridstrukturer i trä och betong innebär utmaningar i sig. Aspekter att beakta är olika materialegenskaper, konstruktionsprocesser, bärande system och deras komponenter samt infästningar mellan komponenter. Detta examensarbete syftade till att analysera vertikala deformationer i en hög hybridbyggnad av huvudsakligen trä, med en betongkärna. Deformationsskillnader som uppstår när trä kopplas till betong har undersökts för tidsberoende laster och omgivande klimatförhållanden. Möjliga lösningar för att minimera skillnaderna har föreslagits.

Examensarbetet startade med en litteraturstudie för att få förståelse för ämnet inför analysen av vertikala deformationer. Bärande komponenter dimensionerades och utifrån litteraturstudien togs materialmodeller fram i MATLAB. Slutligen utfördes en parametrisk samt en allmän studie på en fiktiv byggnad. Den parametriska studien består av sex olika scenarier och resultatet visar deformation över tid i diagram för en pelare eller vägg i botten av byggnaden.

Den allmänna analysen visar att skillnader i deformationer mellan trä- och betongkomponenterna kommer att uppstå längs med byggnadens höjd. Totala antalet våningar avgör hur stora differenserna blir och även var längs höjden den maximala skillnaden kommer att framträda. Utifrån den parametriska studien konstateras det att de mest effektiva parametrarna att justera, för att minska skillnaderna, är hållfasthetsklasserna samt dimensionerna för träelementen.

Nyckelord: hybridkonstruktion, trä, höghus, vertikala deformationer, långtidseffekter

Preface

This master thesis marks the end of our time at Chalmers University of Technology, leaving us with a degree in Structural Engineering, cherished memories and friends to last a lifetime. We are very proud of all we have accomplished and look forward to what the future has to offer.

The thesis has been carried out at COWI in Gothenburg. We want to thank all employees at the Building Division for the warm welcome and the good energy in the group. We would like to give a special thanks to our two supervisors at COWI, Thomas Andersson and Adam Jonsson, for their guidance and support during the thesis work. All the time and effort they have extended our way has been priceless and we are very grateful.

We also want to thank Robert Jockwer, Associate Professor in timber structures at the Division of Structural Engineering, Chalmers University of Technology. His inputs and support along the way has been much appreciated and very important for the progress of our project.

Frida Dahlqvist, Isa Kollberg, Gothenburg, June 2023

List of Acronyms

CLT cross laminated timber

EWP engineered wood product

glulam glued laminated timber

JIT just in time

LVL laminated veneer lumber

MC moisture content

RH relative humidity

SLS serviceability limit state

TCC timber-concrete composite

ULS ultimate limit state

List of Symbols

A_c	cross-sectional area of concrete (gross section) [mm^2]
D	diffusion coefficient [mm^2/h]
E_c	elastic modulus of concrete [Pa]
E_{cm}	mean elastic modulus of concrete [GPa]
E_{ms}	stiffness of mechano-sorptive Kelvin-Voigt element [GPa]
$E_{t,0,mean}$	mean elastic modulus of timber, parallel to grain [MPa]
$E_{t.0}$	modulus of elasticity of timber at time t=0 [Pa]
G_k	characteristic value for permanent load [N]
Q_d	design value for variable load acting on element [N]
Q_k	characteristic value for variable load [N]
RH	ambient relative humidity [%]
$\Delta T(t)$	change of temperature [K]
$\Delta\sigma_i$	difference in stress from decreased or increased load [Pa]
$\Delta u(t)$	difference of moisture content at time t from t=0 [%]
Δu_{max}	maximum difference of moisture content in timber [%]
α_L	original hygroexpansion coefficient [%/%]
α_{Temp}	temperature expansion coefficient of timber [1/K]
α_{ds1}	coefficient depending on the type of cement [-]
α_{ds2}	coefficient depending on the type of cement [-]
$\bar{\alpha}_L$	adapted hygroexpansion coefficient [%/%]
$\beta(f_{cm})$	factor considering the concrete strength [-]
$\beta(t_0)$	factor considering the age when the concrete was loaded [-]
β_H	coefficient depending on the ambient relative humidity and the notional size of the section [-]
β_{MC}	surface emission factor [1/h]
β_{RH}	factor that considers the ambient relative humidity [-]
$\beta_{as}(t)$	time function of autogenous shrinkage [-]
$\beta_c(t, t_0)$	time function of the creep coefficient [-]
$\beta_{ds}(t, t_s)$	time function of drying shrinkage [-]
$\gamma_{g,SLS}$	partial safety factor for permanent load in SLS [-]
$\gamma_{g,ULS}$	partial safety factor for permanent load in ULS [-]
$\gamma_{q,SLS}$	partial safety factor for variable load in SLS [-]
$\gamma_{q,ULS}$	partial safety factor for variable load in ULS [-]
ψ_0	factor for combination value of a variable load [-]
ψ_1	factor for frequent value of a variable load [-]
ψ_2	factor for quasi-permanent value of a variable load [-]
ψ_i	factor for a variable load [-]

ρ_c	effective density of concrete with reinforcement [kN/m^3]
$\rho_{t,k}$	characteristic density of timber [kg/m^3]
σ_0	stress parallel to fibres [Pa]
$\sigma_c(t)$	stress in concrete at time t [Pa]
$\sigma_t(t)$	stress in timber at time t [Pa]
σ_{90}	stress perpendicular to fibres [Pa]
$\sigma_{t,c,0,d}$	design compressive stress in timber [MPa]
$\sigma_{t,c,0}$	compression stress in timber, parallel to grain [MPa]
$\sigma_{t,m}$	bending moment stress in timber [MPa]
$\sigma_{t,t,0}$	tension stress in timber, parallel to grain [MPa]
$\sigma_{t.0}$	stress in timber from initial load [Pa]
τ_k	creep factor [-]
$\varepsilon_{c.ca}(\infty)$	final value of autogenous shrinkage [-]
$\varepsilon_{c.ca}(t)$	autogenous shrinkage strain [-]
$\varepsilon_{c.cdi}$	starting value to determine the drying shrinkage strain [-]
$\varepsilon_{c.cd}(\infty)$	final value of drying shrinkage [-]
$\varepsilon_{c.cd}(t)$	drying shrinkage strain [-]
$\varepsilon_{c.cs}(t)$	shrinkage strain of concrete [-]
$\varepsilon_{c.c}(t, t_0)$	creep strain of concrete [-]
$\varepsilon_{c.el}(t)$	elastic strain of concrete [-]
$\varepsilon_{c.tot}(t, t_0)$	total strain of concrete [-]
$\varepsilon_{el.vs.ms}$	total strain in timber from elastic, visco-elastic and mechano-sorptive behaviour [-]
$\varepsilon_{t.T}(t)$	shrinkage/swelling strain in timber from temperature differences [-]
$\varepsilon_{t.el}(t)$	linear elastic strain in timber [-]
$\varepsilon_{t.ms}(t)$	mechano-sorptive strain in timber [-]
$\varepsilon_{t.sw}(t)$	shrinkage/swelling strain in timber from moisture content differences [-]
$\varepsilon_{t.tot}(t)$	total strain in timber from long-term effects [-]
$\varepsilon_{t.vs}(t)$	visco-elastic strain in timber [-]
$\varphi(t, t_0)$	creep coefficient [-]
φ_k	characteristic retardation time [-]
φ_0	notional creep coefficient [-]
φ_{RH}	factor considering the relative humidity [-]
$f_{c,0,k}$	characteristic compressive strength of timber, parallel to grain [MPa]
$f_{c,90,k}$	characteristic compressive strength of timber, perpendicular to grain [MPa]
f_{ck}	strength class of concrete [MPa]
f_{cm}	mean compressive strength of concrete at an age of 28 days [MPa]
$f_{m,k}$	characteristic bending strength of timber [MPa]
$f_{t,c,d}$	design compressive strength of timber, parallel to grain [MPa]

$f_{t,m,d}$	design bending strength of timber [MPa]
$f_{t,t,d}$	design tension strength of timber, parallel to grain [MPa]
g_k	characteristic value for permanent load [N/m]
h_0	notional size [mm]
k_c	strength related reduction factor [-]
k_h	coefficient depending on the size of the section [-]
q_k	characteristic value for variable load [N/m]
t_0	age of concrete when load is applied [days]
t_s	age of the concrete when drying shrinkage starts (normally at the end of curing) [days]
t	actual age of material [days]
$u(t)$	average moisture content at time t [%]
u_{RH}	equilibrium of moisture content in timber [%]
u_{max}	Maximum moisture content [%]
u_{min}	minimum moisture content [%]
u	perimeter of cross-section which is exposed to drying [mm]
w_{creep}	creep deflection [mm]
w_c	precamber deflection (if applied) [mm]
w_{fin}	final deflection [mm]
w_{inst}	instantaneous deflection [mm]
$w_{net,fin}$	net final deflection [mm]

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1 Introduction

The introductory chapter introduces the project with a background and problem description, explains aim and objectives and presents the chosen method as well as limitations of the study.

1.1 Background

The population of Sweden increases every year and according to Statistics Sweden (SCB) the population size is approximated to increase with 50,000 people per year (SCB, 2022). An increasing population requires more housing and a further developed infrastructure, thus densifying the cities. Meanwhile, the effect of global warming results in a need of stricter usage of resources such as land and materials. In addition, the importance of preserving green areas for recreation and biodiversity within the cities counteract the sprawl of the cities into all available gaps between buildings. One solution to increase the amount of housing while preserving green areas within the city is to build upwards, i.e., taller buildings.

The interest of using timber in buildings has increased in recent years. The increase of interest is mainly due to the environmental benefit with timber being a renewable material as well as the timber industry having lower climate change impact in comparison to steel and concrete (Skullestad et al., 2016). However, a pure timber high rise building present limitations and challenges. For example, its rather high strength-to-weight ratio can lead to a tall building being too light for its size, i.e., allowing wind loads to cause a swaying motion of the building as well as risk of negative consequences from seismic action (Ramage et al., 2017). To increase the structural mass of the building and providing horizontal stability, concrete could be used where it would be more efficient compared to timber, for example in the core. A building with a mix of different materials is called a hybrid building. The hybrid structure allows for designing and creating a structure where strengths from different materials are utilized. Introducing concrete in a timber structure will not only brace the structure horizontally but also improve the dynamic response. It will also widen the possibilities of building high, which might encourage the industry to choose timber as a primary building material, and thus decreasing the environmental impact of construction.

1.2 Problem description

Hybrid structures have several positive effects, but also challenging aspects to consider when two different materials are interacting. One aspect to consider is how the properties of different materials will affect the vertical deformations. Namely how the outcome will vary depending on the mixture of concrete and type of timber material, i.e., different stiffnesses and different long-term behaviours. Two long-term factors are creep and shrinkage, both of which are material dependent.

Another aspect to consider is the construction phase. Either the entire core is cast first to full building height and thereafter the wooden structure is assembled around the core (serial process), or each level is built simultaneously, with one level of concrete core followed by one level of timber structure (parallel process). The two processes enables the possibility of adjustments differently which will affect the final result.

Depending on if the vertical load bearing timber elements are columns or walls, the vertical deformations will differentiate. This as columns will have a concentrated applied load compared to the distributed load along the walls, as well as the general difference between engineered wood products. Additionally, the joints between structural elements will impact the final deformations. When considering all the mentioned aspects, problems such as inclination of floor levels might appear.

1.3 Aim and objectives

The aim of the thesis work is to analyse vertical deformations in tall hybrid buildings of mainly timber, with a core of concrete. Deformation differences that arises when timber is connected to concrete are to be investigated for time dependent loads and surrounding climate conditions. Additionally, possible solutions to minimize the differences are to be suggested.

In order to achieve the aim, a general as well as a parametric study is to be performed on a fictitious case study building. The building has a core of concrete and a remaining structural system of timber. The general study investigates different heights of the building and their impact on deformations. Regarding the parametric study, the following parameters are to be analysed:

- Vertical load bearing timber systems and elements.
- Concrete mixtures and different timber materials.
- Dimensions of timber elements.
- Construction process: Serial process or parallel process.
- Initial moisture content of timber products.

1.4 Method

The thesis work starts with a literature study to acquire understanding of the topic and to establish a basis for further analysis of vertical deformations. The literature study covers relevant materials and their properties as well as structural systems and constructions processes suitable for a hybrid structure. Existing projects of timber and timber-concrete structures are studied for further understanding and inspiration of tall buildings. Standards and guidelines according to Eurocode, accounting for long term effects, are also included in the literature study. Material models are created for each material based on standards and guidelines.

Timber elements are dimensioned through an iterative process to optimize the material usage and the conception of the building. A parametric study is carried out in the software MATLAB and is based on material models where time and moisture dependent behavior is analysed. The study investigates how the final vertical deformations are affected by different parameters. Furthermore, a general analysis of the fictitious building is carried out, for timber as well as for concrete, where vertical deformations for each level are calculated and compared with each other. The analysis is conducted for three different building heights. Through the parametric study and the general analysis, as well as the literature study, conclusions regarding long-term effects in mixed material buildings are made.

1.5 Limitations

The focus of the thesis is to conduct a parametric study as well as a general analysis of a hybrid high rise building of timber and concrete, only studying vertical deformations. The analyzes are conducted on a fictitious building of 20mx20m with a concrete core in the centre. The building is located in Gothenburg, Sweden, and the construction of the building starts January 1st. Its occupants are office workers, hence the building is adapted to these conditions. Regulations are according to Swedish standards.

Timber components such as columns, beams and walls are dimensioned in ultimate limit state (ULS) and serviceability limit state (SLS), considering compressive and bending capacity, as well as buckling and deflection. Slab, core and roof element dimensions are approximated based on the literature study. As vertical deformations are of interest in this study, no dynamic responses are accounted for. With the centered core, the building is provided with horizontal stability. All connections in the building are assumed to be without errors.

1.6 Ethical, ecological and social aspects

Ethical and ecological aspects are linked to the thesis work through the United Nations' Global Goals for Sustainable Development. For example, biodiversity, preserving ecosystems and green areas for recreation in the cities by building higher as well as the environmental aspects of implementing more timber in structures. Social aspects are not included in this thesis as the focus is on deformations in high rise hybrid structures.

2
Construction material

The construction materials relevant in this thesis work are timber and concrete. In this chapter, each material is thoroughly introduced and their properties are highlighted. Different timber products suitable for the project are presented.

2.1 Timber

Timber is a building material that has been of great importance for a long time. The material is well known for its great quality of being a renewable resource which distinguishes it from two other commonly used building materials, concrete and steel. Another great and important quality of timber is its ability of storing carbon dioxide, amplifying its importance in an environmental point of view (Fröbel & Bergkvist, 2020).

Timber is an orthotropic material, meaning it has three main directions with varying material characteristics (Swedish Wood, 2016b). The three directions are Longitudinal (L), Tangential (T) and Radial (R), see Figure 2.1. Although, the difference in behaviour tangentially and radially is usually disregarded as they are very similar, leaving the directions to be expressed as parallel and perpendicular to grain, σ_0 and σ_{90}. Shortly, this entails that the material will behave differently depending on which direction is loaded, with timber generally having high strength loaded parallel to grain and very low strength loaded perpendicular to grain. The mechanical properties will vary between different engineered wood products (EWPs) as well as within the same type of timber product. The variations within the same type depend mainly on the surrounding conditions during the growth of the tree and are usually shown as differences in density, stiffness and strength. These values are determined through grading.

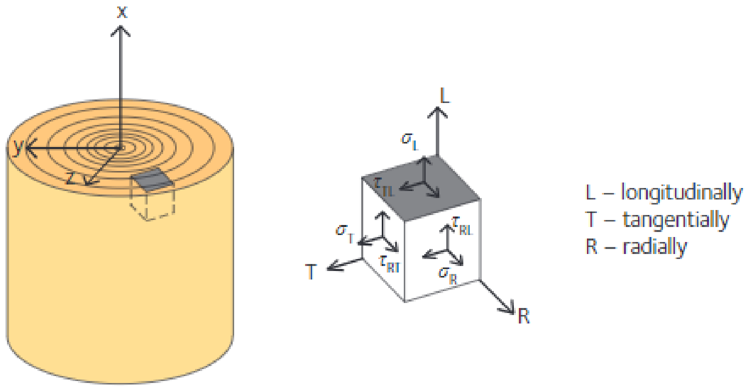


Figure 2.1: Longitudinal, tangential and radial direction of wood. From *Design of timber structures: Structural aspects of timber construction, Volume 1*, 2016b, Swedish Wood.

Factors influencing the mechanical properties of timber are i.a., moisture, temperature and time, etc (Swedish Wood, 2016b). The moisture content (MC) of timber, until reaching its saturation point, will affect the strength and stiffness. The ratio is usually linear where low MC equals higher strength and stiffness. Regarding the influence of temperature, the strength and stiffness of timber will increase with decreasing temperature. Of major impact on the properties is the time factor where the loading period has shown to greatly affect the bending stiffness by it decreasing with increasing time of loading.

Knots in timber are formed from branches on trees (Fröbel & Bergkvist, 2020). In sawn timber boards, these knots will influence the quality of the timber as the knots creates discontinuity of the fibres, making the area surrounding the knot weaker. The determined strength class of the sawn timber depends on the quantity of knots, their appearances and position. The strength classes differentiate depending on the EWP. EWPs are beneficial in several aspects as they i.a. more efficiently can utilize the material compared to solid timber (Thelandersson & Larsen, 2003). They also allow for many different sizes and can thereby be adapted to market requirements. Different EWPs are presented in the following sub chapters.

2.1.1 Structural timber

Structural timber, or sawn timber, is pure solid timber with no modification done to its material properties, i.e., not an EWP but still commonly used in the industry. In Sweden, trees are normally harvested after approximately 80 years of growth (Swedish Wood, 2016b). The retrieved saw logs are after harvesting transported to a sawmill where they are graded based on quality, length and diameter. Finally, when the logs are cut, the specimen are each evaluated and strength graded to be placed into different strength classes. The properties of the material can vary quite significantly depending on which part of the tree is utilized and if anomalies such as knots are present. Therefore, the grading of structural timber is done by visual and mechanical strength testing. The strengths of structural timber are divided into classes which ranges between C14-C40, where the number represents the bending strength in MPa. Due to structural timber being unmodified and only conducted of pure timber, cross-sections are bound to the size of the tree. In Sweden, standard cross-sections are normally limited up to 100x225mm, with the standard initial MC of 12%.

2.1.2 Glulam

Glued laminated timber (glulam) is the oldest form of EWP (Swedish Wood, 2016b). The product consists of finger jointed laminated structural timber boards, glued together, with the fibre direction in the axial direction, see Figure 2.2 (Svenskt Trä, 2018a). Commonly used in Sweden, in glulam products, is spruce as timber material with as adhesive. MUF is a synthetic glue which, according to SS-EN 14080 (2013), fulfils Climate Class I-III, meaning that the glue is climate independent (Fröbel & Bergkvist, 2020).

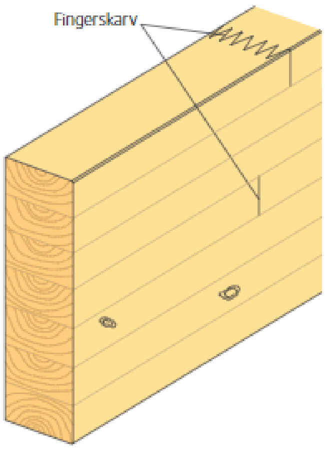


Figure 2.2: Part of glulam beam showing the laminated boards and the finger joint. From *Limträhandbok Del 1 - Fakta om limträ*, 2018a, Svenskt Trä.

The most common cross-section shape of a glulam member is rectangular. The height of straight cross-sections are usually multiples of 45 mm, the normal thickness of a single lamella, and the widths are regulated to standards in the sawmills, normally 90, 115, 140, 165, 190 and 215 mm (Svenskt Trä, 2018b). The widths are limited to 215 mm, sometimes 240 mm, as lumber of greater width is unusual. If larger widths are desired, an option is joining smaller timber components.

Glulam elements are produced straight or with a curvature and the high strength to weight ratio enables large spans making glulam a suitable choice as a structural element (Swedish Wood, 2016b). Due to the effect of lamination, a glulam element has higher effective strength compared to structural timber of the same size. Higher effective strength is achieved through homogenisation by defects such as knots being distributed along the glulam beam and its layers. This compared to a solid timber beam where a knot can constitute a larger part of a cross section.

The different glulam strength classes are sorted as GL22h-GL32h and GL22c-GL32c as well as GL28hs and GL28cs (Svenskt Trä, 2018a). The existing classes in Sweden are GL28hs, GL28cs, GL30c and GL30h, with GL30c being the dominating one. The number in the strength class corresponds to the characteristic value of bending strength in MPa, for example where the lamellas of GL30h corresponds to the strength class C30 of structural timber, whilst the letter explains whether the glulam is combined (c), homogeneous (h) or split/resawn (s). A combined (c) glulam beam consist of at least four lamellas with the lamellas having different strength classes. Lamellas with higher strength are placed at the outer edges whilst lower strength lamellas are placed in the mid zone of the beam. Homogeneous (h) glulam beams consists of at least three lamellas, all with the same strength class. Resawn (s) glulam beams are split from wider glulam beams. According to Swedish standard SS-EN 14080 (2013) (referred to in Svenskt Trä, 2018a), the strength class of the beam will be downgraded when resawn, e.g., GL28hs consists of corresponding structural member with strength class C30. The standard initial MC of glulam is 12%.

Glulam is not only beneficial regarding its somewhat evenly distributed strength and geometrical possibilities but is also visibly pleasing, often leaving it exposed as an architectural feature (Thelandersson & Larsen, 2003).

2.1.3 CLT

Cross laminated timber (CLT) is an EWP consisting of sawn timber boards glued on top of each other, layered alternately perpendicular to the adjacent layers, see Figure 2.3 (Swedish Wood, 2016b). CLT products are made with an odd number of layers (minimum three) resulting in the top and bottom layers being oriented in the same direction. With the alternating layers, the variation of strength and stiffness between each layer are concluded to balance each other out, providing an overall higher strength (Swedish Wood, 2019). Spruce and pine are the most common choices of wood species for CLT used in Sweden. The material properties for strength graded CLT products ranges between C14-C30 and the CLT panel can either consist of one strength class or a combination of several. CLT products have a standard initial MC of 12%.

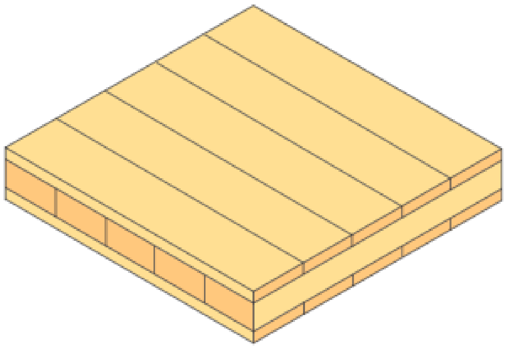


Figure 2.3: A CLT board illustrating the cross laminated layers. From *Design of timber structures: Structural aspects of timber construction, Volume 1*, 2016b, Swedish Wood.

cross laminated timber products allows for many different shapes and sizes, but are currently most commonly used as wall or slab components (Swedish Wood, 2019). A CLT panel is composed of 20-60 mm thick planks or boards. Panels are available in various sizes, with the largest dimension being 0.5 m thick, 4.8 m wide and 30 m long. Furthermore, CLT enables large cross-section areas which entails high strength and stiffness making cross laminated timber panels well suited as bracing of a building. CLT, compared to other structural materials, has a very high strength to weight ratio, qualifying the EWP as a structural material for high-rise structures. Moreover, the possibilities of prefabrication are great and the low weight of CLT enables smooth transport and assembly.

2.1.4 LVL

Laminated veneer lumber (LVL) is an EWP normally used as load carrying columns, beams and panels in buildings and other structures (Fröbel & Bergkvist, 2020). The product is usually made of spruce or pine and consists of at least five layers of laminated veneer sheets, with the fibres in all layers normally oriented in the longitudinal direction of the beam, see Figure 2.4. Generally, LVL members have high strength regarding bending, tension, compression and shear (Swedish Wood, 2016b). In some cases, when mechanical properties in numerous directions are wished to be utilized, some sheets can be placed perpendicular to the length of the beam. This is beneficial in for example slab structures where also high stiffness across the panel is desired.

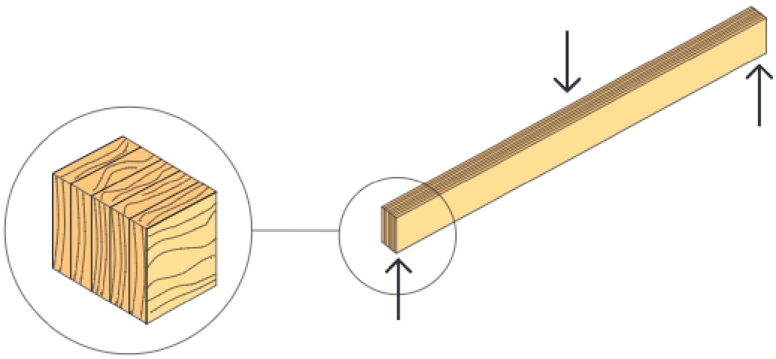


Figure 2.4: A laminated veneer lumber beam. From *Design of timber structures: Structural aspects of timber construction, Volume 1*, 2016b, Swedish Wood.

Similarly to glulam, through the effect of lamination, LVL is a reliable EWP with less varied strength and stiffness along the beam compared to a solid timber beam (Thelandersson & Larsen, 2003). LVL are produced as two types, Kerto-S and Kerto-Q, with different material properties (Swedish Wood, 2016a). The thickness of Kerto S, S referring to all veneers being oriented in the same direction, ranges between 21-90 mm. Meanwhile, Kerto Q consists of some veneers being cross grained and are available in thicknesses 21-69 mm, where the properties differ depending on the thickness of the product. A LVL beam is available with a depth of up to 600 mm and a length of approximately 24 m (Canadian Wood Council, n.d.). The standard initial MC in a LVL product is 6%.

2.2 Concrete

Concrete is a material consisting of a mixture of cement, aggregates, water and admixtures (Al-Emrani et al., 2019). Admixtures can significantly modify the properties of concrete and are therefore added to achieve desired characteristics. Concrete has a very high compressive strength whilst the tensile strength is relatively low, measured to approximately 10% of the compressive strength. The material is divided into different strength classes that ranges between C12/15-C90/105, with the first and second number representing the characteristic compressive cylinder strength in MPa as well as the characteristic cube strength in MPa (Al-Emrani et al., 2019).

Cracking is inevitable when it comes to concrete and can not be prevented due to its low tensile strength. However, the addition of steel reinforcement can controllingly distribute applied loads and thus control cracking of the concrete (Al-Emrani et al., 2019). The reinforcement is anchored straight in the concrete resulting in loads being transferred from the concrete to the steel rebars by bond and contact pressure. By controllingly assure that crack formations in the early stages remains acceptably small and spread out, problems such as corrosion of the reinforcement can be prevented.

Prestressing is another way of controlling cracking (Al-Emrani et al., 2019). Prestressing entails the rebars being subjected to tensile forces during production. The compressive stresses in the concrete from the prestressing will be relieved progressively in step with increasing load. The tensile stresses occurring in the concrete from loading, the stresses that causes cracking, are thereby delayed. Either the prestressing is done before or after anchorage, i.e., pretensioned or post-tensioned.

Concrete is a world wide dominating structural material. Its endless possibilities in shapes and sizes, when combined with rebars, allows for many different structural elements. The material is very durable and is available all over the world. It can either be constructed as prefabricated elements or cast at site. Additionally, it is also relatively easy to handle and is considered being a cheap material. Although, from a negative perspective, concrete is a rather heavy material compared to its strength and is weak when loaded in tension. Concrete is an ideal material from a climate-resilience perspective, but from an overall climate perspective it is not, this due to its large carbon footprint. The amount of carbon dioxide that is released during production of cement answers for approximately 8% of human caused global emissions (Ellis et al., 2020).

3
Construction

In this chapter, common structural systems suitable for tall timber buildings are introduced. Moreover, timber components and connections are briefly presented as well as the construction and production process. Since the building is a hybrid of timber and concrete, the response to loads will differentiate and the final deformations will be different. This results in the need of tolerances which is also mentioned in this chapter. Finally, some relevant information on existing and future timber/hybrid buildings is summarized.

3.1 Structural systems in timber

The structural system of a building determines how loads are handled and transferred to the foundation. The system should be designed according to load carrying capacity, stability and performance (Al-Emrani et al., 2019), where verification of the structural elements are to be done in limit state according to Swedish regulations. The limit state is divided into SLS, verifying that the building fulfills requirements of function regarding normal usage, and ULS, verifying the ultimate load carrying capacity.

A braced structure is a system where lateral loads are resisted by bracing elements, stabilizing the structure (Al-Emrani et al., 2011). Some examples of bracing units are cores and shear walls. However, in this thesis, the considered primary bracing system is a concrete core.

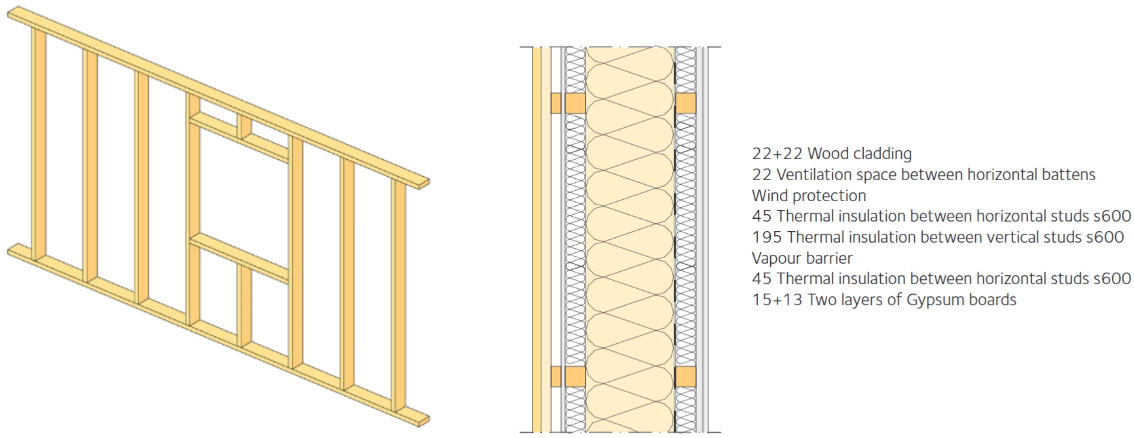
Choosing a structural system for a specific building is based on what type of building is to be constructed. The most suitable system will differentiate depending on requirements, which in turn differs between e.g., small houses and tall office buildings. The structural system of a timber building can be categorised as three different systems: panel system, modular system and beam-column system (Swedish Wood, 2016b).

3.1.1 Panel system

A panel system in timber entails plane elements such as wall and floor elements, utilizing a system of light frames or solid timber elements (Swedish Wood, 2016b). Both systems are built on the same principle, with the walls resisting vertical and horizontal loads and the floors distributing the horizontal loads to the bracing walls through diaphragm action. These two different systems can be utilized in modular systems, which are prefabricated box units containing walls, slabs, inner cladding, ceiling, and installations (Swedish Wood, 2016b). Although, this type of structural system is not considered in this project.

3.1.1.1 Light frame panel system

A light frame panel system consists of light frame wall elements, see Figure 3.1. Light frame wall elements are usually made of structural timber, but can also be made of glulam and LVL. The wall element is composed of equally distanced vertical lightweight studs, clad with panels for stabilization and with the gaps being filled with insulation (Swedish Wood, 2016b). The system is most commonly used in single-family houses but can be utilized in taller buildings, but with a limit of 6-8 levels due to difficulties with deformations perpendicular to grain.

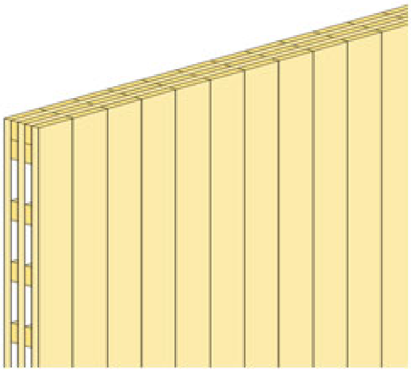


(a) 3D-view of wall element. **(b)** Section view of outer wall element.

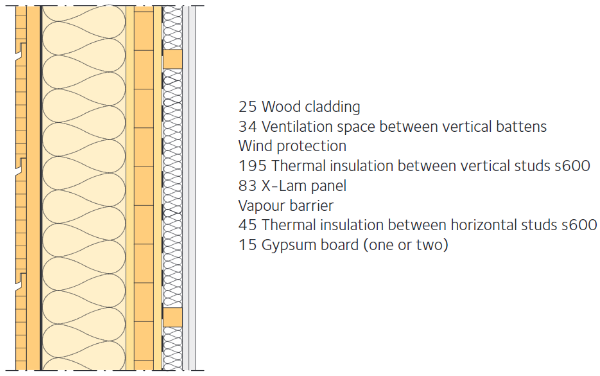
Figure 3.1: Light frame wall element. From *Design of timber structures: Structural aspects of timber construction, Volume 1*, 2016b, Swedish Wood.

3.1.1.2 Solid timber panel system

Solid timber panel systems consist of solid timber wall elements, see Figure 3.2. The wall element is normally composed of CLT but is sometimes also as a mechanically assembled element with nails (TräGuiden/Svenskt Trä, 2003). Whenever the wall element is an outer wall it should be complemented with an light weight stud layer with insulation on the outside as well as facade, doors and windows (Swedish Wood, 2016b). The solid timber wall element provides more stability to a structure compared to a light frame wall.



(a) 3D-view of wall element.



(b) Section view of outer wall element.

Figure 3.2: Solid timber wall element. From *Design of timber structures: Structural aspects of timber construction, Volume 1*, 2016b, Swedish Wood.

3.1.2 Beam-column system

Beam-column systems are commonly used in industrial as well as commercial and office buildings, where open spaces are desired (Swedish Wood, 2016b). The system is a grid system composed of beams and columns resisting vertical loads. The horizontal loading is resisted by diaphragm action in the slabs as well as by diagonal bracing or shear wall action. Using glulam in the system enables spans of 80-100 meters. Although, large spans as such are mostly used in spaces allowing for beams with large cross sections, i.e., bandy and ice hockey arenas. Figure 3.3 shows a column-beam system.

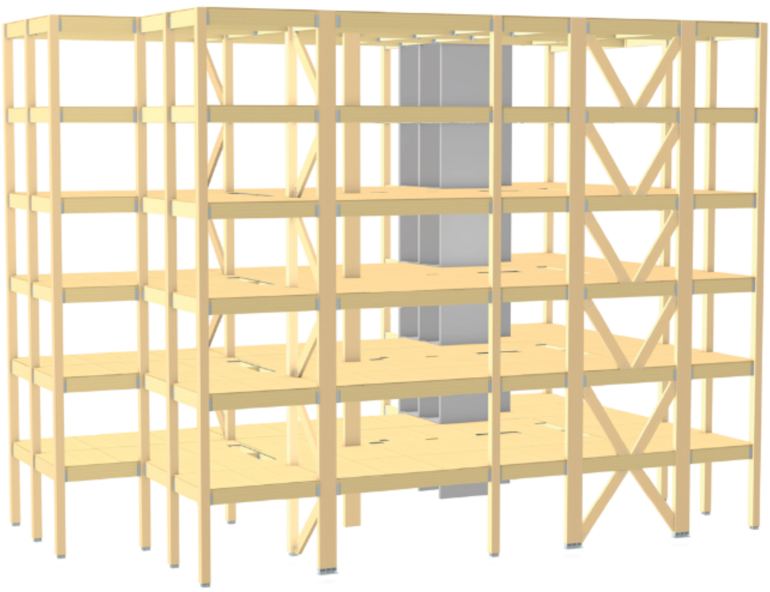


Figure 3.3: Beam-column system. From *Flervåningshus Trä8*, n.d., Moelven.

3.1.2.1 Timber columns

Timber columns are usually part of a primary structural beam-column system (TräGuiden/Svenskt Trä, 2019b). The columns do normally have a rectangular or circular cross-section, rectangular being the most common one, and are usually composed of glulam or structural timber. Glulam enables larger cross-sections compared to structural timber, i.e., higher load carrying capacity, and a variation of different shapes if desired. Columns are axially loaded components often designed rather slender and thus prone to buckling (Swedish Wood, 2016b). It is therefore crucial to consider stability when designing columns.

3.1.2.2 Timber beams

Timber beams are often composed of solid sections of structural timber, glulam or LVL (TräGuiden/Svenskt Trä, 2019a). Structural timber is usually used for smaller spans because longer spans require larger cross-sections. Glulam and LVL are more suitable for spans larger than five meters. This as both of them allows for larger cross-sections, with LVL providing slender beams with larger height, where the size of the elements primarily being limited to transportation possibilities. Light weight beams, normally a combination of timber boards and solid wood produced as I-sections and box-sections, are also suitable for larger spans (TräGuiden/Svenskt Trä, 2020). Beams are designed in ULS, loaded with bending moment, and verified in SLS according to Swedish requirements regarding deflection.

3.1.3 Connections

An important part of the structural system is the connection between different components. The different solutions of connections are many, but with timber being an orthotropic material, there are also several aspects to take into consideration when choosing a specific connection. With timber having very low compressive strength perpendicular to grain it is important to avoid transferring vertical loads through joints subjected to compression stresses in this direction (WoodWorks, 2021). If this cannot be avoided, the connection should be bridged with steel components.

Another aspect to consider when designing connections is the interface of tolerances between different relevant materials. Tolerances for timber are usually quite small whilst for other materials such as steel and concrete, the tolerances are larger (WoodWorks, 2021). When working with different materials, like in a hybrid building, it is necessary and important to account for these differences of fabrication tolerances. There are three different connections that are of interest for this project; column to column, column/wall to beam and timber component to core.

3.1.3.1 Column to column

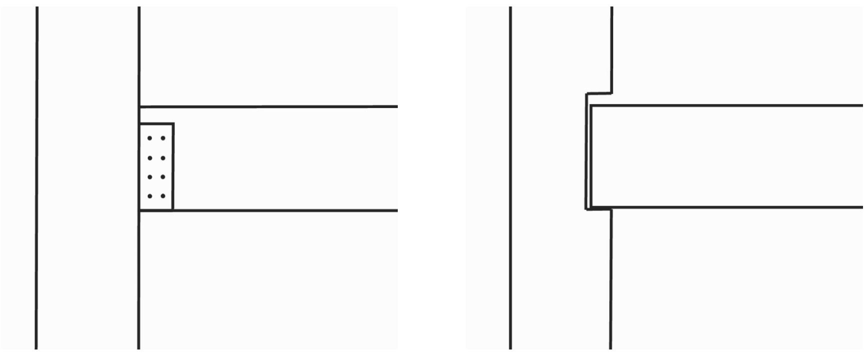
Normally, when a structural system consists of columns, the columns are continuous over at least two storeys. In some cases, the columns can even be designed as continuous throughout the entire height of the building. Continuous columns are preferred over discontinuous ones due to the high costs associated with splicing columns.

The joint between columns often consist of a steel plate with concealed brackets but the elements can also be placed directly on top of each other. If the joint includes a steel component, the bracket is inserted into pre-sawn notches in each column, connecting them, and thereafter fastened with dowels. Wall elements, compared to columns, are normally produced as single story panels. The connection between walls is the same as for columns.

3.1.3.2 Column/wall to beam

There are several ways of connecting beams to columns/walls. Which type of connection is used depends on architectural preferences as well as load types (Kolb, 2008). Various suggestions of connections for beams include continuous and oversailing beams, compound columns, forked connections, etc. For taller buildings, the most suitable and common way of connecting beams to columns/walls are with beams spanning between the vertical elements. This to avoid compressive stresses perpendicular to grain.

The connectors are usually mechanical fasteners such as sheet metal components. One connection type is a joist hanger, also know as beam shoe, where the beam is hung and fastened to the column/wall through a metal hanger, see Figure 3.4a. Concealed connections are also commonly used in column/wall to beam connections. Examples of such are concealed brackets and concealed hook plates. Another column to beam connection is the beam being supported on a carved out notch in the column, see Figure 3.4b. This solution lets the column support the beam without any mechanical fastener and without the occurrence of compressive stresses perpendicular to grain. Although, this type of connection will reduce the cross-section of the column where the beam is attached, leading to higher compressive stresses in the column and thereby larger deformations. This connection is only appropriate to use if the cross-section is large enough to carry the load through the reduced cross-section.



(a) Metal hanger. **(b)** Notch.

Figure 3.4: Column to beam connection details.

3.1.3.3 Timber component to concrete core

The connection between slab or beam to core is complicated due to the difference of materials in the core and the rest of the structure. First of all, when a timber element (slab or beam) is connected to a concrete element (core), it is important to provide a moisture barrier between the timber and the concrete to prevent moisture from penetrating the timber (Svenskt Trä, 2018c). It is also important to design a detail with the capacity of transferring vertical as well as the horizontal loads from the slab to the core.

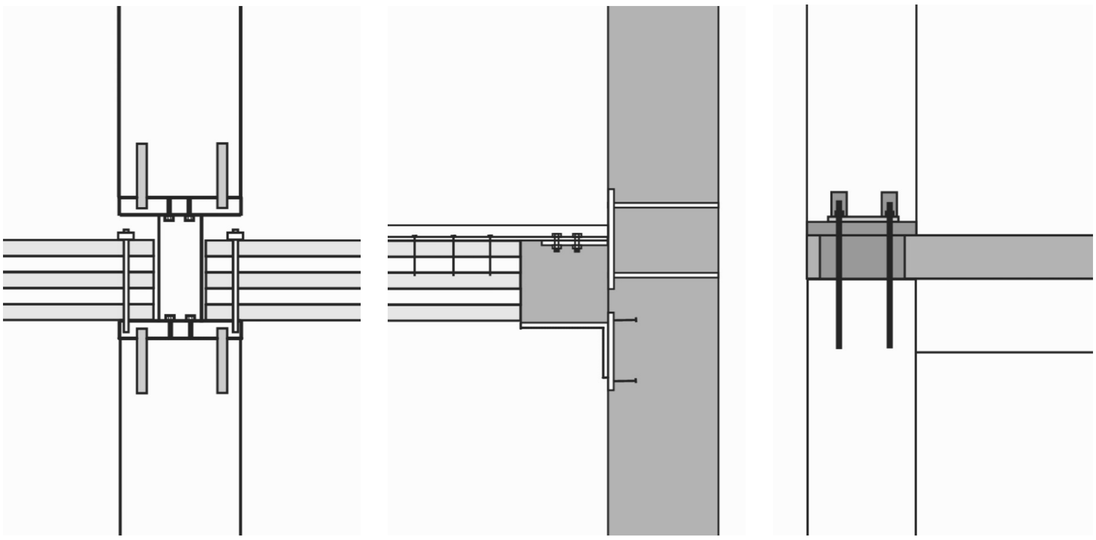
One solution of connecting a glulam beam to a concrete core is to weld a steel hanger to an embedded steel plate. A suggestion of how to connect a CLT slab to a concrete core is to bolt a longitudinal steel or timber ledger to the core and thereafter place the slab on top of the ledger. A relatively new approach is a connection called Lock Floor, by Rothoblaas, where the slab is connected to a wall through a concealed hook along the slab edge as well as the wall (Rothoblaas, n.d.).

3.1.3.4 Connections used in buildings outside of Sweden

The previously mentioned connection details are examples commonly used in Sweden. Less conventional connection details in Sweden are presented in the following section. For example, Brock Commons is a building in Canada (further presented in Section 3.4) having a column to slab connection according to Figure 3.5a. A steel connector is fastened to two steel plates attached with rods at the ends of the columns (naturally:wood, 2016). The CLT slab is placed and supported by the bottom column, and is bolted to the steel plate at the column edge. The connection transfers vertical loads to the bottom column from the slab and the upper column.

Brock Commons also has an interesting connection detail between the CLT slab and the concrete core, see Figure 3.5b, with the slab being vertically supported by a steel hanger bolted to the core. To anchor the slab in the horizontal direction, a drag strap is screwed to the slab and bolted to a steel tab which is welded to an embedded steel plate in the core (naturally:wood, 2016).

Figure 3.5c shows an example of a connection between a column and slab in a building called Arbo (further presented in Section 3.4) located in Switzerland. The slab is supported on the bottom column and fastened with steel rods (Pirmin Jung Ingenieure, n.d.). A steel plate is attached to the edge of the top column and is positioned on screw nuts connected to the steel rods, providing tolerance in the joint and allowing for adjustments if needed. The remaining space between the upper column and the slab is filled with concrete.



(a) Brock Commons: Column - CLT slab.
(b) Brock Commons: CLT Slab - Concrete core.
(c) Arbo: Column - TCC slab.

Figure 3.5: Different non-conventional connections from projects in Canada and Switzerland.

3.2 Production

The assembly of a building is affected by the production of element components. Factors influenced by the way of production are i.a., time, cost and physical labour. The different elements can either be produced in a factory or on-site, both of which briefly explained in the following subsections.

3.2.1 Prefabrication

Prefabrication is a method where elements are manufactured in a factory and then transported to the site ready for assembly. Prefabrication of all materials has many advantages with it being an effective and less costly production method compared to building on-site. Although, with prefabricating, additional costs regarding transport need to be considered and might end up being more costly in the end.

Another positive aspect to emphasize is that manufacturing occurs under controlled conditions, hence prefabrication can ensure higher utilization of the materials as well as maintain a good and safe work environment for the employees. The method also enables just in time (JIT) delivery, meaning less need for storage at the site. A downside of prefabrication is the risk of elements being wrongly manufactured, either the dimensions being wrong or with misplacement of openings. Such setback may cause stop of the construction process for an uncertain amount of time before the prefabricated element is corrected.

3.2.2 On-site

On-site production is carried out at the location of the building. The production method is more time-consuming compared to prefabrication where, for example, curing of the concrete has to be done before other elements can be attached to the concrete element. In addition, when building on site, nearby storage of material and equipment is usually required. Although, unlike prefabrication, production on-site allows for last-minute changes or adjustments regarding building dimensions or openings, etc. In addition, on-site production enables installations in the structure and minimizes the risk of cold bridges compared to prefabricated elements (Svensk Betong, n.d.-a).

3.3 Construction process

When building a conventional tall timber building, the predominant approach is building with prefabricated elements, whether it is panels, beams or columns. When building with concrete, the elements can be either prefabricated or cast on-site. Which approach is chosen depends on the building's structural system and what is considered more suitable for the specific case. Regardless the approach of production, the construction process will also affect the outcome of the building.

Two construction processes are relevant to consider for a hybrid timber-concrete building and they are hereby referred to as serial construction process and parallel construction process. These processes can be utilized for any material and components in a building and all individual structural elements will perceive the same load in the end for both processes. A detailed description of each of the two processes is presented in the following subsections where the concrete is cast-in-situ.

3.3.1 Serial construction process

Assembling a building by a serial construction process means that the entire core of the building is cast first to full building height and thereafter the wooden structure is assembled around the core. The execution of this type of construction process gives the concrete a chance to set before the timber structure is erected, which enables the possibility to adjust to early on deformation in the concrete. Although, a big drawback with the serial process is the difference of tolerances between the materials. The timber elements are prefabricated according to precise drawings whereas the concrete structure is built on site, and therefore more prone to irregularities and discrepancies. When applying the serial process, possible deviations will be hard to detect until the concrete is cast to full height. The summarized differences can be very large and will need to be compensated for. Therefore, the joints between members need to be designed to provide adequate tolerances to account for differences that might occur during the construction.

3.3.2 Parallel construction process

A parallel construction process implies that each level is built simultaneously, with one level of concrete core being cast followed by one level of timber structure being erected. On the contrary to the serial process, the parallel process will not to the same extent enable early on adjustments between the concrete and timber regarding long term deformations. However, by applying the parallel process, the tolerance differences between the materials can be smaller than for the serial process. When constructing one floor at a time, possible deviations can be detected floor by floor, allowing for adjustments to be done to the concrete. With the tolerances needed for compensation being limited to one individual floor, the connections between timber members can be designed for less tolerance.

A challenge with the parallel process is the interaction of two different professions where the concrete as well as timber industry has to cooperate during construction. Ideally, the work would be facilitated if the two building processes of concrete and timber are controlled by the same contractor.

3.4 Review of projects

The construction of taller pure timber and timber-concrete buildings is becoming more and more common over the world. In Sweden, approximately 90% of all single-family houses are timber-made whilst the market share of larger buildings built in timber is only about 15% (Fröbel & Bergkvist, 2020). Although, this percentage is expected to increase in the coming years with the great increase in interest. The increase in interest is largely due to the fact that timber is a renewable and environmentally sound choice of material but also due to the recent success of development in timber manufacturing and engineering (Jockwer et al., 2021). In this chapter, a few projects of tall timber buildings from around the world are presented.

3.4.1 Primarily timber elements

Mjøstårnet and Sara Kulturhus are two tall timber buildings positioned in the Nordic countries. They are primarily built with timber and are two of the highest timber buildings in the world. Mjøstårnet is composed of a timber beam-column structural system, with stabilizing trusses, and with a few touches of concrete elements. Sara Kulturhus, on the other hand, consists of a mix of a timber beam-column system with steel trusses, and solid timber modules with a centered core. The two buildings are further presented in the following subsections.

3.4.1.1 Mjøstårnet, Norway

Mjøstårnet was finished in 2019, built to a height of 85.4 meters, which at the time of writing is the second tallest timber building in the world, see Figure 3.6 (Moelven, n.d.). The building, located in Brumunddal, Norway, is designed by Voll Arkitekter AS and constructed by Hent AS as well as Moelven Limtre AS. Mjøstårnet is a hybrid building with primarily timber structural elements, with concrete slabs only on the upper floors managing comfort criteria (by adding weight) and acoustics (Abrahamsen, 2017a, 2017b). The structural timber elements are glulam trusses along the façade, resisting horizontal and vertical loads, in addition to columns, beams and CLT walls for vertical loading.

(a) Completed building. [Photograph] by Voll Arkitekter AS, 2018

(b) Close-up during construction. [Photograph] by Ricardo Foto, 2018.

Figure 3.6: Mjøstårnet, Norway.

3.4.1.2 Sara Kulturhus, Sweden

Sara Kulturhus is located in Skellefteå, Sweden, and is a 74 meter tall and 20-storey timber building, being the tallest timber building in Sweden (White Arkitekter Sverige, n.d.). The architect responsible for the design is White Arkitekter and the structural design was conducted by TK Botnia (today part of the consulting firm Tyréns) as well as Martinsons (Sara Kulturhus, n.d.-a, n.d.-b). The building was finished in 2021 and is a cultural centre as well as a hotel.

The city of Skellefteå has a long history of building in timber, inspiring the design of Sara Kulturhus. The building consists of two parts of which the bottom part houses the cultural centre and the top being the hotel. The bottom part is made of prefabricated CLT and glulam structural components, with timber and steel trusses allowing for large spans and thereby large open spaces, see Figure 3.7. The hotel, on the other hand, is made of prefabricated solid timber modules with centred CLT cores.

Figure 3.7: Sara Kulturhus, Skellefteå. From Sara Kulturhus [Photograph] by Jonas Westling, 2021.

3.4.2 Timber structure with concrete core

Tall timber buildings can also be found around Northern America and Central Europe. In the following subsections a few tall timber-concrete hybrid buildings are presented. All buildings are built with centered concrete cores, with a variation of timber structural systems.

3.4.2.1 Brock Commons Tallwood House, Canada

In Vancouver, Canada, at the University of British Colombia (UBC), a student resident building was constructed and finished in 2017 (naturally:wood, n.d.). *Brock Commons Tallwood House* is a mass timber hybrid building, with 18 stories and a height of 53 meters, housing more than 400 students, see Figure 3.8 (naturally:wood, 2016).

The majority of the building consists of mass timber columns and slab panels, and only the bottom podium as well as the two cores are made of reinforced concrete. The cores were cast to full height before the timber structure was erected, i.e. a serial construction process, see Figure 3.8a. All timber components were prefabricated and could therefore be assembled quite fast well at the site, this making Brock Commons Tallwood House assembled and finished approximately four months faster than other projects of its size. The architect responsible for the design is Acton Ostry Architects, Inc. and the structural engineer were Fast+Epp (Council on Tall

Buildings and Urban Habitat, n.d.-b).

(a) Construction process. **(b)** Completed building.

Figure 3.8: Brock Commons Tallwood House, Vancouver. From naturallywood [Photograph] by KK Law, 2016.

3.4.2.2 HoHo Wien, Austria

The second tallest concrete-timber hybrid building at the time of writing is *HoHo Wien*, located in Vienna (Council on Tall Buildings and Urban Habitat, 2022; SIGA, n.d.). HoHo measures 84 meters in height and was completed the same year as Mjøstårnet, 2019, see Figure 3.9 (HoHo Wien, n.d.). The building consists of CLT walls, glulam columns and timber-concrete composite (TCC) slabs with a core of concrete (Triple Wood, n.d.). The prefabricated CLT walls as well as the TCC slabs were supplied by JIT delivery (HoHo Wien, n.d.). The building is designed by the architect firm Rudiger Lainer + Partner and structurally designed by RWT+ZT GmbH (Council on Tall Buildings and Urban Habitat, n.d.-c).

(a) Close-up of completed building. [Photograph] by Thomas Lerch, 2019.

(b) Completed building. [Photograph] by DERFRITZ, 2020.

Figure 3.9: HoHo Wien, Vienna.

3.4.2.3 Ascent, USA

In 2022, *Ascent* was completed and became the world's tallest timber as well as timber-concrete hybrid building with a height of 86.6 meters, see Figure 3.10 (Council on Tall Buildings and Urban Habitat, 2022). The building is positioned in Milwaukee, USA and has a mixed-use of retail and residential apartments, with a pool and an amenity area on the top floor (Carlsson, 2022). The structural design of Ascent is made by Thornton Tomasetti and the architect is Korb + Associates Architects (Thornton Tomasetti, n.d.). The structural system consists of a five-storey underground concrete parking space, which above ground tapers off into two cores throughout the entire height of the building. The rest of the structural system is a beam-column system, made of glulam columns and beams, and CLT slabs.

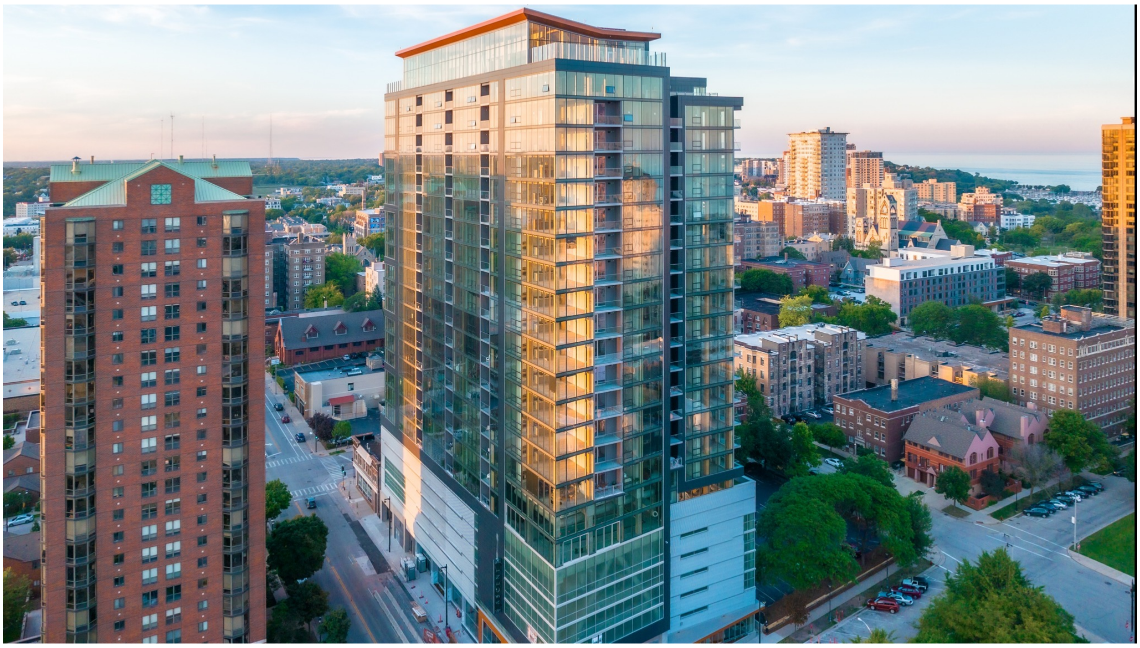


Figure 3.10: Ascent, Milwaukee. [Photograph] from Ascent, n.d.

3.4.2.4 Suurstoffi 22 and Arbo, Switzerland

The first built tall timber building in Switzerland is called *Suurstoffi 22* and was completed in 2018, see Figure 3.11 (Swiss Krono, n.d.). The building is located in Risch Rotkreuz, has 10 storeys, and is 36 meters tall (Jockwer et al., 2018). The structural elements consist of glulam columns and beams, TCC slabs and two stabilizing reinforced concrete cores, see Figure 3.11a. Suurstoffi 22 was designed by Burkard Meyer Architekten and its timber construction was carried out by ERNE AG Holzbau (Council on Tall Buildings and Urban Habitat, n.d.-e).

(a) Photo showing structural elements such as columns, beams and TCC slab.

(b) Completed building.

Figure 3.11: Suurstoffi 22, Risch Rotkreuz. [Photograph] by Roger Frei, n.d.

Built closely in time, following Suurstoffi 22, was *Arbo*, completed in 2019, see Figure

3.12 (Jockwer et al., 2021). Arbo, which is an office building also located in Risch Rotkreuz, is almost twice as tall as Suurstoffi 22 with 15 storeys and a height of 60 meters. The structural elements consist of beech LVL and glulam columns as well as beams of glulam, TCC slabs and a reinforced concrete core, see Figure 3.12a (Jockwer et al., 2018). Manetsch Meyer Architekten AG and Büro Konstrukt AG are responsible for the design and the timber construction was carried out by ERNE AG Holzbau and PIRMIN JUNG Schweiz AG (Council on Tall Buildings and Urban Habitat, n.d.-a). As seen in Figure 3.12a, the concrete core was built and finished prior to that the enclosed timber construction was erected.

(a) Construction process. [Photograph] by Robert Jockwer, 2018.

(b) Completed building. [Photograph] by Pirmin Jung, n.d.

Figure 3.12: Arbo, Risch Rotkreuz.

3.4.3 Future Projects

Future projects where it is considered building high with timber are many and in this subsection a few of those are briefly presented. A building that is aiming high in height is *W350 Project* in Tokyo, Japan. W350 Project is a research project of a 350 meter tall timber building set to be constructed by 2041 (Nikken Sekkei Ltd, n.d.). It is being investigated whether the building can consist of timber and steel, with the inner frame of the building consisting of timber only, while the outer will be a hybrid frame of timber and steel (Harada et al., 2020).

A few tall hybrid buildings that are more realistic and planned to be built in the near future are projects in Switzerland, Germany and the Netherlands. *Project Pi*, in Zug, is proposed to be 80 meters tall with 28 storeys and set to be the tallest timber structure in Switzerland (Implenia AG, n.d.). The building is planned to consist of a timber frame structure as well as TCC slabs and a core of solid timber frame structure (Implenia AG, 2019; WaltGalmarini AG, 2021). The proposed building is designed by Duplex Architekten with the structural design done by WaltGalmarini

AG (Council on Tall Buildings and Urban Habitat, n.d.-d).

WoHo, in Berlin, is planned to be 98 meters tall and is located in the district of Kreuzberg (Degree of Freedom, n.d.). The building is designed by Mad Arkitekten and structurally designed by Degree of Freedom Engineers. The structural system will consist of CLT walls and a core of reinforced concrete (O'Sullivan, 2021).

Dutch Mountains in Eindhoven, Netherlands, will consist of two buildings, one building of 130 meters and one of 100 meters, joined at the bottom via a "valley" (Studio Marco Vermeulen, n.d.). The structural system will consist of mostly timber elements, CLT walls and glulam beams. Although, the core and heavy-loaded structural elements will be of reinforced concrete. The architect of the building is Studio Marco Vermeulen and Arup is responsible for the structural design.

4
Input data

The analyzes in this project are performed based on a fictitious building with loads based on location and occupancy. In this chapter, all relevant input data are introduced. Climate conditions for the location are presented in addition to the structural system of the building and material properties. ultimate limit state and serviceability limit state are presented as well as their respective load combinations and capacity checks. Finally, a preliminary sizing is presented.

4.1 Fictitious building

The fictitious building is a typical high-rise office building with a flat roof and a square footprint, and is located in Gothenburg, Sweden. Figure 4.1 shows a simplified floor plan with only the facade and centered core presented. The dimensions of the building and the placement of the core are determined according to the floor plan, but depending on the structural system either columns or walls will be added to the final plan.

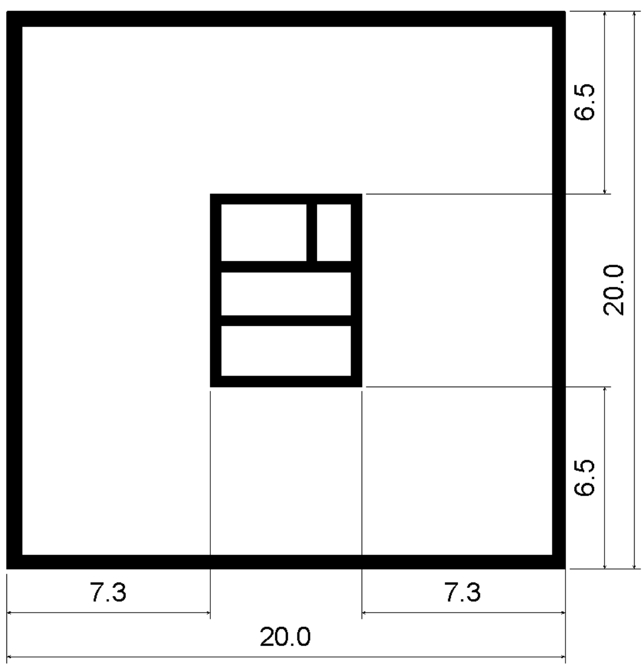


Figure 4.1: Simplified floor plan with dimensions in meter.

The case study building is to be built along the waterfront of Göta River, on the south side, see Figure 4.2. The area surrounding the building will consist of a few similar buildings and lower vegetation. The close distance to the river implies that the building will to a greater extent be exposed to wind and moisture.

Figure 4.2: Map of Gothenburg showing the area of interest (circled).

Information regarding outdoor temperature and relative humidity (RH) for Gothenburg is found at the Swedish Meteorological and Hydrological Institute (SMHI). Collected data from a 10-year period is compiled and a daily average is determined. Two separate sine curves are approximated based on the average values, providing a yearly behaviour of both temperature and RH, see Figure 4.3 and Figure 4.4.

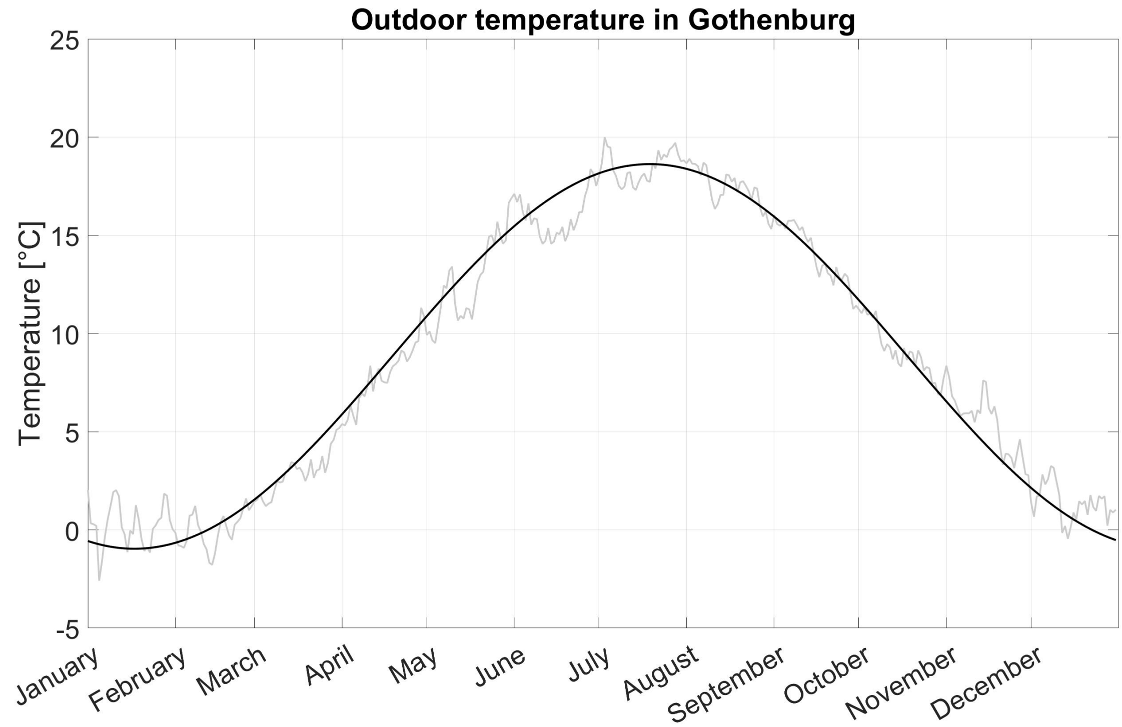


Figure 4.3: Temperature variation in Gothenburg during a year. Based on daily average temperature collected from SMHI between 2008-2018.

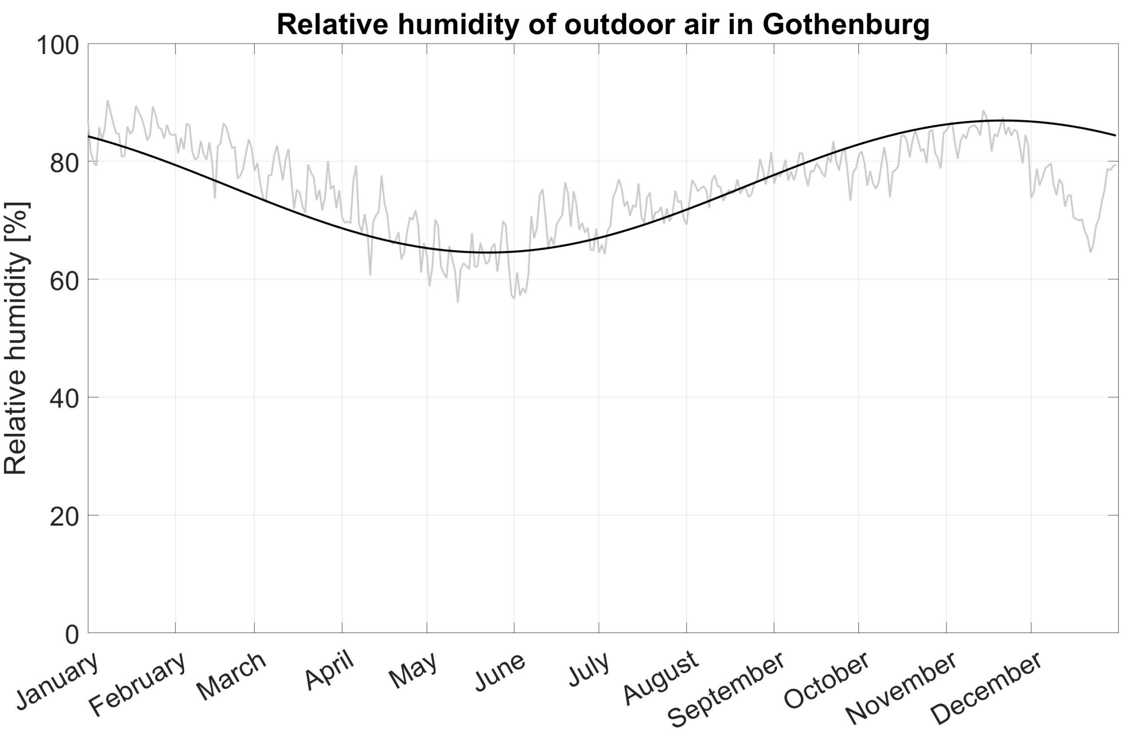


Figure 4.4: Relative humidity variation in Gothenburg during a year. Based on daily average RH collected from SMHI between 2008-2018.

The indoor climate of an office building is regulated according to Swedish requirements on thermal comfort. For this project, the indoor temperature is assumed constant at 22 °C, whilst the relative humidity, the ratio between the current vapour content and the saturation vapour content, is assumed to vary during the year. Cooler air have less capacity of storing moisture compared to warmer air, meaning outdoor air will be dryer during winter with high RH. When dry outdoor air is supplied to the indoor climate and thereafter heated, the RH will decrease as the moisture storage capacity of the warmer air is higher. Based on this, the indoor relative humidity is approximated and shown in Figure 4.5. The mean RH is set to 50% with a small amplitude. The approximation is based on the building being equipped with a well performed ventilation system allowing for a controlled indoor climate.

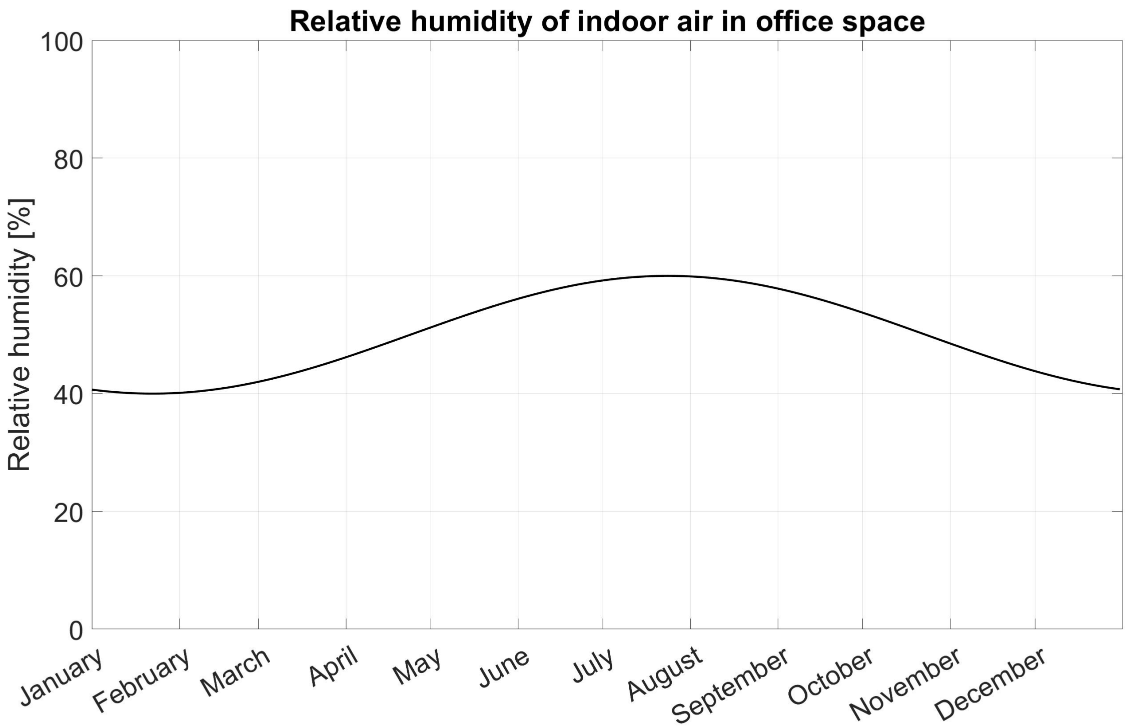


Figure 4.5: Approximated variation of the indoor relative humidity during a year in office space.

4.1.1 Structural systems

This project covers two different structural systems. The different systems are presented as System 1 and System 2, and are illustrated in Figure 4.6 and Figure 4.7. Both systems presented below are complemented with a concrete roof, to add weight to the structure, and a stabilizing cast-in-situ concrete core in the centre of the building.

System 1
System 1 is composed of columns along the façade with beams spanning between

columns and the centered core in one direction, connected with metal hangers. Each column is continuous over two stories and are placed on top of each other. The beams and part of the core are supporting CLT slab elements spanning in the perpendicular direction. The timber material of columns and beams are either glulam or LVL.

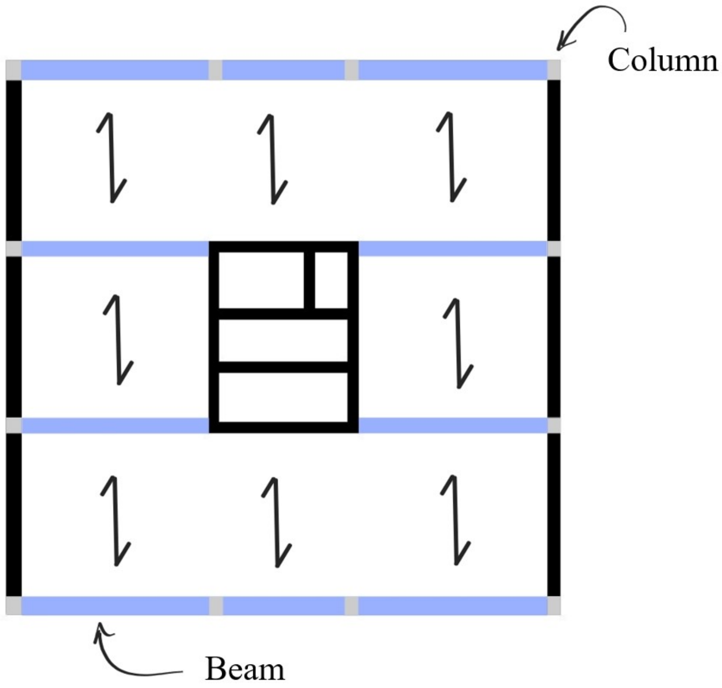


Figure 4.6: A column-beam structural system.

System 2
The second structural system consists of outer load bearing wall elements, with glulam beams spanning between walls and the centered core in one direction. The beams are connected to the vertical elements with metal hangers. The wall elements are single storey panels placed on top of each other. The beams, the outer walls and part of the core are supporting CLT slab elements spanning in the perpendicular direction. The wall elements are CLT and the beams are glulam.

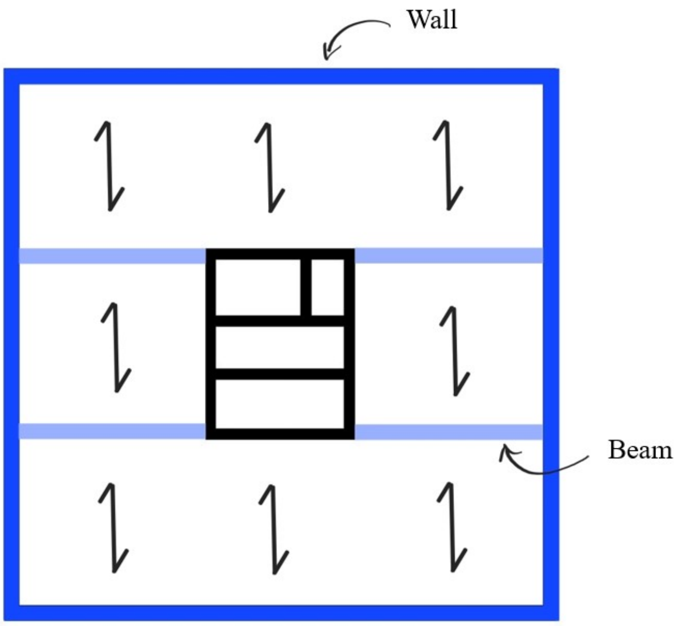


Figure 4.7: A load bearing wall structural system.

4.2 Material properties

The Engineered Wood Products of interest are glulam, CLT and LVL, with strength classes GL30c, GL24c, C14&C30, C24 and Kerto-S of which are chosen to capture a wide range of behaviour. Relevant material properties for each EWP are presented in Table 4.1. The concrete strength classes of interest are C30/37, C40/50 and C50/60, see Table 4.2 for material properties.

Table 4.1: Properties of EWPs and different strength classes. Values of compression perpendicular to grain for LVL refers to edgewise compression.

	Glulam		**CLT**		**LVL**
Class	**GL30c**	**GL24c**	**C14&C30**	**C24**	**Kerto-S**
$f_{c,0,k}$ [MPa]	24.5	21.5	24	21	35
$f_{c,90,k}$ [MPa]	2.5	2.5	2.4	2.0	6.0
$f_{m,k}$ [MPa]	30	24	30	24	44
$E_{t,0,mean}$ [MPa]	11000	12000	7000	11000	13800
$\rho_{t,k}$ [kg/m^3]	390	365	350	350	480

Table 4.2: Concrete properties with different strength classes. Concrete Class N.

Class	**C30/37**	**C40/50**	**C50/60**
f_{ck} [MPa]	30	40	50
E_{cm} [MPa]	33000	35000	37000
ρ_c [kN/m^3]	25	25	25

4.3 ULS

Ultimate limit state (ULS) is a state of when a structure no longer has the capacity of carrying loads and is on the verge of collapse. Designing in ULS means that the elements are to be designed to be able to withstand the stresses they possibly will encounter during their lifetime. ULS is verified by the load-resistant capacity for compression, tension and bending as well as buckling presented as:

$\sigma_{t,c,0} \leq f_{t,c,d}$	Check of compressive strength capacity parallel to grain
$\sigma_{t,t,0} \leq f_{t,t,d}$	Check of tensile strength capacity parallel to grain
$\sigma_{t,m} \leq f_{t,m,d}$	Check of bending capacity
$\sigma_{t,c,0,d} \leq k_c f_{t,c,d}$	Check of buckling parallel to grain

For ULS, the considered load combination is the fundamental combination presented in Equation 4.1 (Swedish Standards Institute, 2010).

$$Q_d = \sum_{j \geq 1} \gamma_{g,ULS,j} G_{k,j} + \gamma_{q,ULS,1} Q_{k,1} + \sum_{i>1} \gamma_{q,ULS,i} \psi_{0,i} Q_{k,i} \tag{4.1}$$

where: $\gamma_{g,ULS}$ = partial safety factor for permanent load in ULS [-]
= 1.35
$\gamma_{q,ULS}$ = partial safety factor for variable load in ULS [-]
= 1.5
ψ_0 = factor for combination value of a variable load [-]

4.4 SLS

Serviceability limit state (SLS) is a state where a structure is on the verge of no longer fulfilling its requirements to function (Al-Emrani et al., 2019). Designing according to SLS implies designing according to normal usage and is the state when deflections are evaluated. Deflections are usually increasing with time and therefore are SLS calculations crucial for long-term behaviour. The net final deflection, $w_{net,fin}$, is calculated for the beams according to Equation 4.2 (Swedish Standards Institute, 2009). The deflections are verified with limiting values according to Swedish standards presented in Table 4.3.

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{4.2}$$

where: $w_{net,fin}$ = net final deflection [mm]
w_{fin} = final deflection [mm]
w_{inst} = instantaneous deflection [mm]
w_{creep} = creep deflection [mm]
w_c = precamber deflection (if applied) [mm]

Table 4.3: Extract of limiting values for deflections of beams according to Eurocode 5 (Swedish Standards Institute, 2009).

	w_{inst}	$w_{net,fin}$	w_{fin}
Beam on two supports	$l/300$ to $l/500$	$l/250$ to $l/350$	$l/150$ to $l/300$

There are two load combinations that are considered for calculations in SLS (Swedish Standards Institute, 2010). For short term conditions, a characteristic combination, showed in Equation 4.3, is applied. When considering long term conditions, a quasi-permanent combination, presented in Equation 4.4, is applied.

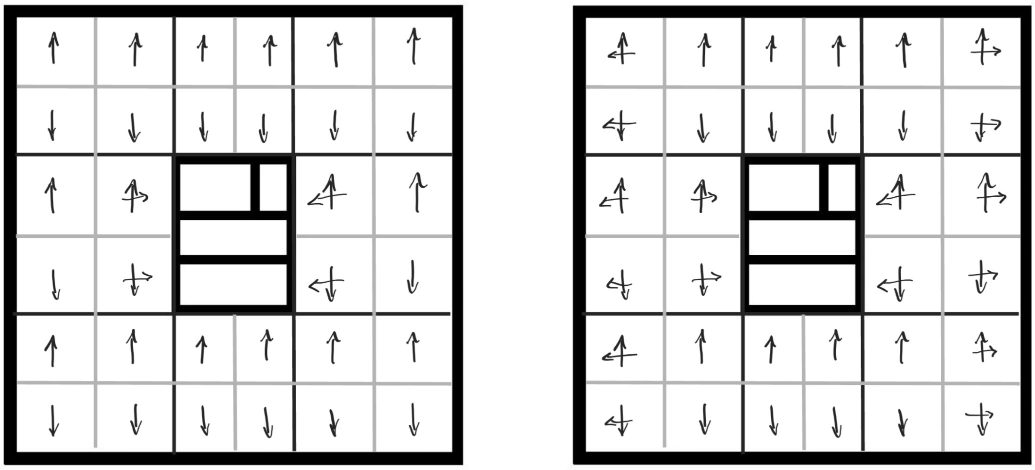
$$Q_d = \sum_{j\geq 1} \gamma_{g,SLS,j} G_{k,j} + \gamma_{q,SLS,1} Q_{k,1} + \sum_{i>1} \gamma_{q,SLS,i} \psi_{0,i} Q_{k,i} \tag{4.3}$$

$$Q_d = \sum_{j\geq 1} \gamma_{g,SLS,j} G_{k,j} + \sum_{i\geq 1} \gamma_{q,SLS,i} \psi_{2,i} Q_{k,i} \tag{4.4}$$

where: Q_d = design value for variable load acting on element [N]
G_k = characteristic value for permanent load [N]
Q_k = characteristic value for variable load [N]
ψ_0 = factor for combination value of a variable load [-]
ψ_2 = factor for quasi-permanent value of a variable load [-]
$\gamma_{g,SLS}$ = partial safety factor for permanent load in SLS [-]
= 1.0
$\gamma_{q,SLS}$ = partial safety factor for variable load in SLS [-]
= 1.0

4.5 Loads

During the building's lifetime it will be subjected to different loads of which are important to consider in the design process. The case study building will be subjected to permanent load and variable load. The loads acting on the slab on each floor will be distributed and transferred to the vertical load bearing elements according to Figure 4.8 and Figure 4.9.



(a) System 1. (b) System 2.

Figure 4.8: Plan view showing load dividing lines for each structural system case.

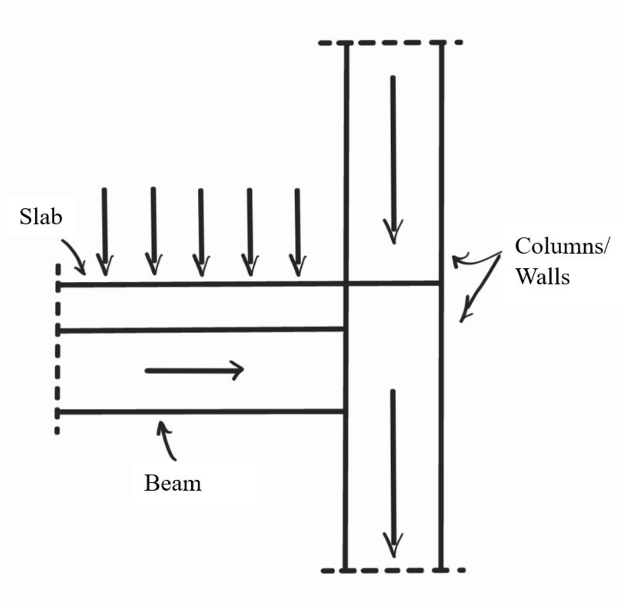


Figure 4.9: Loads transferred to column/wall element.

4.5.1 Permanent load

Permanent load refers to all loads with very little variation over time and therefore assumed constant (Al-Emrani et al., 2019). Permanent load include weight of structural components, also known as dead weight, as well as installations, see Table 4.4. The load is known by the characteristic term G_k [N] or g_k [N/m].

Table 4.4: Permanent loads according to the Application of European Construction Standards, EKS (Boverket, 2011).

		Loads [kN/m^2]
Structural members	Timber column	-
	Timber wall element	-
	Timber beam	-
	Timber slab element	1.2
	Concrete roof element	3.9
	Concrete core element	73.6
Installations		0.3

The weight of structural components such as columns, beams and wall elements are not presented in Table 4.4 as they require further calculations, see Subchapter 4.6. The self weights of slab, core and roof elements has been approximated and not further dimensioned nor optimized. The self weight of the CLT slab is based on data from a material guide for CLT by Martinssons (2022) whilst the concrete roof is estimated as the weight of prestressed hollow core elements (Svensk Betong, n.d.-b). The weight of the concrete core is approximated based on the density of reinforced concrete.

4.5.2 Variable load

Variable load refers to loads that varies in time, i.e., imposed loads, partition walls, snow, and wind (Al-Emrani et al., 2019). The characteristic term for variable load is Q_k [N] or q_k [N/m]. The imposed loads relevant for this project are load from construction workers during construction and office load when the building is finished and in use. The applied imposed load is an overestimation of the actual characteristic load as it will not act directly as a constant load after the building is finished. Although, the full load might appear once in a while and therefore need to be considered. Load reduction factors, ψ_i, are applied to variable loads in calculations of load combinations, see Subsection 4.3 and Subsection 4.4. The factors are applied due to the low probability of all the loads acting with full amplitude at the same time. Generally estimated values of variable loads as well as recommended ψ_i factors are presented in Table 4.5.

Table 4.5: Variable loads and recommended values for ψ_i-factors for variable loads according to the Application of European Construction Standards, EKS (Boverket, 2011).

	Loads [kN/m^2]	ψ_0	ψ_1	ψ_2
Construction	1.0	0.7	0.5	0.3
Office areas	2.5	0.7	0.5	0.3
Partition walls	5.0	0.7	0.5	0.3
Snow	1.2	0.6	0.3	0.1
Wind	-	0.3	0.2	0

Imposed loads are added on every floor, excluding the roof level. The construction load is added step wise with every built storey and when the building is finalized, the load is removed. Once the building is complete and ready for occupancy, the total office load is added at once.

The wind load acting on the building is not presented in Table 4.5 due to it requiring detailed calculations which are presented in Appendix A. Calculations are performed with input data related to Gothenburg with a reference wind velocity of 25 m/s.

4.6 Preliminary sizing

A preliminary sizing is done to determine dimensions of timber columns, beams and walls. The dimensioning is performed in ultimate limit state for all components, considering the worst load case. The columns are dimensioned and verified in compression as well as checked against buckling. The beams are checked in bending and verified in SLS regarding deflection according to Equation 4.2 and Table 4.3. The wall element is dimensioned and verified in bending, axial compression as well as combined bending and axial compression. For detailed calculations of all components, see Appendix A. The final dimensions from preliminary sizing regarding the case study building being 20 stories high are presented in Table 4.6. Table 4.7 presents sizes of components dimensioned for the case study building if being 10 as well as 30 stories tall.

Table 4.6: EWPs of different strength classes with dimensioning sizes of case study building with 20 stories.

	Strength class	Thickness/ Width [mm]	Height [mm]
Columns	GL30c	280	315
	GL24c	280	360
	Kerto-S	288	330
Beams	GL30c	230	675
	GL24c	280	675
	Kerto-S	144	790
Walls	C14&C30	130	1000
	C24	140	1000

Table 4.7: Timber components with dimensioning sizes for building with 10 as well as 30 stories.

	Number of stories	Thickness/ Width [mm]	Height [mm]
Columns (GL30c)	10	190	225
	30	330	405
Beams (GL30c)	10	230	675
	30	230	675
Walls (C24)	10	120	1000
	30	160	1000

5

Material models

Long-term factors such as creep and shrinkage/swelling will affect the materials by causing strains. Creep is a deformation behaviour increasing with time and shrinkage/swelling varies with the change of climate condition. The total strain caused by long-term effects can be calculated for each material according to equations presented in the following subchapters.

5.1 Loading

The construction process is reflected in the models by step wise increasment of the load over time. Each increment of load represent one added storey in the process and is expressed as $\Delta\sigma_i$, see Figure 5.1.

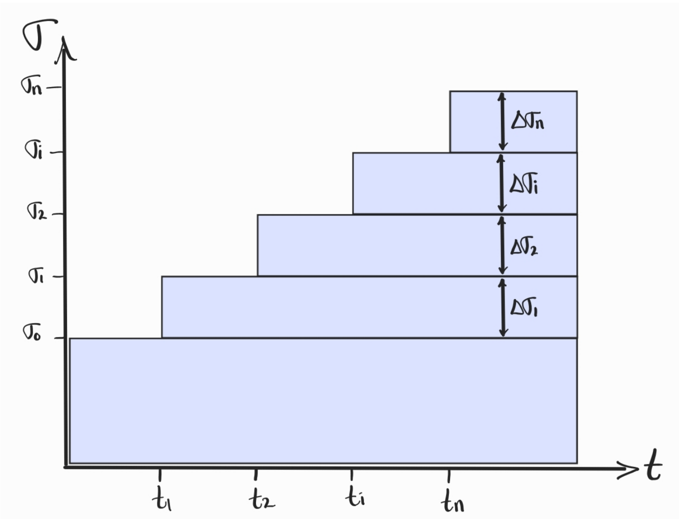


Figure 5.1: Illustration of the step wise increment of loading over the time of construction.

5.2 Long-term effects of timber

The long-term behaviour of timber can be described by a rheological model using Hooke's spring elements and Kelvin-Voigt elements (Jockwer et al., 2018). The model is presented in Figure 5.2 and in Equation 5.1. The total strain of the system can be divided into individual strains such as elastic, visco-elastic, mechano-sorptive, shrinkage/swelling from MC variations as well as temperature induced expansion (Chalmers University of Technology, 2021).

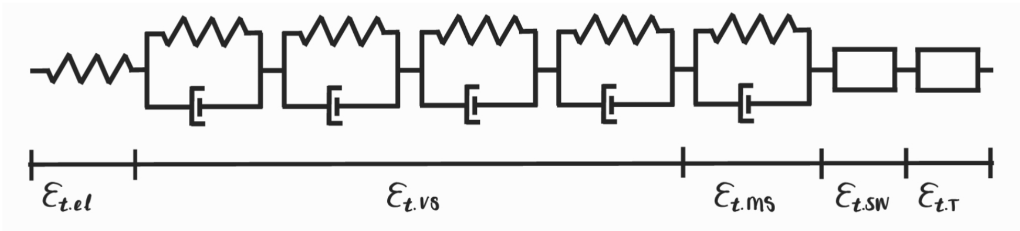


Figure 5.2: Rheological material model of timber.

$$\varepsilon_{t.tot}(t) = \varepsilon_{t.el}(t) + \varepsilon_{t.vs}(t) + \varepsilon_{t.ms}(t) + \varepsilon_{t.sw}(t) + \varepsilon_{t.T}(t) \tag{5.1}$$

where:
- $\varepsilon_{t.tot}(t)$ = total strain in timber from long-term effects [-]
- $\varepsilon_{t.el}(t)$ = linear elastic strain in timber [-]
- $\varepsilon_{t.vs}(t)$ = visco-elastic strain in timber [-]
- $\varepsilon_{t.ms}(t)$ = mechano-sorptive strain in timber [-]
- $\varepsilon_{t.sw}(t)$ = shrinkage/swelling strain in timber from moisture content differences [-]
- $\varepsilon_{t.T}(t)$ = shrinkage/swelling strain in timber from temperature differences [-]

5.2.1 Varying moisture content

The MC of timber affects mechano-sorptive creep as well as shrinkage/swelling. The calculations of the different MC related strains are based on an average MC in the cross-section of the timber member. This average value is approximated by calculating a mean value of the MC distribution within the cross-section. The MC behaviour within the timber member, i.e. the diffusion of moisture, can be described with Fick's 2^{nd} Law, see Equation 5.2. The problem is seen as one dimensional as the moisture transport is assumed limited to parallel to the glued layers.

$$\frac{\partial u}{\partial t} = D\frac{\partial^2 u}{\partial x^2} \tag{5.2}$$

A climate with varying RH entails a variation of MC in the timber member over time. The time factor is accounted for according to Equation 5.3.

$$\frac{u_j^{i+1} - u_j^i}{\Delta t} = D\frac{u_{j+1}^i - 2u_j^i + u_{j-1}^i}{(\Delta x)^2} \tag{5.3}$$

The index i and j refers to the time and the position in the cross-section respectively. Babiak (1995) and Becker (2002) determines the boundary conditions regarding the surface of the timber member which describes how the content of moisture on the surface adapts to the equilibrium MC of the timber. See Equation 5.4 for calculation of MC on the surface of the timber member. The initial MC in the timber is assumed constant.

$$u_0^{i+1} = (u_{RH} - u_0^i)(1 - exp(-\beta_{MC} t)) \tag{5.4}$$

where: β_{MC} = surface emission factor [1/h]
= 0.03

Kollmann (1963) writes in his report on sorption theory that the equilibrium of MC in timber can be determined with Equation 5.5.

$$\begin{aligned} u_{RH} = 0.108RH^{0.64} &+ 0.202exp\left[-\frac{1}{2}(2.75(RH-1)-1)^2\right] \\ &+0.10exp\left[-\frac{1}{2}(21(RH-1)-1)^2\right] \end{aligned} \tag{5.5}$$

Symmetry boundary conditions can be applied on the cross-section, i.e. the midpoint MC can be expressed as ($u_{j-1}^i = u_{j+1}^i$). Thus, the MC within the cross-section at time i+1 is determined according to Equation 5.6.

$$u_j^{i+1} = 2D(u_{j-1}^i - u_j^i) \tag{5.6}$$

where: D = diffusion coefficient [mm^2/h]
= $0.5exp(4.0u)$

Boundary and convergence conditions regarding stability, according Neumann and Courant-Friedrichs-Lewy, allows for the distribution of moisture to be determined numerically. With this, the desired average MC in the cross-section can be decided for time i and used in calculations of mechano-sorptive and shrinkage/swelling strains.

5.2.2 Linear elastic behaviour of timber

The linear elastic strain in timber depends on applied stress over time. The elastic strain is instantaneous when stress is applied and is linearly reversible when removing load (ETH Zürich, 2012). The linear elastic strain is calculated according to Equation 5.7 and the behaviour is illustrated in Figure 5.3 together with the step wise increasing stress.

$$\varepsilon_{t.el}(t) = \frac{\sigma_t(t)}{E_{t.0}} \tag{5.7}$$

where: $\sigma_t(t)$ = stress in timber at time t [Pa]
$E_{t.0}$ = modulus of elasticity of timber at time t=0 [Pa]

Figure 5.3: Top plot: Stress in timber. Bottom plot: Linear elastic behaviour of timber.

5.2.3 Creep of timber

Long-term load depended strain in timber, for non-varying MC, is called visco-elastic strain, often defined as creep, and is made out of elastic and viscous components (ETH Zürich, 2012). The visco-elastic strain is expressed as a series connection of a linear elastic spring and a linear elastic damper, known as Kelvin-Voigt elements (Jockwer et al., 2018). The visco-elastic strain is calculated according to Equation 5.8 and an example of how the visco-elastic strain varies over time is shown in Figure 5.4. The timber is subjected to step wise loading which is also seen in the figure.

$$\varepsilon_{t.vs}(t) = \sigma_{t.0} \sum_{k=1}^{4} \frac{\varphi_k}{E_{t.0}} \left[1 - exp\left(\frac{t}{\tau_k}\right)\right] + \sum_{i=1}^{n} \Delta\sigma_i \left[\sum_{k=1}^{4} \frac{\varphi_k}{E_{t.0}} \left[1 - exp\left(\frac{t-\tau_k}{\tau_k}\right)\right]\right] \tag{5.8}$$

where: $\sigma_{t.0}$ = stress in timber from initial load [Pa]
φ_k = characteristic retardation time [-]
t = actual age of material [days]
τ_k = creep factor [-]
$\Delta\sigma_i$ = difference in stress from decreased or increased load [Pa]

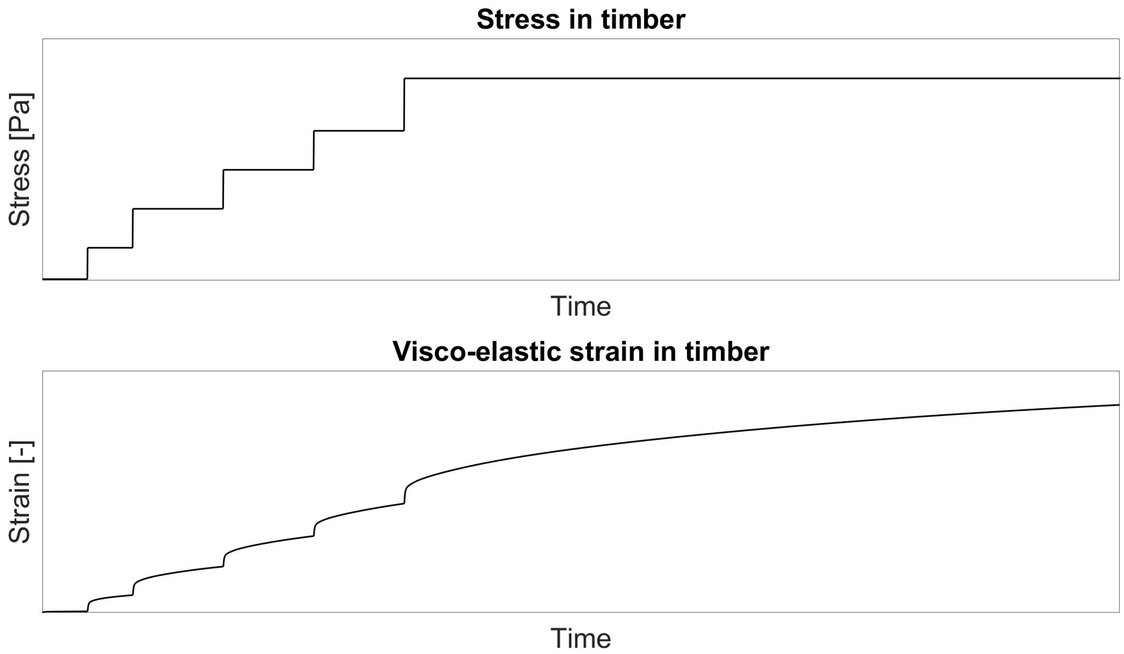


Figure 5.4: Top plot: Stress in timber. Bottom plot: Visco-elastic behaviour of timber.

The total creep behaviour depends not only on time and loading but also the surrounding conditions. Mechano-sorptive creep is a phenomenon of creep in timber where deformations occur not only due to stress but also under varying climate conditions such as change of MC (Hanhijärvi & Mackenzie-Helnwein, 2003). The mechano-sorptive strain is calculated according to Equation 5.9 (Becker, 2002). The behaviour if the stress is constant and the MC varies is illustrated in Figure 5.5. On the contrary, if the stress is increasing with time and the MC is constant, a behaviour shown in Figure 5.6 will occur. When both stress and MC is changing, the behaviour is expected as shown in Figure 5.7.

$$\varepsilon_{t.ms}(t) = \frac{\sigma_t(t)}{E_{ms}} \cdot exp\left(-\frac{u(t)}{\Delta u_{max}}\right) \tag{5.9}$$

$$E_{ms} = \frac{E_{t.0}}{\alpha_L} \cdot 1.25 \cdot 10^{-3} \cdot \frac{1}{\Delta u_{max}} \tag{5.10}$$

where: $u(t)$ = average moisture content at time t [%]
Δu_{max} = maximum difference of moisture content in timber [%]
$= u_{max} - u_{min}$
α_L = original hygroexpansion coefficient [%/%]

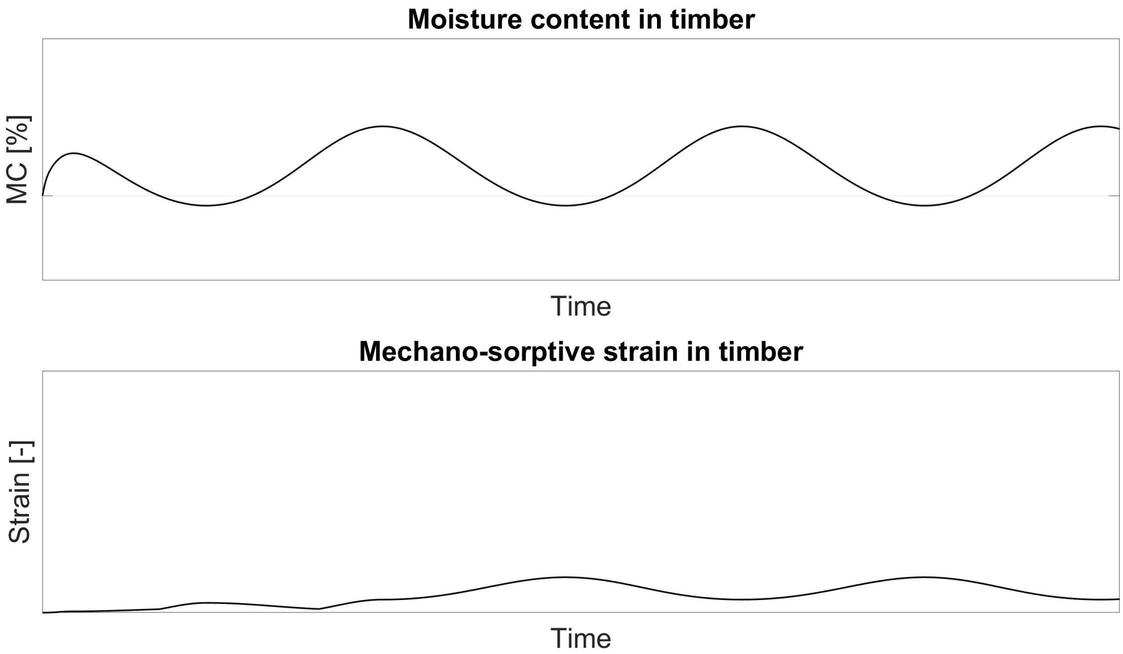


Figure 5.5: Top plot: Moisture content in timber which depends on the relative humidity. The horizontal line represents the initial moisture content. Bottom plot: Mechano-sorptive behaviour of timber with constant loading and varying moisture content.

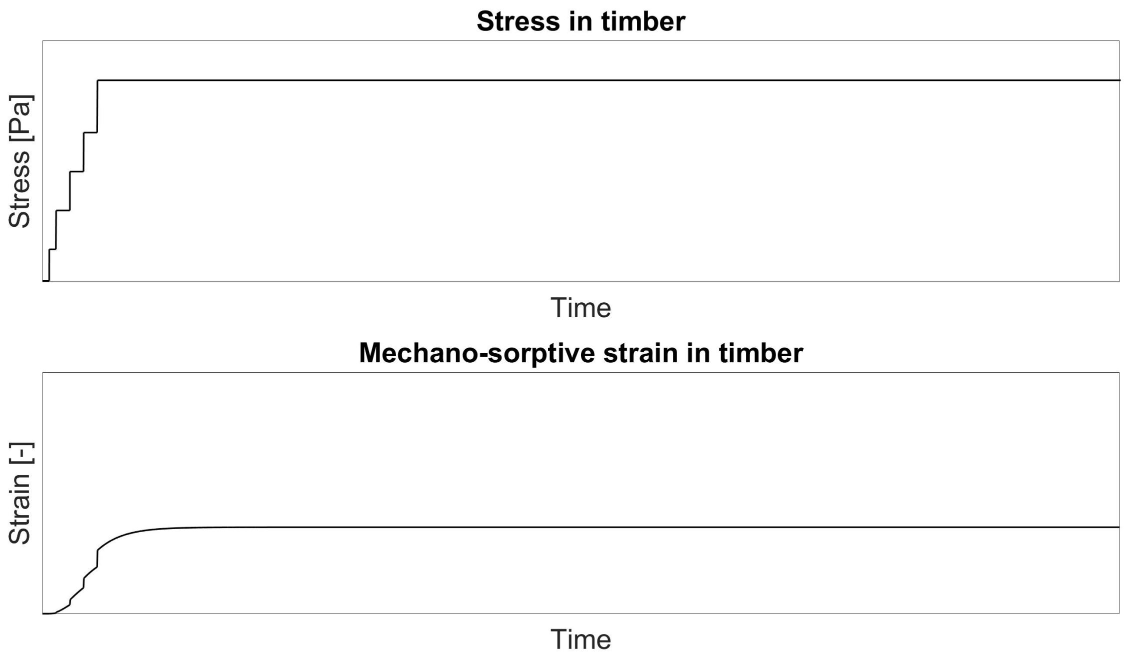


Figure 5.6: Top plot: Stress in timber. Bottom plot: Mechano-sorptive behaviour of timber with constant moisture content and multiple loading steps.

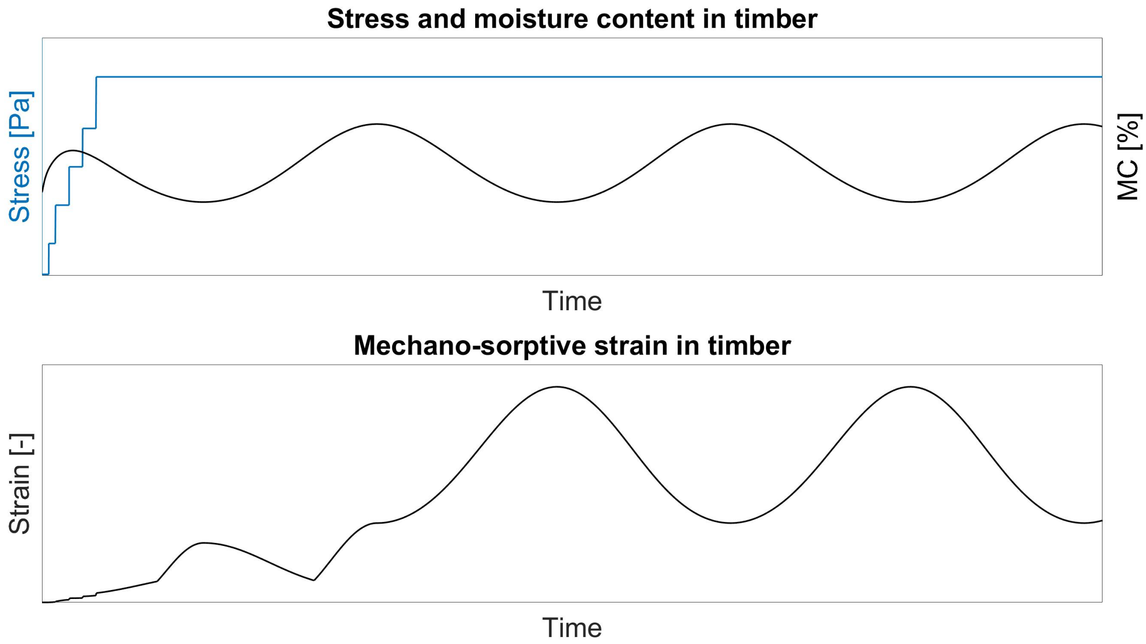


Figure 5.7: Top plot: Stress and moisture content in timber. Bottom plot: Mechano-sorptive behaviour of timber with varying moisture content and multiple loading steps.

5.2.4 Shrinkage/swelling of timber

Shrinkage and swelling occurs in timber due to i.a. variations of moisture content and/or temperature. Strains occurring when the moisture content changes in the material can be calculated according to Equation 5.11 and the behaviour can be seen in Figure 5.8 together with the varying moisture content (Becker, 2002).

$$\varepsilon_{t.sw}(t) = \bar{\alpha}_L \cdot \Delta u(t) \tag{5.11}$$

where:

$\bar{\alpha}_L$	= adapted hygroexpansion coefficient [%/%]	
	$= \alpha_L \cdot exp(180\varepsilon_{el.vs.ms})$	$\varepsilon_{el.vs.ms} \geq 0$
	$= \alpha_L \cdot exp(-180\varepsilon_{el.vs.ms})$	$\varepsilon_{el.vs.ms} < 0$
$\varepsilon_{el.vs.ms}$	= total strain in timber from elastic, visco-elastic and mechano-sorptive behaviour [-]	
$\Delta u(t)$	= difference of moisture content at time t from t=0 [%]	

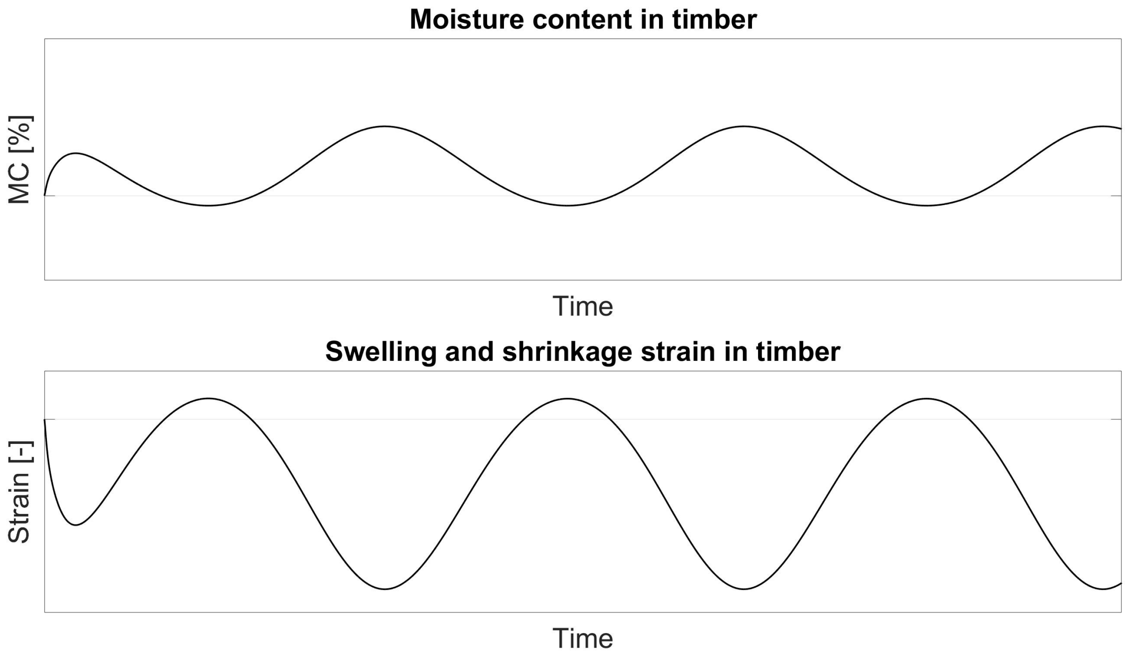


Figure 5.8: Top plot: Moisture content in timber which depends on the relative humidity. The horizontal line represents the initial moisture content. Bottom plot: Behaviour of timber from shrinkage/swelling due to variations of moisture. The horizontal line represents zero strain.

Temperature variations can also affect the timber by causing shrinkage or swelling strains. The strain from temperature variations is calculated according to Equation 5.12 and the behaviour is illustrated in Figure 5.9 together with the varying temperature (Jockwer et al., 2018). Although, for temperatures between -30°C and +90°C the influence is proven very small and can usually be neglected (Swedish Wood, 2016b).

$$\varepsilon_{t.T}(t) = \alpha_{Temp} \cdot \Delta T(t) \tag{5.12}$$

where: α_{Temp} = temperature expansion coefficient of timber [1/K] $= 3 \backsim 6 \cdot 10^{-6}$
$\Delta T(t)$ = change of temperature [K]

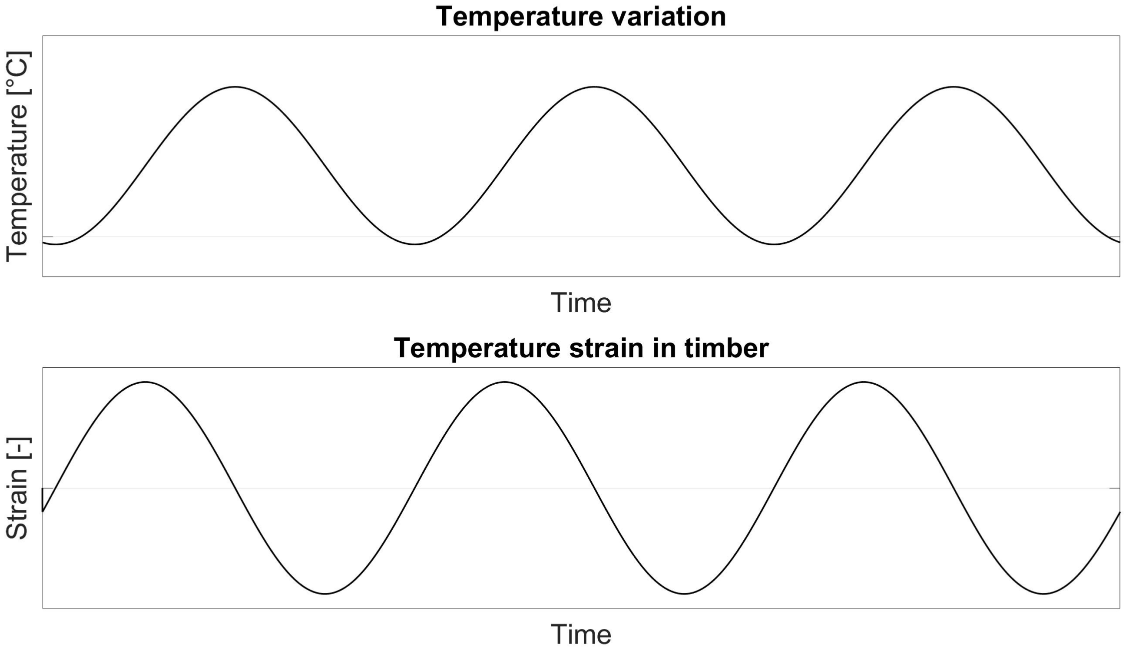


Figure 5.9: Top plot: The temperature variation of the surrounding air with the horizontal line representing zero degrees. Bottom plot: Behaviour of timber from temperature strains. The horizontal line represents zero strain.

5.3 Long-term effects of concrete

The long-term effects of concrete are creep and shrinkage. The following equation determines the total strain of concrete including the elastic strain.

$$\varepsilon_{c.tot}(t, t_0) = \varepsilon_{c.el}(t) + \varepsilon_{c.c}(t, t_0) + \varepsilon_{c.cs}(t) \tag{5.13}$$

where: $\varepsilon_{c.el}(t)$ = elastic strain of concrete [-]
$\varepsilon_{c.c}(t, t_0)$ = creep strain of concrete [-]
$\varepsilon_{c.cs}(t)$ = shrinkage strain of concrete [-]

5.3.1 Linear elastic behaviour of concrete

The linear elastic behaviour of concrete depends on the applied stress over time and the E-modulus of the material. The linear elastic strain is calculated according to Equation 5.14 and the behaviour is illustrated in Figure 5.10 together with the step wise increasing stress.

$$\varepsilon_{c.el}(t) = \frac{\sigma_c(t)}{E_c} \tag{5.14}$$

where: $\sigma_c(t)$ = stress in concrete at time t [Pa]
E_c = elastic modulus of concrete [Pa]

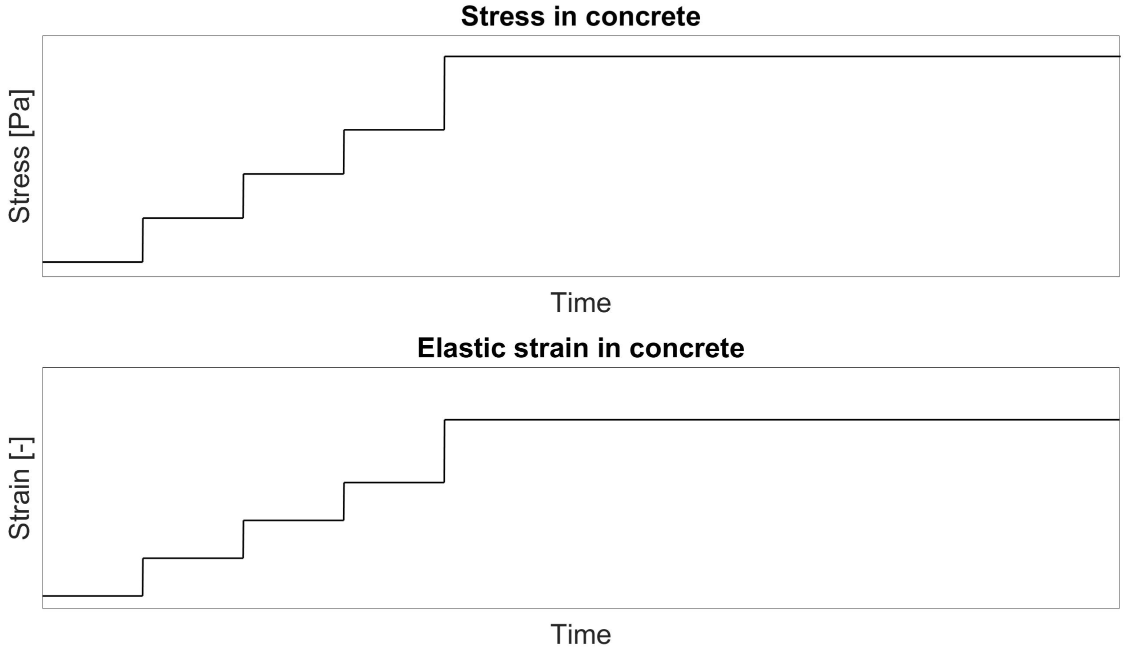


Figure 5.10: Top plot: Stress in concrete. Bottom plot: Linear elastic behaviour of concrete.

5.3.2 Creep of concrete

Creep of concrete is assumed to be proportional to the stress and is determined by considering the creep coefficient, $\varphi(t, t_0)$, see Equation 5.15. The creep coefficient considers both the concrete age, t, and the concrete age at loading, t_0, since concrete is highly influenced by stresses at an early age. Other parameters that also influence creep deformations are concrete composition, ambient RH and size of the cross-section. An example of the creep behaviour over time, with constant stress and a specific RH, is shown in Figure 5.11, where the strain is shown to stabilize after a long time.

The creep deformations are determined according to Eurocode 2 as:

$$\varepsilon_{c.c}(t, t_0) = \varphi(t, t_0)\frac{\sigma_c(t)}{E_c} \tag{5.15}$$

where: $\varphi(t, t_0)$ = creep coefficient [-]

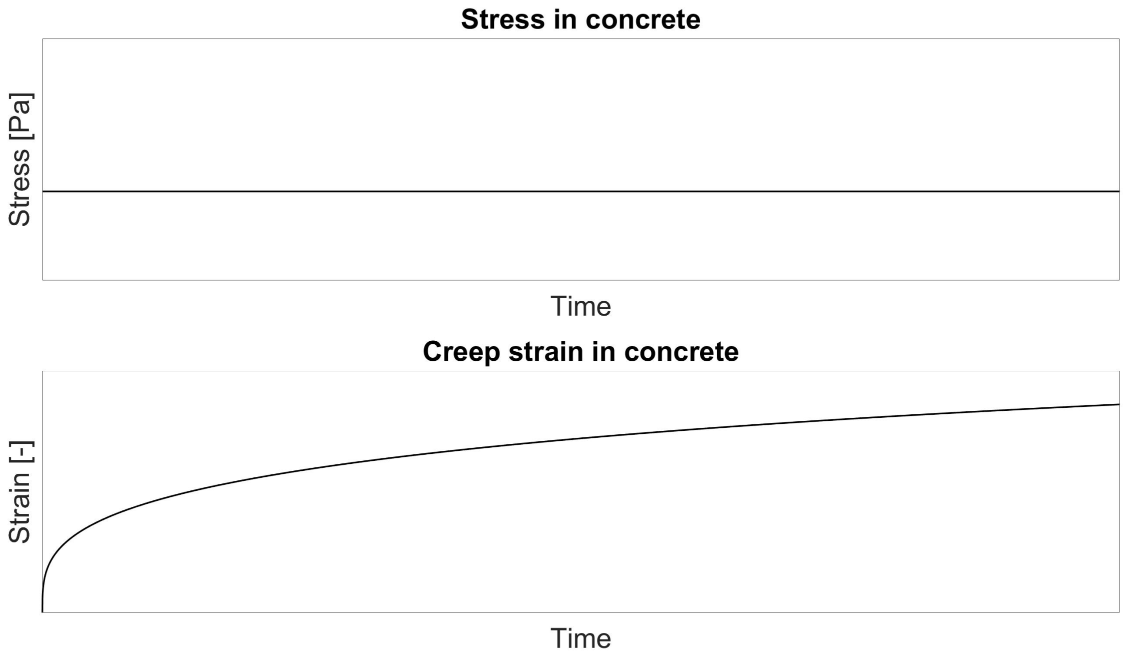


Figure 5.11: Top plot: Stress in concrete. Bottom plot: Behaviour of concrete from creep strains.

The creep coefficient can be calculated according to Eurocode 2 as:

$$\varphi(t, t_0) = \beta_c(t, t_0) \cdot \varphi_0 \tag{5.16}$$

where: $\beta_c(t, t_0)$ = time function of the creep coefficient [-]
φ_0 = notional creep coefficient [-]

The notional creep coefficient represent the final value of creep, reached after approximately 70 years, while the time function expresses the development (Engström, 2014). The creep coefficient can be described as a 'creep function', where the age at loading is considered for each applied load, giving each applied load one unique 'creep function'. This also applies for changes in RH, where each change results in one unique 'creep function'.

An estimation of the notional creep coefficient is:

$$\varphi_0 = \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \tag{5.17}$$

where: φ_{RH} = factor considering the relative humidity [-]
$\beta(f_{cm})$ = factor considering the concrete strength [-]
$\beta(t_0)$ = factor considering the age when the concrete was loaded [-]

The factor considering relative humidity, φ_{RH}, is determined as:

$$\begin{aligned} \varphi_{RH} &= 1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h_0}} && \text{for } f_{cm} \leq 35 \text{ MPa} \\ \varphi_{RH} &= \left[1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h_0}} \cdot \left[\frac{35}{f_{cm}}\right]^{0.7}\right] \left[\frac{35}{f_{cm}}\right]^{0.2} && \text{for } f_{cm} > 35 \text{ MPa} \end{aligned} \tag{5.18}$$

where: RH = ambient relative humidity [%]
h_0 = notional size [mm]
f_{cm} = mean compressive strength of concrete at an age of 28 days [MPa]
f_{cm} = f_{ck} + 8 MPa

The notional size, considering the concrete section and the area of concrete exposed to drying, is determined as:

$$h_0 = \frac{2 \cdot A_c}{u} \tag{5.19}$$

where: A_c = cross-sectional area of concrete (gross section) [mm^2]
u = perimeter of cross-section which is exposed to drying [mm]

The factor considering concrete strength, $\beta(f_{cm})$, is determined as:

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{ck} + 8}} \qquad \text{with } f_{ck} \text{ in MPa} \tag{5.20}$$

The factor considering concrete age when the load is applied, $\beta(t_0)$, is determined as:

$$\beta(t_0) = \frac{1}{0.1 + {t_0}^{0.20}} \qquad \text{valid for cement Class } N \tag{5.21}$$

where: t_0 = age of concrete when load is applied [days]

For cement Class S or Class R, the concrete age is adjusted by modifying the age at loading t_0 according to:

$$t_0 = t_{0,T} \cdot \left(\frac{9}{2 + t_{0,T}^{1.2}} + 1\right)^{\alpha} \geq 0.5 \tag{5.22}$$

where: $t_{0,T}$ = age of concrete at loading [days] adjusted (or not) with regard to temperature
α = -1 for cement Class S
α = 1 for cement Class R

The time function considers the ambient relative humidity, the concrete class, and the cross-section. The calculation of the time function, $\beta_c(t, t_0)$, is according to:

$$\beta_c(t, t_0) = \left[\frac{(t - t_0)}{\beta_H + (t - t_0)}\right]^{0.3} \tag{5.23}$$

where: t = actual age of material [days]
t_0 = age of concrete when load is applied [days]
β_H = coefficient depending on the ambient relative humidity and the notional size of the section [-]

$$\begin{aligned} \beta_H =& 1.5\left[1 + (0.012RH)^{18}\right] \cdot h_0 + 250 \leq 1500 && \text{for } f_{cm} \leq 35 \text{ MPa} \\ \beta_H =& 1.5\left[1 + (0.012RH)^{18}\right] \cdot h_0 + ... \\ & + 250 \cdot \left[\frac{35}{f_{ck} + 8}\right]^{0.5} \leq 1500 \cdot \left[\frac{35}{f_{ck} + 8}\right]^{0.5} && \text{for } f_{cm} > 35 \text{ MPa} \end{aligned} \tag{5.24}$$

5.3.2.1 Superposition method

Most often the concrete stress varies over time, where the load may increase or decrease during the lifetime of the structure (Engström, 2014). For example, at the removal of the framework the structure will be subjected to dead weight, which will remain during its lifetime. At the end of the construction period, the structure will be subjected to additional loads in form of variable loads. These variations of loads will influence how the creep affects the structure. If the concrete is subjected to varying loads, i.e. varying stress components, at different ages, the creep strain will consist of unique creep functions for each stress component.

Several different approaches account for varying stress at different ages and the superposition method is one accurate method to calculate the creep deformation under varying stress (Engström, 2014). Each unique creep function develop independently and can be superimposed. Equation 5.25 show an example of how the superposition method is used and Figure 5.12 illustrates an example in a graph for step wise loading accounting for both elastic and creep strain. If the load decreases during the time period, it will be accounted for by including the decreasing load, i.e. decreasing stress component, with a negative sign. The model accounts for varying RH through a simplification, where a mean value of the RH, from the first seven days after casting, is calculated. This simplification is considered reasonable as concrete is most easily affected during its early age.

$$\varepsilon_{c.c}(t) = (1 + \varphi(t, t_1)) \cdot \frac{\sigma_1}{E_{cm}} + (1 + \varphi(t, t_2)) \cdot \frac{\sigma_2}{E_{cm}} + (1 + \varphi(t, t_3)) \cdot \frac{\sigma_3}{E_{cm}} \tag{5.25}$$

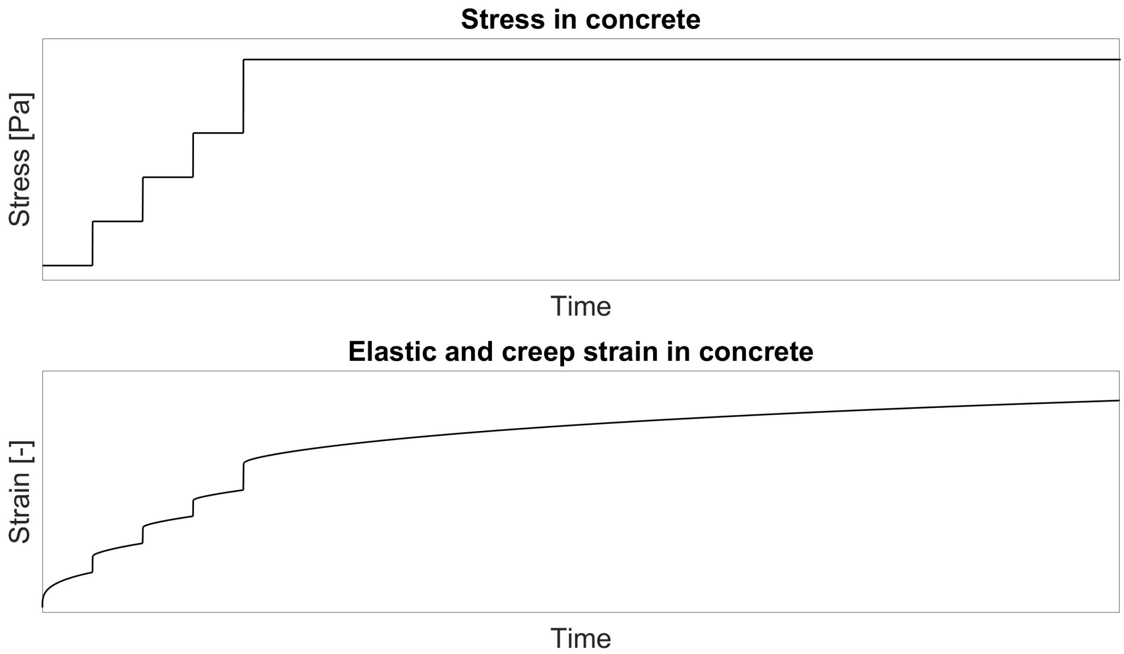


Figure 5.12: Top plot: Stress in concrete. Bottom plot: Behaviour of concrete from elastic and creep strain applying superposition method.

5.3.3 Shrinkage of concrete

Shrinkage strain of concrete is a deformation divided into two components, drying and autogenous shrinkage (Engström, 2014). Eurocode 2 determines the total shrinkage strain at age t [days] to be calculated according to Equation 5.26. An example of the shrinkage strain behaviour is illustrated in Figure 5.13 together with varying RH.

$$\varepsilon_{c.cs}(t) = \varepsilon_{c.cd}(t) + \varepsilon_{c.ca}(t) \tag{5.26}$$

where: $\varepsilon_{c.cd}(t)$ = drying shrinkage strain [-]
$\varepsilon_{c.ca}(t)$ = autogenous shrinkage strain [-]

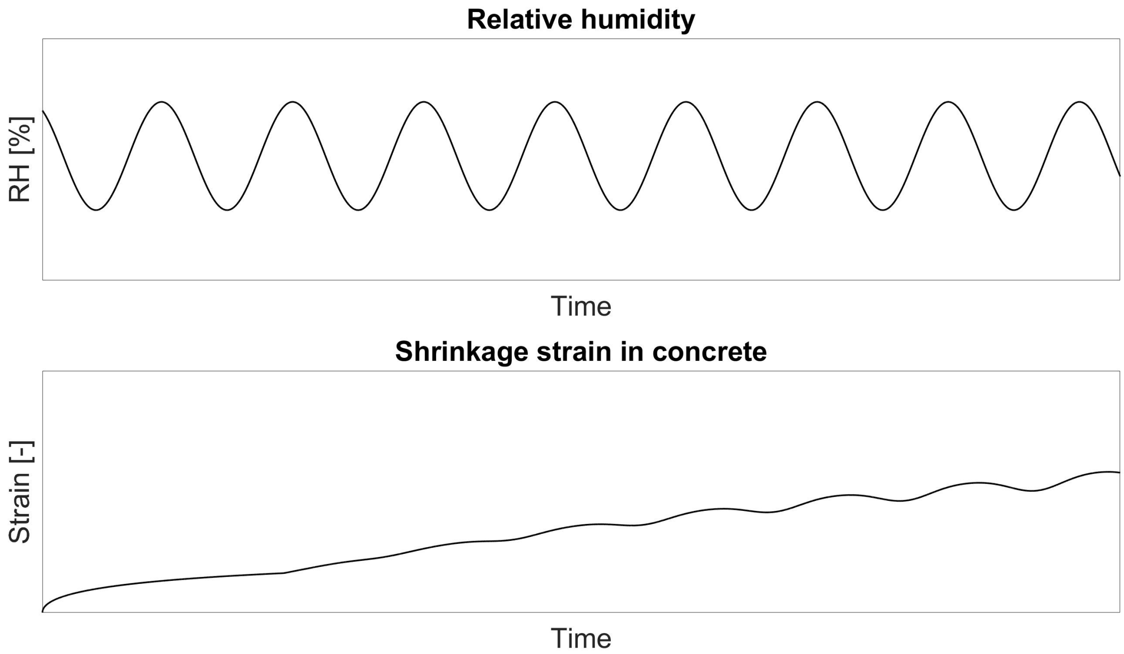


Figure 5.13: Top plot: Varying relative humidity. Bottom plot: Behaviour of concrete from drying and autogenous shrinkage.

The drying shrinkage, $\varepsilon_{c.cd}(t)$, is a slow process which occurs whenever there is an exchange of moisture between the material and the surroundings. The average value of the drying shrinkage across the section is determined according to Eurocode 2 as:

$$\varepsilon_{c.cd}(t) = \beta_{ds}(t, t_s) \cdot \varepsilon_{c.cd}(\infty) \tag{5.27}$$

where: $\beta_{ds}(t, t_s)$ = time function of drying shrinkage [-]
$\varepsilon_{c.cd}(\infty)$ = final value of drying shrinkage [-]

The final value, and thereby the development, of drying shrinkage is largely influenced by the composition of the concrete mixture, i.e. water content, and the ambient relative humidity. In addition, concrete volume and the area of concrete exposed to drying are also influencing the drying shrinkage. The final value of the drying shrinkage can be determined as:

$$\varepsilon_{c.cd}(\infty) = k_h \cdot \beta_{RH} \cdot \varepsilon_{c.cdi} \tag{5.28}$$

where: k_h = coefficient depending on the size of the section [-], see Table 5.1
β_{RH} = factor that considers the ambient relative humidity [-]
$\varepsilon_{c.cdi}$ = starting value to determine the drying shrinkage strain [-]

Table 5.1: The coefficient k_h depending on the size of the cross-section, see Equation 5.19 for calculation of h_0 (Engström, 2014).

h_0 [mm]	k_h
100	1.0
200	0.85
300	0.75
≥500	0.70

The ambient relative humidity is considered in β_{RH} and is defined as:

$$\beta_{RH} = 1.55 \left[1 - \left[\frac{RH}{(RH)_0} \right]^3 \right] \tag{5.29}$$

where: RH = ambient relative humidity [%]
$(RH)_0$ = 100% (reference value)

Eurocode 2 determines the starting value of drying shrinkage strain, $\varepsilon_{c.cdi}$, according to:

$$\varepsilon_{c.cdi} = 0.85 \cdot \left[(220 + 110 \cdot \alpha_{ds1}) \cdot exp\left(-\alpha_{ds2} \cdot \frac{f_{cm}}{10} \right) \right] \cdot 10^{-6} \tag{5.30}$$

where: f_{cm} = mean compressive strength of concrete at an age of 28 days [MPa]
α_{ds1} = coefficient depending on the type of cement [-]
= 3 for cement Class S
= 4 for cement Class N
= 6 for cement Class R
α_{ds2} = coefficient depending on the type of cement [-]
= 0.13 for cement Class S
= 0.12 for cement Class N
= 0.11 for cement Class R

The time function describes the development of drying shrinkage and is determined as:

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \cdot \sqrt{h_0{}^3}} \qquad \text{with } h_0 \text{ in [mm]} \tag{5.31}$$

where: t = actual age of material [days]
t_s = age of the concrete when drying shrinkage starts (normally at the end of curing) [days]

Autogenous shrinkage, $\varepsilon_{c.ca}(t)$, also known as chemical shrinkage, does not depend on moisture exchange with the surroundings (Engström, 2014). This phenomena takes place within the concrete, during hardening, and is more evident in concrete with low water-cement ratios. Thus, the hydration process in the early stages of hardening will be quick and the remaining cement, which has not yet reacted with water, will continue to react with the small amount of moisture left in the concrete. Eurocode 2 determines that the autogenous shrinkage strain at age t [days] can be calculated according to:

$$\varepsilon_{c.ca}(t) = \beta_{as}(t) \cdot \varepsilon_{c.ca}(\infty) \tag{5.32}$$

where: $\beta_{as}(t)$ = time function of autogenous shrinkage [-]
$\varepsilon_{c.ca}(\infty)$ = final value of autogenous shrinkage [-]

The final value of the autogenous shrinkage is calculated according to:

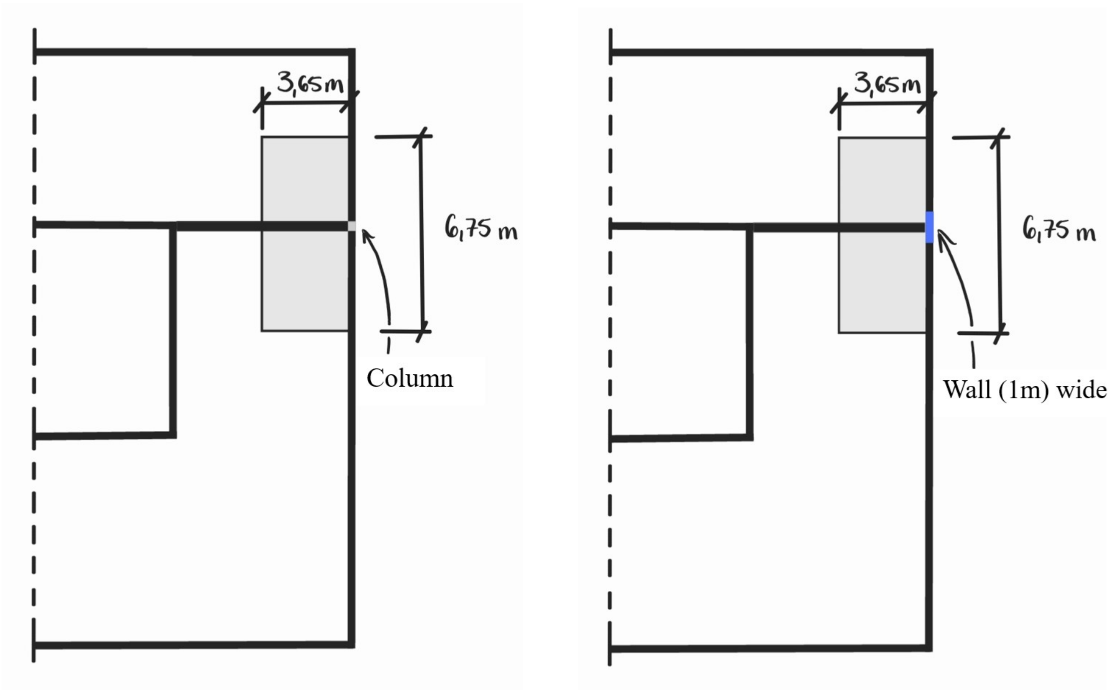
$$\varepsilon_{c.ca}(\infty) = 2.5(f_{ck} - 10) \cdot 10^{-6} \qquad \text{with } f_{ck} \text{ in [MPa]} \tag{5.33}$$

The time function describes the development of the autogenous shrinkage and is determined as:

$$\beta_{as}(t) = 1 - exp(-0.2t^{0.5}) \qquad \text{with } t \text{ in [days]} \tag{5.34}$$

6
Parametric study

A parametric study is performed in MATLAB on the fictitious building, for detailed calculations see Appendix B. A column/wall element on the bottom floor, shown in Figure 6.1, is analysed by determining the strain it will be subjected to during and after construction.



(a) Timber column and its tributary area.

(b) Timber wall element and its tributary area.

Figure 6.1: Illustration of specific timber element of which the parametric study is performed on.

Different scenarios are investigated and analysed to determine which parameters are of large impact regarding the resulting deformations. To be able to compare the deformations of the timber and the concrete components, all elements are considered equal in height. Since the columns are continuous over two storeys, the column deformation is divided in half. The specific parameter that is investigated varies whilst remaining parameters are fixed, meaning if nothing else is stated, glulam components are always GL30c, CLT components C24 and concrete core element C40/50. The thickness of the slab elements and the core elements are 280 mm and 300 mm respectively of which are estimated based on literature and consultation with supervisors. All connections are assumed to be without errors.

The parallel construction process is applied in all scenarios if nothing else is mentioned. Furthermore, all scenarios considers a construction period of approximately four months as well as occupancy one year after construction is finalized. The construction period determines how long the building will be exposed to outdoor conditions. Outdoor temperature and RH is considered during the entire construction period whilst indoor conditions are applied after the building is finished. The time before indoor climate can be fully applied, an approximated transition period is considered where the outdoor and indoor conditions are linearly connected. The considered temperature as well as relative humidity are presented in Figure 6.2 and Figure 6.3.

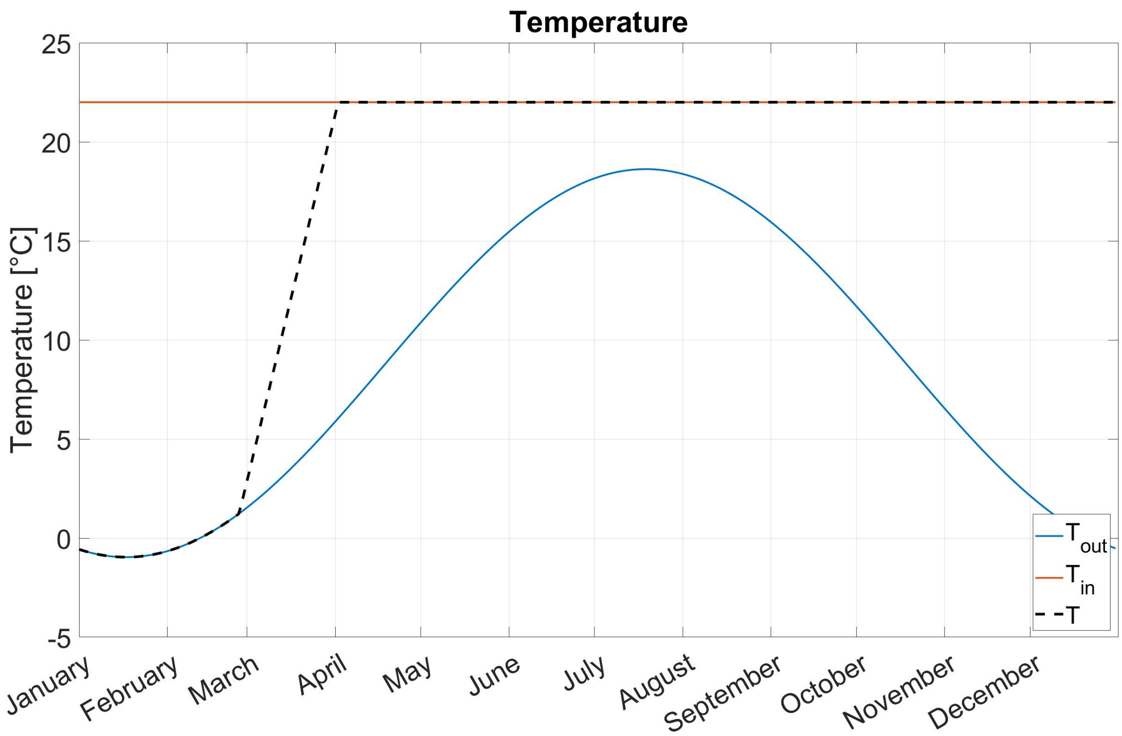


Figure 6.2: The temperature the fictitious building is subjected to during its first year. The dashed line represents the combination of indoor and outdoor conditions.

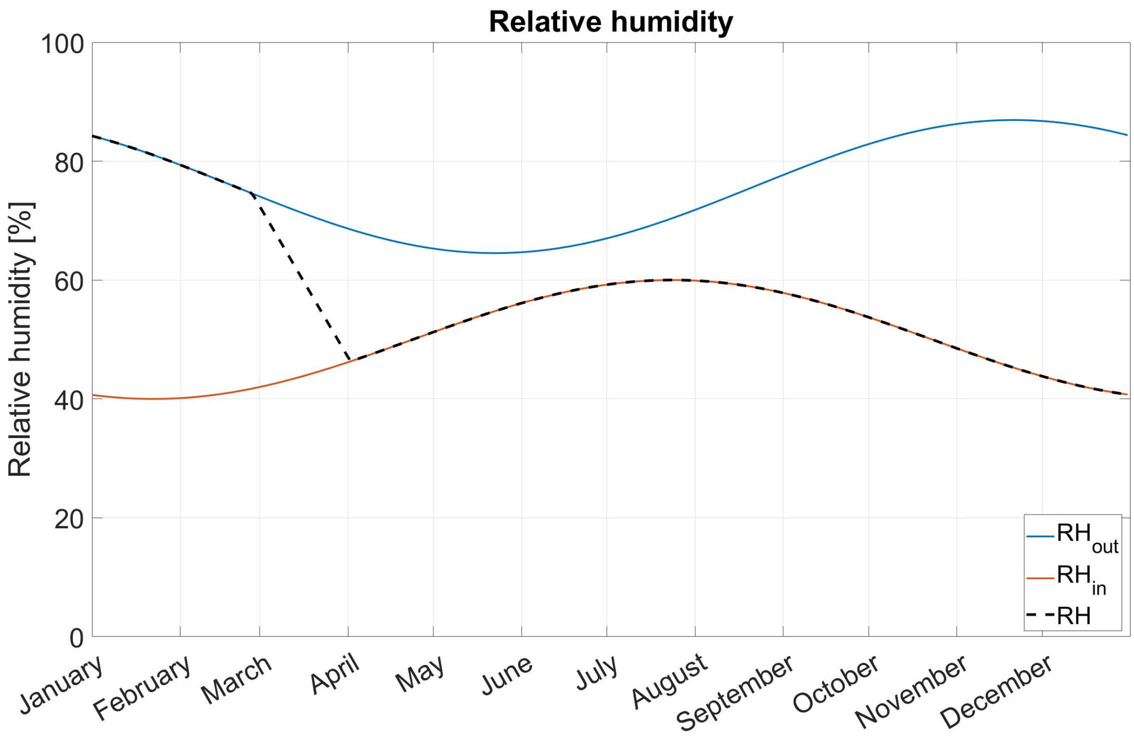


Figure 6.3: The relative humidity the fictitious building is subjected to during its first year. The dashed line represents the combination of indoor and outdoor conditions.

Each individual strain is calculated based on the material models of timber and concrete, see Chapter 5. Figure B.1 and Figure 6.5 illustrates, over a two year period, the individual strains as well as the total strain for each standard case of timber column (GL30c) and concrete core element (C40/50).

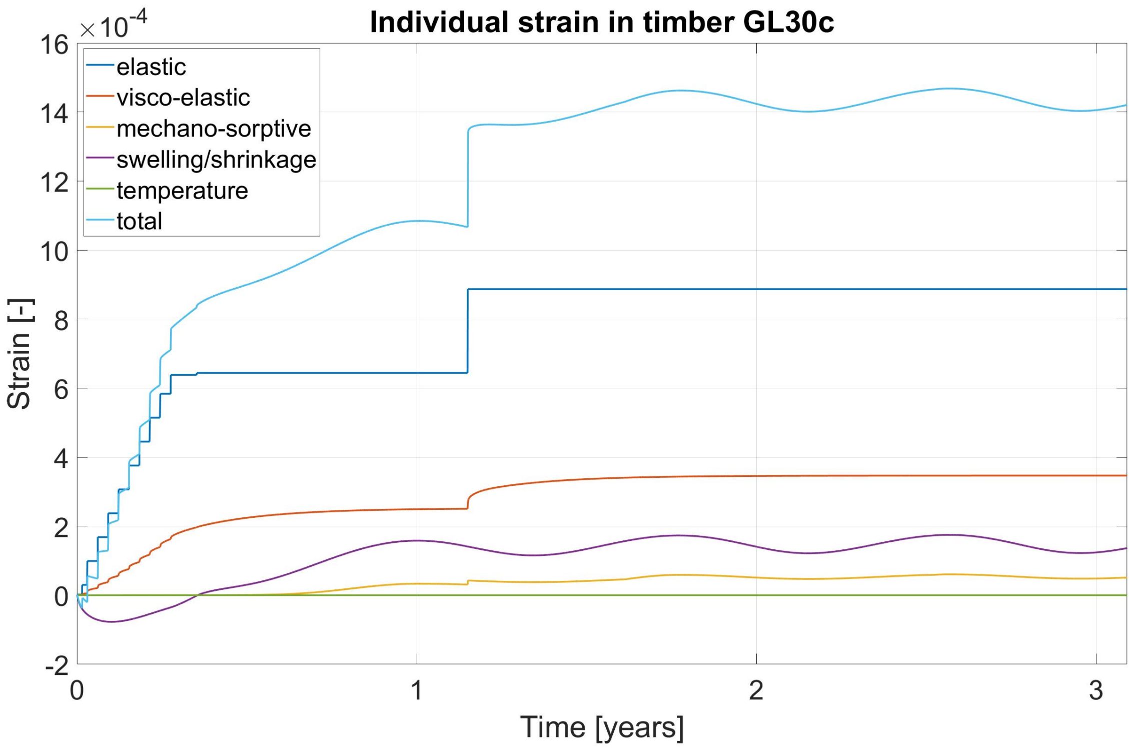


Figure 6.4: The strains of a standard column (GL30c) over time.

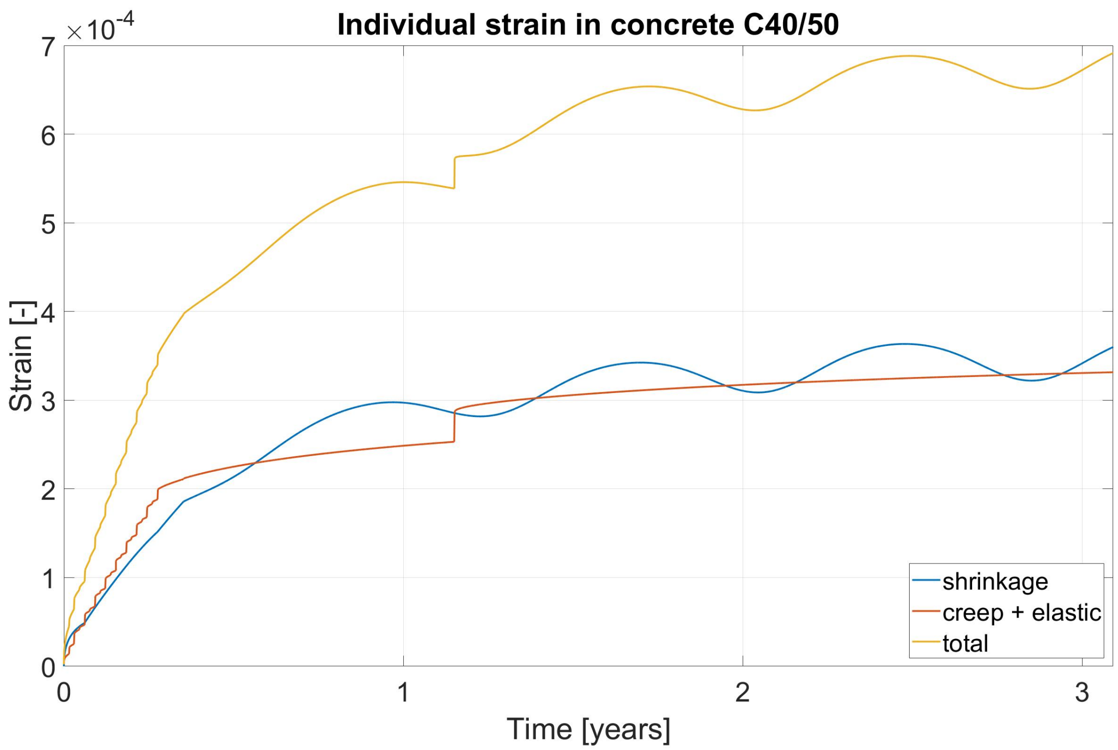


Figure 6.5: The strains of a standard core element (C40/50) over time.

The plot in Figure B.1, showing the individual strains of timber, indicates that some strains have larger impact on timber compared to others. The strain from temperature as well as the mechano-sorptive strain are close to insignificant on the total strain whilst the influence from the visco-elastic strain and the strain from shrinkage/swelling are noticeable. The largest impact is related to the elastic strain. Observe the swelling behaviour early on in the construction phase which influences the total timber strain. The individual strains regarding concrete, see Figure 6.5, are of equally large impact on the total strain. The creep and elastic strain are combined due to applying the superposition method.

The early on behaviour of mechano-sorptive strain, shown as the sudden decrease in strain seen in Figure 5.7, does not seem to be able to account for the variation of moisture content. Although, with the mechano-sorptive strain being small at the early stages, see Figure B.1, this behaviour is overseen.

6.1 Comparing structural systems

Two structural systems are considered in the first scenario, columns and load bearing walls. The analysis and comparison is performed for one type of column and one type of wall, with fixed strength classes as well as dimensions according to Table 6.1.

Table 6.1: Specific parameters for scenario: structural systems.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c	280	315	6000	12
Walls	C24	140	1000	3000	12
Beams	GL30c	230	675	7300	12
Core	C40/50	300	1000	3000	-

Figure 6.6 shows the deformations of each system over a time period of three years, where the system of columns are proven to result in larger deformations.

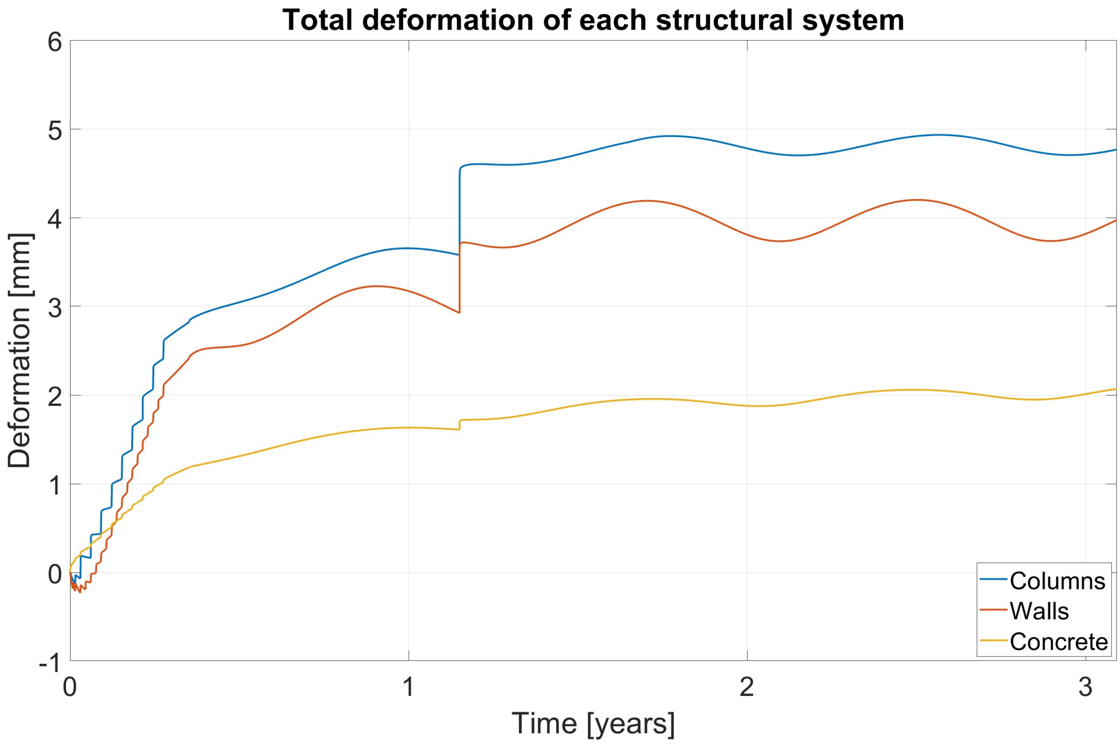


Figure 6.6: Vertical deformations of timber elements for scenario: structural systems. Concrete core element deformation is shown as a reference.

6.2 Varying strength class of timber

Another scenario analyses and compares the impact on vertical deformations with having different strength classes of columns & beams or walls. The analysis of the second scenario is performed by calculating the deformations of the timber component with fixed dimensions, see Table 6.2. The dimensions of the timber elements are selected based on the smallest possible dimensions of the different strength classes. For example, GL24c requires larger dimensions compared to GL30c, resulting in dimensions of GL24c being the selected one. The two CLT walls analysed have a strength class of C24 as well as a combination of C14 and C30. The resulting strains the column/wall element is subjected to are calculated for each different strength class and the resulting vertical deformations are compared.

Table 6.2: Specific parameters for scenario: varying strength class of timber.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c GL24c Kerto-S	280	360	6000	12 12 6
Walls	C14&C30 C24	140	1000	3000	12 12
Beams	GL30c GL24c Kerto-S	230	675	7300	12 12 6
Core	C40/50	300	1000	3000	-

Figure 6.7 and Figure 6.8 shows the resulting deformations for each strength class of the timber column and timber wall element respectively.

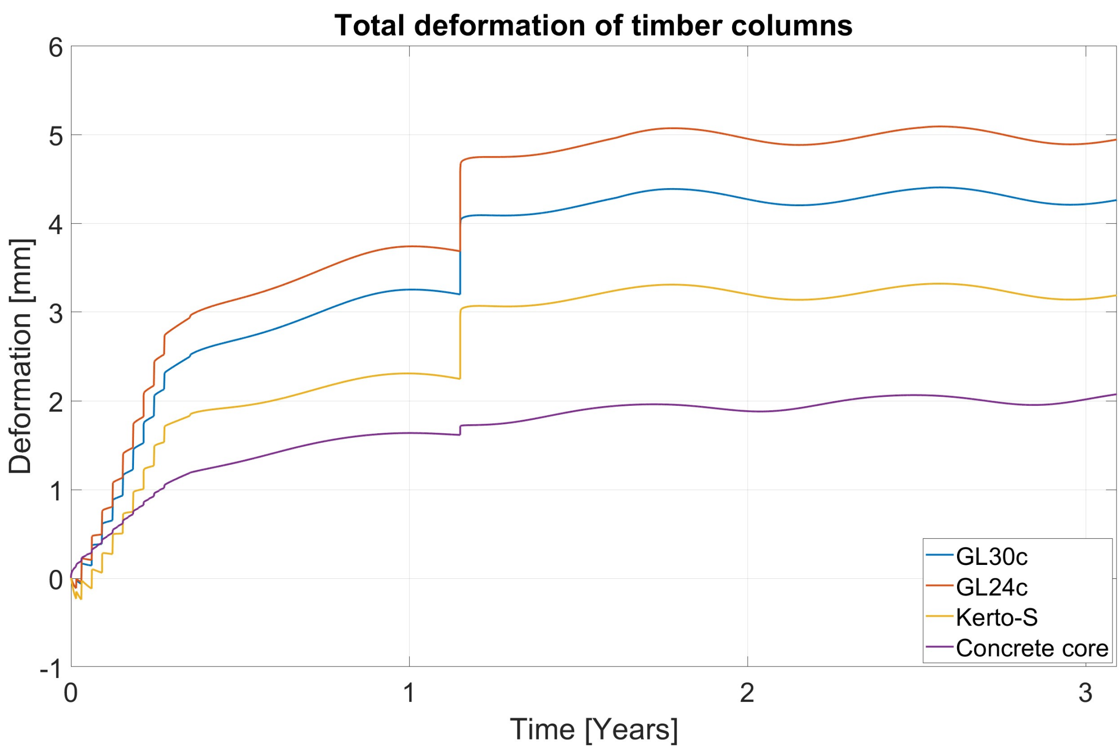


Figure 6.7: Vertical deformations of timber column for scenario: varying strength class of timber. Concrete core element deformation is shown as a reference.

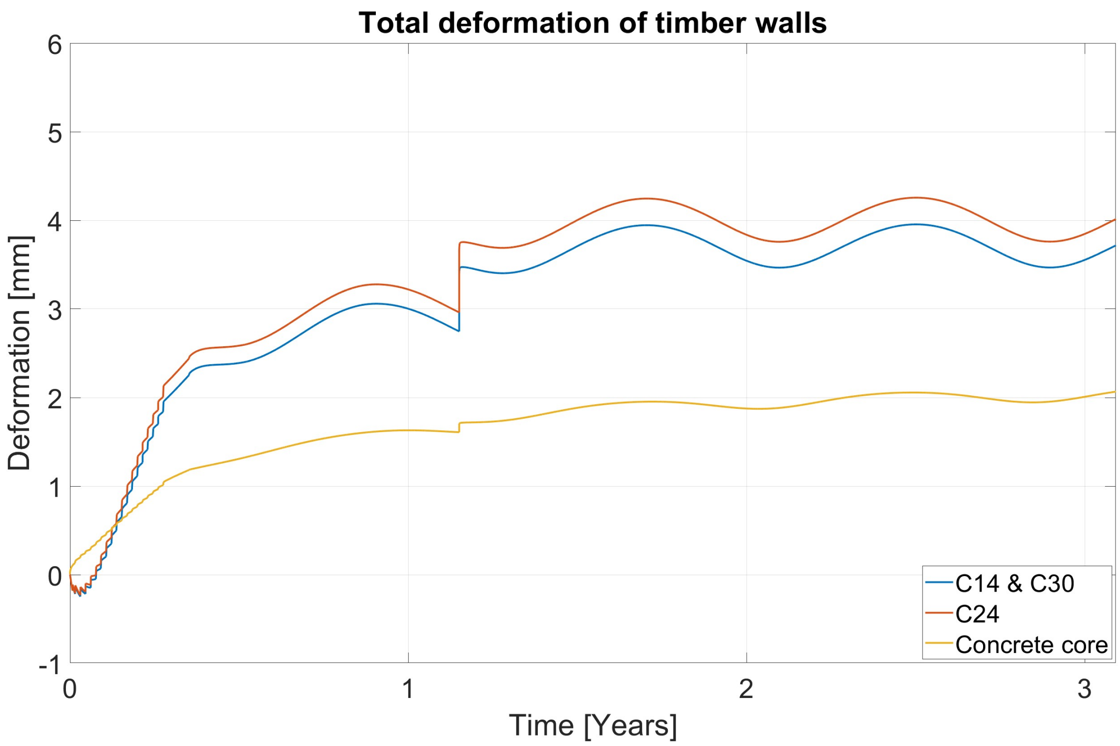


Figure 6.8: Vertical deformations of timber wall element for scenario: varying strength class of timber. Concrete core element deformation is shown as a reference.

The case with columns, presented in Figure 6.7, show a scattered result of deformations where GL24c show the largest change, followed by GL30c and lastly, showing the best result, Kerto-S. The different deformations of wall elements regarding C14&C30 and C24, presented in Figure 6.8, show similar results but with slightly larger deformation of the C24 wall element.

6.3 Varying strength class of concrete

The third scenario analyses and compares the impact on vertical deformations with having different strength classes of concrete core. The dimensions as well as strength classes of columns, beams and walls are fixed whilst the concrete core element is analysed for three different strength classes, see Table 6.3.

Table 6.3: Specific parameters for scenario: varying strength class of concrete.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c	280	315	6000	12
Beams	GL30c	230	675	7300	12
Core	C30/37 C40/50 C50/60	300	1000	3000	-

Figure 6.9 presents the resulting deformation of the concrete core element for each concrete strength class and shows that the impact difference is very small between the cases.

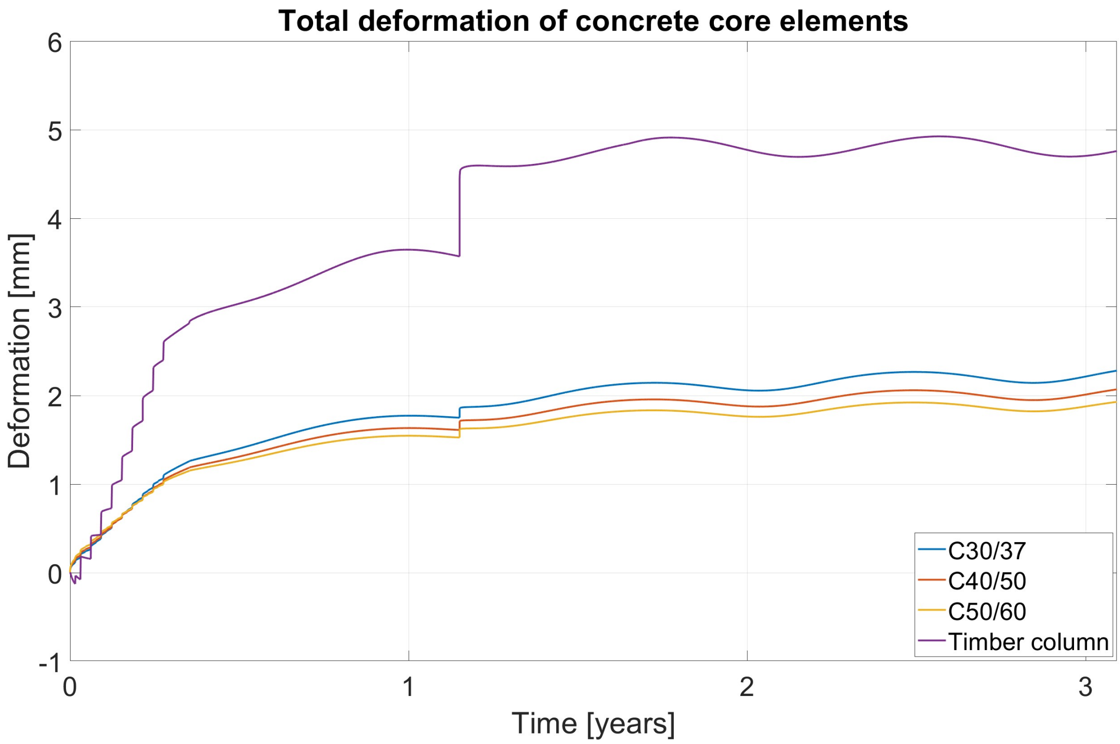


Figure 6.9: Vertical deformations of concrete core element with column & beam system for scenario: varying strength class of concrete. Timber column deformation is shown as a reference.

6.4 Varying dimensions of columns and walls

Different dimensions are analysed in the fourth scenario. The column and wall element vary between three dimensions whilst the strength class of timber elements are fixed, see Table 6.4. The concrete core element dimensions and strength class are fixed.

Table 6.4: Specific parameters for scenario: varying strength class of concrete.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c	280 280 230	315 405 540	6000	12
Walls	C24	140 180 220	1000 1000 1000	3000	12
Beams	GL30c	230	675	7300	12
Core	C40/50	300	1000	3000	-

Figure 6.10 and Figure 6.11 shows the resulting deformation of the timber column as well as the timber wall element for three different dimensions. Both figures show a behaviour stating that larger dimensions and cross-sections equals smaller deformations.

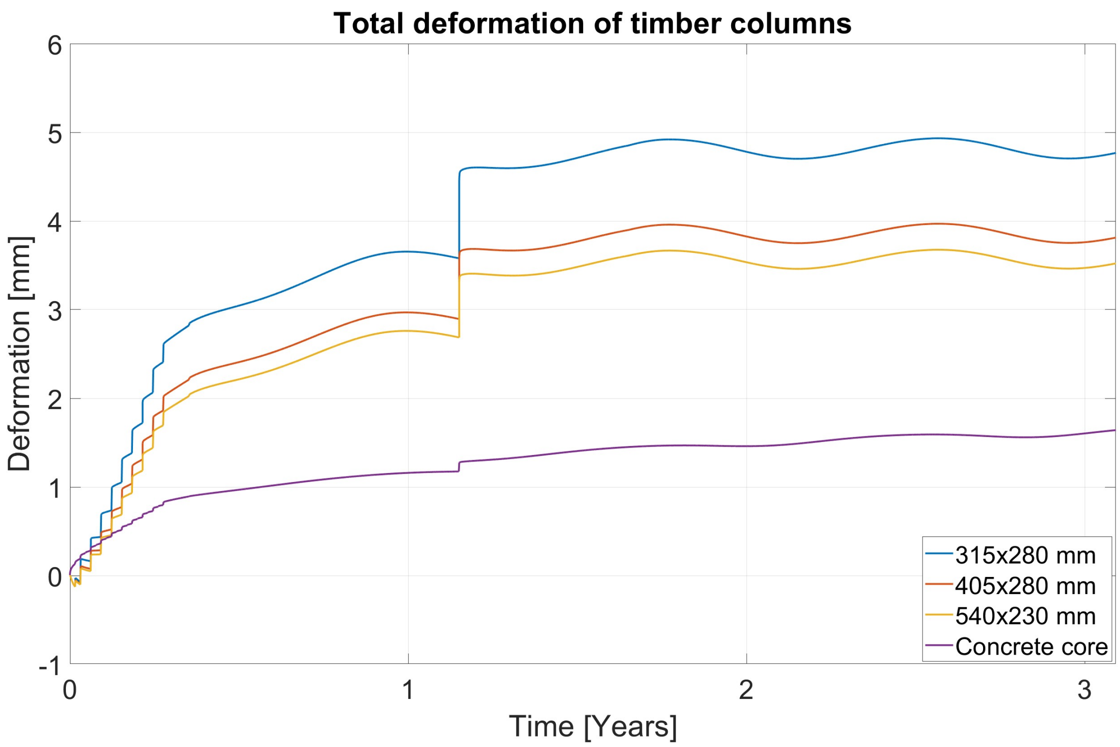


Figure 6.10: Vertical deformations of timber column for scenario: varying dimensions of columns and walls. Concrete core element deformation is shown as a reference.

Figure 6.11: Vertical deformations of timber wall element for scenario: varying dimensions of columns and walls. Concrete core element deformation is shown as a reference.

6.5 Change of initial moisture content in timber

The initial moisture content of timber may differentiate between different EWPs. The fifth scenario analyses how the initial MC influences the resulting vertical deformation of the timber component. Two different EWPs of columns as well as one type of wall, each with three different initial MC, are analysed. The different initial moisture contents analysed in the current scenario are chosen as the standard value as well as a value above and a value below. The different parameters relevant in this scenario are presented in Table 6.5.

Table 6.5: Specific parameters for scenario: different initial moisture content in timber. The standard initial moisture content is listed first.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c	280	315	6000	12 15 8
	Kerto-S	280	315	6000	6 12 4
Walls	C24	140	1000	3000	12 15 8
Beams	GL30c	230	675	7300	-
Core	C40/50	300	1000	3000	-

Figure 6.12 and Figure 6.13 show the resulting deformation of two different timber columns with varying initial MC. Figure 6.14 shows the resulting deformation of a timber wall element with varying initial MC.

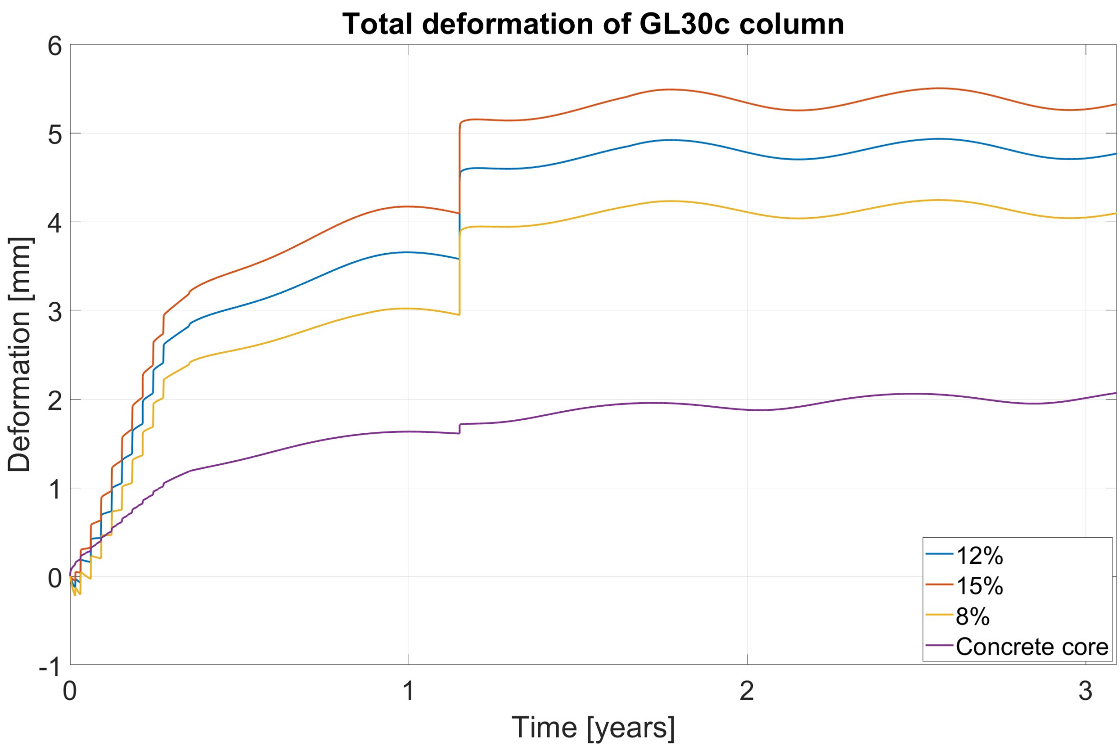


Figure 6.12: Vertical deformations of timber column of GL30c for scenario: varying initial moisture content. Concrete core element deformation is shown as a reference.

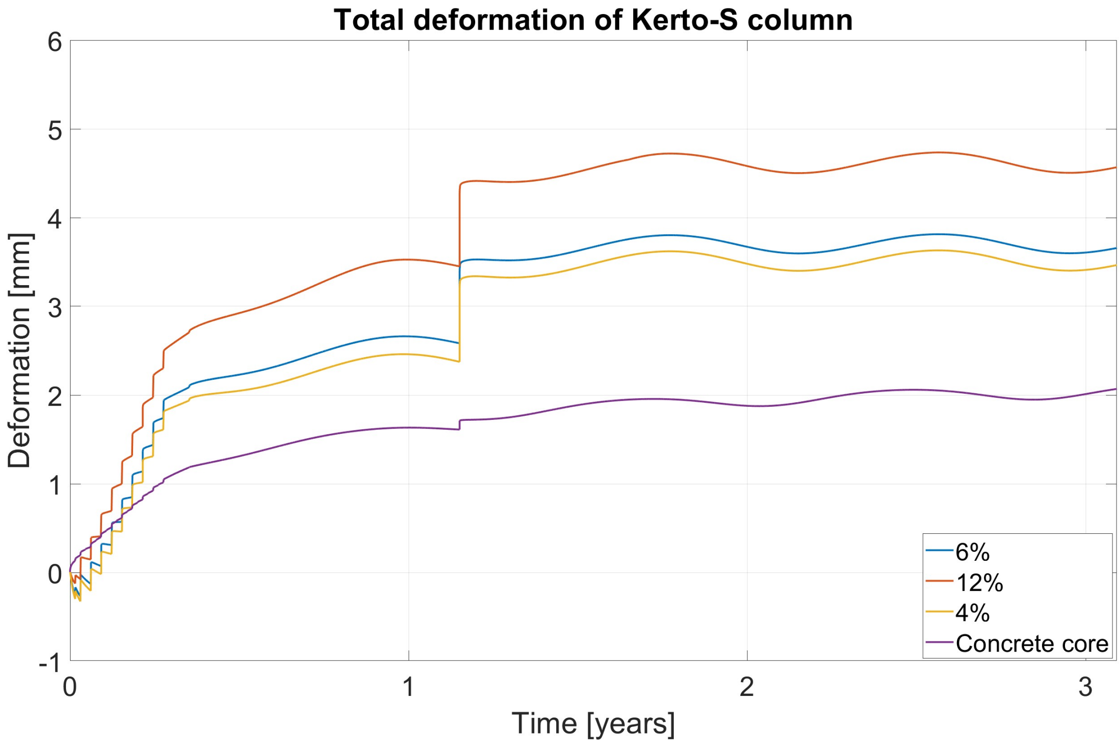


Figure 6.13: Vertical deformations of timber column of Kerto-S for scenario: varying initial moisture content. Concrete core element deformation is shown as a reference.

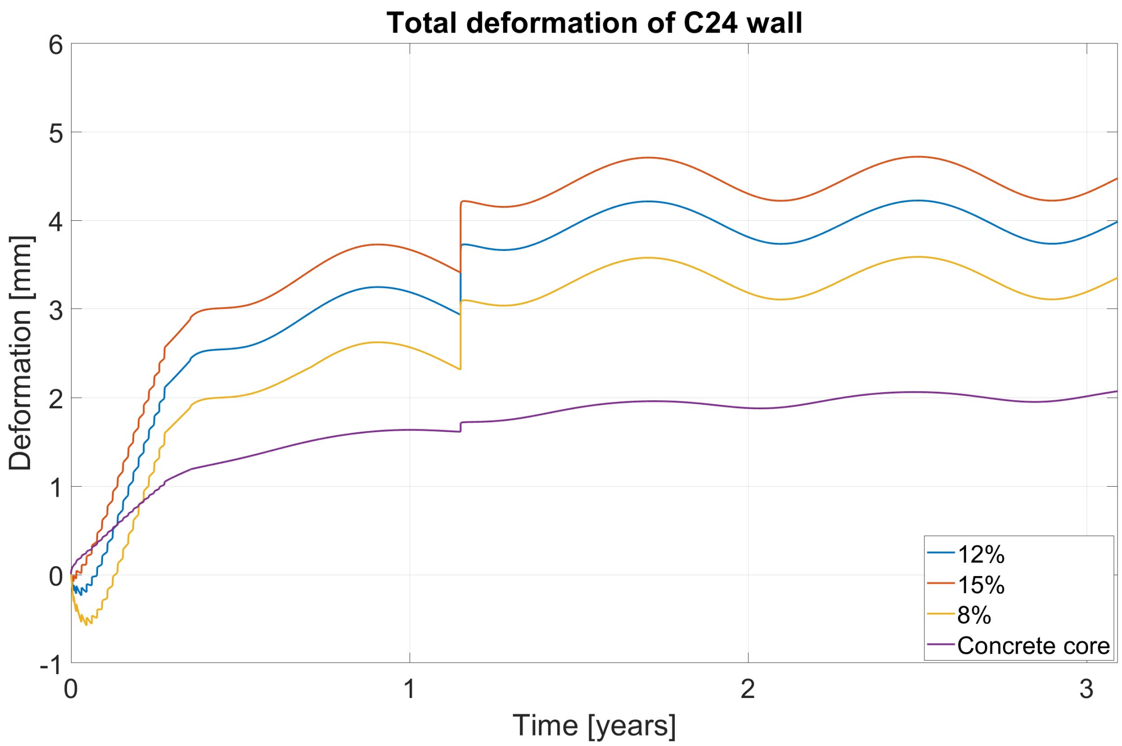
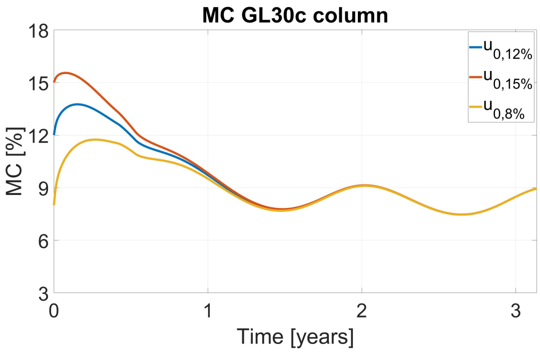


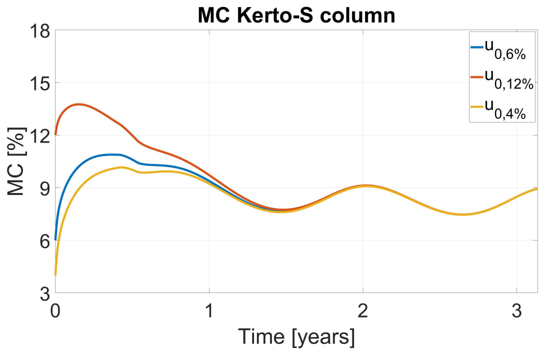
Figure 6.14: Vertical deformations of timber wall element of C24 for scenario: varying initial moisture content. Concrete core element deformation is shown as a reference.

Figure 6.12 contains deformation curves of three different initial moisture contents regarding a GL30c column and it shows a significant difference between the different cases. A similar behaviour is seen in Figure 6.13, where a Kerto-S column is studied, as well as in Figure 6.14, regarding a C24 wall. According to all three figures, a larger initial moisture content equals increase of the final deformation whilst a smaller initial value entails smaller deformations.

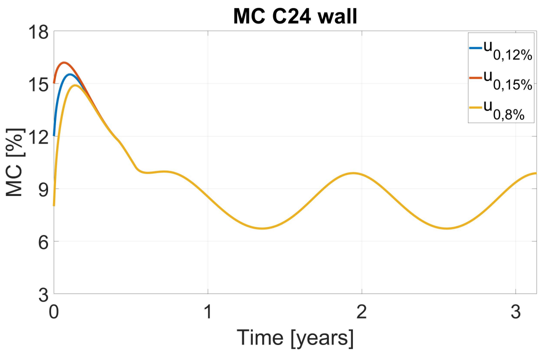
Figure 6.15 show a convergence study performed on each initial moisture content case for each EWP.



(a) GL30c column.



(b) Kerto-S column.



(c) C24 wall element.

Figure 6.15: Convergence study of each EWP's initial moisture content.

The case regarding GL30c, see Figure 6.15a, show a converged behaviour at 1.3 years. Figure 6.15b, showing the different cases of initial MC for a Kerto-S column, present a converged behaviour at approximately 1.5 years. The C24 wall element, see Figure 6.15c, show a converged behaviour at around four months.

6.6 Parallel or serial construction process

The last scenario analyses and compares the impact of applying the parallel construction process versus the serial construction process. For each process, regarding all building components, the same dimensions as well as strength classes are applied, see Table 6.6. The difference is limited to whether the concrete core is constructed parallel with the timber elements or before the timber is erected.

Table 6.6: Specific parameters for scenario: parallel or serial construction process.

	Strength class	Thickness/ Width [mm]	Height [mm]	Length [mm]	Initial moisture content [%]
Columns	GL30c	280	315	6000	12
Beams	GL30c	230	675	7300	12
Core	C40/50	300	1000	3000	-

Figure 6.16 illustrates the deformation behaviour of both processes, for timber as well as concrete elements. The figure shows that the difference of final deformation regarding timber elements is insignificant whilst the construction process shows to have larger impact on concrete elements.

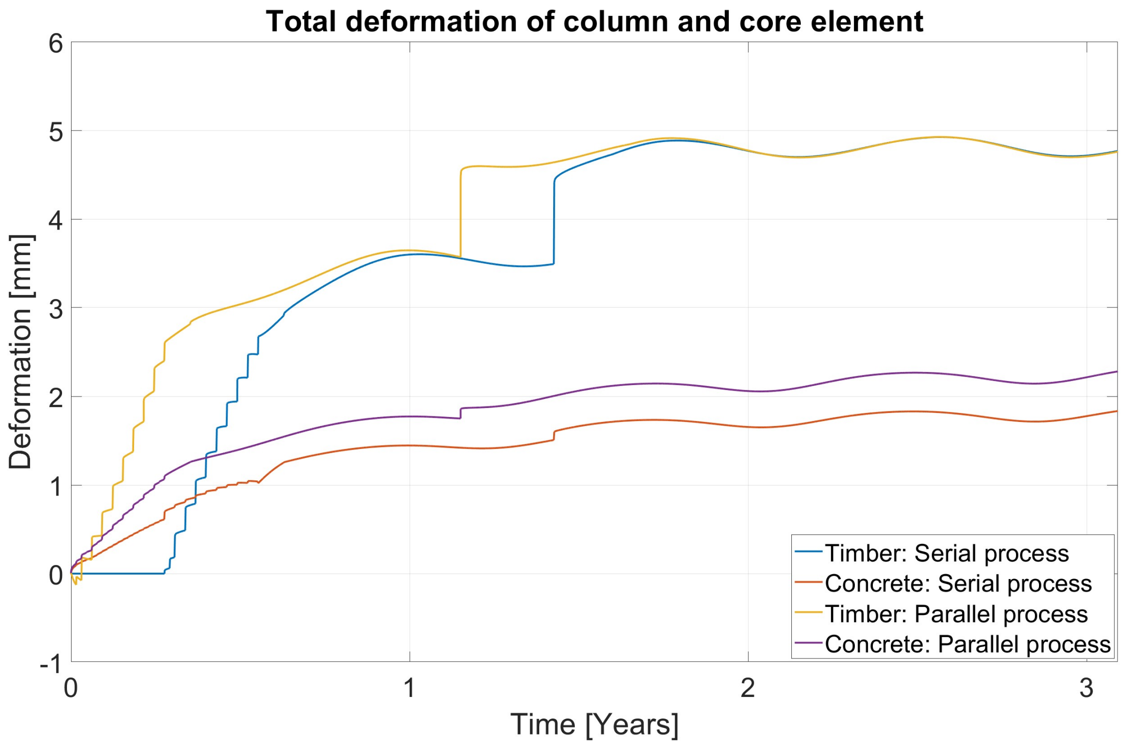


Figure 6.16: Vertical deformations of timber column and concrete core element for scenario: parallel or serial construction process.

6.7 Comparison of parameters

All scenarios are graded against a reference value specific for each scenario to determine the impact of the specific parameter. The reference value is decided according to what is set as standard in this project. For example, the strength class of columns is GL30c and the dimensions are 315x280 mm, according to preliminary sizing. The difference in deformation at specific times are extracted and a percentage difference is calculated. For example, in Scenario 6.1, where the column is considered the reference case, the final wall deformation is approximately 85% of the final column deformation.

Figure 6.17 shows the difference of deformation at time of adding roof load to the building. The loading occurs at the same time for all scenarios except the last one, Scenario 6.6.

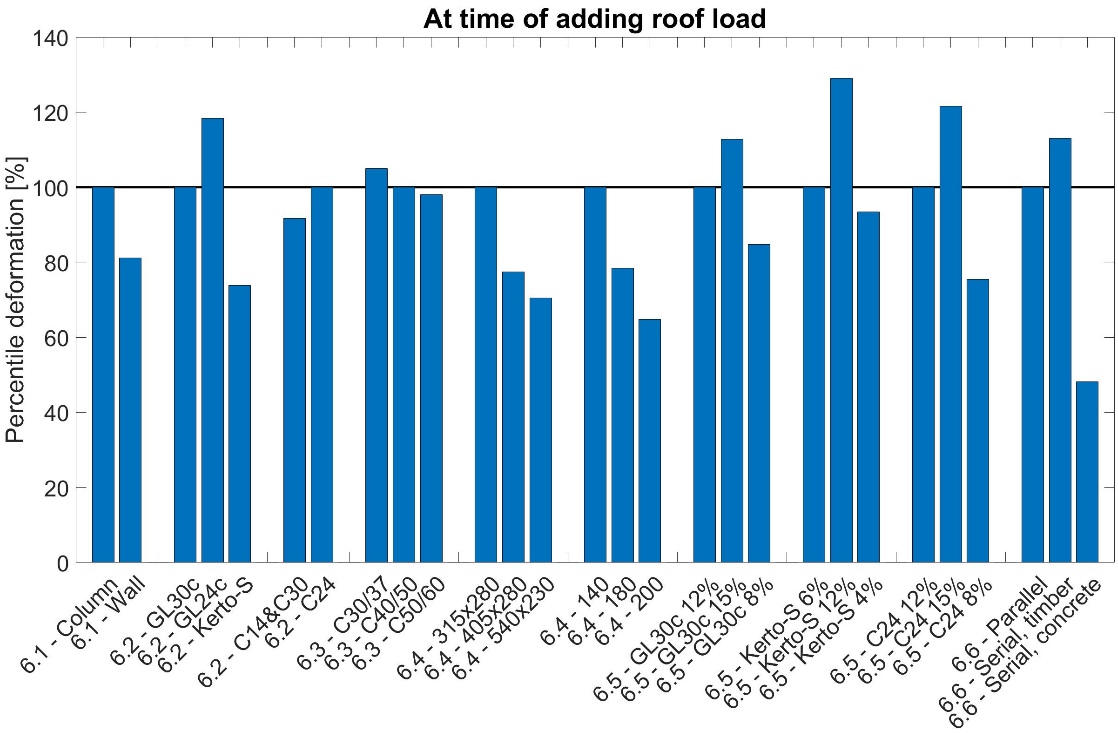


Figure 6.17: Comparison of deformation at time of adding roof load for all scenarios.

Figure 6.18 shows the difference of deformation at time of adding office load to the building. The loading occurs at the same time for all scenarios except the last one, Scenario 6.6. This as the serial construction process shifts the time of loading. Figure 6.19 shows the difference of deformation at three years after the start of construction and is considered a state where the deformation behaviour has converged.

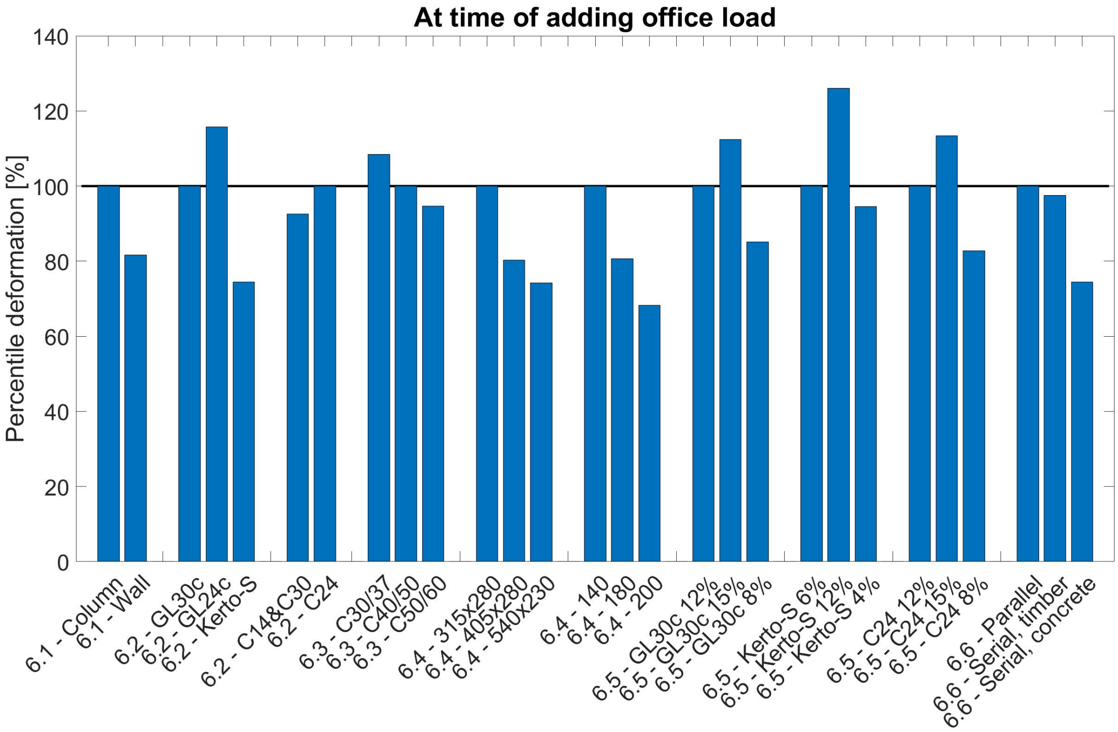


Figure 6.18: Comparison of deformation at time of adding office load for all scenarios.

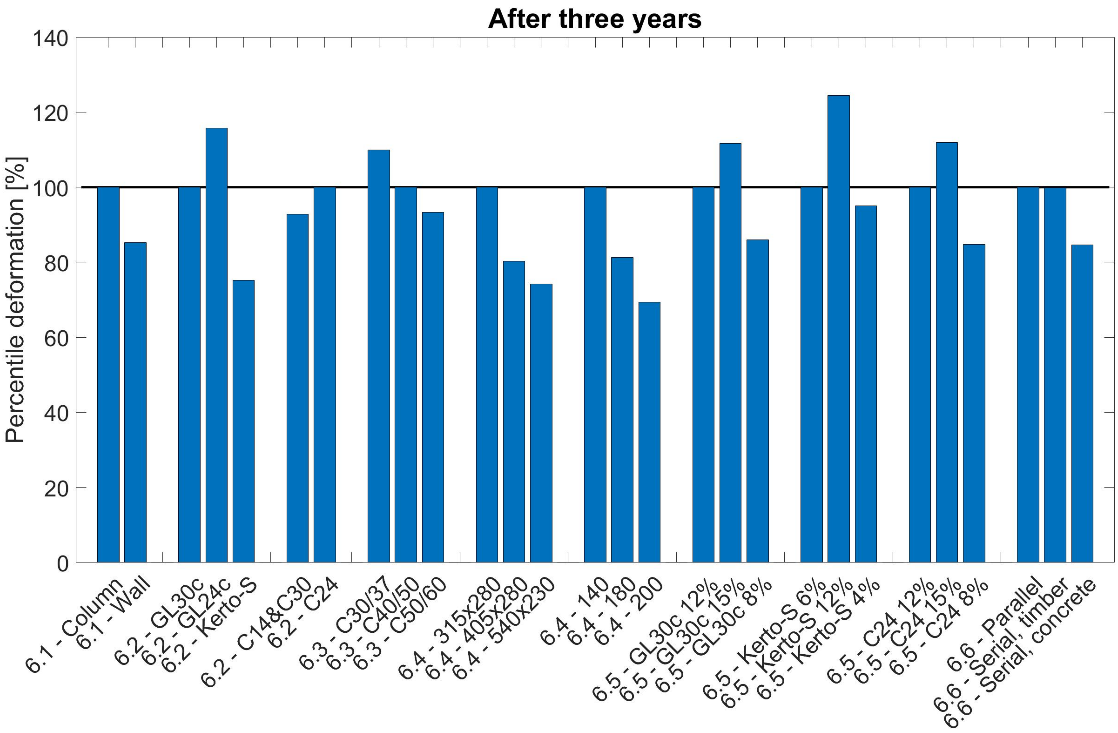


Figure 6.19: Comparison of deformation at three years.

7

General analysis of building

A general analysis is performed on the fictitious building, with a structural system composed of columns and beams, see System 1 in Subchapter 4.1.1. In the analysis, total deformations of timber and concrete elements are compared. To be able to compare the deformations all vertical elements are considered equal in height. Since the columns are continuous over two storeys, the column deformation is divided in half. The analysis is performed for three different building heights, with the building having 10 stories, 20 stories and 30 stories. The columns have been dimensioned in ULS for each specific load case resulting in the cross-section area of the columns varying, i.e. columns are smaller in the 10-storey building compared to the 20-storey and the 30-storey buildings.

7.1 Building with 10 stories

The total building height with 10 stories is approximately 30 meters. The dimensions of different components regarding the specific case are presented in Table 7.1. Figure 7.1 show the accumulated deformation of each column as well as each core element.

Table 7.1: Specific parameters for 10-story building.

	Thickness/ Width [mm]	Height [mm]	Length [mm]
Columns	190	225	6000
Beams	230	675	7300
Core	300	1000	3000

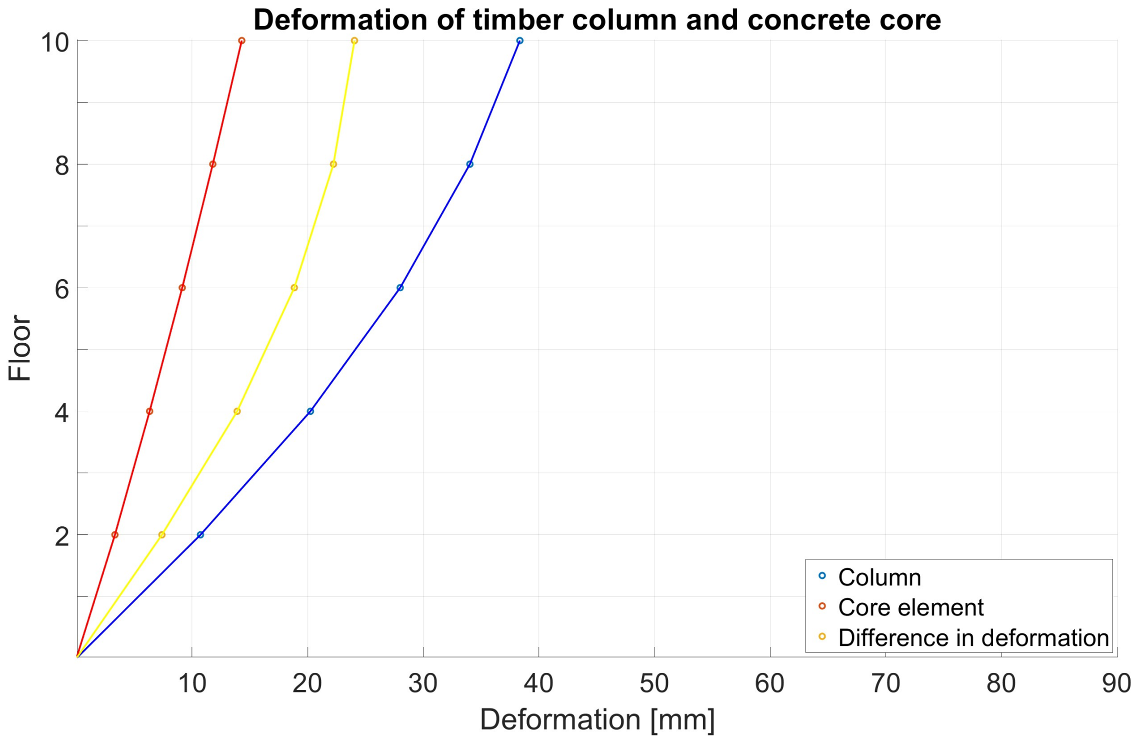
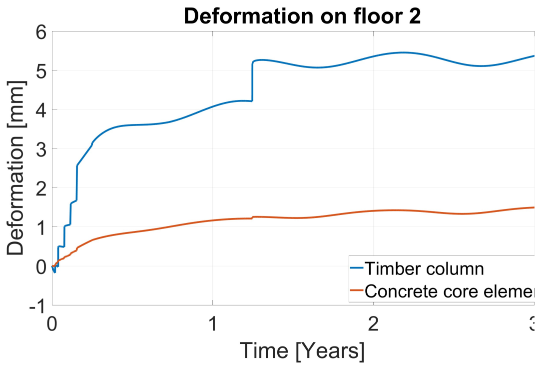
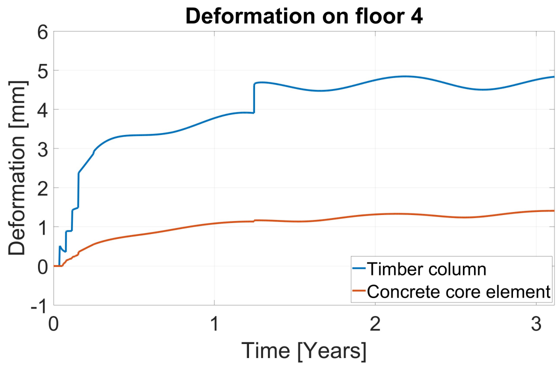


Figure 7.1: The accumulated deformation, floor by floor, of each timber column and concrete core element, i.e. the change in position from original position. The yellow line represents the difference between the components.

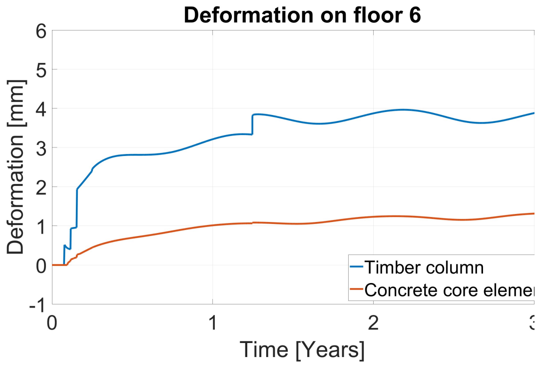
Figure 7.1 show the accumulated deformation, floor by floor, of the different elements. The deformation regarding the concrete elements show a linear behaviour whereas the change of the timber columns are exponential. Therefore, the growth of the difference between timber and concrete will decrease with each increasing floor level, see the yellow line. Further on, Figure 7.2 illustrates the deformation change of selected columns and core elements over time.



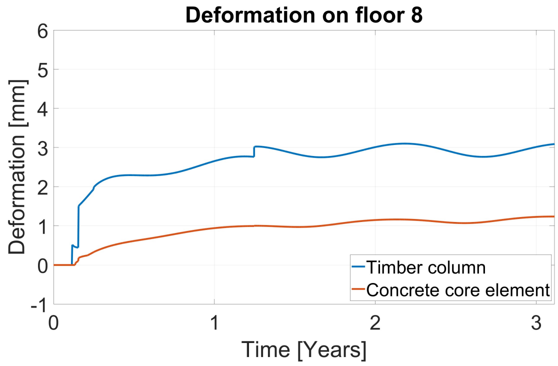
(a) Deformation of timber column and concrete core element on floor 2.



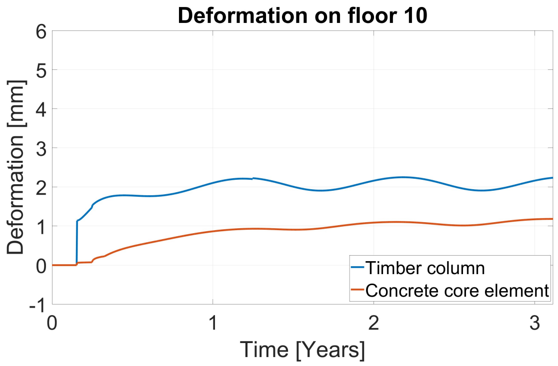
(b) Deformation of timber column and concrete core element on floor 4.



(c) Deformation of timber column and concrete core element on floor 6.



(d) Deformation of timber column and concrete core element on floor 8.



(e) Deformation of timber column and concrete core element on floor 10.

Figure 7.2: The figures illustrate the change of deformation between floors.

The largest deformation considering each individual column in Building 10 (the 10-storey building) occurs in the bottom column, see Figure 7.2a. The higher up in terms of floors, the less load is applied on each column and less deformation is seen. This pattern is presented in Figure 7.2a - 7.2e together with the concrete deformation behaviour, which does not show any noticeable change.

7.2 Building with 20 stories

The total building height with 20 stories is approximately 60 meters. The dimensions of different components regarding the specific case are presented in Table 7.2. Figure 7.3 show the accumulated deformation of each column as well as each core element.

Table 7.2: Specific parameters for 20-story building.

	Thickness/ Width [mm]	Height [mm]	Length [mm]
Columns	280	315	6000
Beams	230	675	7300
Core	300	1000	3000

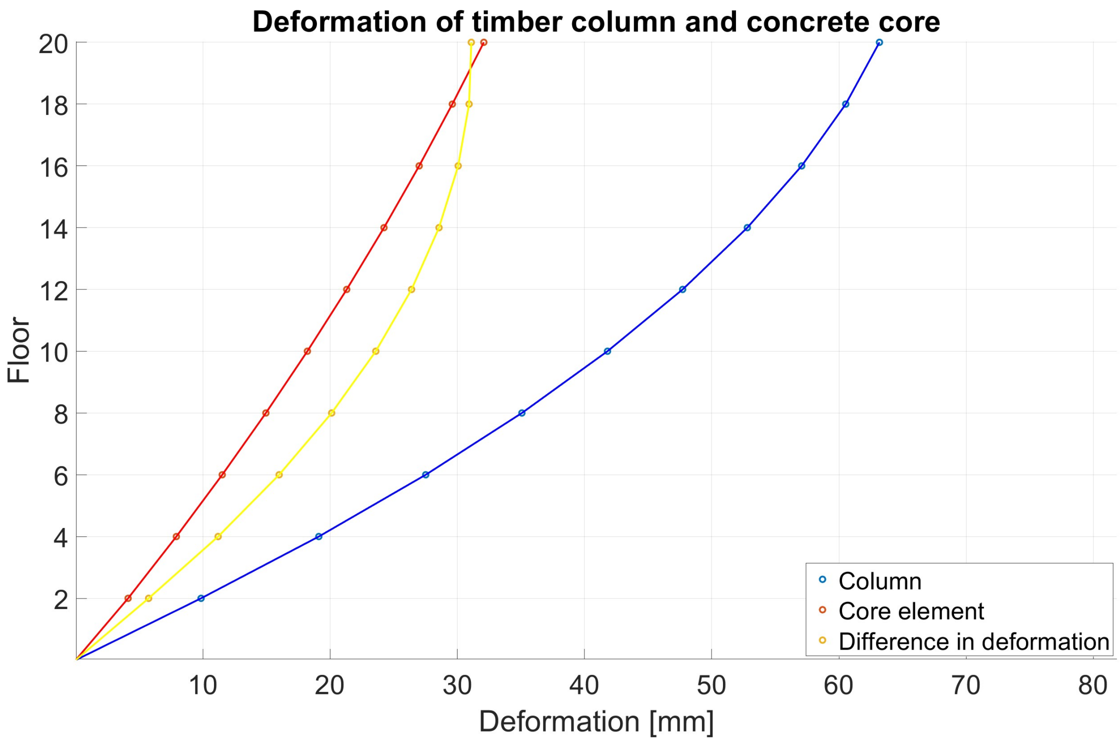
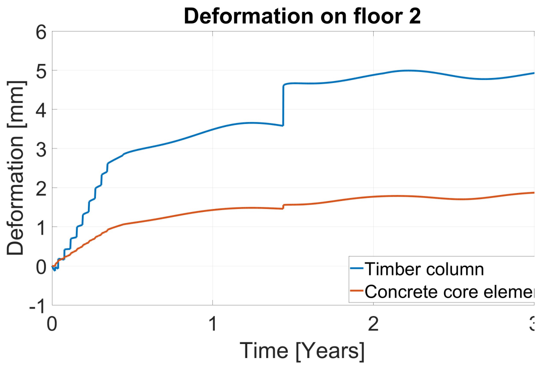
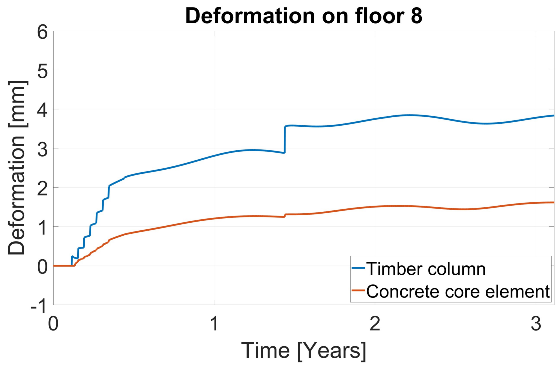


Figure 7.3: The accumulated deformation, floor by floor, of each timber column and concrete core element, i.e. the change in position from original position. The yellow line represents the difference between the components.

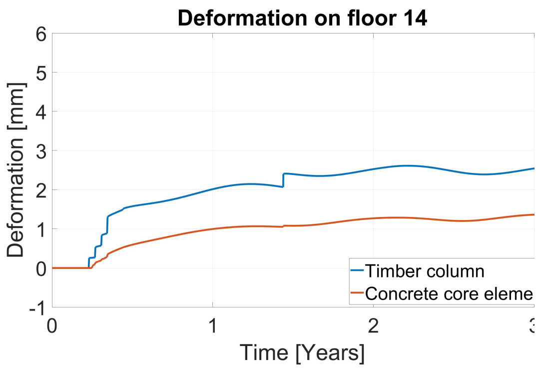
The plot with the accumulated deformations in Figure 7.3 show an exponential behaviour of the timber column, with the concrete showing tendency of a slight bend. The yellow line, the difference between timber and concrete, show a behaviour where the growth of increased difference decreases with each floor and is approximately zero at the top floors. Further on, Figure 7.4 illustrates the deformation change of selected columns as well as core elements over time.



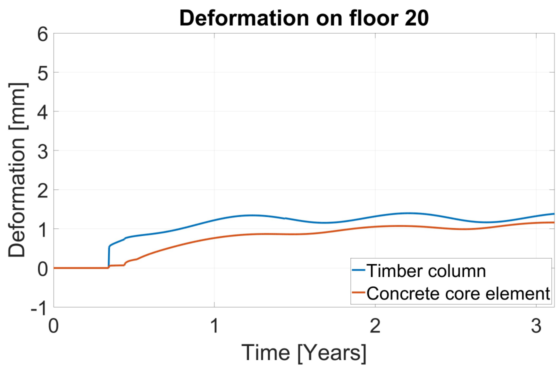
(a) Deformation of timber column and concrete core element on floor 2.



(b) Deformation of timber column and concrete core element on floor 8.



(c) Deformation of timber column and concrete core element on floor 14.



(d) Deformation of timber column and concrete core element on floor 20.

Figure 7.4: The figures illustrate the change of deformation between floors.

The largest deformation considering each individual column in Building 10 (the 10-storey building) occurs in the bottom column, see Figure 7.4a. The higher up in terms of floors, the less load is applied on each column and less deformation is seen. This pattern is presented in Figure 7.4a - 7.4d together with the concrete deformation behaviour, which does not show any noticeable change.

7.3 Building with 30 stories

The total building height with 30 stories is approximately 90 meters. The dimensions of different components regarding the specific case are presented in Table 7.3. Figure 7.5 show the accumulated deformation of each column as well as each core element.

Table 7.3: Specific parameters for 30-story building.

	Thickness/ Width [mm]	Height [mm]	Length [mm]
Columns	330	405	6000
Beams	230	675	7300
Core	300	1000	3000

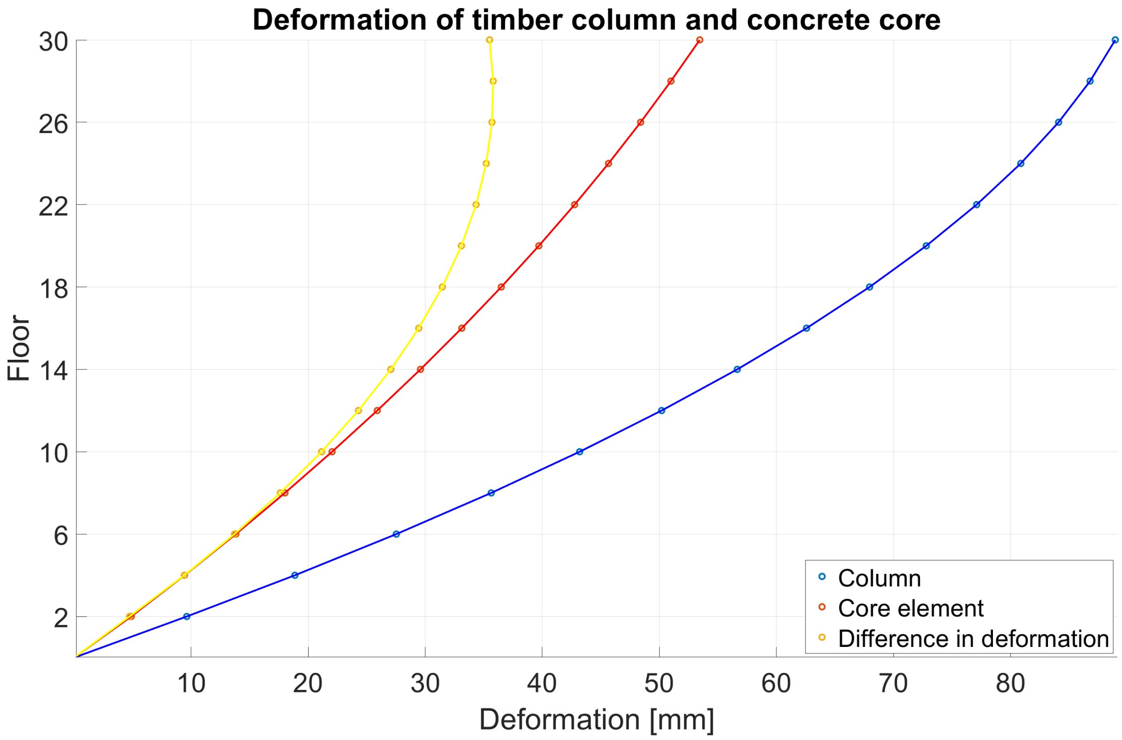
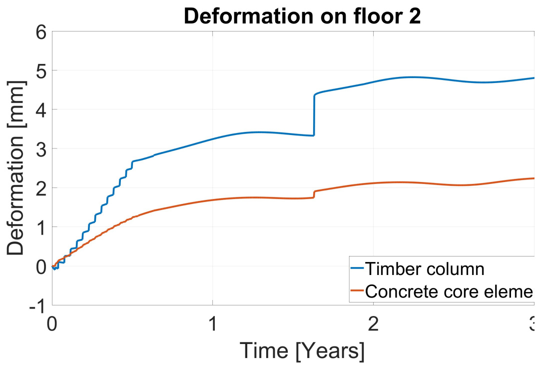
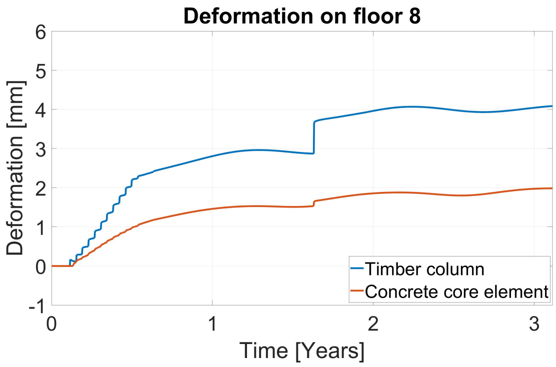


Figure 7.5: The accumulated deformation, floor by floor, of each timber column and concrete core element, i.e. the change in position from original position. The yellow line represents the difference between the components.

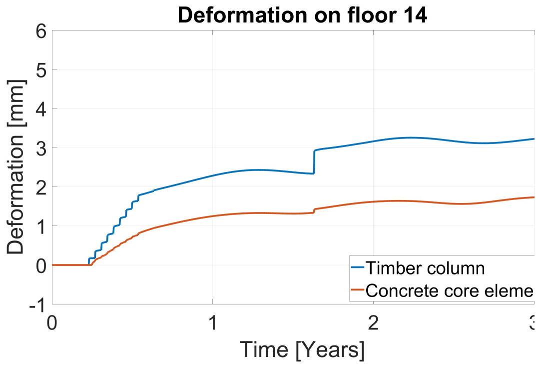
In Figure 7.5, an exponential behaviour is experienced regrading the timber columns. The behaviour of the core elements present a slight, convex bend. This results in the growth of increased difference reaching its peak at approximately floor 28. Further on, Figure 7.6 illustrates the deformation change of selected columns as well as core elements over time.



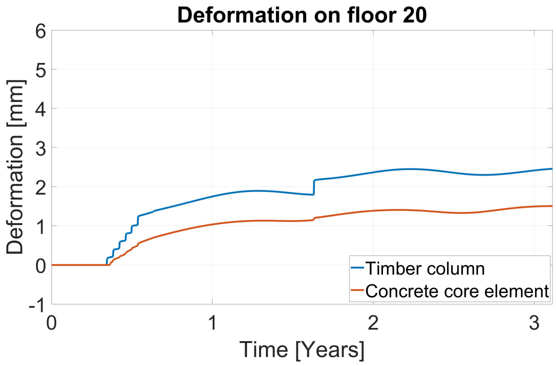
(a) Deformation of timber column and concrete core element on floor 2.



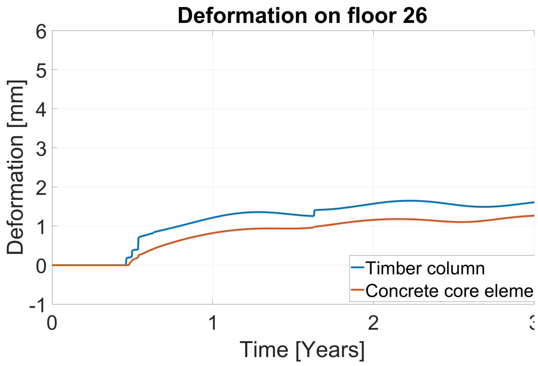
(b) Deformation of timber column and concrete core element on floor 8.



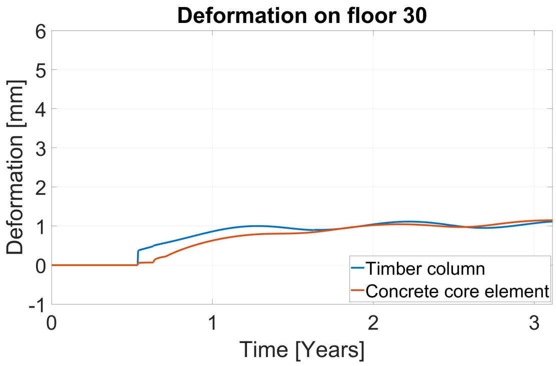
(c) Deformation of timber column and concrete core element on floor 14.



(d) Deformation of timber column and concrete core element on floor 20.



(e) Deformation of timber column and concrete core element on floor 26.



(f) Deformation of timber column and concrete core element on floor 30.

Figure 7.6: Figures illustrating the change of deformation between floors.

The largest deformation considering each individual column in Building 10 (the 10-storey building) occurs in the bottom column, see Figure 7.2a. The higher up in terms of floors, the less load is applied on each column and less deformation is seen. This pattern is presented in Figure 7.6a - 7.6f together with the concrete deformation behaviour, which does not show any noticeable change.

8

Discussion

The parametric study and the general analysis are in this chapter discussed, separately and in their entirety. The scenarios are compared to each other and evaluated, along with a brief discussion regarding connections. The part covering the general analysis compares the three buildings whilst also drawing parallels to the parametric study.

8.1 Parametric study

The parametric study considers six different scenarios which are compared individually and to each other, and show variation of impact on the final deformation. All scenarios show a negative deformation early on which means that the timber element stretches. This is due to swelling strain that depends on an increasing moisture content at the start of construction.

In Subchapter 6.7, the different scenarios are presented in bar charts and compared to each other at three specific times. Rather small differences are located in Scenario 6.3, varying concrete strength classes, for all time steps and it is concluded that the material properties of the concrete do not significantly impact the deformation. Although, the concrete seems to be more influenced by the time aspect according to Scenario 6.6 which shows a difference between parallel process and serial process. A parameter showing significant differences in deformations is Scenario 6.4, the analysis regarding variation of dimensions. The difference in deformation behaviour applies for both the case with columns and the case with wall elements proving that dimensioning can have large impact on the final result.

Another parameter showing larger differences is the initial moisture content, analysed in Scenario 6.5, where an increased initial MC entails larger deformation and vice versa. In practice, the initial moisture content varies between different EWPs. The parameter was therefore analysed separately for three different timber materials to gain understanding of how large of an impact the initial value has. The study shows that a lot less deformation could be obtained if the initial MC was lower, meaning that a decreased initial MC would be beneficial. Although, this must be further investigated as lowering the initial MC might not be applicable in production.

Each case of initial MC regarding columns, see Figure 6.15a and Figure 6.15b, show a difference of behaviour until approximately 1.5 years meaning that the initial MC influences the deformation up until this time. After 1.5 years, the MC has converged and the expected deformation behaviour will be the same for all different initial MCs. The curves regarding the wall element, shown in Figure 6.15c, converges much earlier compared to the columns, showing a converged behaviour at approximately four months. This could be explained by the wall being thinner than the column, meaning it is expected to reach equilibrium with the surrounding conditions at an earlier stage.

In the second scenario, considering different strength classes of timber, the differences could be explained by the variation in modulus of elasticity. As previously stated, the largest impact on the total strain is the elastic part, calculated as stress divided by the E-modulus. This concludes, a larger E-modulus equals a smaller strain. For example, Kerto-S shows, in the scenario, the smallest deformation and has the largest E-modulus. Additionally, if also considering scenario five, where the impact of having the standard initial moisture content changed is studied, a standard initial MC of 6% for the Kerto-S column can be a contributing factor to its overall smaller deformation.

The part of the parametric study analysing the different construction processes, see Figure 6.16, show an insignificant difference of the final deformation in the timber columns whilst the concrete behaviour differentiate. The serial process allows the concrete to harden before additional load is applied which is why this process show the smallest deformation. A result with smaller deformations is usually sought, however, when working with two different materials the aim is to achieve similar behaviours leaving the parallel process being more suitable for the case.

8.1.1 Connections

The parametric study does not cover, nor does the general analysis, the importance and impact of connections between different elements. All scenarios consider a simple column to column or wall to wall connection where the elements are joint together solely end to end grain, with the assumption of the connection acting without additional deformation or tolerance. But if considering errors, a possible source could be the fibres of each element penetrating each other, namely creating a local breakage around the connection. To avoid this type of problem, a steel plate can be added between the elements, providing the connection with a stronger foundation.

One problem with a hybrid structure of timber and concrete is that the materials have different tolerances. Therefore, the connection between beam and either timber column/wall or concrete core is crucial. This can be managed in different ways, for example as they have done in Brock Commons and Arbo, see Subchapter 3.1.3.4. Those connections provide adjustments possibilities minimizing the initial differences between timber and concrete elements.

The connection concerning column/wall to beam can also affect the outcome of the final vertical deformation. The project has considered the beam spanning between vertical elements connected with a metal hanger. The connection is assumed to be without any errors and not affecting the vertical deformations. If instead the chosen type is a notch connection, see Figure 3.4b, the column will have a larger deformation locally at the notch. This as the column will experience higher stresses at the attachment of the beam due to the smaller cross-section.

8.2 General analysis

According to the analysis of the two structural systems, Scenario 6.1, building with load bearing walls are the better option in relation to deformations. However, the general analysis is performed on a structural system composed of columns & beams. This as the differences are larger between timber columns and concrete core elements, constituting a worst case scenario. Further, as flexibility is an important factor for an office building, columns are the more suitable choice as it enables a varied floor plan optimal for its occupants. Additionally, nowadays, newly built office buildings tend to have larger windows or glass facades which also benefit from a system of columns rather than walls.

Another advantage of a column & beam-system is the material efficiency. The total volume of timber is likely to be smaller for a column & beam-system compared to a system of load bearing walls. For this project, optimizing the column dimensions on floors higher up would contribute to less material usage. This would lead to less self weight and thereby smaller deformations on the bottom columns. As of now, the columns are dimensioned according to the highest load, i.e. the load on the bottom column. As the loads decrease with the height of the building, the column dimensions could be reduced.

In the general analysis three different building heights are analysed and compared to each other, all of which having a structural system composed of columns & beams. When analysing the figures in Subchapter 7.1 - 7.3, the bottom column, with the largest individual deformation, shows the smallest change of position while the opposite applies for the top column of the building. This due to the accumulated deformation of all columns affecting the top column, in contrast to the bottom column, which is only affected by its own deformation.

Comparing all three buildings, an increased number of floors entails increased accumulated deformation. The timber behaviour for all cases are exponential whilst the concrete show a linear behaviour for Building 10 and show tendency of a convex appearance for Building 20 and 30. The changed concrete behaviour between buildings leads to a shifted behaviour of the yellow line, i.e. difference in deformation between the column and core element. The maximum differences seem to appear at the top floor in Building 10, whilst the maximum is reached one or a few floors down from the top regarding Building 20 and 30. Considering the pattern shown for all three buildings, it is to assume that the pattern will continue for larger building heights. For example, a building with 40 stories will therefore likely have the largest difference appear at a floor level further down proportionally from the top floor compared to Building 30. The differences between timber columns and concrete core elements implies an inclination of the connected beam, meaning that the largest inclination will appear on the floors with maximum differences.

When studying the accumulated deformation-plots for Building 10, 20 and 30, it is shown that an inclination more or less can be expected throughout the height of all three buildings. The effect of this problem might be minimized by for example adapting the connecting joint between the beam and the concrete core element. If installing a joint allowing for vertical movements, the beam could adjust along with the deformations and remain horizontal.

Another factor of which could minimize the difference of deformation is to extend the initial length of the timber column/wall, making it longer than the height of the concrete core element. Taking the instant deformation into consideration, the result of linear elastic strain, an initial larger height would allow the timber component to deform into the desired height. Although, this does not account for the long-term behaviour deformations meaning that the final column height will be even less. However, as the elastic strain invokes the largest deformation, the difference from long-term behaviour is small in comparison.

The parametric study can be utilized to see which parameters are most profitable to adjust to achieve smaller deformations. For example, choosing a lager dimension will minimize the deformation, and so will selecting an EWP of higher strength class, i.e. higher E-modulus. Both these changes will reduce the deformation in timber with approximately 25%. Combining these two, or adjusting another parameter, will likely reduce the deformation further but not necessarily add up to 25% plus 25%. If decreased standard initial moisture content is implemented in the manufacturing, the final deformation could be reduced as well.

9

Conclusion

The vertical deformations of a tall timber and concrete hybrid building are influenced by multiple factors. This thesis work analyzes the long-term effects in timber as well as concrete. It shows that the total strains in timber mainly are influenced by the elastic, creep and shrinkage/swelling strain and that the total strains in concrete are equally influenced by elastic & creep and shrinkage.

Calculations performed in the general analysis regarding long-term effects show that differences of vertical deformations will appear between timber and concrete regardless of the height of the structure. How these differences are managed or minimized have been analysed through a parametric study. The study concludes that increasing the dimensions or choosing an EWP with a higher strength class will most effectively provide the timber component with smaller deformations. Additionally, changing the initial moisture content of the timber is also shown to impact the deformations where a lower initial MC is proven to result in smaller deformations and vice versa.

Further, the analysis regarding different structural systems conclude that load bearing wall elements equal smaller deformations compared to columns. Although, as other aspects contribute to the choice of structural system the deformations are necessarily not the crucial aspect.

It is also concluded that changing the concrete to a higher strength class does not improve the final deformation significantly. However, in some cases it might be beneficial to consider a lower strength class that results in higher deformations to achieve a smaller difference to the timber. This would not only be beneficial through a deformation point of view but also environmentally as a lower concrete strength class entails less cement, which has a large carbon footprint.

Moreover, whilst the timber remains unaffected by the choice of construction process, the concrete obtains smaller deformations when applying the serial process. It is therefore concluded that the parallel process is favourable when considering deformation differences between timber and concrete.

The general analysis indicates a pattern where a building with increased number of floors will have its largest difference between timber and concrete further down proportionally from the top floor. This infers that the taller the building, the lower down in terms of floors the peak difference is expected to appear. The largest adjustments will be needed where the maximum difference appears to account for the deformations.

9.1 Design recommendation

The following statements are recommendations regarding design and construction.

- Choose an engineered wood product with a reasonable high strength class.
- Consider a larger dimension of the timber cross-section.
- If possible, strive towards a lower initial moisture content in timber.
- From a computationally efficient perspective, most important strains to consider in timber are elastic, visco-elastic and shrinkage/swelling.
- Choose a concrete with low strength class to reduce deformation differences between the two materials.
- Apply parallel process as it is favourable regarding deformation difference.

9.2 Further studies

In the following section, a few topics are presented which might be of interest to investigate in further studies. The in detail investigation of how connections impact the structure and its individual components are one aspect to look further into. Also how the connections can be utilized to adjust the difference in floor height and avoid inclinations. Second order effects need to be investigated due to inclinations and eccentricity.

Strains related to creep and shrinkage are dependant on RH as well as temperature variations. In this project, their behaviours are approximated as sinus curves meaning the materials will react differently depending on when during the year construction starts. The construction start of this project is January 1st and the deformations might decrease if the construction starts later during the year.

The parametric study only considered changing one parameter at a time. Therefore, several parameters could be investigated at the same time to gather understanding of how they interact with each other and what result can be achieved.

As mentioned in previous subchapters, the initial MC is proven to have large impact on the final deformation and decreasing it would minimize the deformations. However, the application in practice may not be realistic and would have to be investigated through literature study and interviews with production companies.

The timber components in this project has been dimensioned and designed considering only permanent and variable loads. Aspects such as accidental loads and fire has not been accounted for and should be included in further studies.

When calculating the creep strains in concrete, a mean value of the relative humidity was assumed. In reality, the RH varies with time and will result in new creep functions with each change. Due to the complexity of applying this in the superposition method, using a mean value was considered a reasonable simplification. In further studies, this complex problem could be investigated.

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A

Appendix

A.1 Preliminary sizing: 20 floors

Input data

Building dimensions

Number of floors	n_{floors}	18	plus additional bottom and top floor
Width	w [m]	10	
Height	h [m]	57	

Material

Timber					
	Glulam		CLT		LVL
	GL30c	GL24c	C14 and C30	C24	Kerto-S
f_{c0k} [Pa]	2,45E+07	2,15E+07	2,40E+07	2,10E+07	3,50E+07
$f_{m.x.k}$ [Pa]	3,00E+07	2,40E+07	3,00E+07	2,40E+07	4,40E+07
$E_{0.x.05}$ [PA]	1,08E+10	1,91E+10	8,00E+09	7,40E+09	-
E_{t0mean} [Pa]	1,30E+10	1,10E+10	1,20E+10	1,10E+10	1,38E+10
ρ_{tk} [kg/m³]	390	365	350	350	480
u_0 [%]	12	12	12	12	6
k_{mod}	0,8	0,8	0,8	0,8	0,8
γ_M	1,25	1,25	1,25	1,25	1,2

Concrete			
	C30/37	C40/50	C50/60
f_{ck} [Pa]	3,00E+07	4,00E+07	5,00E+07
E_{cm} [Pa]	3,30E+10	3,50E+10	3,70E+10
ρ_c [kg/m³]	2500	2500	2500

Concrete components

Core

Height	h [m]	3
Thickness	t [m]	0,3
Length	l [m]	1

Roof

Span length	l [m]	7
Thickness	t [m]	0,265
Mass	m [kg/m²]	400
Self weight	g_k [N/m²]	3924

Timber components

Slab

Span length	l [m]	7
Thickness	t [m]	0,28
Mass	m [kg/m²]	126
Self weight	g_k [N/m²]	1236,06

Columns

		GL30c	GL24c	Kerto-S
Width	w [m]	0,28	0,28	0,288
Height	h [m]	0,315	0,36	0,33
Length	l [m]	3	3	3
Cross section	A [m²]	0,0882	0,1008	0,09504
Mass	m [kg/m²]	1170	1095	1440
Self weight	g_k [N/m²]	11477,70	10741,95	14126,40

Column check

Tributary area	A_t [m²]	24,6375	
Reduction factor	k_{mod}	0,7483	climate class 2
Partial coef.	γ_M	1,25	

Total load	F_{Ed} [N]	1,18E+06	1,19E+06	1,17E+06
Compressive stress	σ_d [Pa]	1,34E+07	1,18E+07	1,23E+07
Requirement	f_{c0d} [Pa]	1,47E+07	1,29E+07	1,44E+07
Check	$\sigma_d < f_{c0d}$	OK!	OK!	OK!
Utilization		91,28%	91,67%	85,50%

Ckecking slenderness of columns

Cracking length	l_e [m]	3	3	3
Equivalent width	w [m]	0,297	0,317	0,308
Slenderness ratio	λ	34,993	32,733	33,710
Relative slenderness	λ_{rel}	0,531	0,350	0,540
Instability factor	k_y	0,652	0,564	0,658
Reduction factor	k_c	0,969	0,994	0,968
	β_c	0,1	0,1	0,1
Requirement	k_c*f_{c0d} [Pa]	2,37E+07	2,14E+07	3,39E+07
Check!	$\sigma_d < k_c*f_{c0d}$	OK!	OK!	OK!

Beam

Beam lengths at different columns [m]				
At 0105		7,3		
		GL30c	GL24c	Kerto-S
Height	h [m]	0,675	0,675	0,79
Width	w [m]	0,23	0,28	0,144
Cross section	A [m²]	0,15525	0,189	0,11376
E-modulus	E_{mean} [Pa]	1,30E+10	1,10E+10	1,38E+10
Reduction factor	k_{def}	0,8	0,8	0,8
Reduction factor	ψ_2	0,3	0,3	0,3
Final E-modulus	E_{fin} [Pa]	1,05E+10	8,87E+09	1,11E+10
Inertia	I [m^4]	5,89E-03	7,18E-03	5,92E-03

ULS: Stress control

Reduction factor	k_{mod}	0,7
Partial coef.	γ_M	1,25
Tributary area	A_t [m²/m]	6,75
Lenght	l [m]	7,3

Total load	$q_{d.beam}$ [N/m]	3,96E+04	3,97E+04	3,95E+04
Moment	M_{Ed} [Nm]	2,64E+05	2,64E+05	2,63E+05
Compressive stress	$\sigma_{c.d}$ [Pa]	1,51E+07	1,24E+07	1,76E+07
Tensile stress	$\sigma_{t.d}$ [Pa]	-1,51E+07	-1,24E+07	-1,76E+07
Requirment	$f_{m.d}$ [Pa]	1,68E+07	1,34E+07	2,46E+07
Check!	$\sigma_d < f_{m.d}$	OK!	OK!	OK!
Utilization ratio		89,86%	92,50%	71,31%

SLS: Deflection check

Total load	$q_{d.quasi.beam}$ [N/m]	1,50E+04	1,51E+04	1,50E+04
Final E-modulus	E_{fin} [Pa]	7,22E+09	6,11E+09	7,67E+09

Instant deflection	w_{inst} [m]	0,0130	0,0127	0,0122
Requirement	L/500	0,0146	0,0146	0,0146
Check!	w_{inst} < L/500	OK!	OK!	OK!
Utilization ratio		89,31%	87,18%	83,50%
Creep deflection	w_{creep} [m]	0,0104	0,0102	0,0098
Total deflection	w_{tot} [m]	0,0235	0,0229	0,0219
Requirement	L/300	0,0243	0,0243	0,0243
Check!	w_{tot} < L/300	OK!	OK!	OK!
Utilization ratio		96,45%	94,15%	90,18%

Wall

		C14 and C30	C24
Width	w [m]	1	1
Height	h [m]	3	3
Thickness	t [m]	0,13	0,14
Cross section	A [m²]	0,13	0,14
Tributary area	At [m²/m]	3,65	3,65
Density	ρ [kg/m³]	350	350
Reference E-modulus	E_{ref} [Pa]	1,20E+10	1,10E+10

Inertia calculation for wall with C14 & C30

Layers	Class	t_i	E_i	$E_i/E_{ref}*b*t_i^3/12$	a_i	$E_i/E_{ref}*b*t_i*a_i$
1	C30	0,03	1,20E+10	2,25E-06	-0,05	7,50E-05
2	C14	0,02	7,00E+09	-	-	-
3	C30	0,03	1,20E+10	2,25E-06	0	0
4	C14	0,02	7,00E+09	-	-	-
5	C30	0,03	1,20E+10	2,25E-06	0,05	7,50E-05

Inertia calculation for wall with C24

Layers	Class	t_i	E_i	$E_i/E_{ref}*b*t_i^3/12$	a_i	$E_i/E_{ref}*b*t_i*a_i$
1	C24	0,03	1,10E+10	2,25E-06	-0,055	9,08E-05
2	C24	0,03	1,10E+10	-	-	-
3	C24	0,02	1,10E+10	6,67E-07	0	0,00E+00
4	C24	0,03	1,10E+10	-	-	-
5	C24	0,03	1,10E+10	2,25E-06	0,055	9,08E-05

		C14&C30	C24
Net moment of inertia	$I_{x.net}$ [m^4]	1,57E-04	1,87E-04

levels w/ P	2
length of dist	4
Slab length [m]	6,75

Load from beam	C14 and C30		C24	
	g [N/m]	G [N]	g [N/m]	G [N]
selfweight	5,94E+02	2,17E+03	5,94E+02	2,17E+03
slab	6,49E+03	2,37E+04	6,49E+03	2,37E+04
office	16875	61593,75	16875	61593,75
partition wall	3375	12318,75	3,38E+03	1,23E+04
	Σ	1,36E+05	Σ	1,36E+05

Beam on roof

selfweight	5,94E+02	2,17E+03	5,94E+02	2,17E+03
roof	2,65E+04	9,67E+04	2,65E+04	9,67E+04
installations	2,03E+03	7,39E+03	2,03E+03	7,39E+03
	Σ	1,27E+05	Σ	1,27E+05

Walls

selfweight	1339,065	-	1442,07	-
tot (dim)	1606,878	-	1730,484	-

Total load

$q_{d.tot}$ [N/m]	8,77E+05	-	8,79E+05	-

Bendning stress control

Reduction factor	k_{mod}	0,7	
Partial coef.	γ_M	1,3	
		C14 and C30	C24
Crackling length	l_e [m]	3	3
Moment	$M_{y.d}$ [Nm]	5,19E+03	5,19E+03
Stress	$\sigma_{m.y.d}$ [Pa]	2,15E+06	1,94E+06
Requirement	$f_{m.d}$ [Pa]	1,62E+07	1,29E+07
Check	$\sigma_{m.y.d} < f_{m.d}$	OK!	OK!
Utilization ratio		13,31%	15,05%

Axial compression control

Net area of x-dir. layers	$A_{x.net}$ [m²]	9,00E-02	8,00E-02
	$i_{x.ef}$	0,042	0,048
Slenderness ratio	λ_y	71,885	62,106
Rel. slenderness ratio	$\lambda_{rel.y}$	1,253	1,053
Instability factor	k_y	1,333	1,092
Reduction factor	$k_{c.y}$	0,560	0,724
Load	N_d [N/m]	8,77E+05	8,79E+05
Stress	$\sigma_{c.x.d}$ [Pa]	9,75E+06	1,10E+07
Requirement	$k_{c.y}*f_{c.0.xlay.d}$ [Pa]	1,34E+07	1,52E+07
Check	$\sigma_{c.x.d} < f_{c.0.xlay.d}$	OK!	OK!
Utilization ratio		72,59%	72,32%

Combined bending and axial compression control

Requirement	$\sigma_{c.x.d}/(k_{c.y}*f_{c.0.xlay.d}) + \sigma_{m.y.d} / f_{m.d}$	0,859	0,874
Check	≤ 1	OK!	OK!
Utilization ratio		85,90%	87,37%

Loads

Gravity	g [m/s²]	9,81

Safety factors

SLS		ULS	
$\gamma_{g.SLS}$	1	$\gamma_{g.ULS}$	1,2
$\gamma_{q.SLS}$	1	$\gamma_{q.ULS}$	1,5

Permanent loads

Timber

	Strength class	g_k [N/m²]
Column	GL30c	1,15E+04
	GL24c	1,07E+04
	Kerto-S	1,41E+04
Beam	GL30c	2,58E+03
	GL24c	2,42E+03
	Kerto-S	3,72E+03
Wall	C14 & C30	1,03E+04
	C24	1,03E+04
Slab		1,24E+03
Installations		3,00E+02

Concrete

	Strength class	g_k [N/m²]
Core	C30/37	7,36E+04
	C40/50	7,36E+04
	C50/60	7,36E+04
Roof		3,92E+03

Variable loads

	q_k [N/m²]	ψ_0	ψ_1	ψ_2
Construction	1,00E+03	0,7	0,5	0,3
Office	2,50E+03	0,7	0,5	0,3
Partition walls	5,00E+02	0,7	0,5	0,3
Snow	1,20E+03	0,6	0,3	0,1

Wind load Category II

Velocity	v_b [m/s]	25	
Reference velocity	q_b [N/m²]	390,63	
Terrain coef.	k_r	0,19	
Height considered	z [m]	20	
Terrain coef.	z_0 [m]	0,05	
Exposure factor	c_e	2,810	
Peak velocity pressure	q_p [N/m²]	1097,64	
Reference area	A_{ref} [m²/m]	3	
External pressure	c_{pe}	-1,2	h/d
Internal pressure	c_{pi}	0,2	
Wind load	q_w [N/m]	4610,07	

A.2 Preliminary sizing: 10 floors

Input data

Building dimensions

Number of floors	n_{floors}	8	plus additional bottom and top floor
Width	w [m]	10	
Height	h [m]	27	

Material

Timber	
	Glulam: GL30c
f_{c0k} [Pa]	2,45E+07
$f_{m.x.k}$ [Pa]	3,00E+07
$E_{0.x.05}$ [PA]	1,08E+10
E_{t0mean} [Pa]	1,30E+10
ρ_{tk} [kg/m³]	390
u [%]	12
k_{mod}	0,8
γ_M	1,25

Concrete	
	C40/50
f_{ck} [Pa]	4,00E+07
E_{cm} [Pa]	3,50E+10
ρ_c [kg/m³]	2500

Concrete components

Core

Height	h [m]	3
Thickness	t [m]	0,3
Width	w [m]	1

Roof

Span length	l [m]	7
Thickness	t [m]	0,265
Mass	m [kg/m²]	400
Self weight	g_k [N/m²]	3924

Timber components

Slab

Span length	l [m]	7
Thickness	t [m]	0,28
Mass	m [kg/m²]	126
Self weight	g [N/m²]	1236,06

Columns

		GL30c
Width	w [m]	0,19
Height	h [m]	0,225
Length	l [m]	3
Cross section	A [m²]	0,04275
Self weight	m [kg/m²]	1170
Self weight	g [N/m²]	11477,70

Column check

Tributary area	A_t [m²]	24,6375	
Reduction factor	k_{mod}	0,7483	climate class 2
Partial coef.	γ_M	1,25	

Total load	F_{Ed} [N]	6,10E+05
Compressive stress	σ_d [Pa]	1,43E+07
Requirement	f_{c0d} [Pa]	1,47E+07
Check	$\sigma_d < f_{c0d}$	OK!
Utilization		97,27%

Ckecking slenderness of columns

Cracking length	l_e [m]	3
Equivalent width	w [m]	0,207
	λ	50,262
Relative slenderness	λ_{rel}	0,762
	k_y	0,813
	k_c	0,911
	β_c	0,1
Requirement	k_c*f_{c0d} [Pa]	2,23E+07
Check!	$\sigma_d < k_c$*f_{c0d}	OK!

Beam

Beam lengths at different columns [m]	
At 0105	7,3
At 0109	6,35

		GL30c
Height	h [m]	0,675
Thickness	t [m]	0,23
Cross section	A [m²]	0,15525
E-modulus	E_{mean} [Pa]	1,30E+10
Reduction factor	k_{def}	0,8
Reduction factor	ψ_2	0,3
Final E-modulus	E_{fin} [Pa]	1,05E+10
Inertia	I [m^4]	5,89E-03

ULS: Stress control

Reduction factor	k_{mod}	0,7
Partial coef.	γ_M	1,25
Tributary area	A_t [m²/m]	6,75
Lenght	l [m]	7,3

Total load	$q_{d.beam}$ [N/m]	3,96E+04
Moment	M_{Ed} [Nm]	2,64E+05
Compressive stress	$\sigma_{c.d}$ [Pa]	1,51E+07
Tensile stress	$\sigma_{t.d}$ [Pa]	-1,51E+07
Requirment	$f_{m.d}$ [Pa]	1,68E+07
Check!	$\sigma_d < f_{m.d}$	OK!
Utilization ratio		89,86%

SLS: Deflection check

Total load	$q_{d.quasi.beam}$ [N/m]	1,50E+04
Final E-modulus	E_{fin} [Pa]	7,22E+09

Instant deflection	w_{inst} [m]	0,0130
Requirement	L/500	0,0146
Check!	$w_{inst} <$ L/500	OK!
Utilization ratio		89,31%
Creep deflection	w_{creep} [m]	0,0104
Total deflection	w_{tot} [m]	0,0235
Requirement	L/300	0,0243
Check!	$w_{tot} <$ L/300	OK!
Utilization ratio		96,45%

Loads

Gravity	g [m/s^2]	9,81

Safety factors

SLS		ULS	
$\gamma_{g.SLS}$	1	$\gamma_{g.ULS}$	1,2
$\gamma_{q.SLS}$	1	$\gamma_{q.ULS}$	1,5

Permanent loads

Timber

	Strength class	g_k [N/m²]
Column	GL30c	1,15E+04
Beam	GL30c	2,58E+03
Slab		1,24E+03
Installations		3,00E+02

Concrete

	Strength class	g_k [N/m²]
Core	C40/50	7,36E+04
Roof		3,92E+03

Variable loads

	q_k [N/m²]	ψ_0	ψ_1	ψ_2
Construction	1,00E+03	0,7	0,5	0,3
Office	2,50E+03	0,7	0,5	0,3
Partition walls	5,00E+02	0,7	0,5	0,3
Snow	1,20E+03	0,7	0,5	0,2

Wind load Category II

Velocity	v_b [m/s]	25	
Reference velocity	q_b [N/m²]	390,63	
Terrain coef.	k_r	0,19	
Height considered	z [m]	20	
Terrain coef.	z_0 [m]	0,05	
Exposure factor	c_e	2,810	
Peak velocity pressure	q_p [N/m²]	1097,64	
Reference area	A_{ref} [m²/m]	3	
External pressure	c_{pe}	-1,2	h/d
Internal pressure	c_{pi}	0,2	
Wind load	q_w [N/m]	4610,07	

A.3 Preliminary sizing: 30 floors

Input data

Building dimensions

Number of floors	n_{floors}	28	plus additional bottom and top floor
Width	w [m]	10	
Height	h [m]	87	

Material

Timber	
	Glulam: GL30c
f_{c0k} [Pa]	2,45E+07
$f_{m.x.k}$ [Pa]	3,00E+07
$E_{0.x.05}$ [PA]	1,08E+10
E_{t0mean} [Pa]	1,30E+10
ρ_{tk} [kg/m³]	390
u [%]	12
k_{mod}	0,8
γ_M	1,25

Concrete	
	C40/50
f_{ck} [Pa]	4,00E+07
E_{cm} [Pa]	3,50E+10
ρ_c [kg/m³]	2500

Concrete components

Core

Height	h [m]	3
Thickness	t [m]	0,3
Width	w [m]	1

Roof

Span length	l [m]	7
Thickness	t [m]	0,265
Mass	m [kg/m²]	400
Self weight	g [N/m²]	3924

Timber components

Slab

Span length	l [m]	7
Thickness	t [m]	0,28
Mass	m [kg/m²]	126
Self weight	g [N/m²]	1236,06

Columns

		GL30c
Width	w [m]	0,33
Height	h [m]	0,405
Length	l [m]	3
Cross section	A [m²]	0,13365
Self weight	m [kg/m²]	1170
Self weight	g [N/m²]	11477,70

Column check

Tributary area	A_t [m²]	24,6375
Reduction factor	k_{mod}	0,7483
Partial coef.	γ_M	1,25

Total load	F_{Ed} [N]	1,77E+06
Compressive stress	σ_d [Pa]	1,33E+07
Requirement	f_{c0d} [Pa]	1,47E+07
Check	$\sigma_d < f_{c0d}$	OK!
Utilization		90,46%

Ckecking slenderness of columns

Cracking length	l_e [m]	3
Equivalent width	w [m]	0,366
	λ	28,427
Relative slenderness	λ_{rel}	0,431
	k_y	0,599
	k_c	0,984
	β_c	0,1
Requirement	k_c*f_{c0d} [Pa]	2,41E+07
Check!	$\sigma_d < k_c*f_{c0d}$	OK!

Beam

Beam lengths at different columns [m]	
At 0105	7,3
At 0109	6,35

		GL30c
Height	h [m]	0,675
Width	w [m]	0,23
Cross section	A [m²]	0,15525
E-modulus	E_{mean} [Pa]	1,30E+10
Reduction factor	k_{def}	0,8
Reduction factor	ψ_2	0,3
Final E-modulus	E_{fin} [Pa]	1,05E+10
Inertia	I [m^4]	5,89E-03

ULS: Stress control

Reduction factor	k_{mod}	0,7
Partial coef.	γ_M	1,25
Tributary area	A_t [m²/m]	6,75
Lenght	l [m]	7,3

Total load	$q_{d.beam}$ [N/m]	3,96E+04
Moment	M_{Ed} [Nm]	2,64E+05
Compressive stress	$\sigma_{c.d}$ [Pa]	1,51E+07
Tensile stress	$\sigma_{t.d}$ [Pa]	-1,51E+07
Requirment	$f_{m.d}$ [Pa]	1,68E+07
Check!	$\sigma_d < f_{m.d}$	OK!
Utilization ratio		89,86%

SLS: Deflection check

Total load	$q_{d.quasi.beam}$ [N/m]	1,50E+04
Final E-modulus	E_{fin} [Pa]	7,22E+09

Instant deflection	w_{inst} [m]	0,0130
Requirement	L/500	0,0146
Check!	w_{inst} < L/500	OK!
Utilization ratio		89,31%
Creep deflection	w_{creep} [m]	0,0104
Total deflection	w_{tot} [m]	0,0235
Requirement	L/300	0,0243
Check!	w_{tot} < L/300	OK!
Utilization ratio		96,45%

Loads

Gravity	g [m/s²]	9,81

Safety factors

SLS		ULS	
$\gamma_{g.SLS}$	1	$\gamma_{g.ULS}$	1,2
$\gamma_{q.SLS}$	1	$\gamma_{q.ULS}$	1,5

Permanent loads

Timber

	Strength class	g_k [N/m²]
Column	GL30c	1,15E+04
Beam	GL30c	2,58E+03
Slab		1,24E+03
Installations		3,00E+02

Concrete

	Strength class	g_k [N/m²]
Core	C40/50	7,36E+04
Roof		3,92E+03

Variable loads

	q_k [N/m²]	ψ_0	ψ_1	ψ_2
Construction	1,00E+03	0,7	0,5	0,3
Office	2,50E+03	0,7	0,5	0,3
Partition walls	5,00E+02	0,7	0,5	0,3
Snow	1,20E+03	0,7	0,5	0,2

Wind load		Category II
Velocity	v_b [m/s]	25
Reference velocity	q_b [N/m²]	390,63
Terrain coef.	k_r	0,19
Height considered	z [m]	20
Terrain coef.	z_0 [m]	0,05
Exposure factor	c_e	2,810
Peak velocity pressure	q_p [N/m²]	1097,64
Reference area	A_{ref} [m²/m]	3
External pressure	c_{pe}	-1,2 h/d
Internal pressure	c_{pi}	0,2
Wind load	q_w [N/m]	4610,07

A.4 Parametric study: Scenarios

6.1 Structural systems

Columns and beams (continous columns (two floors))

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,315	0,28	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,63E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Load bearing walls

	Strength class	h or t [m]	w [m]	l [m]
Walls	C24	0,14	1	3
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Wall								
	Q_{wall} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1442,07	4,49E+04	4,64E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1442,07	3,63E+04	3,78E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	6,70E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	5,84E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

6.2

Varying strength class of columns and beams

	Strength class	h or t [m]	w [m]	l [m]
Columns	Varying	0,36	0,28	6
Beams	Varying	0,675	0,28	7,3
Core	C40/50	0,3	1	3
A_t [m^2]	24,6375			

GL30c

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2313,90	4,54E+04	9,31E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2313,90	3,68E+04	7,59E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,54E+04	1,13E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,68E+04	9,56E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

GL24c

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2165,58	4,52E+04	9,27E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2165,58	3,66E+04	7,54E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,52E+04	1,13E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,66E+04	9,53E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Kerto-S

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2847,88	4,60E+04	9,49E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2847,88	3,74E+04	7,76E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,60E+04	1,14E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,74E+04	9,69E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Varying strength class of wall

	Strength class	h or t [m]	w [m]	l [m]
Walls	Varying	0,14	1	3
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m^2]	24,6375			

C14&C30

Wall								
	Q_{wall} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1442,07	4,49E+04	4,64E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1442,07	3,63E+04	3,78E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	6,70E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	5,84E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

C24

Wall								
	Q_{wall} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1442,07	4,49E+04	4,64E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1442,07	3,63E+04	3,78E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	6,70E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	5,84E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

6.3

Varying concrete strength class

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,315	0,28	6
Beams	GL30c	0,675	0,23	7,3
Core	Varying	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,41E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

C30/37, C40/50, C50/60

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

6.4

Varying dimensions of timber column

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	Varying	Varying	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Columns	h [m]	w [m]
Case 1	0,315	0,28
Case 2	0,405	0,28
Case 3	0,54	0,23

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Case 1: 315x280 mm

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,63E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Case 2: 405x280mm

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2603,14	4,49E+04	9,25E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2603,14	3,63E+04	7,52E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Case 3: 540x230mm

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2851,06	4,49E+04	9,27E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2851,06	3,63E+04	7,55E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Varying dimensions of timber wall

	Strength class	h or t [m]	w [m]	l [m]
Walls	C24	Varying	Varying	3
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Walls	h [m]	w [m]
Case 1	0,14	1
Case 2	0,18	1
Case 3	0,22	1

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	6,70E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	5,84E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

140 mm

Wall								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1442,07	4,49E+04	4,64E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1442,07	3,63E+04	3,78E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

180 mm

Wall								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1854,09	4,49E+04	4,68E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1854,09	3,63E+04	3,82E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

220 mm

Wall								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2266,11	4,49E+04	4,72E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	2266,11	3,63E+04	3,86E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

6.5

Change of inital moisture content in timber column and beam

	Strength class	h or t [m]	w [m]	l [m]
Columns	Varying	0,315	0,28	6
Beams	Varying	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Timber	GL30c	Kerto-S
u_1	12	6
u_2	15	12
u_3	8	4

GL30c

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,63E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	$\Delta Q_{2.storeys}$ [N	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Kerto-S

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2491,90	4,54E+04	9,34E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2491,90	3,68E+04	7,61E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	$\Delta Q_{2.storeys}$ [N	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,54E+04	1,13E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,68E+04	9,57E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Change of inital moisture content in timber walls

	Strength class	h or t [m]	w [m]	l [m]
Walls	C24	0,14	1	3
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Timber	C24
u_1	12
u_2	15
u_3	8

Wall								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	1442,07	4,28E+04	4,62E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	1442,07	3,41E+04	3,75E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,54E+04	6,75E+04	9,67E+04	7,39E+03	2,46E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,68E+04	5,89E+04	9,67E+04	7,39E+03	7,39E+03	1,85E+04	2,96E+03

6.6

Parallel or Serial construction process with columns

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,315	0,28	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,63E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	ΔQ_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	8,99E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	7,26E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Serial process: The core is assembles one storey at a time. Timber is thereafter assembled two stories at a time
=> the core will be loaded with timber two storeys at a time

A.5 General Analysis

20 floors

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,315	0,28	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	2024,67	4,49E+04	9,19E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	2024,67	3,63E+04	7,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	2,96E+03

10 floors

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,225	0,19	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	981,34	4,49E+04	9,09E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	981,34	3,63E+04	7,36E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	5,91E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	5,91E+03

30 floors

	Strength class	h or t [m]	w [m]	l [m]
Columns	GL30c	0,405	0,33	6
Beams	GL30c	0,675	0,23	7,3
Core	C40/50	0,3	1	3
A_t [m²]	24,6375			

Column 0105								
	Q_{column} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	3067,99	4,49E+04	9,29E+04	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	3067,99	3,63E+04	7,57E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	5,91E+03

Core								
	Q_{core} [N]	$\Delta Q_{bot.floor}$ [N]	ΔQ_{storey} [N]	Q_{roof} [N]	$Q_{installations}$ [N]	Q_{const} [N]	ΔQ_{office} [N]	Q_{snow} [N]
Characteristic	22072,50	4,49E+04	1,12E+05	9,67E+04	7,39E+03	4,93E+04	6,16E+04	2,96E+04
Quasi-permanent	22072,50	3,63E+04	9,47E+04	9,67E+04	7,39E+03	1,48E+04	1,85E+04	5,91E+03

B
Appendix

B.1 MATLAB code - main script

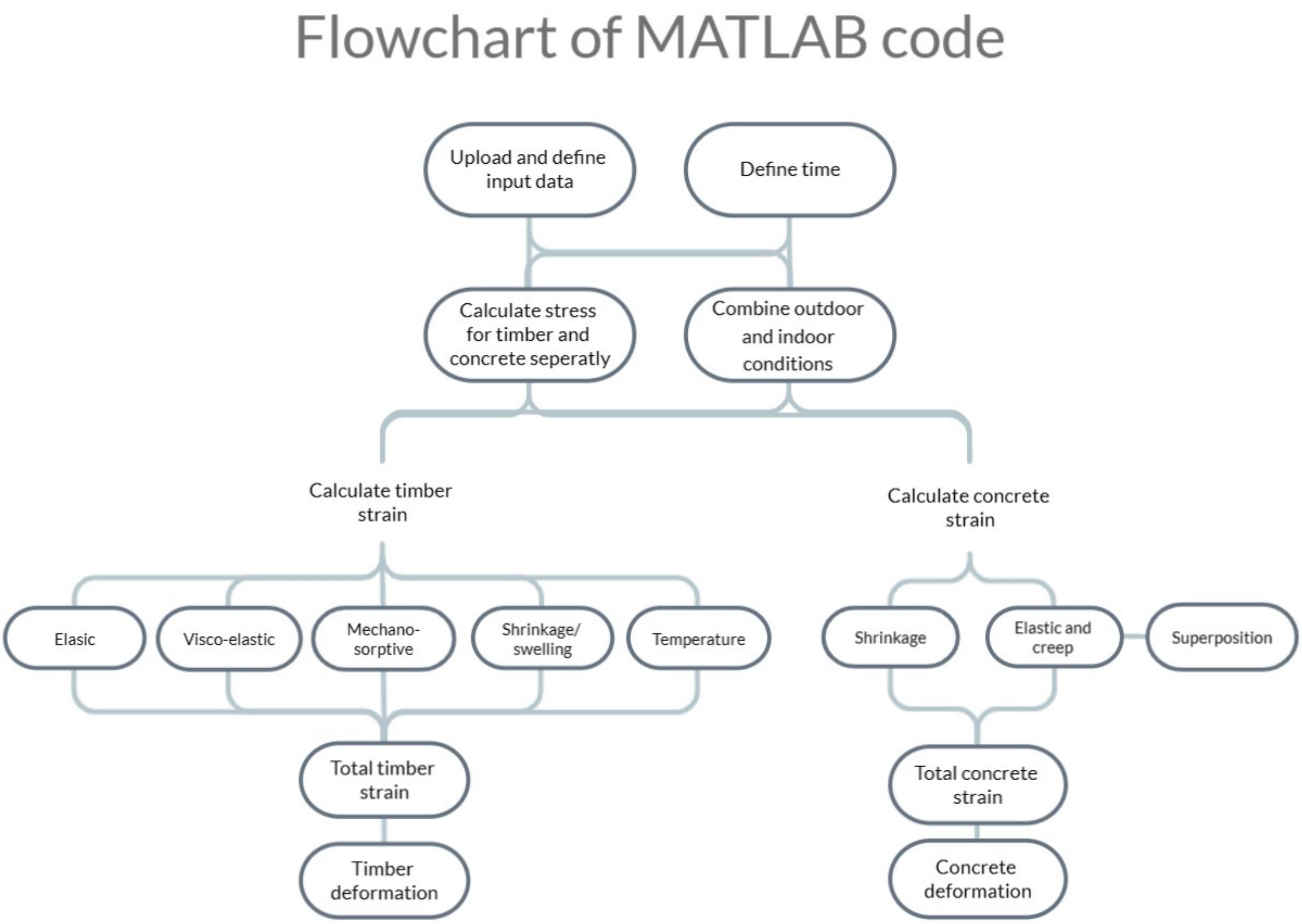

Figure B.1: Flowchart of the course of action regarding MATLAB-code

```
% The script should be modified for each specific scenario.

% Functions:
    % I - Columns and beams: stress_timber
    % II - Walls: stress_timber_wall

% Scenario 6.1 - comparing structural system
    % I - Timber columns and beams: GL30c.
```

```
    % II - Timber walls: C24.
    % Concrete core: C40/50.
% Scenario 6.2 - varying strength classes of columns and
% beams/walls
    % I - Three different load cases: GL30c, GL24c, Kerto-S.
    % II - Two different load cases: C14&C30, C24.
    % Concrete core: C40/50.
% Scenario 6.3 - varying concrete strength classes
    % Three different classes of concrete:30/37,40/50,50/60.
    % Timber columns: GL30c.
% Scenario 6.4 - varying dimensions of columns/walls
    % I - Three different load cases:315x280,405x280,540x230.
    % II - Three different load cases: 140, 180, 220.
% Scenario 6.5 - varying initial moisture content
% columns/walls
    % I - Two different load cases: three different initial
    %     MC of GL30c and Kerto-S.
    % II - One load case: three different initial MC of C24.
% Scenario 6.6 - construction processes
    % Timber columns: GL30c, Concrete core: C40/50.
    % I - serial process.
    % II - parallel process.
% Scenario stories - different total number of stories
    % Three different building heights: 10, 20 and 30 levels.
    % One load case per height: GL30c columns&beams, C40/50
    % concrete core.
    % Stresses calculated for each column and time.
    % Functions: stress_timber_level, stress_concrete_level

close all
clear; clc; clearvars;

% Add funktions to the script
addpath('functions_concrete')
addpath('functions_timber')

%% Example of Scenario 6.2
% Input data
% Wait bar
            wend = 10;
            wait1 = waitbar(0,'Reading input data...');
            w = 1;
            waitbar(w/wend)
            w = w+1;
```

```
filename = 'Input_data.xlsx'; % name of file
dimension = readmatrix(filename,'Sheet', ...
    'Different scenarios','Range','C40:E42');
case_1 = readmatrix(filename,'Sheet', ...
    'Different scenarios','Range','C48:J54');
case_2 = readmatrix(filename,'Sheet', ...
    'Different scenarios','Range','C59:J65');
case_3 = readmatrix(filename,'Sheet', ...
    'Different scenarios','Range','C70:J76');
material_data = readmatrix(filename,'Sheet', ...
    'Calc (20 levels)','Range','B11:F24');
    waitbar(w/wend)
    w = w+1;

% Dimensions; Thickness/height, width and length of timber
% and concrete elements
Thickness = [dimension(1,1) dimension(3,1)];      % [m]
Width = [dimension(1,2) dimension(3,2)];          % [m]
Length = [dimension(1,3) dimension(3,3)];         % [m]

A_office = 20*20;                       % Office area per floor
                                        % (incl. core) [m2]
A_column = Thickness(1)*Width(1);       % Area of column [m2]
A_core = Thickness(2)*Width(2);         % Area of core [m2]
u = 2*(Width(2))*1000;                  % Perimeter of cross-
                                        % section which is
                                        % exposed to drying [mm]
h_0 = 2*A_core*1000^2/u;                % Notional size [mm]

% Number of additional loads, 1st load (column/core wall)
% not included, i.e. total stories = n_tot + 2
n_tot =  xlsread(filename,'Calc (20 levels)','D3');
    waitbar(w/wend)
    w = w+1;

% Common loads for both materials (both timber and concrete)
Q_const = [case_1(2,6), ...             % Imposed load during
    case_2(2,6), case_3(2,6)];          % construction phase
Q_office = [case_1(2,7), ...            % Imposed load (office)
    case_2(2,7), case_3(2,7)];
Q_roof = [case_1(2,4), ...              % Permanent load from
    case_2(2,4), case_3(2,4)];          % roof
Q_installations = [case_1(2,5), ...     % Permanent load from
    case_2(2,5), case_3(2,5)];          % installtions
Q_snow = [case_1(2,8), ...              % Snow load
    case_2(2,8), case_3(2,8)];
```

```
% Loads acting on column 0105
Q_column_t = [case_1(2,1), ...     % Self weight of column
    case_2(2,1), case_3(2,1)];     % (continous two stories)
dQ_botfloor_t = [case_1(2,2), ... % Self weight of bot. floor
    case_2(2,2), case_3(2,2)];     % incl. partition walls
dQ_storey_t = [case_1(2,3), ...    % Permanent load from:
    case_2(2,3), case_3(2,3)];     % slab (trib. area), 1/2
                                   % beam, 1 column

% Loads acting on core
Q_core_c = [case_1(7,1) case_2(7,1) case_3(7,1)];
dQ_botfloor_c = [case_1(7,2) case_2(7,2) case_3(7,2)];
dQ_storey_c = [case_1(7,3) case_2(7,3) case_3(7,3)];
    waitbar(w/wend)
    w = w+1;

% Stresses on timber column
sigma_column_t = Q_column_t/A_column;
dsigma_botfloor_t = dQ_botfloor_t/A_column;
dsigma_storey_t = dQ_storey_t/A_column;
sigma_const_t = Q_const/A_column;
sigma_roof_t = Q_roof/A_column;
sigma_installations_t = Q_installations/A_column;
sigma_office_t = Q_office/A_column;
sigma_snow_t = Q_snow/A_column;

sigma_all_t = [sigma_column_t; dsigma_botfloor_t;
    dsigma_storey_t; sigma_const_t; sigma_roof_t;
    sigma_installations_t; sigma_office_t; sigma_snow_t];

% Stresses on concrete core element
sigma_core_c = Q_core_c/A_core;
dsigma_botfloor_c = dQ_botfloor_c/A_core;
dsigma_storey_c = dQ_storey_c/A_core;
sigma_const_c = Q_const/A_core;
sigma_office_c = Q_office/A_core;
sigma_roof_c = Q_roof/A_core;
sigma_installations_c = Q_installations/A_core;
sigma_snow_c = Q_snow/A_core;

sigma_all_c = [sigma_core_c; dsigma_botfloor_c;
    dsigma_storey_c; sigma_const_c; sigma_roof_c;
    sigma_installations_c; sigma_office_c; sigma_snow_c];
    waitbar(w/wend)
    w = w+1;
```

```
% Material properties
E_0_t = [material_data(4,1), material_data(4,2), ...   % [Pa]
    material_data(4,5)];
E_0_c = material_data(13,2); % [Pa]
alfa_L = 0.005;  % Hygroexpansion coeff. in long.dir [%/%]
beta = 0.03;     % Surface emission factor [1/h]
u0 = [material_data(6,1), ...                 % Initial mositure
material_data(6,2), material_data(6,5)];  % content of
                                              % timber [%]

f_ck = material_data(12,2);
f_cm = f_ck + 8e6;
    waitbar(w/wend)
    w = w+1;

% Time related variables
y = 5;                   % No. of years simulated
f = 2;                   % No. of floors built at a time, column
t_build_timber = 2;      % No. of weeks building each floor
t_build_concrete = 1;    % No. of weeks building each floor
t_fin = 5*7*24;          % Time of finishing the envelope [h]
t_office = 52*7*24;      % Time past t_fin, occupants move in[h]

time=linspace(0,y*365*24,y*365*24+1);          % Time [h]
t_i_timber=linspace(t_build_timber*7*24,...    % Time of
    n_tot/f*t_build_timber*7*24,n_tot/f);      % construction[h]
t_loading_timber=[0, t_i_timber, ...           % Loading steps
    t_i_timber(end)+t_fin, ...                 % combined[h]
    t_i_timber(end)+t_fin+t_office];
t_i_concrete=linspace(t_build_concrete*7*24,... % Time of
    n_tot*t_build_concrete*7*24,n_tot);        % construction[h]
t_loading_concrete=[0, t_i_concrete, ...       % Loading steps
    t_i_concrete(end)+t_fin, ...               % combined[h]
    t_i_concrete(end)+t_fin+t_office];
t_s=28*24;              % Age of concrete when drying shrinkage
                        % starts [h]
t_RH=7*24;              % Duration of time where RH is
                        % considered for creep [h]
duration=y*24*365;      % Length of plots [h]
    waitbar(w/wend)
    w = w+1;

% Relative humidity
RH = zeros(1,length(time));
RH_out = readmatrix('Klimatdata.csv.xlsx', ... % [%]
    'Sheet','v, v_s','Range','W3:W8762');
```

```
RH_out = RH_out';
RH_in = readmatrix('Klimatdata.csv.xlsx', ...   % [%]
    'Sheet','Inomhus RH','Range','I2:I8761');
RH_in = RH_in';
RH_in = repmat(RH_in,1,y+1);

% Temperature
T = zeros(1,length(time));
T_out = readmatrix('Klimatdata.csv.xlsx', ...% [degC]
    'Sheet','v, v_s','Range','AB3:AB8762');
T_out = T_out';
T_in = 22;          % Assumed mean indoor temperature
                    % after construction [degC]

% Combining the outdoor and indoor conditions
for i=1:length(time)
    if time(i) <= t_i_timber(end)
        T(i) = T_out(1,i);
        RH(i) = RH_out(1,i);
    elseif (time(i) > t_i_timber(end) && ...
            time(i) < (t_i_timber(end) + t_fin))
        % Transition RH
        RH_start = RH_out(t_i_timber(end));
        RH_end = RH_in(t_i_timber(end)+t_fin);
        k_RH = (RH_end-RH_start)/t_fin;
        m_RH = RH_start - (RH_end-RH_start)/t_fin* ...
            t_i_timber(end);
        RH(i) = k_RH*time(i)+m_RH;
        % Transition temperature
        T_start = T_out(t_i_timber(end));
        T_end = T_in;
        k_T = (T_end-T_start)/t_fin;
        m_T = T_start - (T_end-T_start)/t_fin* ...
            t_i_timber(end);
        T(i) = k_T*time(i)+m_T;
    else
        T(i) = T_in;
        RH(i) = RH_in(1,i);
    end
end
    waitbar(w/wend)
    w = w+1;

% Stress variations in timber element [Pa]
sigma_t = zeros(length(E_0_t),length(time));
delta_sigma_t = [];
```

```
for k=1:length(E_0_t)
    [sigma_t(k,:),delta_sigma_t_new]=stress_timber(time, ...
        t_i_timber, t_fin,t_office,sigma_all_t(:,k),n_tot);
    delta_sigma_t = [delta_sigma_t; delta_sigma_t_new];
    waitbar(w/wend)
    w = w+1;
end

delta_sigma_t = [delta_sigma_t(1,:); delta_sigma_t(2,:);
                 delta_sigma_t(4,:); delta_sigma_t(6,:)];

% Stress variations in concrete element [Pa]
sigma_c = zeros(length(E_0_c),length(t_loading_concrete));
sigma_c_el = zeros(length(E_0_c),length(time));
time_delta_sigma_c = [];
for k=1:length(E_0_t)
    [sigma_c(k,:),sigma_c_el(k,:), ...
        delta_sigma_c]=stress_concrete(time,t_i_concrete, ...
        t_loading_concrete,t_loading_timber,t_fin, ...
        t_office,sigma_all_c(:,k),n_tot);
    time_delta_sigma_c = [time_delta_sigma_c; delta_sigma_c];
    waitbar(w/wend)
    w = w+1;
end

time_delta_sigma_c = [time_delta_sigma_c(1,:);
                      time_delta_sigma_c(2,:);
                      time_delta_sigma_c(4,:);
                      time_delta_sigma_c(6,:)];
    waitbar(w/wend)
    w = w+1;
    close(wait1);

%% Calculating strains - timber
wait2 = waitbar(0,'Calculating timber strain...');

e_el = zeros(length(E_0_t),length(time));
e_vs = zeros(length(E_0_t),length(time));
e_ms = zeros(length(E_0_t),length(time));
e_sw = zeros(length(E_0_t),length(time));
e_T = zeros(length(E_0_t),length(time));
e_t = zeros(length(E_0_t),length(time));
```

```
% Equilibrium timber moisture content
u_RH = zeros(1,length(time));
for i=1:length(time)
    u_RH_ = 100*moisture_content_RH(RH(1,i));          % [%]
    u_RH(1,i) = u_RH_;
end

% Discretisation of half of cross-section for MC calculations
for k=1:length(E_0_t)
    waitbar(k/length(E_0_t))
    n=5;  % Dividing half timber cross-section in n elements
    x = zeros(1,ceil(Thickness(1)*1000/(2*n)+1));
    for i = 1:length(x)
        x(i) = (n*i-n)/1000;
    end
    uold = u0(k)*ones(1,length(x));
    unew = zeros(1,length(x));
    u = zeros(1,length(time));
    u(1) = uold(1);
    du_max = zeros(1,length(time));

% Calculation of time dependent behaviour - MC
% Calculation of MC distribution in cross-section
    du = zeros(1,length(time));
    for i=2:length(time)
        unew(1)=(u_RH(i)-uold(1))*(1-exp(-beta*(time(i)- ...
        time(i-1))))+uold(1);
        D = 0.5*exp(4*uold(end)/100);
        unew(end)=D*((time(i)-time(i-1))/n)*(2* ...
        uold(end-1)-2*uold(end))+uold(end);
        for j = 2:length(x)-1
            D = 0.5*exp(4*uold(j)/100);
            unew(j)=D*(time(i)-time(i-1))/n*(time(i)- ...
            time(i-1))*(uold(j-1)-2*uold(j)+ ...
            uold(j+1))+uold(j);
        end
        u_ = mean(unew);
        uold = unew;
        u(1,i) = u_;

% Max. difference of MC in timber at time t(i) for
% mechano-sorptive creep
        if i==2
            du_ = u(1) - u0(k);
        else
            du_ = u(i-1) - u(i-2);
```

```
            end
            du_max_ = max(u)-min(u(1,1:i));
            du_max(1,i) = du_max_;
            du(i) = du_;
        end

% Temperature difference
        dT = zeros(1,length(time));
        for i = 1:length(time)
            if i-1 == 0
                dT(i)=0;  % Temperature diff. equals 0 at time=0
            else
                dT(i) = T(i)-T(i-1);
            end
        end

% Calculating all strains in timber
% Elastic strain
        e_el(k,:) = elastic_strain(sigma_t(k,:),E_0_t(k));

% Visco-elastic strain
        sigma_column_t = sigma_column_t';
        delta_sigma_k = [delta_sigma_t(1,:);
                         delta_sigma_t(k+1,:)];
        e_vs(k,:) = visco_elastic_strain(time, ...
            sigma_column_t(k), delta_sigma_k,E_0_t(k));

% Mechano-sorptive strain
        e_ms(k,:) = mech_sorp_strain(time,sigma_t(k,:), ...
            E_0_t(k), u,du_max,alfa_L);

% Shrinkage/Swelling strain: moisture
        e_el_vs_ms = e_el+e_vs+e_ms;
        e_sw(k,:) = shrink_swell_strain(time,du,alfa_L, ...
            e_el_vs_ms);

% Shrinkage/Swelling strain: temperature
        e_T(k,:) = temp_strain(dT);

% Total strain
        e_t = e_el + e_vs + e_ms + e_sw + e_T;
end
        close(wait2);
```

```
%% Calculating strains - concrete
wait3 = waitbar(0,'Calculating concrete strain...');

e_el_c = zeros(length(E_0_t),length(time));
e_cc = zeros(length(E_0_t),length(time));
e_cs = zeros(length(E_0_t),length(time));

for k = 1:length(E_0_t)
    waitbar(k/length(E_0_t),wait3)

% Elastic
    e_el_c(k,:) = elastic_strain(sigma_c_el(k,:),E_0_c);

% Creep inlc. elastic strain
    RH_creep_c = mean(RH(1,1:t_RH)); % Creep is considered
                                      % for a mean value of
                                      % RH from the first week
    e_cc(k,:) = creep_strain(RH_creep_c,h_0,f_ck,time, ...
        time_delta_sigma_c(1,:), ...
        time_delta_sigma_c(k+1,:),E_0_c);

% Shrinkage
    e_cs(k,:) = shrinkage_strain(h_0,time,t_s,RH,f_ck);
 end

% TOTAL STRAIN (creep incl. elastic strain and shrinkage)
e_c = e_cc + e_cs;
    close(wait3);

%% Calculating deformations of timber and conrete elements
w_t=zeros(length(E_0_t),length(time));
w_c=zeros(length(E_0_t),length(time));

% For scenarios: 0-E
for k = 1:length(E_0_t)
% Timber column
    w_t(k,:) = e_t(k,:)*Length(1)*1000; % [mm]

% Concrete core
    w_c(k,:) = e_c(k,:)*Length(2)*1000; % [mm]
end

% For scenario: stories
w_t=zeros(n_tot/2+1,length(time));
w_c=zeros(n_tot+2,length(time));
w_fl_t=zeros(n_tot/2+1,1);
```

```
w_fl_c=zeros(n_tot+2,1);

% Timber column
n=0;
for k = 1:n_tot/2+1
    n=n+1;
    % Deformation of individual element [mm]
    w_t(k,:) = e_t(k,:)*Length(1)*1000;

    % Change of position of each column at t=3yrs
    if k == 1
        w_fl_t(k,1) = w_t(k,end);
    else
        w_fl_t(k,1) = w_t(k,end)+sum(w_t(1:k-1,end));
    end
end

% Concrete core
for k=1:n_tot+2
    w_c(k,:) = e_c(k,:)*Length(2)*1000; % [mm]

    % Change of position of each core element at t=3yrs
    if k == 1
        w_fl_c(k,1) = w_c(k,end);
    else
        w_fl_c(k,1) = w_c(k,end)+sum(w_c(1:k-1,end));
    end

end

% Adjusting vector of conrete deformations to match
% timber vector
w_fl_c = [w_fl_c(2,1);  w_fl_c(4,1);  w_fl_c(6,1);
          w_fl_c(8,1); w_fl_c(10,1); w_fl_c(12,1);
          w_fl_c(14,1); w_fl_c(16,1); w_fl_c(18,1);
          w_fl_c(20,1)];

% Calculating difference in height between column and core
% element on each floor after deformation
w_fl_diff = w_fl_c-w_fl_t;
```

B.2 MATLAB code - functions

B.2.1 Stress in timber columns

```
function [sigma_t, delta_sigma_t_new]=stress_timber(time, ...
    t_i,t_fin,t_office,sigma,n_tot)

sigma_t=zeros(1,length(time));
delta_sigma_t=zeros(1,length(time));
delta_sigma_t_new = [];

for t=1:length(time)
    n=0;
    for i=1:length(t_i)
        if time(t) >= t_i(i)
            n=n+1;
        else
            break
        end
    end
    if time(t) == 0
        sigma_t(1,t)=sigma(1);
    elseif time(t) > 0 && time(t) < t_i(end)
        if time(t) >= t_i(1)/2 && time(t) < t_i(1)
            sigma_t(1,t)=sigma(1)+sigma(2);
        elseif time(t) >= t_i(1)
            sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*n+ ...
            sigma(4)*n;
        else
            sigma_t(1,t)=sigma(1);
        end
    elseif time(t) >= t_i(end) && time(t) < t_i(end) + t_fin
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*n+ ...
        sigma(4)+sigma(5)+sigma(8);
    elseif time(t) >= t_i(end)+t_fin && ...
        time(t) < t_i(end)+t_fin+t_office
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*n+ ...
        sigma(4)+sigma(5)+sigma(6)+sigma(8);
    elseif time(t) >= t_i(end)+t_fin+t_office
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*n+ ...
        sigma(5)+sigma(6)+sigma(7)*n_tot+sigma(8);
    end
end

delta_sigma_t(1,1) = sigma_t(1,1);
for h = 2:length(time)
```

```
    delta_sigma_t(1,h) = sigma_t(1,h)-sigma_t(1,h-1);
end

time_delta_sigma_t = [time; delta_sigma_t(1,:)];

for i = 1:length(time)
    if time_delta_sigma_t(2,i) ~= 0
        delta_sigma_x = time_delta_sigma_t(:,i);
        delta_sigma_t_new = [delta_sigma_t_new, ...
        delta_sigma_x];
    end
end
```

B.2.2 Stress in timber walls

```
function [sigma_t, ...
    delta_sigma_t_new]=stress_timber_wall(time,t_i, ...
    t_fin,t_office,sigma,n_tot)

sigma_t=zeros(1,length(time));
delta_sigma_t=zeros(1,length(time));
delta_sigma_t_new = [];

for t=1:length(time)
    n=0;
    for i=1:length(t_i)
        if time(t) >= t_i(i)
            n=n+1;
        else
            break
        end
    end
    if time(t) == 0
        sigma_t(1,t)=sigma(1);
    elseif time(t) > 0 && time(t) < t_i(end)
        if time(t) >= t_i(1)/2 && time(t) < t_i(1)
            sigma_t(1,t)=sigma(1)+sigma(2);
        elseif time(t) >= t_i(1)
            sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*n+ ...
            sigma(4)*n;
        else
            sigma_t(1,t)=sigma(1);
        end
    elseif time(t) >= t_i(end) && time(t) < t_i(end) + t_fin
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*(n+1)+ ...
```

```
        sigma(4)+sigma(5)+sigma(8);
    elseif time(t) >= t_i(end)+t_fin && ...
        time(t) < t_i(end)+t_fin+t_office
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*(n+1)+ ...
        sigma(4)+sigma(5)+sigma(6)+sigma(8);
    elseif time(t) >= t_i(end)+t_fin+t_office
        sigma_t(1,t)=sigma(1)+sigma(2)+sigma(3)*(n+1)+ ...
        sigma(5)+sigma(6)+sigma(7)*n_tot+sigma(8);
    end
end

delta_sigma_t(1,1) = sigma_t(1,1);
for h = 2:length(time)
    delta_sigma_t(1,h) = sigma_t(1,h)-sigma_t(1,h-1);
end
time_delta_sigma_t = [time; delta_sigma_t(1,:)];

for i = 1:length(time)
    if time_delta_sigma_t(2,i) ~= 0
        delta_sigma_x = time_delta_sigma_t(:,i);
        delta_sigma_t_new = [delta_sigma_t_new, ...
        delta_sigma_x];
    end
end
```

B.2.3 Stress in concrete with timber columns

```
function [sigma_c, sigma_c_el, ...
    time_delta_sigma_c] = stress_concrete(time,t_i, ...
    t_loading,t_loading_other,t_fin,t_office,sigma,n_tot)

sigma_c_old = 0;
sigma_c_el_old = 0;
sigma_c = zeros(1,length(t_loading));
sigma_c_el = zeros(1,length(time));
delta_sigma_c=zeros(1,length(t_loading));

for t=1:length(t_loading)
        n=0;
        for i=1:length(t_i)
            if t_loading(t) >= t_i(i)
                n=n+1;
            else
                break
            end
```

```
        end
        if t_loading(t) == 0
            sigma_c(1,t) = sigma(1);
        elseif t_loading(t) == t_i(1)
            sigma_c(1,t) = sigma_c_old+sigma(1)+sigma(2)+ ...
            sigma(4)/2;
        elseif t_loading(t) < t_i(end)
            if any(t_loading(t)==t_loading_other(1,:)) && ...
                n ~= 0
                sigma_c(1,t)=sigma_c_old+sigma(3)+sigma(4);
            else
                sigma_c(1,t)=sigma_c_old+sigma(1);
            end
        elseif (t_loading(t) >= t_i(end)) && ...
                (t_loading(t) < t_i(end) + t_fin)
            sigma_c(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(4)+sigma(8);
        elseif (t_loading(t) >= t_i(end) + t_fin) && ...
                (t_loading(t) < t_i(end) + t_fin + t_office)
            sigma_c(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(6)+sigma(4)+sigma(8);
        elseif t_loading(t) >= t_i(end) + t_fin + t_office
            sigma_c(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(6)+sigma(7)*(n_tot+1)+ ...
                sigma(8);
        end
        sigma_c_old = sigma_c(1,t);
end

delta_sigma_c(1,1) = sigma_c(1,1);
for h = 2:length(t_loading)
    delta_sigma_c(1,h) = sigma_c(1,h)-sigma_c(1,h-1);
end

time_delta_sigma_c = [t_loading; delta_sigma_c(1,:)];

for t=1:length(time)
        n=0;
        for i=1:length(t_i)
            if time(t) >= t_i(i)
                n=n+1;
            else
                break
            end
        end
        if time(t) == 0
```

```
            sigma_c_el(1,t) = sigma(1);
        elseif time(t) == t_i(1)
            sigma_c_el(1,t) = sigma_c_el_old+sigma(1)+ ...
                sigma(2)+sigma(4)/2;
        elseif time(t) < t_i(end)
            if any(time(t)==t_loading_other(1,:)) && n ~= 0
                sigma_c_el(1,t)=sigma_c_el_old+sigma(3)+ ...
                    sigma(4);
            elseif any(time(t) == t_loading(1,:))
                sigma_c_el(1,t)=sigma_c_el_old+sigma(1);
            else
                sigma_c_el(1,t)=sigma_c_el_old;
            end
        elseif (time(t) >= t_i(end)) && ...
                (time(t) < t_i(end) + t_fin)
            sigma_c_el(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(4)+sigma(8);
        elseif (time(t) >= t_i(end) + t_fin) && ...
                (time(t) < t_i(end) + t_fin + t_office)
           sigma_c_el(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(6)+sigma(4)+sigma(8);
        elseif time(t) >= t_i(end) + t_fin + t_office
            sigma_c_el(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(6)+ ...
                sigma(7)*(n_tot+1)+sigma(8);
        end
        sigma_c_el_old = sigma_c_el(1,t);
end
```

B.2.4 Stress in concrete with timber walls

```
function [sigma_c, sigma_c_el, ...
    time_delta_sigma_c] = stress_concrete_wall(time,t_i, ...
    t_loading,t_loading_other,t_fin,t_office,sigma,n_tot)

sigma_c_old = 0;
sigma_c_el_old = 0;
sigma_c = zeros(1,length(t_loading));
sigma_c_el = zeros(1,length(time));
delta_sigma_c=zeros(1,length(t_loading));

for t=1:length(t_loading)
        n=0;
        for i=1:length(t_i)
            if t_loading(t) >= t_i(i)
```

```
                n=n+1;
            else
                break
            end
        end
        if t_loading(t) == 0
            sigma_c(1,t) = sigma(1);
        elseif t_loading(t) == t_i(1)
            sigma_c(1,t) = sigma_c_old+sigma(1)+sigma(2)+ ...
                sigma(4);
        elseif t_loading(t) < t_i(end)
            if any(t_loading(t)==t_loading_other(1,:)) && ...
                n ~= 0
                sigma_c(1,t)=sigma_c_old+sigma(3)+sigma(4);
            else
                sigma_c(1,t)=sigma_c_old+sigma(1);
            end
        elseif (t_loading(t) >= t_i(end)) && ...
                (t_loading(t) < t_i(end) + t_fin)
            sigma_c(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(4)*2+sigma(8);
        elseif (t_loading(t) >= t_i(end) + t_fin) && ...
                (t_loading(t) < t_i(end) + t_fin + t_office)
            sigma_c(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(6)+sigma(4)*2+sigma(8);
        elseif t_loading(t) >= t_i(end) + t_fin + t_office
            sigma_c(1,t)=sigma(1)*(n+2)+sigma(2)*(n+1)+ ...
                sigma(5)+sigma(6)+sigma(7)*(n_tot+1)+ ...
                sigma(8);
        end
        sigma_c_old = sigma_c(1,t);
end

delta_sigma_c(1,1) = sigma_c(1,1);
for h = 2:length(t_loading)
    delta_sigma_c(1,h) = sigma_c(1,h)-sigma_c(1,h-1);
end
time_delta_sigma_c = [t_loading; delta_sigma_c(1,:)];

% Calculation of stress for elastic strain
for t=1:length(time)
        n=0;
        for i=1:length(t_i)
            if time(t) >= t_i(i)
                n=n+1;
            else
```

```
                break
            end
        end
        if time(t) == 0
            sigma_c_el(1,t) = sigma(1);
        elseif time(t) == t_i(1)
            sigma_c_el(1,t) = sigma_c_el_old+sigma(1)+ ...
                sigma(2)+sigma(4);
        elseif time(t) < t_i(end)
            if any(time(t)==t_loading_other(1,:)) && n ~= 0
                sigma_c_el(1,t)=sigma_c_el_old+sigma(3)+ ...
                    sigma(4);
            elseif any(time(t) == t_loading(1,:))
                sigma_c_el(1,t)=sigma_c_el_old+sigma(1);
            else
                sigma_c_el(1,t)=sigma_c_el_old;
            end
        elseif (time(t) >= t_i(end)) && ...
                (time(t) < t_i(end) + t_fin)
            sigma_c_el(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(4)*2+sigma(8);
        elseif (time(t) >= t_i(end) + t_fin) && ...
                (time(t) < t_i(end) + t_fin + t_office)
           sigma_c_el(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(6)+ ...
                sigma(4)*2+sigma(8);
        elseif time(t) >= t_i(end) + t_fin + t_office
            sigma_c_el(1,t)=sigma(1)*(n+2)+ ...
                sigma(2)*(n+1)+sigma(5)+sigma(6)+ ...
                sigma(7)*(n_tot+1)+sigma(8);
        end
        sigma_c_el_old = sigma_c_el(1,t);
end
```

B.2.5 Equilibrium timber moisture content

```
function u_RH = moisture_content_RH(RH)

    RH = RH/100;
    a = 0.108;
    n = 0.64;
    c1 = 0.202;
    b1 = 2.75;
    c2 = 0.1;
    b2 = 21.0;
```

```
    u_RH = a*RH^n + c1*exp(-0.5*(b1*(RH-1)-1)^2) + ...
        c2*exp(-0.5*(b2*(RH-1)-1)^2);
```

B.2.6 Elastic strain

```
function e_el = elastic_strain(sigma,E)

e_el = sigma/E;
```

B.2.7 Visco-elastic strain in timber

```
function epsilon_vs = visco_elastic_strain(time,sigma_0, ...
    delta_sigma,E)

phi_k = [0.085 0.035 0.07 0.2];   % Characteristic retardation
                                   % time
tau_k = [2.4 24 240 2400];         % Creep factor
epsilon_vs = zeros(1,length(time));

for t=1:length(time)
    epsilon_vs1 = 0;
    epsilon_vs2 = 0;
    for k=1:4
            epsilon_vs1 = epsilon_vs1 + sigma_0* ...
                phi_k(k)/E*(1-exp(-time(t)/tau_k(k)));
    end
    n=0;
    for i=1:length(delta_sigma)
        if time(t) >= delta_sigma(1,i)
            n=n+1;
        else
            break
        end
    end
    for i=1:n
        for k=1:4
                epsilon_vs2 = epsilon_vs2 +
                    delta_sigma(2,i)*(phi_k(k)/E* ...
                    (1-exp(-(time(t)-delta_sigma(1,i))/ ...
                    tau_k(k))));
        end
    end
    epsilon_vs(t) = epsilon_vs1 + epsilon_vs2;
end
```

B.2.8 Mechano-sorptive strain in timber

```
function e_ms = mech_sorp_strain(time,sigma,E,u, ...
    du_max,alfa_L)

e_ms = zeros(1,length(time));
e_ms_ = 0;
for i=1:length(time)
    if du_max(i) == 0
        e_ms(1,i) = 0;
    else
    E_ms = E/alfa_L*1.25e-3* ...      % Depending on timber
        1/(du_max(i)/100);            % material only

    e_ms_= sigma(i)/E_ms.*exp(-u(i)/du_max(i));
    e_ms(1,i) = e_ms_;
    end
end
```

B.2.9 Shrinkage/swelling strain in timber

```
function e_sw = shrink_swell_strain(time,du,alfa_L, ...
    e_el_vs_ms)

alfa_L = alfa_L/100; % [-]
e_sw = zeros(1,length(time));
e_sw_ = 0;
for i = 1:length(time)
    if e_el_vs_ms(1,i) > 0
        alfa_L_ = alfa_L*(1-180*e_el_vs_ms(1,i));
    else
        alfa_L_ = alfa_L*exp(-180*e_el_vs_ms(1,i));
    end

    e_sw_ = e_sw_ - du(1,i)*alfa_L_;
    e_sw(1,i) = e_sw_;
end
```

B.2.10 Temperature strain in timber

```
function e_T = temp_strain(dT)

alfa_temp = 5e-6;  % temperature expansion coeff. timber,
                   % longitudinal direction [1/K]
```

```
e_T = alfa_temp*dT;
```

B.2.11 Creep and elastic strain in concrete

```
function e_cc = creep_strain(RH,h_0,f_ck,time,t_loading, ...
     sigma,E)
time = time/24;
t_loading = t_loading/24;

f_cm = f_ck + 8e6; % [Pa]

wait = waitbar(0,'Calculating concrete creep strain...');

% beta_H, Coefficient depending on the ambient
% relative humidity and the notional size of the section
if f_cm <= 35e6
     beta_H = 1.5*(1+(0.012*RH)^18)*h_0+250;
     if beta_H > 1500
          beta_H = 1500;
     end
else
     beta_H = 1.5*(1+(0.012*RH)^18)*h_0+250*(35e6/f_cm)^0.5;
     if beta_H > 1500*(35e6/f_cm)^0.5
          beta_H = 1500*(35e6/f_cm)^0.5;
     end
end

beta_c = zeros(length(t_loading),length(time));
beta_t0 = zeros(length(t_loading),1);
e_cc_indi = zeros(length(t_loading),length(time));

for p=1:length(t_loading)
     waitbar(p/length(t_loading),wait)
     beta_t0(p) = 1/(0.1+t_loading(p)^0.2);

     beta_fcm = 16.8/sqrt(f_cm/10^6);          % f_cm in [Pa]

     if f_cm <= 35e6
          phi_RH = 1 + (1-RH/100)/(0.1*h_0^(1/3));
     elseif f_cm > 35e6
          phi_RH=(1 + (1-RH/100)/(0.1*h_0^(1/3))* ...
               (35e6/f_cm)^0.7)*(35e6/f_cm)^0.2;
     end
```

```
    phi_0 = beta_t0*phi_RH*beta_fcm;

    for t=1:length(time)
        if time(t)<t_loading(p)
        beta_c0=0;
        else
        beta_c0=((time(t)-t_loading(p))/ ...  % Time function
            (beta_H+(time(t)-t_loading(p)))).^0.3;
        end
        beta_c(p,t)=beta_c0;

        phi = beta_c.*phi_0;

        if time(t)<t_loading(p)
        e_cc_indi(p,t) = 0;
        else
        e_cc_indi(p,t) = sigma(p)*(1+phi(p,t))/E;
        end
    end
end

e_cc = sum(e_cc_indi);
close(wait)
```

B.2.12 Shrinkage strain in concrete

```
function e_cs = shrinkage_strain(h_0,time,t_s,RH,f_ck)
time = time/24;    % Time [days]
t_s = t_s/24;      % Age of concrete when drying
                   % shrinkage starts [days]

f_cm = f_ck + 8e6; % [Pa]
alfa_ds1 = 4;                        % For cement class N
alfa_ds2 = 0.12;                     % For cement class N



% Drying shrinkage
k_h = k_h_func(h_0);     % Coeff. considering notional size

beta_RH = zeros(1,length(time));

beta_ds = zeros(1,length(time));
e_cd = zeros(1,length(time));
beta_as = zeros(1,length(time));
e_ca = zeros(1,length(time));
```

```
for i=1:length(time)
    if time(i) < t_s
        beta_ds(i) = 0;
    else
        beta_ds(i)=(time(i)-t_s)/((time(i)-t_s)+ ...
            0.04*sqrt(h_0^3));
    end

    beta_RH_ = 1.55*(1-(RH(i)/100)^3);
    beta_RH(i) = beta_RH_;

    e_cdi = 0.85*((220+110*alfa_ds1)* ...
        exp(-alfa_ds2*f_cm/10^6/10))*10^-6;

    e_cd_inf = k_h*beta_RH*e_cdi;

    e_cd(i) = beta_ds(i)*e_cd_inf(i);

% Autogenous shrinkage
    e_ca_inf = 2.5*(f_ck/10^6-10)*10^-6; % With f_ck in [Pa]

    beta_as(i) = 1-exp(-0.2*time(i)^0.5);

    e_ca(i) = beta_as(i)*e_ca_inf;
end

% Total shrinkage
e_cs = e_cd + e_ca;
```

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