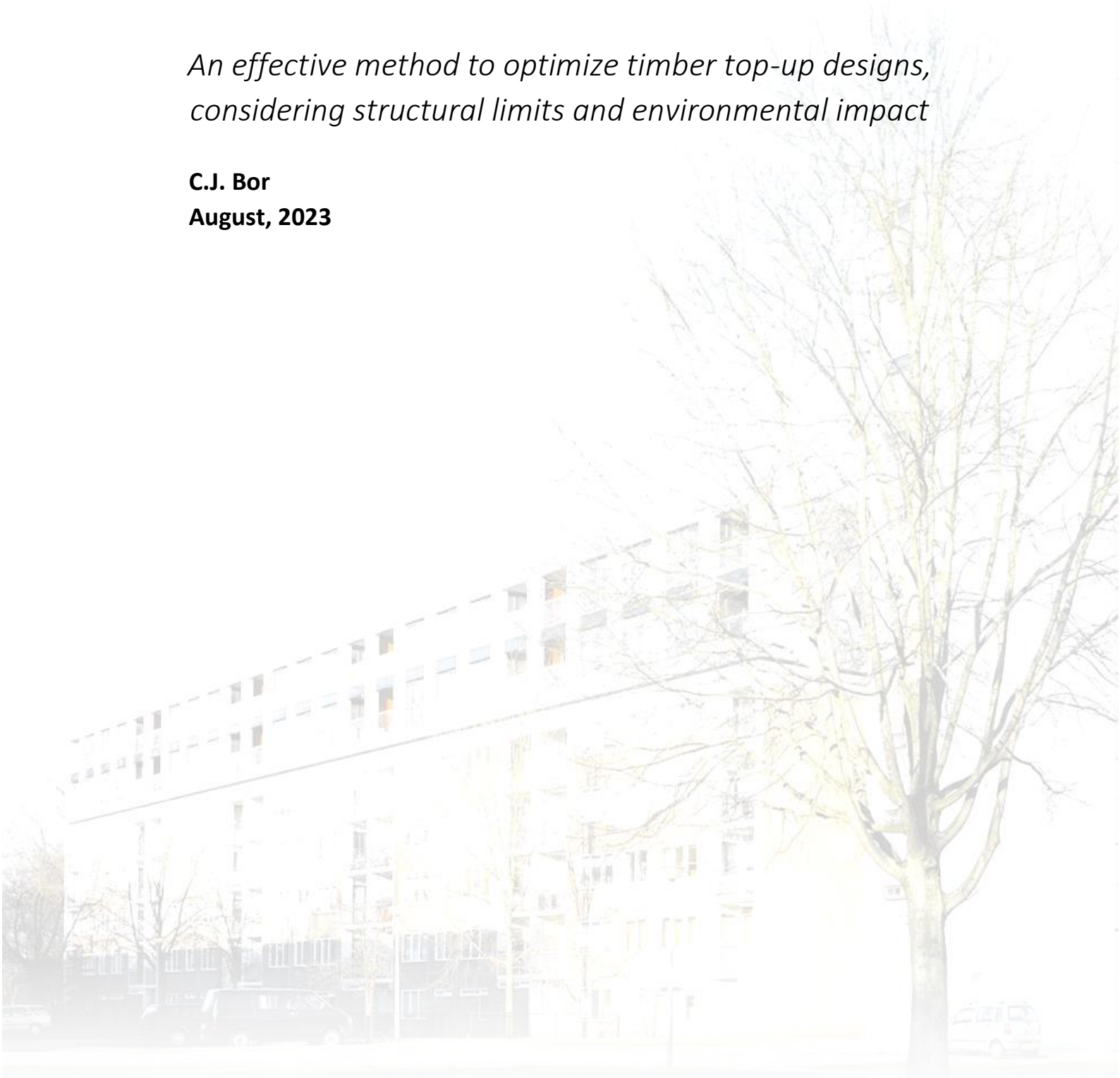


Vertical Extension of Council Estates

*An effective method to optimize timber top-up designs,
considering structural limits and environmental impact*

C.J. Bor

August, 2023



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Vertical Extension of Council Estates

An effective method to optimize timber top-up designs,
considering structural limits and environmental impact

By

C.J. Bor

*to obtain the degree of Master of Science in Civil Engineering
at the Delft University of Technology,
to be defended publicly on August 1, 2023*

Student number:	4892968	
Thesis committee:	Dr. Ir. H.R. Schipper	TU Delft
	Ir. A.C.B. Schuurman	TU Delft
	Prof. Dr. H.M. Jonkers	TU Delft
	Ing. M. Freriks	Van Rossum Raadgevende Ingenieurs B.V.

An electronic version of this thesis is available at <http://repository.tudelft.nl/>

Cover image: Vertical extension of council estate at the IJdoornlaan Amsterdam
(Hans van Heeswijk Architecten, 2003)

VAN RAADGEVENDE
INGENIEURS
ROSSUM

 **TU**Delft

Contact information

Author

Name *C.J. (Hanco) Bor*
Student number *4892968*
Institution *Delft University of Technology*
Faculty *Civil Engineering and Geoscience*
Master track *Building Engineering*
Specialization *Structural design*

Graduation committee

Name *Dr. ir. H.R. (Roel) Schipper (Chair)*
Institution *Delft University of Technology*
Faculty *Civil Engineering and Geoscience*
Department *Materials, Mechanics, Management & Design*
Section *Applied mechanics*

Name *Ir. A.C.B. (Marco) Schuurman*
Institution *Delft University of Technology*
Faculty *Civil Engineering and Geoscience*
Department *Materials, Mechanics, Management & Design*
Section *Applied mechanics*

Name *Prof. dr. H.M. (Henk) Jonkers*
Institution *Delft University of Technology*
Faculty *Civil Engineering and Geoscience*
Department *Materials, Mechanics, Management & Design*
Section *Materials and Environment*

Name *Ing. M. (Mark) Freriks (External)*
Company *Van Rossum Raadgevende Ingenieurs B.V.*
Function *Project leader/ Structural engineer*

Preface

This thesis was written as final part of my studies at Delft University of Technology, to obtain a Master's degree in Civil Engineering, Building Engineering track with specialization Structural Design. I have been working for a year on researching and writing my thesis about an effective method to optimize timber top-up designs for council estates. It has been an enjoyable time, where I could apply my obtained knowledge into scientific research.

The research is primarily intended for structural engineers, involved in top-up projects (on council estates). Moreover, the research is deemed useful for housing-related policy makers in the Netherlands, showing the potential of using upwards extensions on council estates to solve the housing demand. All others interested in council estates, top-ups, and environmental impact of timber structures are invited to peruse the report.

I would like to thank my graduation committee for their guidance and support during my research. I am grateful to dr. ir. H.R. Schipper for being the chair of the project, having the graduation procedure clear and providing effective feedback, to ir. A.C.B. Schuurman for his structural input and guiding recommendations as the parameter study and produced flow charts, and to prof. dr. H.M. Jonkers for his wide-ranging, shared knowledge on sustainability. My sincere thanks to my daily supervisor ing. M. Freriks for his practical knowledge and guidance during the project, being always open for questions.

In addition, I would like to acknowledge my colleagues of Van Rossum B.V., providing me advice and sociability during the graduation process. Special thanks to M. Tromp and T. Jonkers for providing interviews, as well as A. van der Sluis for suggesting the topic of council estates. Thanks to my family for their support and to my study mates for making these study years an unforgettable period.

Hanco Bor
Lelystad, August 2023

Abstract

The challenge of global warming is an increasing concern of the 21st century. The concept of sustainability is therefore considered of high importance as it aims to protect the environment and, more specific, reduce the emission of greenhouse gasses. This has been underlined by signature of the Paris agreements by many countries, including the Netherlands. A major problem of the Dutch government is the growing housing demand. The current deficiency is listed at 325,000 dwellings (Capital Value, 2023). Also after the WWII, the Netherlands faced a housing crisis, resulting in approximately 847,000 council houses (TU Delft, 2022). Due to aging, these flats are currently in an outdated condition (Singelenberg & Van Vlaenderen, 2011). The massive number of flats combined with the aforementioned deficiency of houses and the climate problem prohibit large scale demolition and reconstruction, as this would lead to both a further short-term deficit of houses and emission of construction-related greenhouse gasses.

Mishra et al. (2022) showed the potency of timber buildings to meet the global housing demand and the need for low-carbon construction, by using careful forest planting. Hence, a timber top-up on council estates constitutes a sustainable approach, which accounts for densification and the huge building stock in city centres. The structural similarities in council estates allow the development of a general method, which could assist structural engineers in a sustainable decision-making process to top-up council estates. Therefore, this research is aimed to develop a method to optimize timber top-up designs for council estates, using the following **research question**:

What is an effective method to optimize timber top-up designs for council estates, considering their structural limits and environmental impact?

The scope of the research is delineated to a specific top-up method: of timber (1), for council estates (2), whereas: strengthening for vertical weight (3), and building physic related aspects (4) fell outside of the scope of this thesis.

The theoretical part of the thesis discusses council estates, vertical extensions, and environmental impact.

It is found that the building period impacts the structural variety in council estates. In this research, the scope was narrowed down to the post-war period (1946-1965), as the interwar period (1916 – 1926) shows too wide variation. Hence unfortunately no universal structural lay-out can be defined. Uncertain is the brickwork quality of the flats.

Strengthening for vertical load is complicated and expensive and is therefore excluded from this research as viable option. The structural overcapacity must be determined based on norm comparison. Council estates contain, according to this research, approximately 21% overcapacity (compared to NEN8700 load definition). Horizontal linking of the dilatations is beneficial for structure and environment in case stability is insufficient. Both Timber Frame (Dutch: Houtskeletbouw (HSB)) and Cross Laminated Timber (CLT) construction can be used to top-up a building. HSB is preferred for its lightweight, CLT for its robustness. Two flowcharts are produced, providing the structural overcapacity for distinct council estates.

The carbon footprint method must be used to determine the impact of the extension in [kg CO₂-eq./y/m² new floor area]. Due to the CO₂-capture of wood, more wood use results in an increased

negative Global Warming Potential (GWP), which seems to be positive. Unfortunately, excessive wood logging has detrimental effects on the environment. Therefore, material use must be minimized.

The numerical part is aimed for the structural design and Life Cycle Analysis (LCA) of a modular CLT unit and a non-modular HSB-construction, as well as a definition of the top-up method.

The CLT-construction has a weight of 6.4, or 12.3 kN/m², for 1 or 2 storeys, respectively. The technical lifespan (75 years) of the unit can be fully developed, resulting in a negative GWP of -32.58 (1 storey) or -30.23 (2 storeys) [10⁻¹ kg CO₂-eq./y/m² new floor area]. The HSB-construction has a weight of 5.5, 10.1, and 13.4 kN/m² for 1 till 3 storeys, respectively. The technical lifespan of the HSB top-up (75 years) cannot be fully developed, as it is limited by the remaining realistic service life of the council estate itself. The result are linear graphs indicating the GWP as function of the residual lifespan of a council estate.

Five requirements are defined for an effective method:

1. Allows for spatial variation in structural plans;
2. Adds (if structurally possible) at least two floors;
3. Applies no strengthening for additional vertical loads;
4. Negative global warming potential for the product stage;
5. Minimizes material use per year of service life.

As both variants fulfil the first four requirements, the method optimizes based on material use per year. Consequently, a 3-step method is developed, which considers the flat characteristics, structural overcapacity and minimalization of material use [Section 9.3]. It is found that a large residual lifespan, and number of addable layers increase the potency to apply a HSB top-up.

The answer to the research question is that an effective method is dependent on the structural characteristics of the council estate (defined in section 9.3). The developed method optimizes timber top-up designs for council estates, considering a spread in overcapacity. The optimization produces CO₂-negative extensions (Stage A1-A3), having a minimized material use per year.

To conclude, it is generally not possible to tell a priori which of the two considered timber top-up methods is the most effective for a particular case, considering the structural limits and environmental impact. The residual lifespan of a building determines which method is the most effective. As an exception, the extreme residual lifespans of council estates have a predictable top-up method. A modular CLT extension forms the most fitted solution for buildings with a residual lifespan less than 25 years, while a non-modular HSB extension is most fitted for buildings with a residual lifespan more than 40 years. Further, it can be concluded that timber vertical extensions form a sustainable replacement for combined demolition and reconstruction of buildings. Moreover, these facilitate the realization of a substantial number of houses. Therefore, a vertical extension of council estates forms a sustainable solution to the current housing demand in the Netherlands.

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List of abbreviations

A	New realized floor area
Add. room	Additional room
CFL	Connected Floor Length
CLT	Cross Laminated Timber
CO₂-eq.	Carbon dioxide equivalent
Contr. width	Contributing width
ECI	Environmental Cost Indicator
EPD	Environmental Product Declaration
GWP	Global Warming Potential total
HSB	Timber Frame Construction (Dutch translation: Houtskeletbouw)
LCA	Life Cycle Analysis
LCI	Life Cycle Inventory
LCIA	Life Cycle Impact Assessment
NFL	Normative Floor Length
RSL	Reference Service Life
S	Stability
SLS	Serviceable Limit State
UC	Unity Check
UE	Unit Equivalent
ULS	Ultimate Limit State
V	Volume structural material
VL	Vertical Load
γ_{load}	Historically applied safety factor on loads
γ_{total}	Historically used comprehensive safety factor, includes safety on loads and material

1

Introduction

This master thesis is a research on the vertical extension of council estates, as a sustainable solution for the present housing demand. It marks the final part of the study at Technical University of Delft, to obtain a Master's degree in Civil Engineering, Building Engineering with the specialization Structural Design. The thesis is performed in collaboration with Van Rossum Raadgevende Ingenieurs B.V.

The challenge of global warming is an increasing concern of the 21st century. The concept of sustainability is therefore considered of high importance as it aims to protect the environment and, more specific, reduce the emission of greenhouse gasses. This has been emphasized by signature of the Paris agreements by many countries, including the Netherlands. Another major problem of the Dutch government is the rising housing demand. The current deficiency is listed at 325,000 dwellings (Capital Value, 2023). Also after the WWII, the Netherlands faced with a housing crisis, resulting in approximately 847,000 apartments (TU Delft, 2022). These council estates have similar structural principle as a consequence of a rapid, industrial, and nationally driven building process. Due to aging, these flats are currently in an outdated condition (Singelenberg & Van Vlaenderen, 2011). The massive number of flats combined with the aforementioned deficiency of houses and the climate problem prohibit large scale demolition and reconstruction, as this would lead to both a further short-term deficit of houses and emission of construction-related greenhouse gasses.

Mishra et al. (2022) showed the potency of timber buildings to meet the global housing demand and the need for low-carbon construction, by using careful forest planting. Hence, a timber vertical extension on council estates constitutes a sustainable approach, which accounts for densification and the huge building stock in city centres. The structural similarities in council estates allow the development of a general top-up method, which will help structural engineers make a sustainable, preliminary decision. Therefore, the author posits to develop an effective method to optimize timber top-up designs for council estates, as a solution to the housing and environmental crisis, as well as to the number of council houses.



Figure 1.1: Three-fold problem: global warming, housing demand, and large council estate stock (self-produced).

2

Reading guide

The research will consist of two parts: a theoretical and numerical part. Both parts will be outlined below to assist the reader in understanding the structure of the report.

2.1 Theoretical part

The theoretical part contains research to council estates, vertical extensions, and environmental impact. The preliminary study and research plan span [chapter 3](#) and [4](#), respectively.

The further theoretical frame covers [chapter 5](#) till [7](#).

- ❖ [Chapter 5](#) studies the typology of council estates, based on a literature study, archive inspection, and parameter study. [Appendix A](#) provides background information to [chapter 5](#).
- ❖ [Chapter 6](#) explores the timber top-up possibilities, based on a literature review, case studies, interviews, and norm study. The chapter concludes with two flowcharts, providing an estimated overcapacity for council estates. [Appendix B](#) provides background information to [chapter 6](#).
- ❖ [Chapter 7](#) explores the LCA-calculation procedure and executes exploratory calculations. [Appendix C](#) provides background information to [chapter 7](#).

2.2 Numerical part

The numerical part of the thesis contains the structural design and (LCA-) calculation of two variants, as well as the presentation of the top-up method.

- ❖ [Chapter 8](#) explores and verifies, based on the definition for an effective method, two variants, by structural and environmental calculations. [Appendix D](#) provides background information to [chapter 8](#).
- ❖ [Chapter 9](#) defines the top-up method. This method includes the flowcharts of [chapter 6](#). [Appendix E](#) provides background information to [chapter 9](#).
- ❖ [Appendix X](#) forms an additional appendix, containing excel sheets and hand validations of the calculations.

[Chapter 10](#) discusses the research and provides proposals for further research. [Chapter 11](#) contains the conclusions of the research.

3

Preliminary research

Chapter 3 will contain the preliminary research. It will provide context over environmental changes (international), Dutch housing situation (national), council estates, and vertical extensions (national/regional). This chapter will conclude with a problem definition.

3.1 Context

The context describes the development of the building industry, environmental situation, Dutch urbanisation, council estates and provides information about vertical extensions.

3.1.1 Evolution of the building industry

For centuries, the environmental impact was not considered. During the prehistoric period, perishable building materials as leaves, branches and animal hides were used. Later in human civilisation, more natural materials as clay, stone and timber has been used (Rowan University, n.d.). The ancient Greeks were the first builders who used wood, to build permanent structures. The Romans started to create the structural material concrete (Joe, 2020). In the industrial revolution, steel became a mass-produced material (Rowan University, n.d.). During the 20th century, materials as steel, wood and timber were optimized, while new materials were developed as plastics and polymers (Joe, 2020).

Until the 20th-century, the development of construction industry was driven by three factors: durability of materials, increasing structural dimensions, and the degree of control over interior conditions in buildings (Rowan University, n.d.). From the beginning of the present century, a new parameter has become increasingly pressing: **environmental impact**. People become increasingly environmental conscious. There is more focus on ecology protection, energy conservation and environmentally conscious building materials (Joe, 2020).

3.1.2 Environmental change

The current environmental situation is outlined to understand the new parameter. Europe's annual temperature has increased with an average rate of 0.15 °C/decade since 1910. This is tripled to 0.45 °C/decade since 1981 (NOAA National Centers for Environmental Information, 2021). This is caused by increased carbon emissions (USGCRP, 2017). Figure 3.1[a] presents the earth's CO₂-flux. The increased CO₂-emissions caused by fossil fuels and industry must be compensated by the atmospheric, land and

ocean sink. The insufficient capacity of the ocean sink, results in an increased CO₂-concentration in the atmosphere (causing global temperature rises). By 2101, the expected global temperature rise equals 2.5 – 8.0 °F [Fig. 3.1b] (USGCRP, 2017).

The Global Status Report for Buildings and Construction (2019) states that 39% of the energy- and process-related emissions in 2018 can be linked to the building and construction sector (Global Alliance for Buildings and Construction et al., 2019). The Dutch government has signed the Paris accord (Nederlandse Emissieautoriteit, 2022). 49% of the CO₂ emissions, relative to 1990, must be reduced by 2030 (95% by 2050) (Ministerie van Economische Zaken en Klimaat, 2022).

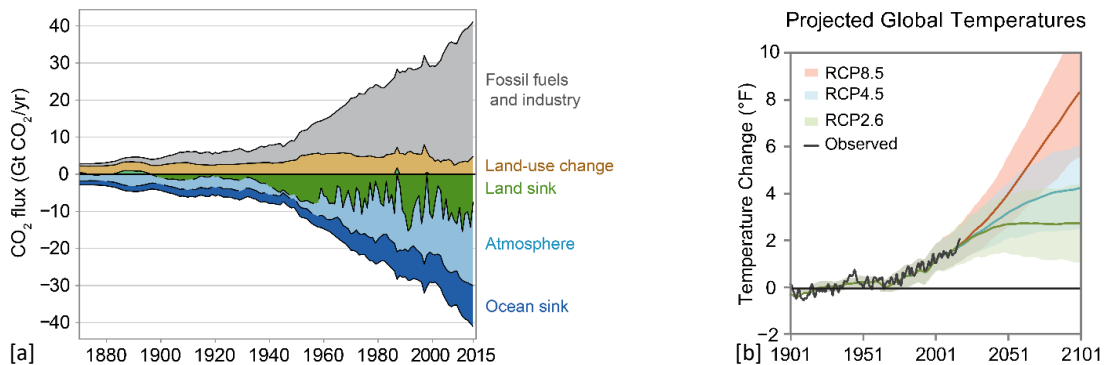


Figure 3.1: Environmental change; [a] CO₂ flux; [b] temperature rise (USGCRP, 2017).

3.1.3 Urbanisation in the Netherlands

On a national level, the Netherlands faces a population surge, leading to pressure on housing demand in both rural and urban areas. Broitman and Koomen (2019) investigated the effect of the densification in the city centres compared to the edges. The research was performed on four cities: Amsterdam, Rotterdam, The Hague, and Utrecht.

Figure 3.2 provides their results. The graphs show vertically the differences in houses/inhabitants per hectare/house between 2000 and 2017. A positive value indicates an increased ratio. The horizontal axis specifies the distance from the city centre. Both the population and dwelling density increase in the city centres. This in contrast to average number of inhabitants per house. In conclusion, the density in the Dutch city centres is increasing in contrast to other world cities (Broitman and Koomen, 2019).

Despite the urbanisation, the housing demand is rising in the Netherlands. The current demand is listed at 325,000 houses (Capital Value, 2023). This shortage is expected to rise, and major efforts are needed to make up for the growing deficit (Ministerie van Binnenlandse Zaken en Koninkrijksrelaties, 2021). To generate more liveable space, several options can be thought of:

- **New construction** of housing is an option only when there is enough space. Due to increased land prices, housing demand and limited space, municipalities start to build high-rise (NL times, 2019).
- **Rebuild of existing residential areas** forms another option. Current houses will be demolished, and more modern residential areas will be reconstructed.
- **Vertical extension** is a more and more investigated option. Vertical extension saves material use and space, compared to other proposed options.

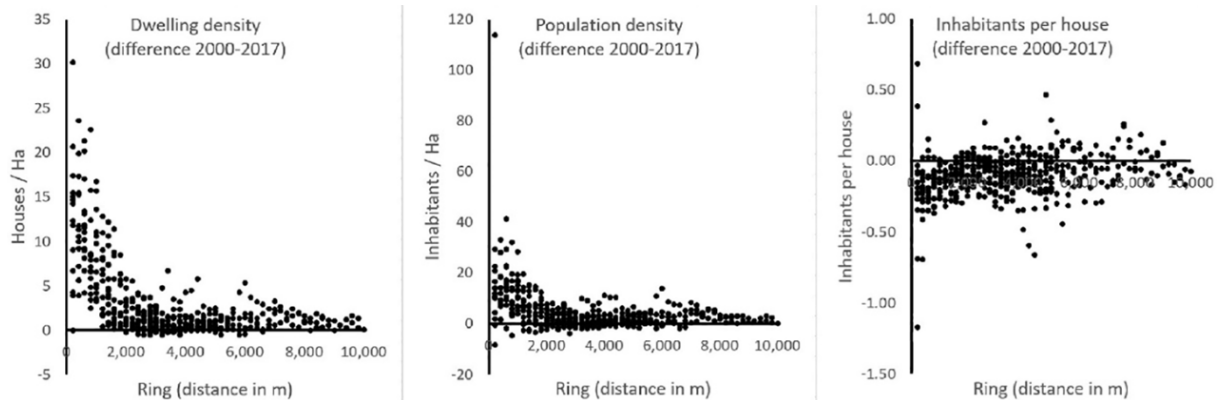


Figure 3.2: Densification in the Netherlands (Broitman & Koomen, 2019)

3.1.4 Council estates

The period after the World War II is marked as a period of large economic growth and expansion (Fels, 2017). The government struggled with a housing crisis, a result of the WWII. As the Netherlands functioned as a **'welfare state'**, the government was expected to provide enough liveable provisions for their population. The city centres formed the normative location for economic growth and expansions (Fels, 2017).

The government signed the **'Reconstruction Law'**, to provide money for more housing. Residential areas were preferred to develop small communities. The government desired diversity in both people and housing. Low-, middle- and high rise had to be built jointly. Due to the housing demand, the building process must happen faster, cheaper, and more industrial. That, at the consequence of diversity. A dominant middle-rise building developed: the council estate (Fels, 2017).

The council estates consist mostly of three or four storeys. More floors would exceed the elevator boundary, significantly increasing the price. The apartments include three to four rooms, with a total area of about sixty square meters (Singelenberg & Van Vlaenderen, 2011).

Present, the districts deal with liveability problems and social deprivation. The flats are used by immigrants and starters on the housing market (Singelenberg & Van Vlaenderen, 2011). The small houses, stairs instead of elevators, and a poor indoor climate, results in wretched living conditions. The bad reputation of these districts makes it undesirable for corporations to invest in these apartments. Fels (2017) concludes that the market position of these neighbourhoods is weak.

Demolition of all council estates will be a huge task and unsustainable. Consequently, a lot of research has been done on renovation scenarios. Singelenberg and Van Vlaenderen (2011) did research to change the target audience by transforming the building composition. Loukopoulou (2012) studied a building envelope refurbishment. Van der Drift (2019) investigated renovation scenarios considering costs, energy prestaton, lower temperature heating, and implementation difficulty. The distinct studies show future potential for the council estates.

Structural improvements can be made as well. By changing towards a more innovative building strategy, the potential of these flats will rise. The liveable space can be increased by vertical extensions on council estates. This solution constitutes a sustainable approach, which accounts for densification and the huge building stock in the city centres, improving the commercial value of council estates. More than 500,000 new dwellings could be realized by applying this strategy (Doodeman, 2021). That underlines the large potential.

3.1.5 Rooftop extensions

Topping-up is defined as: *adding (at least one) additional layer(s) to an existing building* (Encyclo, n.d.). Within the literature, three additional terms exist for a top-up: a rooftop, upward, and vertical extension. These terms are used interchangeable.

3.1.5.1 Reference projects

Several Dutch examples exist of vertical extensions. Most famous is the **Karel Doorman** extension in Rotterdam. Sixteen layers were added on top of the existing building. The extension was realized by designing a lightweight steel structure [Fig. 3.3 a, b]. Two concrete cores were added to reinforce the existing structure. The horizontal load has been taken by the core, to increase the bearing capacity of the columns (Bouwwereld, 2013).

Another project is the three added floors on the former **Smit-Tak** headquarter. The new added structure has been made of a lightweight steel structure and hollow core slabs [Fig. 3.3c, d]. The existing structure used a portal structure for stability. To strengthen for the new wind load, concrete cores were constructed. New piles were added to the foundation (Pieters Bouwtechniek, n.d.).

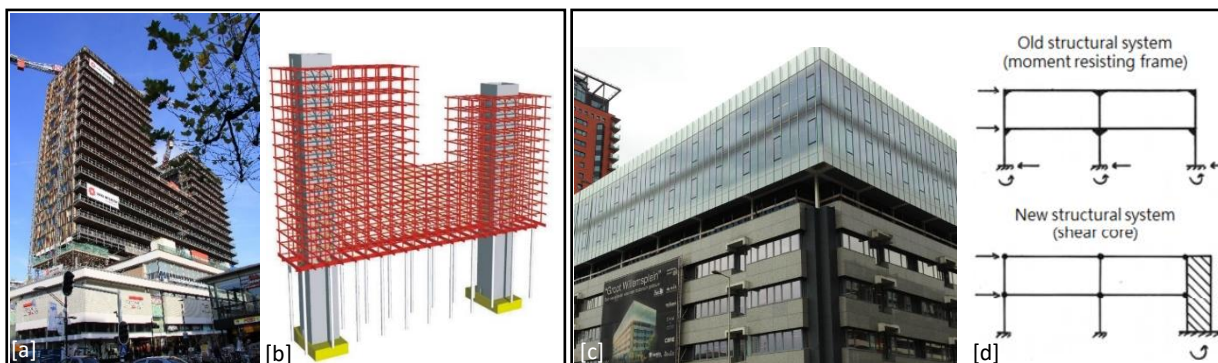


Figure 3.3: [a] Karel Doorman building (Wind, 2012) and [b] corresponding structural system of the top-up (Wind, 2012). [c] Groot Willeplein extension (Pieters Bouwtechniek, n.d.), and [d] related structural system change (Papageorgiou, 2016).

3.1.5.2 Structural methods

When considering a structural method for a vertical extension, Papageorgiou (2016) considered three different methods:

1. **Vertical extension without strengthening the existing building.** This is a feasible solution in case the existing structure is over-dimensioned (limited number of layers will be added).
2. **Local strengthening of the structural system.** Strengthening a construction is highly project specific. Considering strengthening methods for concrete elements, two methods could be applied: concrete jacking and application of externally bonded steel strips. By concrete jacking, the cross-section is enlarged with high performance concrete (Marini & Meda, 2009). Externally bonded steel strips increase the ultimate shear strength by approximately 22% (Altin et al., 2013).
3. **Radical changes in the structural system.** The main focus is to change the stability system.

3.1.5.3 Structural material

The structural material use of a top-up forms a crucial consideration. Three structural aspects must be evaluated: strength, stiffness, and weight. The latter in particular has a major impact on vertical extensions. Concrete is excluded in the material evaluation due to its large self-weight, and carbon footprint. A literature review on steel and timber is conducted.

Unfortunately, there is no absolute answer which material is more effective for vertical extensions.

- **Papageorgiou (2016)** stated that steel is the most optimal material. Steel has architectural flexibility, easy and fast construction, lightweight, precise, and reliable products, thermal comfort possibilities, compliance with fire regulations, and long lifespan. Steel can be recycled without downgrading (but with a CO₂ footprint). The production of steel is polluting.
- **Bikulčiūtė (2021)** stated that timber has a light weight, high strength to weight, and stiffness to weight ratio. Timber is a sustainable material, having a large CO₂-storage capacity, and being a renewable material with a cradle-to-cradle potential.
- **Rissetto (2018)** had conducted research on the performance of structural material for top-ups. The main aspects of Rissetto (2018) were: ecological, thermal, and cost performance. A light-weight wood structure reduces the environmental impact by 50 percent, compared to a light-weight steel structure. Additionally, Rissetto (2018) concluded that light-weight wooden structures are less expensive.
- **Mishra et al. (2022)** showed the potency of timber. Their thorough research addressed that there are good opportunities to meet global housing demand using careful forest planting, and gradually shift away from CO₂-unfriendly building materials.

For this research, the decision was made to focus on timber, based on four factors:

1. Three of the four presented studies show the sustainable potential of timber above steel;
2. Timber has appropriate mechanical properties for top-ups;
3. More research is already performed in steel top-ups;
4. Author's personal interest in timber structures.

3.2 State of art

This section will provide the main scientific reports and information considering top-ups and council estates. This will indicate the knowledge gap related to council estates and top-ups.

3.2.1 Council estates

A lot of research was conducted on the renovation of council estates [Section 3.1.4]. However, no literature is found within the structural solutions for council estates, related to vertical extensions. Recently, SUM (student team of the TU Delft), presented a solution for a sustainable transformation of council estates (TU Delft, 2022). The solution contains a vertical extension. It must be mentioned that no report was yet available describing the structural underpinning of the design.



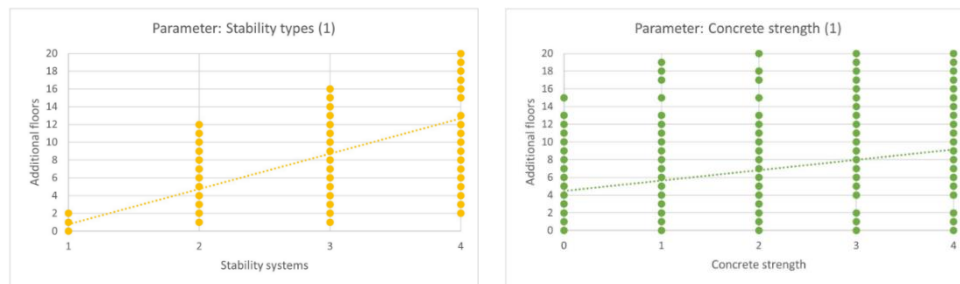
Figure 3.4: Proposal for sustainable transformation of council estates by SUM (TU Delft, 2022).

3.2.2 Rooftop extensions

Bikulčiūtė (2021) performed research in timber upwards extensions, combined with parametric modelling and machine learning. During this study, two main obstacles blocked her progress:

1. The available data about top-upped buildings. Bikulčiūtė constructed a fictive database.
2. The project specificity of vertical extensions. Consequently, a parametric study is performed to create a tool that is more general applicable.

Bikulčiūtė (2021) defined two factors that influence the number of layers that can be added to buildings: the stability system and concrete strength class. Adding more stability elements does not directly contribute to more stability [Fig. 3.5]



(a) Stability type influence on the number of additional floors. (1 - no stability walls and rigid frame structure; 2 - stability walls and rigid frame; 3 - stability walls and supported column structure; 4 - no stability walls and supported column structure)

(c) Concrete strength influence on the number of additional floors. (0 - C12/15, 1 - C16/20, 2 - C20/25, 3 - C15/30, 4 - C30/37)

Figure 3.5: Influencing factors on additional floors for vertical extension (Bikulčiūtė, 2021).

Papageorgiou (2016) performed an optimization study to identify the design parameters for a vertical extension, considering both costs and environmental impact. Based on the study, five parameters were defined: municipal policies; foundation of existing building; absence or not of testing methods; floor system of the new block, and the performance of a feasibility study. The latter reveals the relation between value and costs. The parameters were applied on a structural design and compared with the shadow costs of a rebuilt structure [Fig. 3.6]. A new structure result in a higher ECI value compared to a vertical extension. In conclusion, extension of buildings is a sustainable method to increase liveable space. On this study, a critical remark must be made. The report states: *The advantages of steel upon concrete in vertical extension have been emphasized in both the literature research and the example case studies.* Timber was not considered as a structural material. Most presumable this is a result of the time in which the research is conducted, 2016. Timber is an upcoming structural material.

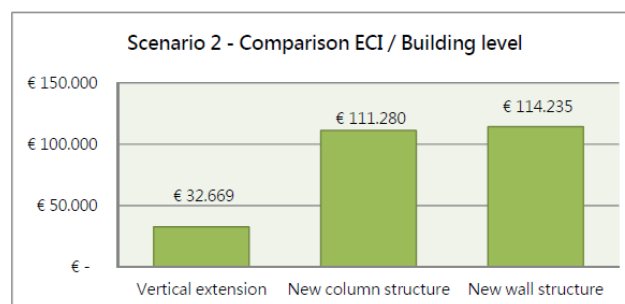


Figure 3.6: Comparison ECI value of a steel top-up and a new constructed building (Papageorgiou, 2016).

González (2017) performed an integral study to tall timber extensions. In this study the architectural and structural aspects were combined. González made a timber extension by considering all design aspects individually. It appeared that a conventional design approach results in complexities since different disciplines do not work together. There is need for an integral design.

3.2.3 Conclusion 'State of Art'

A wide range of information is available on rooftop extensions. Top-ups are project specific and lacking data (Bikulčiūtė, 2021). Bikulčiūtė's research provided general, instead of project specific, information. Papageorgiou (2016) presented project specific possibilities for a steel extension on the Astoria building. Both, Bikulčiūtė and González, emphasised the potency of timber vertical extensions. Papageorgiou (2016) indicated the sustainable potency of steel extensions, instead of timber. Regarding council estates, a lot of literature is available about the interior climate, but research in extending these flats is lacking. The design of SUM indicated the potency a top-up on council estates.

The following points summarizes the current State of Art:

- General (parametric) studies in timber extensions are available (Bikulčiūtė & González). These studies do not consider strengthening of the substructure.
- Project specific studies in steel top-ups (Papageorgiou). Especially the relation between costs and environment is defined.
- Research to the interior climate optimisations of council estates.

The relationship between the structural possibilities of council estates, and the corresponding environmental impact is not examined yet.

3.3 Problem definition

The challenge of **global warming** is an increasing concern of the 21st century. The concept of sustainability is therefore considered of high importance as it aims to protect the environment and, more specific, reduce the emission of greenhouse gasses. This has been underlined by signature of the Paris agreements by many countries, including the Netherlands. A major problem of the Dutch government is the **rising housing demand** and densification. Also, after the WWII, the Netherlands faced a housing crisis, resulting in approximately 847,000 apartments located in council estates (TU Delft, 2022). The **massive number of flats** combined with the aforementioned deficiency of houses and the climate problem prohibit large scale demolition and reconstruction, as this would lead to both a further short-term deficit of houses and emission of construction-related greenhouse gasses.

Mishra et al. (2022) showed the potency of timber buildings to meet the global housing demand, by using careful forest planting, and gradually shift away from CO₂-unfriendly building materials. Hence, a timber top-up on council estates constitutes a sustainable approach, which accounts for densification and the huge building stock in city centres.

Thorough research was performed on improvements for the interior climate of council estates or spatial reorganisations. Data about the structural possibilities is not available. The structural similarities in council estates allow the development of a general method, which could assist structural engineers in a sustainable decision-making process to top-up council estates. Therefore, this research aims to develop a method to optimize timber top-up designs for council estates.

4

Research plan

Chapter 4 will provide the research plan. The research question will be defined, which will be supported by five sub questions. Additionally, a method will be described how to perform the research. The research will be delineated by a scope.

4.1 Research questions

The main research question is:

What is an effective method to optimize timber top-up designs for council estates, considering their structural limits and environmental impact?*

To answer the research question, a theoretical and numerical research are performed.

The **theoretical part** is aimed for researching and learning about council estates, vertical extensions, and environmental impact. The sub-questions related to the theoretical frame are provided in question Q1 till Q3.

The **numerical part** is aimed to develop and calculate for two variants, based on the theoretical frame, the structural and environmental aspects. The sub-questions related to the numerical calculations are provided in sub question Q4 and Q5.

Sub questions:

Q1) What defines a council estate?

Q2) What are possible strategies to top-up a council estate?

Q3) How does the building code count for vertical extensions?

Q4) What are the structural possibilities for the distinct top-up methods?

Q5) What is the environmental impact of the realized variants?

**Effective method will be defined in section 8.1*

4.2 Methodology

The research starts with the preliminary study, followed by the main research (theoretical and numerical part), and ends with the discussion and conclusions.

- The **preliminary research** forms a motivation for the research. Based on an initial literature review, the context, State of Art, problem, and research question(s) are defined.
- The **theoretical part** forms the basis for the numerical exploration. Information is generated based on a literature study, archive inspection, case studies, and interviews. The literature study provides scientific knowledge on council estates, rooftop extensions and codes. The archive inspection provides an overview of the structural layout(s) of council estates. Additionally, it is used for case studies, which broadens the author's vision on top-up possibilities. The interviews provide practical knowledge about top-ups, as well as background information on case studies.
- The **numerical part** develops a method which considers both environmental impact and limitations. The goal is to provide a preliminary design direction that simplifies the decision-making process for top-ups. The software for the calculations is defined in chapter 8. The environmental impact calculation is done by an LCA-calculation.
- The **discussion and conclusions** provide answer to the research question and sub questions. Additional discussion points and recommendations are provided.

4.3 Scope

The scope of this project is narrowed down by choosing the following four restrictions:

1. The method only considers timber top-ups, as motivated in section 3.1.5.3.
2. The research is specifically for council estates. These buildings form a large and relatively homogeneous group, and are therefore suitable for a general top-up method.
3. The method excludes strengthening for additional vertical load. Strengthening for vertical load is complicated and costly, what reduces the applicability of the top-up method.
4. The research does not consider the building physic related aspects. The author will graduate in the structural design specialization, what means that building physic related aspects receive less attention.

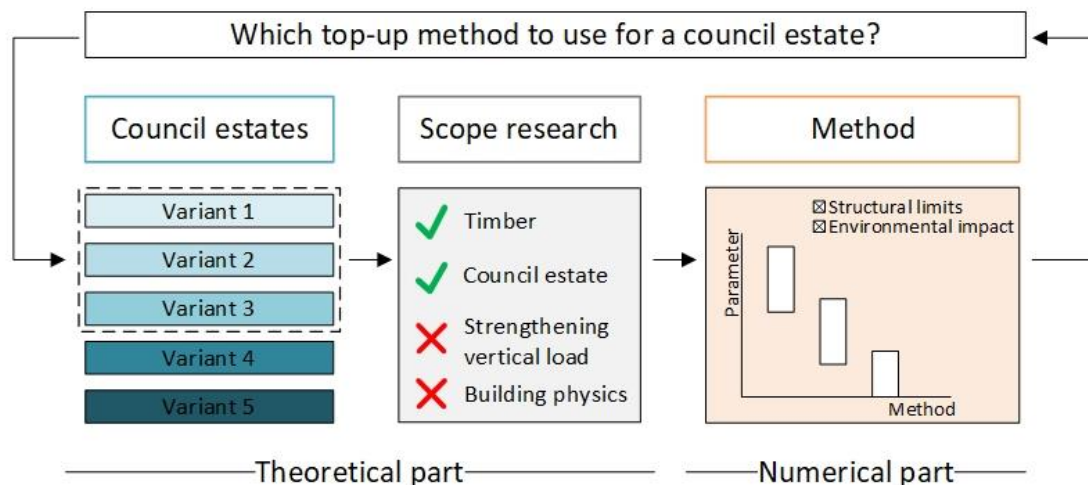


Figure 4.1: Schematization of this study (Self-produced).

5

Council estate

Chapter 5 will describe the typology of council estates in the Netherlands. The chapter will contain an investigation based on a literature study and an archive inspection. The flats will be structurally explored by a parameter study.

5.1 Definition of a council estate

Council estates (Dutch: “portiekflat”) consists of separated houses with their entrance located at a porch. A porch is an open space, with an opening to the road. A stairwell is in the porch to allow the residents to reach their house on different storeys. The porch can be seen as an additional layer of protection against crime. The location of the entrance forms a noticeable difference with standard housing (Watismijnhuiswaard, 2022).

A perfect English translation of the Dutch term **portiekflat** does not exist. This report will use the translation **council estate** as it resembles the Dutch concept the closest. The following definition will be used to classify a council estate:

“A council estate is a multi-storey building, where residents enter their houses by making use of a common route through a porch. A porch is an open space behind the façade with an entrance to the road.”

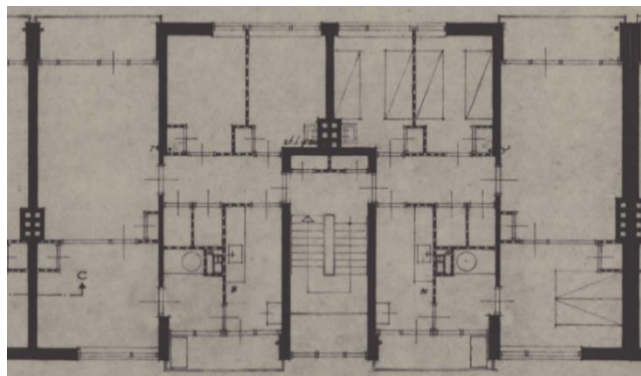


Figure 5.1: Typical layout of a council estate (Archive municipality Amsterdam, 2022).

5.2 Building period

Majority of the council estates were constructed during two periods: interwar period (1916 – 1926) and post-war period (1946 – 1965) (Oorschot et al., 2018). The main driving factor for these buildings was the rapidly growth of urban regions. These buildings are one of the mark points of the Dutch welfare state.

During the **interwar period**, the Housing Act of 1901-1902 was one of the driving protocols that provided affordable housing for the Dutch population. The housing act addressed three parts: building regulation, urban planning, and financial support for housing (Bakker, n.d.). Resultingly, the inter-war council estates were constructed.

The **post-war period** is marked as a period of war damage renovation, city formation, solving housing crisis, and a period of large population expansion (Fels, 2017). The government strived for full employment to prevent the unemployment of 1930 and 1940. To do so, the government provided social security (by the Reconstruction Law). This law provided additional money for housing (Fels, 2017). During this period, the government dominated the decisions regarding the housing market. The government determined numbers of houses, construction quality, production costs, accessibility for houses due to spatial planning, and finally the rent of houses. The housing demand formed in this period a large crisis, what stimulated the building process to become faster, cheaper, and more industrial (Fels, 2017). Resultingly, the post-war council estates were constructed.

5.3 Typology of council estates

Oorschot (2021) performed a study in the typology of council estates. The interwar and post-war period form the normative building periods. The study focussed on four cities: Amsterdam, Rotterdam, The Hague, and Utrecht. In 2016, Amsterdam still counted 155,456 apartments, Rotterdam 104,014, The Hague 107,253, and Utrecht 29,482, located in council estates (Oorschot, 2021). In total, these cities counted approximately 400,000 council houses. Note that a lot of the built apartments were demolished.

Figure 5.2 provides an overview of the literature study in typology of council estates. Section 5.3.1 will provide information about the inter-war period. Section 5.3.2 will outline the post-war period.

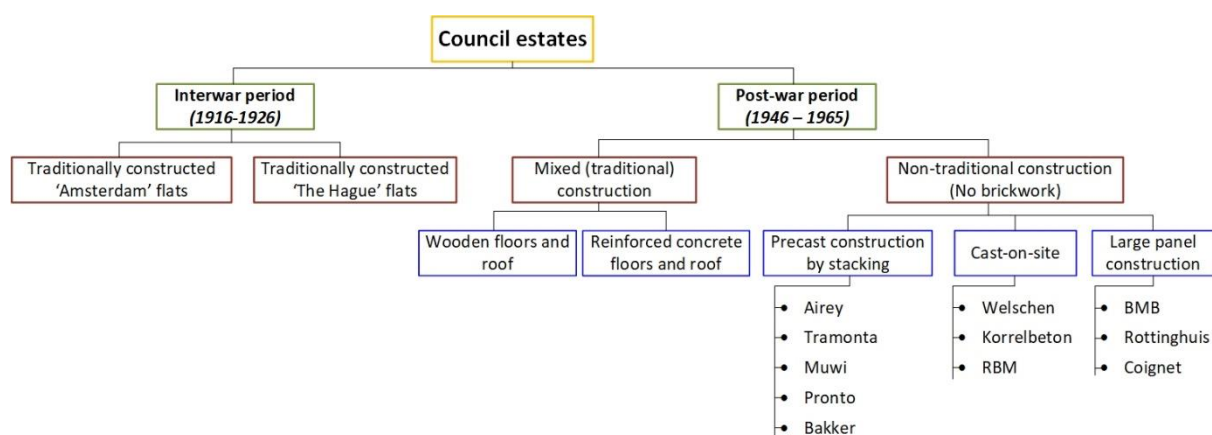


Figure 5.2: Typology of council estates in the Netherlands (Self-produced).

5.3.1 Inter-war period

The interwar buildings were usually constructed as part of a perimeter city block. Following of the urban spaces resulted in irregular floorplans (Oorschot et al., 2018). During the interwar period, Amsterdam built 115,991, The Hague 69,422, Rotterdam 63,299, and Utrecht just 14,406 apartments in council estates (Oorschot, 2021). A lot of these do not exist anymore (due to demolition).

In **Amsterdam** usually a stacking of four apartments is located on both sides of the porch. The porch consists of a closed stairwell and a shared service shaft. Remarkable for these council estates are the pitched roofs, located on top of the buildings (Oorschot et al., 2018).

In **The Hague**, there are usually six units located around a porch. This porch consists, in contrast to Amsterdam, of an open stairwell and two service shafts. That means that the council estates in The Hague are only three storeys tall. The roofs of these council estates are flat. Note that in The Hague the foundations are directly embedded in the subsoil as a “shallow foundation”, whereas the other three cities due to the softer subsoil require deep (pile) foundations (Oorschot et al., 2018).

In **Rotterdam**, the council estates show a similar typology as The Hague. Usually, three storey tall buildings with a flat roof (Oorschot, 2021). In Rotterdam, every apartment has its own stairwell, meaning that the flats become deeper (Oorschot, 2021).

In these inter-war buildings, no elevator is used. The buildings have a traditional construction system with loadbearing walls and facades, floors made of wooden beams (spruce), windows and doors were made of pine. The foundations were traditionally made of brickwork, supported by wooden piles. Sometimes reinforced concrete was used in Amsterdam (Oorschot et al., 2018).

Based on the analysis above, Oorschot et al. (2018) defined two typologies from the interwar period.

1. Traditionally constructed council estates of Amsterdam, with pitched roofs and four stacked apartments mirrored at the shared porch [Fig. 5.3a]. This structural method is equal to the European metropolises of the 19th century.
2. Traditional constructed council estates of The Hague, derived from the Dutch ‘*boven-beneden-woning*’, making sure that each apartment has its own door. Usually three storeys high [Fig. 5.3b].



Figure 5.3: Typology of council estates in the inter-war period; [a] Amsterdam; [b] The Hague (Oorschot et al., 2018: 6).

Despite the subdivision of Oorschot et al. (2018), still there is a lot of variety in dimensions. This is caused by a local urban planning instead of a global organised housing regulation. For this research, it was decided that the large variety of these inter-war buildings will not be applicable for a universal method. There is too much differentiation in shape, floorplan and building material. Therefore, this report will focus further on the post-war council estates.

5.3.2 Post-war period

After the WWII, the building regulations were determined by the government. Resultingly, **local deviations disappeared**. The Prescriptions and Wenken, and the Housing Act provided more standardization in the build-up of council estates. Most of the population had the desire for building on large scale with industrial construction methods. Due to the WWII, there were insufficient money, materials, and skilled construction workers (Oorschot, 2021). That resulted in a continuation of traditional mixed construction methods. At locations with a strong need for houses, the prefabricated system industry developed faster. As a result, in the West of the Netherlands more system constructions were applied (Battum, 2002). The current number of post-war apartments that still remained is about 847,000 (TU Delft, 2022). The post-war flats can be characterized based on construction method.

5.3.2.1 Definition of two main categories

In the period 1946 till 1965, Lijbers et al. (1984) defined five different construction methods.

1. Traditionally built houses in the period 1946 till 1952. Built according to the provisional Wenken (1947), marked by a traditional brickwork structure, combined with a wooden or stony floor (Lijbers et al., 1984).
2. Traditionally built houses in the period 1953 till 1965. Constructed according to the 'Prescriptions and Wenken.' Till around 1956 wooden floors were used (Lijbers et al., 1984).
3. More special construction methods and alternative material use. Some examples of applied systems are: Bredero and Bredero '55, Elementenbouw and Welschen (Lijbers et al., 1984).
4. System building using a continue contract (1948 – 1961). 'Ratiobouw' should provide permission for construction. Examples of these systems are: Airey, Korrelbeton, RBM I, Rottinghuis, Bakker, Muwi, Pronto, Tramonta and BMB (Lijbers et al., 1984).
5. New systems developed in the period 1960 till 1970. Most of these systems are prefabricated systems (Schokbeton-GBS, Coignet, Elementum, VAM, Simplex and Smit II), but also pouring systems exist (Bitcon and EBA) (Lijbers et al., 1984).

Oorschot et al. (2018) summarized the findings of Lijbers et al. (1984) into two main groups (A and B).

- A. Post-war buildings using **traditional mixed construction methods**. Traditionally brickwork loadbearing walls and façade combined with prefabricated construction elements as piles, beams, floor elements and stairs. Approximately 53.4% were built according to the traditional method in the period 1946 till 1965.
- B. Post-war buildings using **non-traditional construction methods**, by making use of (prefabricated) construction components. Approximately 46.6% were built according to this method in the period 1948 till 1961.

The standardization resulted in **more linear structural systems**. Usually, six to eight apartments were equally divided in two stacks, which are mirrored on both sides of the porch (Oorschot, 2021). The two main groups will be set out in the sections [5.3.2.2](#) till [5.3.2.4](#).

5.3.2.2 Traditional construction

Traditional construction can be subdivided into three groups, provided below. No standardization of floor height can be observed before 1961 (Thijssen and Meijer, 1988).

- **Buildings having wooden floors excluding a façade with cavity.** Mainly built in the first half of the years '50 (least applied traditional system). Instead of a cavity, a cling layer of stone was used (Battum, 2002). No crawl space is below the building (Thijssen & Meier, 1988).
- **Buildings having stony floors and façades without cavity.** Built in the period 1946 till 1960 (most applied traditional system) (Battum, 2002). Significant differences could be observed in number of storeys, presence of an attic, angle of the roof, foundation, and façade construction. Roofs were mostly constructed of light-weight concrete (Thijssen and Meijer, 1988).
- **Buildings having stony floors including a façade with cavity.** Constructed in the period 1946 till 1965 (Second largest group of the three) (Battum, 2002). Three to four storey flats were constructed during this period. Both inclined and flat roofs are used. After 1959, the flat roof became normative (Thijssen and Meijer, 1988).

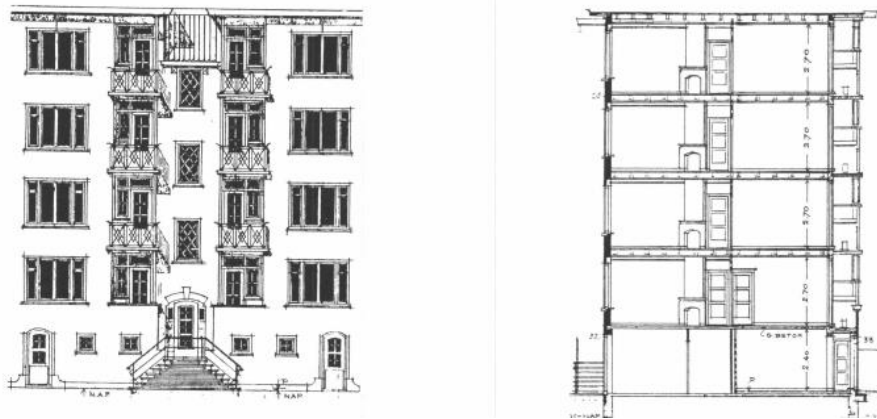


Figure 5.4: Traditionally build council estate with wooden floors (Thijssen & Meier, 1988: 25).

5.3.2.3 Prefabrication methods applied in non-traditional structural methods

Oorschot (2021) defined three main used construction methods in the non-traditional methods: precast construction by stacking, cast-on-site, or large panel precast construction. Till 1961, no standardization in storey height can be observed (Thijssen & Meijer, 1988).

- **Precast construction by stacking**

Construction by stacking means that large blocks were used for the construction. That accelerated the construction process. Different systems were used within the precast construction: Airey, Tramonta, Muwi, Pronto and Bakker. These systems were applied in North and South Holland.

Airey used a combination of concrete or steel columns that are filled with B2-blocks. The skeleton is load bearing. The floors consist of wood on a steel lattice girder. The system was applied from 1949 till 1968 (Thijssen & Meijer, 1988).

Tramonta combined traditional and non-traditional construction. The façade was constructed of brickwork, while on halfway depth a concrete column structure is located. The floors rest on both. The interior walls do not fulfil a loadbearing function, except the stairwell walls (Thijssen & Meijer, 1988).

Muwi, Pronto and Bakker were systems containing their own stacking blocks. Muwi was the most applied stacking method (≈36,000 apartments) (Thijssen & Meijer, 1988). The Muwi system used

hollow blocks, which were dry stacked. Afterwards, these blocks were filled with mortar. Both exterior as interior walls are load bearing (Battum, 2002).

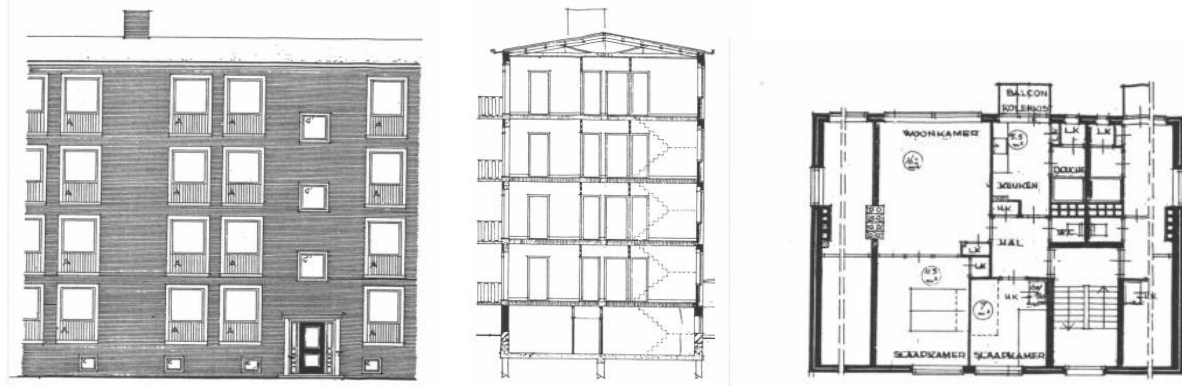


Figure 5.5: Council estate using Tramonta principle (Thijssen & Meijer, 1988: 91).

- **Cast on-site**

Cast on-site uses standardized formwork. Different methods were used: Welschen, Korrelbeton and RBM. Mostly, this system was applied in North and South Holland. Cast on-site was applied after 1960 (Thijssen & Meier, 1988).

Welschen casted all needed elements on-site. The structural system consists of a concrete skeleton and walls, which were stacked concrete blocks, filled afterwards with concrete (Battum, 2002).

Korrelbeton was the first which experimented with standardized formwork for concrete casting. Most of the apartments ($\approx 14,000$) were built before 1962. The structural system is formed by in-situ formed, loadbearing walls. As floors, plates were used (Battum, 2002).

RBM used in-situ cast walls and floors. The walls have standardized sizes (Battum, 2002). Most apartments ($\approx 12,000$) were built in the period 1957 till 1968.

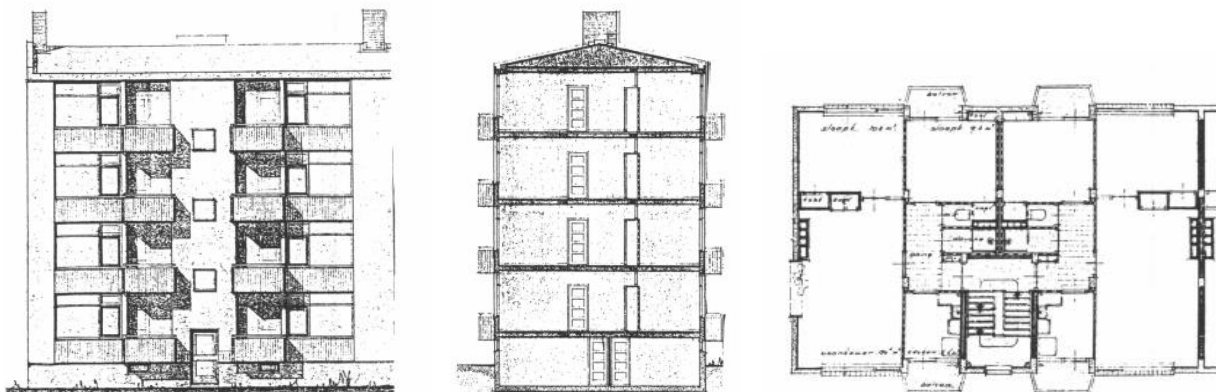


Figure 5.6: Council estate using Welschen method (Thijssen & Meijer, 1988: 122).

- **Large panel precast construction**

Large panel construction used (half) storey-high panels. The large panels resulted in stability problems for tall buildings. Three systems can be distinguished: BMB, Rottinghuis and Coignet (Battum, 2002).

BMB used prefabricated panels of a half storey-height (maximum length of 3.5 meters). A mechanically produced brickwork wall was finished with loadbearing concrete. Floors were constructed of reinforced concrete. BMB was applied from 1950 in Amsterdam and Haarlem (Thijssen & Meier, 1988).

Rottingshuis used prefabricated elements of one storey-height. The loadbearing prefabricated elements were constructed of gravel concrete. The non-loadbearing inner walls were constructed of lightweight hollith concrete. Rottingshuis had own prefabricated concrete floors, placed on top of the walls (Thijssen & Meier, 1988). Rottingshuis was applied in 1965 and 1966 (Battum, 2002).

Coignet used loadbearing concrete walls, with prefabricated concrete floors on top of it, to form six stiff boxes. Stiff connections were needed (Battum, 2002). The system was mainly applied from 1960 till 1966 (Oorschot, 2021).

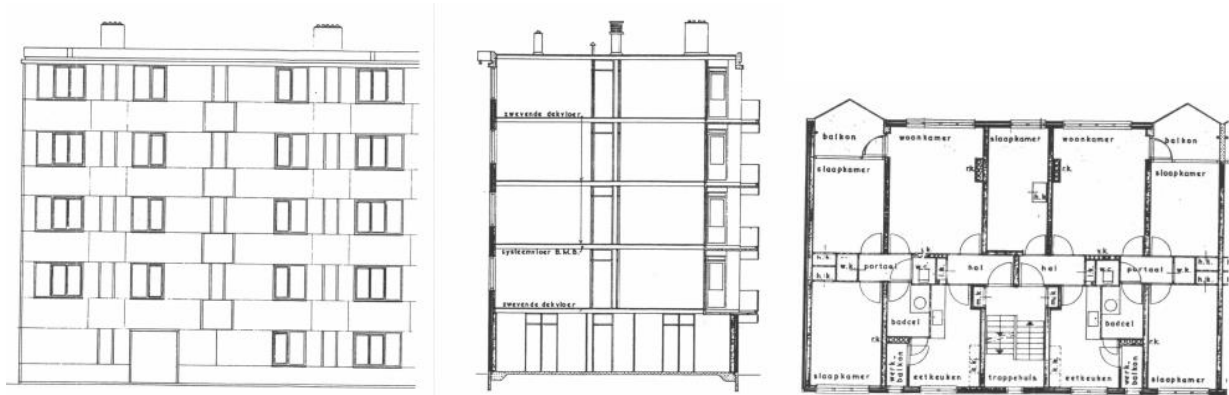


Figure 5.7: Council estate with the BMB method (Thijssen & Meier, 1988: 153).

5.3.2.4 Evaluation different post-war methods

Thijssen and Meier (1988) estimated the percentages of the distinct construction methods (in the Netherlands) [Table 5.1]. The percentage are based on the 22,000 inspected council estates by Thijssen and Meijer (1988).

Table 5.1: Construction methods applied in council estates (Thijssen & Meier, 1988)

Structural method	Percentage (sample) [%]	Location	Building period
Trad. with wooden floor	1.6	Amsterdam, Rotterdam, The Hague	1946-1955
Trad. stony without cavity	33.2	Amsterdam, Rotterdam, The Hague	1946-1960
Trad. stony with cavity	27.1	Widespread in the Netherlands	1946-1965
Precast stack building	13.4	Depends on system, but generally spread over the Netherlands, mostly in North and South Holland	1949-1968
Cast-on-site	17	Mostly North and South holland	1946-1968
Large panel precast construction	7.6	Spread over Netherlands	1950-1966 (Most 1965– 1966)

Table 5.1 indicates that the ratio: **Traditional - Non-traditional** equals approximately **60-40**. Traditional construction with stony floors and no cavity is normative for the traditionally mixed construction. Cast-on-site is the normative category for the non-traditional variants. The traditional constructed council estates with wooden floors are neglectable.

Comparing traditional with non-traditional, the material usage becomes a normative difference. Within the research of Thijssen and Meijer (1988), a variety can be observed in: dimensions, number of storeys, presence of an attic, angle of the roof, foundation, and façade construction.

5.3.3 Basic floor plans

The Prescriptions and Wenken 1946, respectively 1951, formed the basic rules for the mapping of council estates during the post-war period. These norms defined minimum volumes, and room dimensions. Basic floorplans were designed for a maximum of six beds; the study group concluded that families with more than four children are not suitable for council estates (Battum, 2002).

5.3.3.1 Localisation of the stairwell

Important is the location of the stairwell, which can be located (a)symmetrical. In case of an **asymmetrical orientation**, the stairwell is in one of the two apartments (type Ma) [Fig. 5.8a]. It results in two distinct apartments at one floor level, which was occasionally applied (Battum, 2002). The **symmetrical variant** can be subdivided into two variants:

- Type M – The space behind the stairwell is equally subdivided over the two apartments [Fig. 5.8b]. Two equally mirrored apartments on one floor level exist.
- Type Mw – The space behind the stairwell is devoted to one of the two apartments: ‘Additional room’ or ‘Bajonet connection’ [Fig. 5.8c]. This is the most often applied system, as it anticipates better on the family size.

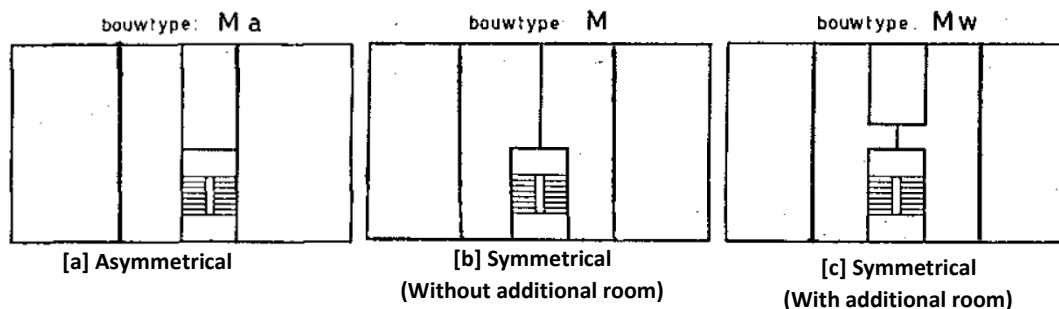


Figure 5.8: Possible orientation of stairwell in council estates (Battum, 2002: 42).

Symmetry was preferred for both parallel and perpendicular to the façade. That would provide more flexibility in the future. The study group efficient housing determined minimum dimensions for the stairwell: minimal depth x width = 3.6 x 2.3 m. Also, a standardized storey height of 2.8 m was introduced (Battum, 2002).

5.3.3.2 Standard floorplans

Within the floorplans, four main variants can be observed. The presented variants include the bajonet connection but also exist without. The differentiation in floorplans is dominated by the relation between kitchen and living room (Battum, 2002).

- Variant A – The living room has the largest dimensions along the façade. The kitchen is placed along the other façade [Fig. 5.9a].
- Variant B – The living room has the largest dimensions perpendicular to the façade. The kitchen is placed in the same bay to the other façade [Fig. 5.9b].
- Variant C – The living room has the largest dimension perpendicular to the façade. The kitchen is located at the other façade in the other bay [Fig. 5.9c].
- Variant D – The kitchen is located at the same façade, next to the living room [Fig. 5.9d].

Battum (2002) performed an archive inspection to determine the division between the different variants. An overview is presented in table 5.2. Variant C including additional room is the normative floorplan for council estates.

Table 5.2: Division of maps in [%] (Battum, 2002: 53-54).

Variant	Traditional				Non-traditional			
	A	B	C	D	A	B	C	D
Incl. additional room	2.4	2.7	29.6	9.5	3.7	3.8	10.9	7.0
Excl. additional room	0	6.6	5.8	9.4	0.7	0	1.5	3.4
Total	2.4	9.3	35.4	18.9	4.4	3.8	12.4	10.4

**Figure 5.9:** Variants of floorplans (Battum, 2002: 46-47). Variant A - D are [a] - [d], respectively.

5.3.4 Number of storeys

Lijbers et al. (1984) performed research to the housing stock in the Netherlands during the period 1945 – 1975 (includes Post-war period). The period after 1965 was dominated by an increased building height, using elevators. These (gallery-) flats are different compared to council estates. Lijbers et al. (1984) divided the post-war period (1946 – 1965) into two parts: 1949 till 1952 and 1953 till 1965. This due to variation in registration methods.

The period of 1949 till 1952 used a registration of the residential layers. This layer was defined as every building layer that contains a living or bedroom. The period 1953 till 1965 used a registration of the number of building layers. This layer was defined as a layer with a storey height larger than 2.3 m, excluding the ground floor. It can be concluded that **most of the council estates have four storeys**. Approximately 31% is classified as gallery flat. These are the ones with more than four building layers (Lijbers et al., 1984).

Table 5.3: Number of storeys in historical stock (Lijbers et al., 1984: 42-43).

1949 - 1952			1953 - 1965		
2	Residential layers	11%	2	Building layers	8%
3	"	52%	3	"	24%
4	"	28%	4	"	38%
>4	"	10%	5	"	18%
			6	"	1%
			7	"	2%
			8	"	1%
			>8	"	8%

5.4 Archive inspection

The archive of Amsterdam will be used to inspect distinct council estates. An overview of the inspection, schematized structural floorplans, and actual floorplans will be provided in appendix A.

5.4.1 Data collection

The purpose of the inspection is to collect structural data about council estates. General information, dimensional data, and structural aspects are collected. The archive of Amsterdam is used as it is online accessible. Due to the Housing Act, the construction process was organized by the government, what eliminated the local deviations (Oorschot, 2021). Inspecting only Amsterdam is sufficient for general information about council estates. Table 5.4 provides an overview of the inspected variants.

Most of the inspected flats were traditionally built: 61.5%. The structural drawings and calculations are often excluded from the archive. Resultingly, the author derived structural systems from the architectural drawings [Appendix A.1].

Table 5.4: Overview of inspected council estates.

Inspection number	Location	Apartments	Construction year
1	Helena Mercierstraat, Amsterdam	168	1960
2	Willem van Weldammelaan, Amsterdam	90	1961
3	Comeniusstraat, Amsterdam	489	1956
4	Maassluisstraat, Amsterdam	120	1959
5	Anske Lamingastraat, Amsterdam	256	1954
6	Petrus Blokhof, Amsterdam	384	1954
7	Grasrijk, Amsterdam	128	1957
8	Loevenstein, Amsterdam	167	1961
9	Jan Voerstraat, Amsterdam	160	1961
10	Saaftingestraat, Amsterdam	324	1961
11	Schoonboomstraat, Amsterdam	459	1962
12	Honselersdijkstraat, Amsterdam	380	1959
13	Steenderenstraat, Amsterdam	143	1981

5.4.2 Standard lay-out of the council estate

A council estate has most often a symmetrical orientated stairwell and floorplan. The stairwell depth equals about 50% of the building depth. Apartments at the same floor are mirrored in the stairwell. All walls around the stairwell are loadbearing. The vertical load bearing system consists of the wall structure. The structural walls are orientated perpendicular to the front façade, while the floors span perpendicular to them. A stability wall is longitudinally located at the end of the stairwell. The structural walls provide stiffness in cross-direction.

The average number of floors equals 3.67. In case of a sloped roof, an attic is often added. The ground floor functions as storage space or parking.

In general, three structural floorplans exist [Fig. 5.10]. Variant 1 includes an additional room, variant 2 and 3 excludes. Variant 3 has slightly shifted inner walls. Only considering the wall orientation perpendicular to the front façade, 38.5% was constructed according to variant 1, 38.5% to variant 2 and 23.0% to variant 3.

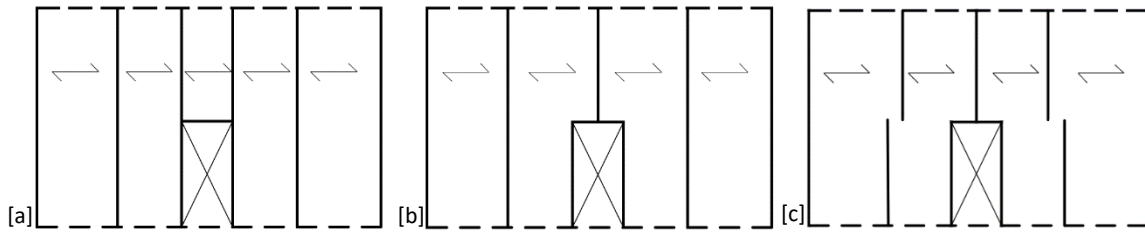


Figure 5.10: Main structural floor plan divisions. [a] Variant 1; [b] Variant 2; [c] Variant 3 (Self-produced).

5.4.3 Structural principles: traditional and non-traditional methods

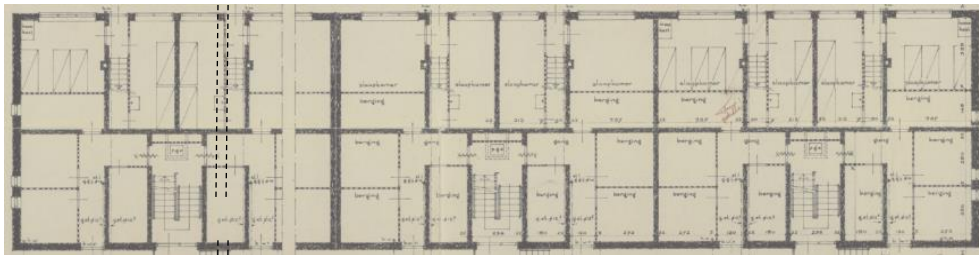
Although the floorplans show a general build-up, variation can be observed between traditional and non-traditional construction. The structural principles of both methods will be described.

5.4.3.1 Traditional construction

The flats were constructed in blocks. Such blocks are divided by dilatations. The ground floor contains more walls, as it is used for storage space [Fig. 5.11]. Brickwork was used to construct the loadbearing walls. These walls transfer both vertical and horizontal loads. The archive indicates a general applied wall thickness of 220 mm. The early traditional flats have load-bearing façades. The floors are mostly continuous slabs, made of reinforced concrete.

In longitudinal direction the stability is realized by a longitudinal-orientated wall at the stairwell. Piers provide additional stiffness in case of a loadbearing façade. In cross-direction, the structural walls provide stiffness. Figure 5.11 shows the council estate located at the Honselersdijk in Amsterdam. It illustrates the additional longitudinal-orientated walls at the ground floor.

Ground floor



1st to 4th storey

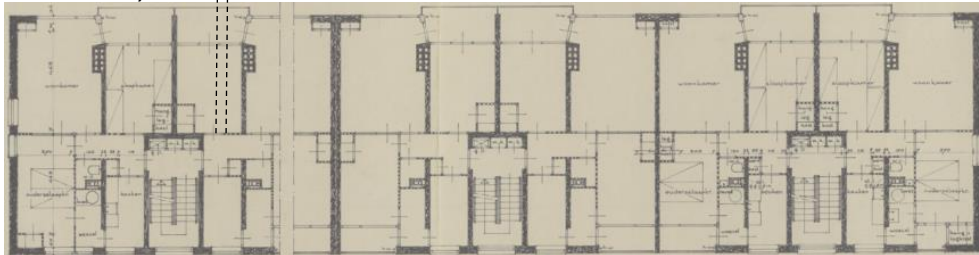


Figure 5.11: Honselersdijk Amsterdam (Archive Municipality Amsterdam, 2022).

5.4.3.2 Non-traditional construction

Non-traditional construction methods have more variety in construction methods, compared to traditional construction. Muwi-, Sintel-, or BB-blocks were used, which are concrete-related materials. The inspected archive contains two Airey variants, and one Muwi, precast stack, and cast in-situ variant. The non-traditional construction methods are defined as one variant. A more elaborate description of the distinct systems can be found in appendix A.2.

The vertical load transfer follows a similar principle as traditional mixed construction. The floors span parallel to the façade and transfer the load towards the loadbearing walls. The thickness of these walls depends on the applied system. Table 5.5 provides a summarized overview of the applied blocks.

Table 5.5: Standard wall thicknesses, non-traditional variants.

Non-traditional system	Type of blocks	Thickness [mm]
Muwi	Muwi-blocks	210
Airey system	Sintel-blocks	205
Precast stack building	BB-blocks	205
Cast in-situ	Concrete	200

The non-traditional variants (excl. cast in-situ variant), have loadbearing façades. The build-up of the façade differs per variant [Appendix A.2]. Similar for the varying internal walls. In cross-direction, the walls provide stiffness. The longitudinal stiffness depends per system. The Muwi, precast stack and in-situ building uses stability walls. The Airey-system has reinforced columns inside the façade, which are monolithic connected to the floor. Figure 5.12 provides a schematic representation of a Muwi, an Airey system and a precast stack variant.

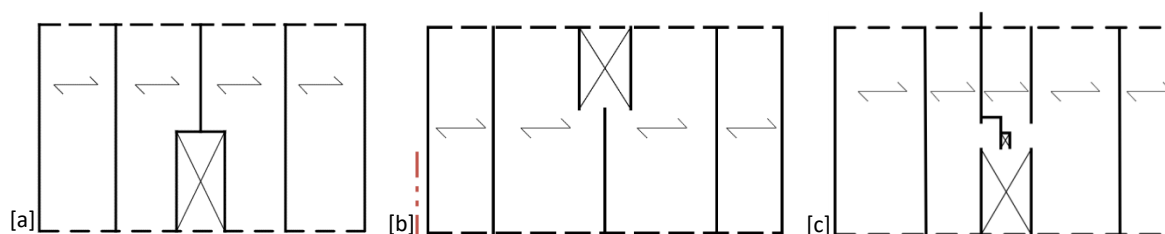


Figure 5.12: Schematization of [a] Muwi, [b] Airey, and [c] precast stack variant (Self-produced).

5.4.4 General aspects

Some general aspects are observed, considering floor system, roof structure and foundation.

- The archive inspection did not contain traditional timber floors (except the attics). The inspected reinforced floors have a varying thickness between 110 and 150 mm, spanning in one direction. The non-traditional systems use their own prefabricated flooring system, which can consist of thicker reinforced floors.
- Flat and pitched roofs exist in the archive research. All pitched roofs were constructed out of a wooden structure. Usually, secondary beams span in one direction, while the top is cladded with timber or stony roof decking. Flat roofs can be constructed similar or as reinforced floor. The reinforced floor thickness varies between 12.5 and 20.5 cm.
- All inspected variants have a piled foundation. These piles were mostly constructed in reinforced concrete (except one building; wooden piles). The foundation strips follow the lines of the loadbearing walls.

5.4.5 Variety in dimensions

Appendix A.1 provides an overview of the investigated council estates and the dimensions. Dimensional variety can be observed in width and length of the stairwell, building depth and height, unit width of a council estate, and the bay width.

The **width of the stairwell** varies between 2.37 and 4.18 m, with an average value of 2.70 m. Note that most staircases have a width of 2.55 m. The **length of the stairwell** is approximately half of the building depth. The **depth of the buildings** in the sample varies from 9.46 till 12.00 m. The **gross floor height** varies between 2.80 till 3.00 m (mostly between 2.80 till 2.90 m). The **unit width** of a council estate is defined as the width of two apartments, located to the porch [Fig. 5.13b]. The unit width varies between 13.36 and 21.10 m. Although the large variation, most buildings have a width smaller than 16.36 m [Fig. 5.13a].

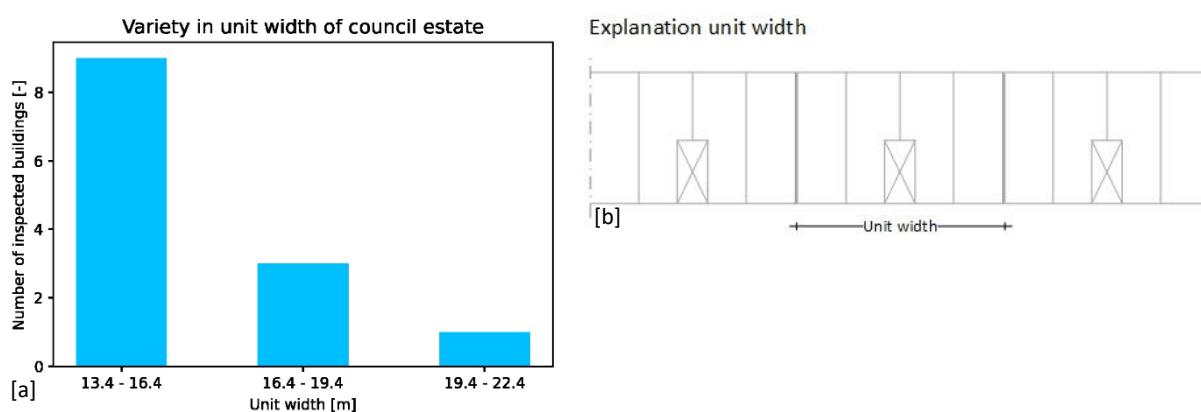


Figure 5.13: Unit width; [a] Histogram. [b] Explanation definition (Self-produced).

The definition of **bay width** strongly depends on the defined variant [Fig. 5.10]. For each variant, bay width 1 and 2 is defined [Fig. 5.14]. Non-traditional variants have in general a larger floor span.

- Variant 1: bay width 1 varies from 3.00 to 4.19 m; bay width 2 varies from 3.31 to 5.13 m.
- Variant 2: bay width 1 varies from 4.04 to 5.42 m; bay width 2 varies from 2.54 to 5.43 m.
- Variant 3: bay width 1 varies from 3.08 to 4.12 m; bay width 2 varies from 2.46 to 3.93 m.

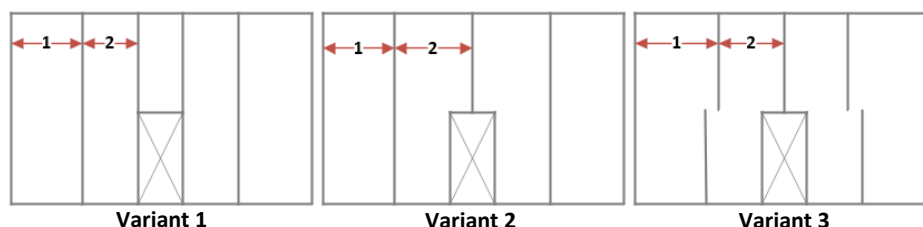


Figure 5.14: Definition 'bay width', variant 1 till 3 (Self-produced).

5.4.6 Evaluation of inspection

From the archive inspection of Amsterdam, a number of remarkable facts can be concluded. Most flats were constructed in a traditional building style: brickwork walls with a reinforced concrete floor system. The brickwork walls show a standardized thickness of 220 mm. The non-traditional variants

show large variation in structural principles: materialization, sizes, and horizontal load transfer strategies.

No standardization can be defined within the council estates, except a general structural floorplan. The Prescriptions and Wenken standardized volumes, which is not interchangeable with the structural dimensions. Most of the buildings have a unit width around 16 meters, while the bay width varies strongly. The found average story number of four is in line with the research of Lijbers et al. (1984).

In most cases, stability is realized by stability walls. In cross-direction, the structural walls provide stiffness. In longitudinal direction, the wall at the stairwell functions as stiffening element. Two to three buildings are structurally connected, followed by a dilatation.

5.5 Exploratory calculations

Exploratory calculations will be performed to inform the author about the vertical and horizontal load transfer in council estates. Section 5.5 will contain the set-up for the calculations of the horizontal load transfer of the flowchart S [Fig. 6.15]. An elaborate description of the vertical load calculations will be provided in appendix A.3. Appendix X.1 and X.2 will provide excel sheets of the calculations.

5.5.1 Set-up

The verifications use actual data from the archive inspection. The calculations use the NEN-EN 1991 as guiding norm, including the safety factors of equation 6.1 and 6.2. For the wind load, wind area II: urban, is normative (using an average building height of 14.00 m). The snow load (0.56 kN/m²), imposed floor load (2 kN/m²) (attic 1.75 kN/m²), and imposed roof load (1 kN/m²) form the other variable loads. Permanent material loads are brickwork (1900 kN/m³), concrete (2400 kN/m³) and gravel (1900 kN/m³).

5.5.2 Vertical load transfer

Five distinct groups are evaluated for the vertical load transfer: high- and low-quality brickwork, prefabricated and in-situ concrete, and a Muwi-system. The Airey system is not normative due to the strong deviating, light-weight floor system. 4-storey buildings are considered. General aspects are described per construction method.

5.5.2.1 Traditional brickwork construction

The brickwork walls have a standardized thickness of 220 mm. **Historically, the walls were dimensioned based on rules of thumb.** No brickwork with a lower compressive strength of 5 N/mm² was used in practise (Belgische baksteenfederatie, 2008). A standard floor and roof build-up is applied for the traditional variants, provided in table 5.6.

Table 5.6: Permanent load definition for traditional construction; roof [left] and floor [right].

Roof build-up	Thickness/ Weight	Load [kN/m ²]	Floor build-up	Thickness / Weight	Load [kN/m ²]
Gravel	30 mm	0.48	Topping	40 kg/m ²	0.4
Schewil insulation plate	70 mm	0.2	Reinforced concrete floor	150 mm	3.6
Reinforced concrete	125 mm	3	Ceiling	20 kg/m ²	0.2
Ceiling	20 kg/m ²	0.2	Non-loadbearing walls	-	0.5
	Total	3.88		Total	4.7

The compressive stress capacity of brickwork can be determined by both mortar and brickwork quality (NPR 9096). There is a large uncertainty in the applied brickwork quality. The characteristic compressive stress capacity varies from low-quality (2.55 N/mm²) to high-quality (11.58 N/mm²). A material factor of 1.7 must be applied. Experts from both Van Rossum B.V. and Nebest B.V. have indicated that **brickwork quality is project specific and must be determined at location**. Variant 3, 7, 8 and 10 of the archive inspection are considered. Details can be found in appendix A.3.

5.5.2.2 Non-traditional variants

Three non-traditional variants are considered: Muwi, prefab and in-situ. Standardized sizes for wall thickness are used: Muwi 210 mm; (prefab) concrete 200 mm. The archive is used for the build-up of a Muwi floor and roof [Table 5.7]. The in-situ variant uses the traditional build-up [Table 5.6]. The prefab variant is calculated with its own floor system 220 kg/m² and a Muwi roof build-up.

Table 5.7: Permanent weights, used for Muwi system; roof [left]; floor [right]

Roof build-up	Weight [kN/m ²]	Floor build-up	Weight [kN/m ²]
Prefab system floor	1.88	Prefab system floor	1.88
Roof finishing	0.92	Roof finishing	0.40
Ceiling	0.22	Ceiling	0.22
Total	3.02	Total	2.50

The design compressive stress capacity of the Muwi variant equals 4.0 N/mm² (Makobeton, 2014). Concrete quality C12/15 is applied, what results in a design compressive stress for the concrete variants of 8.0 N/mm². For both a material factor of 1.5 is considered. The Muwi-system is applied on variant 3 of the archive inspection. Both prefab and in-situ structure on variant 3 and 13. Details about these verifications can be found in appendix A.3.

5.5.2.3 Verification and conclusions

The ground floor walls are used for the verification. The normative wall has the largest sum of floor spans on both sides. The floors are continuous over the walls. This is simplified as a simply supported floor [Fig. 5.15]. In these early design stages this simplification is justified. The load to the wall is calculated based upon 50% of the floor span on both sides. The acting normal stress for traditional variants equals around 1.1 N/mm², while non-traditional variants have a stress around 1.2 (prefab) to 1.5 (in-situ) N/mm². The verification is performed, based on equation 5.1.

$$UC^* = \frac{\sigma_{Ned}}{f_{c,0}} [-] \quad (5.1)$$

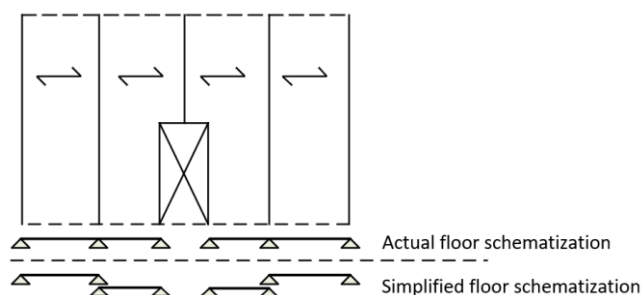


Figure 5.15: Schematization and simplification of the floors (Self-produced).

*UC = Unity Check

Figure 5.16 provides a summary of all individual verifications. A number of aspects are concluded.

- The non-traditional variants have unity checks around 0.2 till 0.3. This is mainly caused by the higher compressive capacity of the non-traditional materials.
- The traditional variants indicate a large spread in unity check. This is caused by the **uncertainty in brickwork quality**. In case high-quality brickwork is used, the check for normal stress equalizes the non-traditional variants. More realistic is a lower quality of brickwork. The lowest quality of brickwork shows a unity check of 0.78. That limits the top-up potential. A top-up on traditional construction requires an inspection on site to determine the quality of brickwork.

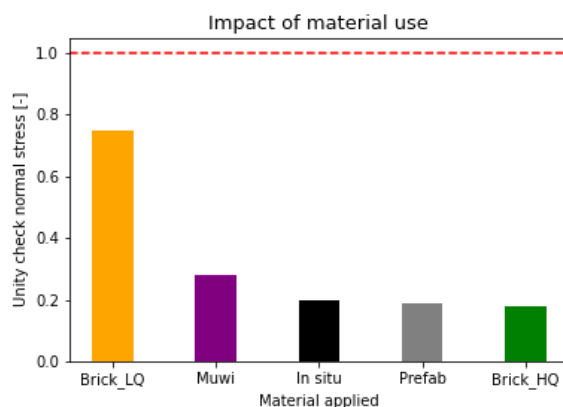


Figure 5.16: Verification normal stress, considering distinct materials (LQ = Low quality brickwork; HQ = High quality brickwork) (Self-produced).

5.5.3 Stability

This section contains the applied principles, as well as the results for the stability of the Helena Mercierstraat. That building is a normative variant due to its traditional construction, ratio: building depth to stability wall length, and 4-storey height. A safety factor of 0.9 is applied on permanent loading, while vertical variable loading is excluded (except wind load; $\gamma_Q=1.5$). Table 5.8 provides an overview of the parameters that is used.

Table 5.8: Parameters Helena Mercierstraat.

Geometrical parameters	Value	Wall parameters	Value
Depth	10.85 m	Length stab. wall	2.62 m
Height	14 m	Thickness	220 mm
Width (one unit building)	18 m	Contr. width top	3.23 m
Storey height	2.80 m	Contr. width bottom	1.62 m
Units between dilatation	2 [-]	Floor weight	4.7 kN/m ²

As a result of the connectivity between the stability wall and the stairwell-/ apartment separation wall, part of these walls is classified as contributing width (Contr. width) (Van Rossum B.V., internal communication). The load that acts on that width, forms an additional load to the stability wall. Figure 5.17 provides the definition of the contributing width. It is defined as an angle of 30 degrees from bottom to the second floor. Figure 5.18 provides the stability situation in both wind directions.

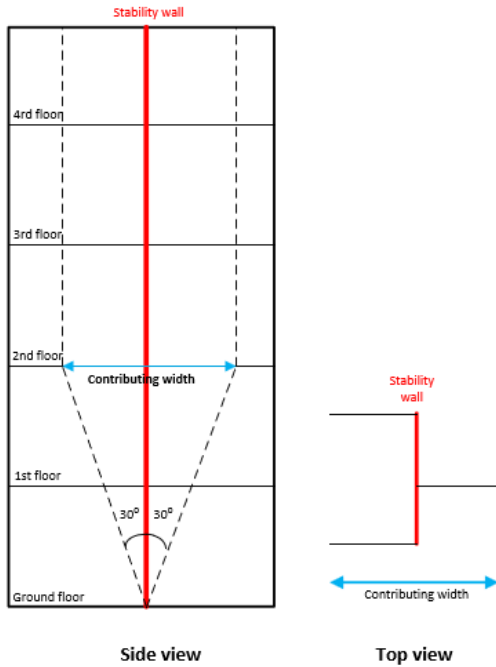


Figure 5.17: Contributing width (Van Rossum B.V.; Internal communication; Self-produced).

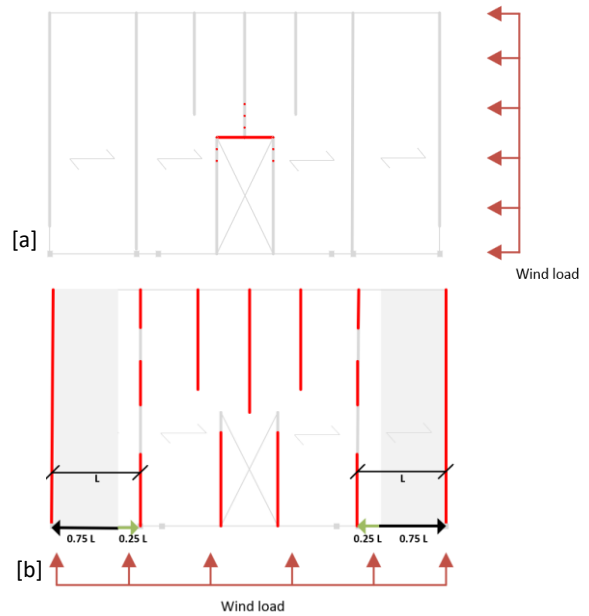


Figure 5.18: Stability situation; [a] Longitudinal direction; [b] Cross-direction (Self-produced).

5.5.3.1 Longitudinal direction

The longitudinal-orientated stability wall provides stiffness in that direction [Fig. 5.18a]. The contributing wall area delivers point-loads on the stability wall [Fig. 5.19]. The maximum compressive force is calculated according to equation 5.2. Equation 5.3 till 5.5 are used to determine the tensile force, caused by the wind. Note that only pressure is used due to dilatation ($C_f = +0.8$).

$$F_{c,max} = F_L + 0.5F_m + 0.5qL = 432.4 \text{ kN} \tag{5.2}$$

$$q_{wind,wall} = \frac{1}{2} \cdot C_s C_d \cdot C_f \cdot q_p(z_e) \tag{5.3}$$

$$M_{wind,ULS} = \frac{1}{2} q_{wind,wall} H^2 \cdot 1.5 \tag{5.4}$$

$$F_{t,max} = M_{wind,ULS} / L_{wall} = 189.9 \text{ kN} \tag{5.5}$$

The unity check is defined as the ratio between the maximum compressive force and tensile force, which equals 0.44.

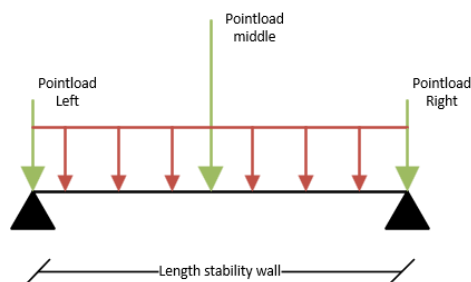


Figure 5.19: Schematization of stability wall in longitudinal direction (Self-produced).

5.5.3.2 Cross direction

The structural walls in cross-direction provide stiffness in that direction [Fig. 5.18b]. The outer wall is normative due to low vertical weight. It is assumed that (due to higher stiffness) $\frac{3}{4}$ of the wind on that bay is transferred to that wall [Fig. 5.18b]. Both compression and suction can occur ($C_{f,tot} = +1.5$). Equation 5.6 calculates the design bending moment per wall. The bending stresses are calculated according to equation 5.7.

$$M_{wall,i,ed} = \frac{1}{2} q_{wall,i} \cdot H^2 \cdot 1.5 \quad (5.6)$$

$$\sigma_{M,i,ed} = \frac{M_{wall,i,ed}}{W_i} \quad (5.7)$$

Based on equation 5.6 and 5.7, the bending stresses equal 0.17 N/mm². The acting compressive stress equals 0.42 N/mm². The unity check equals 0.28.

5.5.3.3 Conclusions stability

Four conclusions can be made on the stability.

- The Helena Mercierstraat has still overcapacity in the stabilizing elements.
- The definition of the contributing width strongly depends on the structural engineer. The code allows an angle till 45 degrees. Additionally, it is allowed to continue the angled line more than two storeys.
- The wall, next to the dilatation, forms the normative wall for cross-directional stiffness. It appears that cross-direction is not normative for the wind-load.
- Historically stability was not calculated. Buildings with a height between 10 and 20 m must have a stabilizing wall at the end of the stairwell with a thickness of at least 200 mm. Spacing between distinct stairwells was limited to 18 m. Floors must be designed to transfer a horizontal load of 2300 kgf/m (=23 kN/m) (MH Bouwontwerp & Advies, 2017).

5.6 Parameter study

A parameter study will provide the author insight in the effect of varying parameters. Both the vertical and horizontal load transfer will be inspected. Excel results will be provided in appendix X.3.

5.6.1 Parameters for vertical load transfer

The Helena Mercierstraat is defined as standard variant. Distinct parameters are varied to indicate the effect of varying dimensions on the normal stress in walls. The following parameters are studied:

- Connected floor length;
- Material use for floors and roof structure;
- Wall opening percentage;
- Number of storeys.

Connected Floor Length (CFL)

As the floor span on both sides of the wall varies, the sum of the two is defined as connected floor length. 50% of the connected floor length transfers load to the wall. A sample, including the outer apartment separation walls, contains a larger connected floor length, compared to the one which excludes them [Fig. 5.20]. The opening percentage of the outer walls equals zero, while the internal walls have one larger than zero. Both combinations are verified: including outer walls with an opening percentage of zero; excluding outer walls with an opening percentage of 16%. A larger connected floor length can be observed by non-traditional constructed buildings [Fig. 5.20].

Material use for floors and roof structure

Two main materializations can be defined in floor and roof system: concrete and timber. For concrete a floor and roof load is defined as 4.70 and 3.88 kN/m², respectively. For timber these are defined as 1.00 and 0.56 kN/m², respectively. These values include non-loadbearing walls and finishings.

Wall opening percentage

The interior walls contain two, three or four openings. This equals an opening percentage of 15, 23 and 31%, respectively. The opening percentage reduces the cross-section of the wall.

Number of storeys

The archive research contains two variants: 3- and 4-storey flats. Using the standardized storey-height (2.80 m), results in a building height of 11.20 and 14.00 m, respectively.

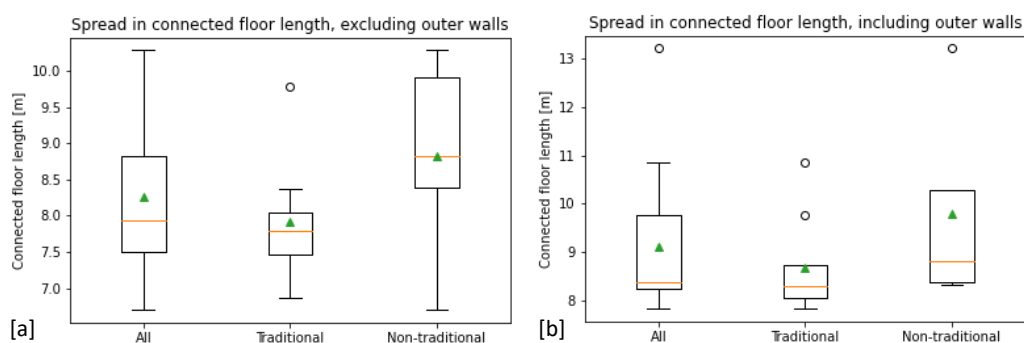


Figure 5.20: Boxplots of spread in connected floor length, [a] excluding or [b] including outer walls (Self-produced).

5.6.1.1 Results

Figure 5.21 presents an overview of the parameter study. Mark that the conclusions only hold in case the walls above the ground are normative for overcapacity instead of the foundation.

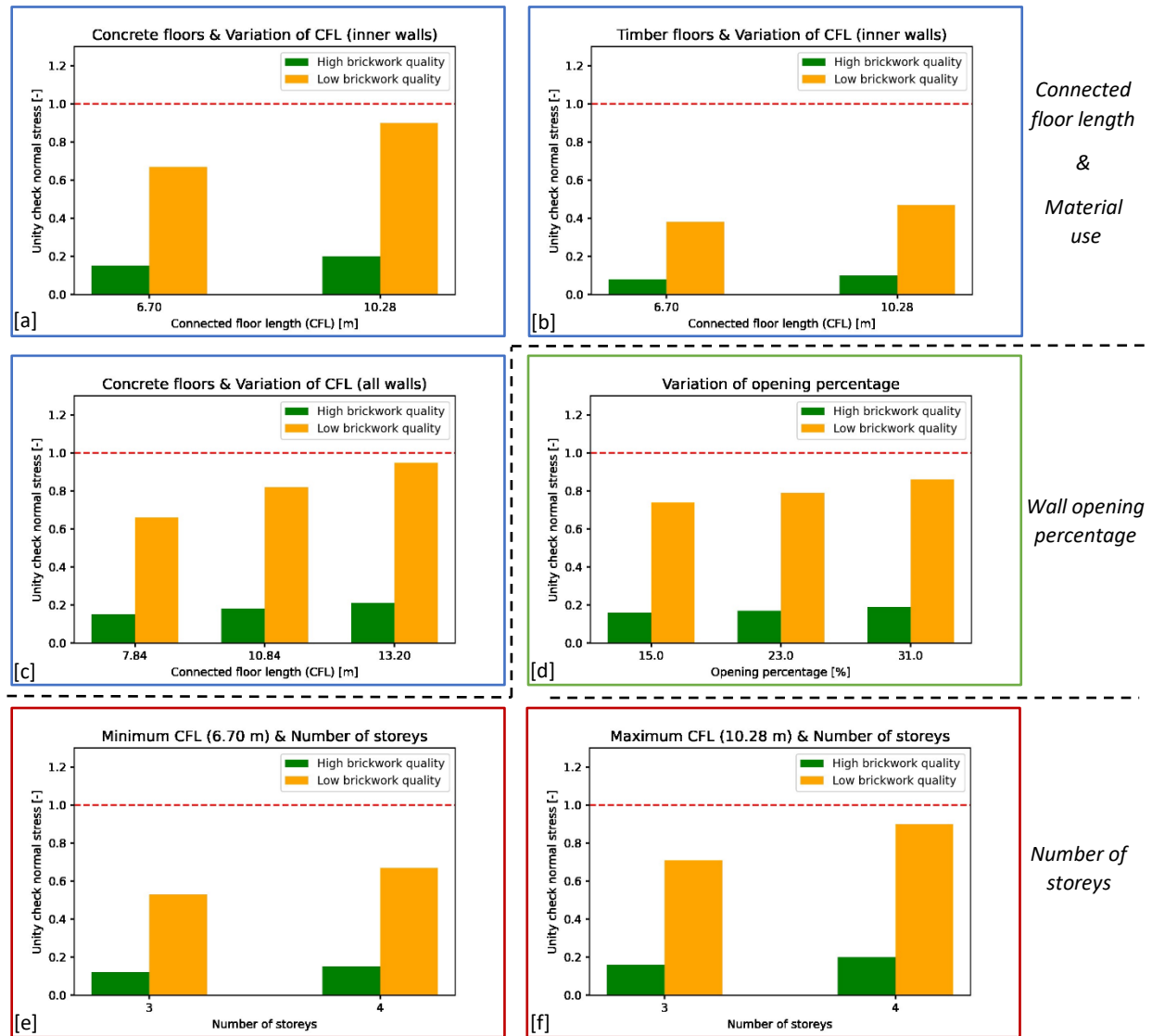


Figure 5.21: Results parameter study on vertical load transfer (Self-produced).

5.6.1.2 Discussion

A larger connected floor length has negative effects on the vertical load transfer [Fig. 5.21a, b, c, e, f]. In case the vertical weight is lower, this effect lowers [Fig. 5.21a, b]. The top-up potential for wooden floors and roofs ($UC^*=0.38 - 0.47$) is larger compared to reinforced concrete construction ($UC=0.67 - 0.90$) [Fig. 5.21a, b]. The internal walls ($UC=0.90$) are normative compared to the outer walls ($UC=0.82$), considering a connected floor length of 10.28 and 10.84 m respectively [Fig. 5.21a, c]. The connected floor length of 13.20 m forms a strong deviating variant in the sample, so is not considered in the conclusion [Fig. 5.21c]. A larger opening percentage has negative effects on the vertical load transfer [Fig. 5.21d]. The difference between two [15%] and four [31%] openings is relatively low ($UC +0.07$) [Fig. 5.21d]. A 3-storey flat ($UC=0.53 - 0.71$) has more top-up potential compared to a 4-storey flat ($UC=0.67 - 0.90$). The effect of the connected floor length should be considered. A 3-storey building with a maximum connected area, has a unity check comparable with a 4-storey building with a minimum connected area [Fig. 5.21e, f].

*UC = Unity Check

5.6.2 Parameters for stability

The Helena Mercierstraat is defined as standard variant. Distinct parameters are varied to indicate the structural effect varying dimensions. For the stability, the following parameters are studied:

- Ratio: depth building / length stability wall;
- Number of storeys
- Material of floors and roof structure;
- Normative floor length.

Ratio: depth building / length stability wall

An undefined relation exists between the depth of the building and the length of the stability wall. To address both, the ratio between the two is defined as parameter. The ratio is on average 4.07, consisting of two extremes [Fig. 5.22]. The ratio varies between 2.87 and 4.50 [-].

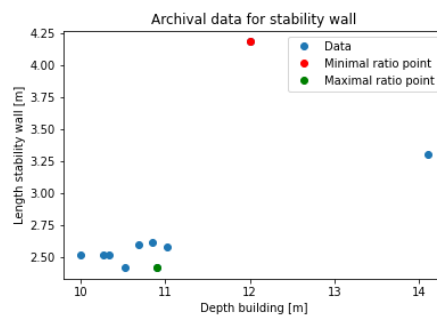


Figure 5.22: Relation between depth building and length stability wall (Self-produced).

Number of storeys

Similar to section 5.6.1. The parameter consists of a 3- and 4-storey building.

Material of floors and roof structure

Similar to section 5.6.1. Both concrete and wooden structures are considered.

Normative Floor Length (NFL)

Normative floor length is defined as the floor length in meters, which transfers vertical load to the stabilizing wall. Multiplication with the contributing width [Section 5.5.3], provides the floor area which transfers load to the wall. A distinction is made between variants in- and excluding an additional room. Below both formula and boundaries are provided.

- Excl. additional room [Fig. 5.23a] $NFL = 0.5a + b$ $2.67 \text{ m} \leq NFL \leq 4.26 \text{ m}$
- Incl. additional room [Fig. 5.23b] $NFL = 2c + 0.5d$ $4.07 \text{ m} \leq NFL \leq 8.25 \text{ m}$

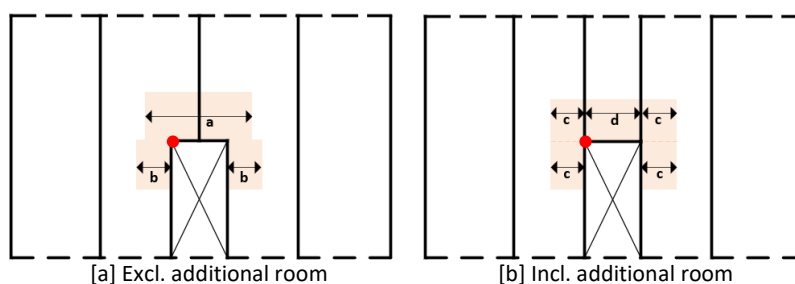


Figure 5.23: Definition normative floor length; Red dot: verification location (Self-produced).

5.6.2.1 Results

Figure 5.24 presents an overview of the parameter study. The distinct colours define different researched parameters.

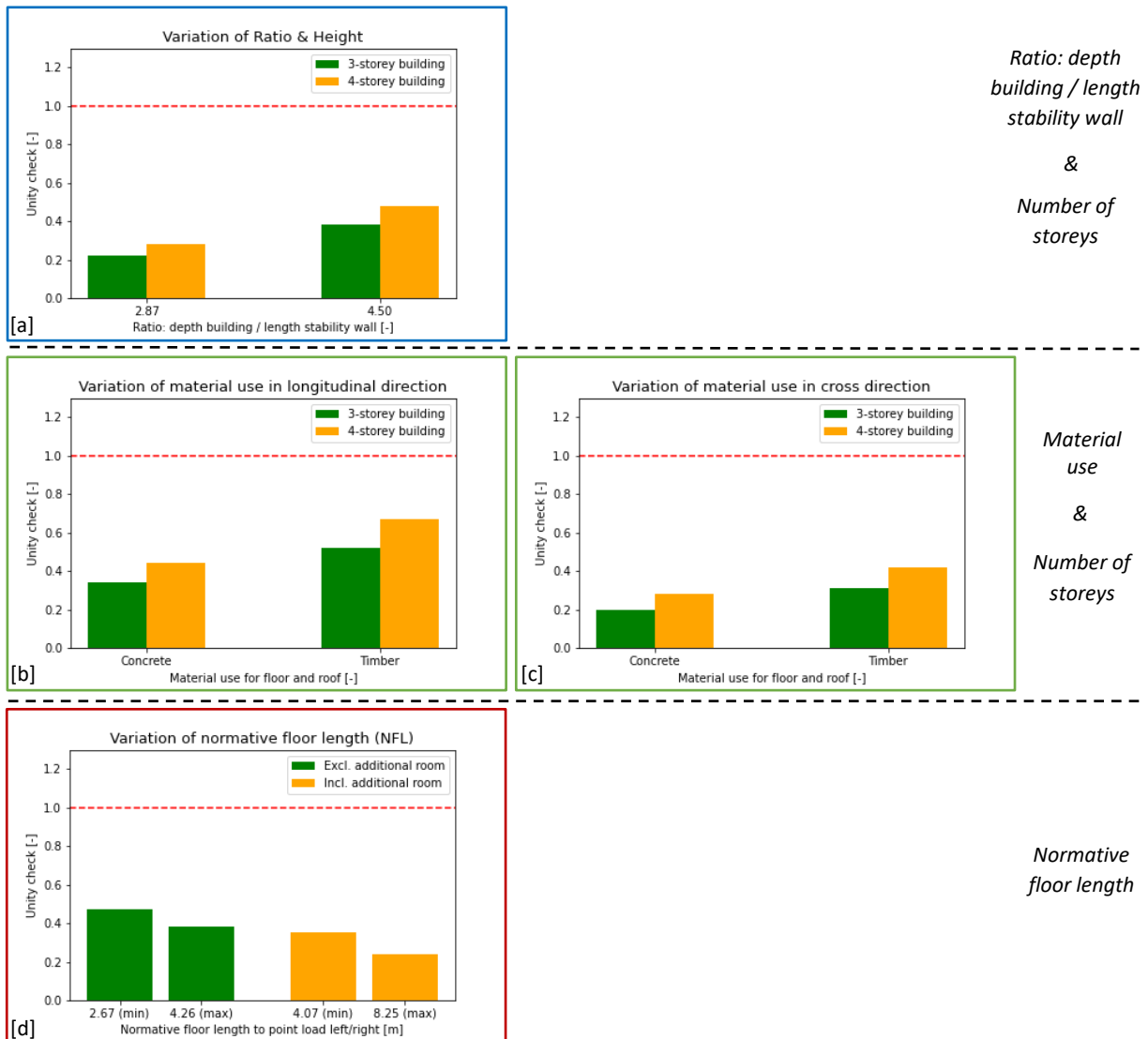


Figure 5.24: Results parameter study on stability (Self-produced).

5.6.2.2 Discussion

An increase in the ratio between building depth and stability wall length has a negative impact on the stability. The effect is smaller for 3-storey (UC*+0.16) compared to 4-storey buildings (UC+0.20) [Fig. 5.24a]. Timber structural floors and roof has a negative impact on the stability compared to concrete slabs. Cross-direction is not normative for stability [Fig. 5.24b, c]. 3- and 4-storey, concrete-slab buildings show potential for at least one layer top-up [Fig. 5.24a, b, c]. For timber buildings it depends on the configuration. A larger normative floor length has positive effects on the stability [Fig. 5.24d]. Presence of an additional room (UC=0.24-0.35) is positive compared to the ones without (UC=0.38-0.47) [Fig. 5.24d].

*UC = Unity Check

5.6.3 Comparison of parameters

Table 5.9 provides an overview of the studied parameters. The structural impact is defined as negative [orange] or positive [green]. A positive structural impact indicates more top-up potential. Note that the parameters for vertical load hold only in case the structural walls have less overcapacity compared to the foundation.

Table 5.9: Comparison building parameters for both vertical load transfer and stability.

Building parameter	Vertical load transfer	Stability
Larger number of floors	Negative	Negative
Larger weight of floor system (positive: timber/ negative: concrete)	Negative	Positive
Larger floor span*	Negative	Positive
Presence of an additional room	No influence	Positive
Larger wall opening percentage	Negative	No influence
Larger ratio depth building to length stability wall	No influence	Negative

*Represents both CFL and NFL

A number of aspects can be concluded, based on table 5.9 (in case structure above ground is normative).

- 4-storey buildings have less top-up potential compared to 3-storey buildings;
- The number of wall openings should be as low as possible, to provide more top-up potential;
- The ratio: 'depth building / length stability wall' should be as low as possible. A larger ratio has negative impact for topping up;
- Vertical weight has an opposed effect on both stability and vertical load. A high vertical weight is beneficial for stability, but negative for the vertical load capacity. Both parameters that are affected are *materialization of floors* and the *floor span*.

Below the ranges of the distinct parameters is summarized.

Min. value	Sign	Parameter	Sign	Max. value
3 [-]	≤	Number of floors	≤	4 [-]
Timber (0.5 kN/m ²)	≤	Floor system	≤	Concrete (4.7 kN/m ²)
6.70 m	≤	CFL (Connected floor length)	≤	10.28 m
4.07 m	≤	NFL (Including add. room)	≤	8.25 m
2.67 m	≤	NFL (excluding add. room)	≤	4.26 m
15.0%	≤	Wall opening percentage	≤	31.0%
2.87 [-]	≤	Ratio: depth building to length stability walls	≤	4.50 [-]

Based on the effects of the parameters, the most (un)favourable variants for both stability and vertical load transfer can be defined. These variants are provided in appendix A.4.

5.7 Functional and technical lifespan of the council estate

This section will provide information about the functional and the technical lifespan of the council estates.

5.7.1 Functional lifespan

The functional lifespan is defined as the time a building fulfil its supposed function. Post-war buildings were constructed in the period 1946 till 1965. Singelenberg and Van Vlaenderen (2011) concluded that technical these buildings are strongly deprecated. Battum (2002) adds that the type of residents is changed due to demographic changes. Currently, the council estates are inhabited by immigrants with low income. Boredom results in vandalism and criminality. Considering renovation of the existing building (and neighbourhood), the **total functional lifespan** can be assumed around **100 years** (Ghattas et al., 2013).

5.7.2 Technical lifespan

The technical lifespan is defined as the time a building is technically capable to function as a loadbearing element. Considering the technical lifespan, a distinction is made between the traditionally and non-traditionally constructed council estates.

Firstly, the traditional council estates are considered. Bown (2007) performed research in the performance of brickwork. Bown concludes that the technical lifespan of a **brickwork building** equals **500 years**. Additionally, under the right conditions these buildings can reach 650 years. Note that a building can contain elements of reinforced concrete, what reduces the technical lifespan strongly. Most of the brickwork buildings are demolished before they reached their technical lifespan. Hopkinson et al. (2019) addressed that most historical brickwork buildings were constructed out of lime-based mortar. The bond strength of this mortar reduces over time, meaning that these materials easily can be separated. That allow for reuse of this brickwork (Hopkinson et al., 2019).

Secondly, **non-traditional systems** are considered. Reinforced (precast) concrete has a technical lifespan of about **100 years** (Kerkhoven, 2022-a; Keulemans, 2016). Corrosion of reinforcement causes the reduced lifespan. More often concrete is crushed for recycling. Considering the concrete-related materials (e.g., Muwi and Airey), no research is available about the technical lifespan of these systems. It is assumed that the technical lifespan is similar to concrete.

5.7.3 Evaluation

The functional lifespan of the current council estates is nearly at the end. Assuming a functional **residual lifespan ranging between 23 and 42 years**. The technical lifespan is larger or equal to the functional lifespan. A new top-up will have a much larger technical lifespan (75 years for timber). The discontinuity between existing functional and new technical lifespan urges for modular construction. In case the existing flat is no longer in proper conditions, the individual units can be placed on other buildings.

5.8 Conclusions on the council estate

A number of council estates were inspected by a literature review and archive study. Consequently, a number of calculations and parameter study were performed. This section will conclude and summarize the findings of the research on council estates.

Variation in construction method

The literature study defined a wide range in typology of council estates. The division is made on construction principles: traditional mixed and non-traditional construction. Traditional construction can be subdivided in wooden or concrete floors. Thijssen and Meier (1988) divided the non-traditional construction in precast stack building, cast in-situ and large panel precast construction. The exploratory calculations addressed that the over-capacity in vertical load of non-traditional construction is much larger compared to traditional construction. Uncertainty in brickwork quality is a dominating parameter.

Standardization but still deviation

In 1946 respectively 1951, the Prescriptions and Wenken were defined. Due to the national housing planning, the local deviations in council estates disappeared. Standardization minimized volumes and dimensions. Unfortunately, this is not interchangeable with a standardized structural plan.

Council estates contain a central orientated stairwell. Cross-directional stability is realized by the stabilizing stairwell-wall. In longitudinal direction, the structural walls provide stiffness. Main standardization can be defined in variants with(out) an additional room. Still a large deviation is observed in bay-width, building depth, height and width, number of floors, stairwell dimensions and materialization.

Opposing structural mechanisms

The parameter study defined six normative parameters in the load transfer: number of floors, flooring system, floor span, presence of an additional room, wall opening percentage and the ratio of the building depth over the wall length. In case the walls above the ground are normative compared to the foundation, it can be concluded that 3-storey council estates have the highest top-up potential. A large vertical weight is preferred for stability. Contrarily, for the vertical capacity, a low weight is preferred. That makes stability and vertical capacity opposing structural mechanisms.

Modular top-up design

The residual lifespan of council estates varies between 23 and 42 years. A new timber top-up has a much larger technical lifespan (75 years). The deviation between functional and technical lifespan urges for a modular top-up.

6

Top-up

Chapter 6 will provide information on the possibilities of top-ups. Based on literature review, case studies, and interviews, a theoretical foundation for top-ups will be formed. The chapter will conclude with two flowcharts, providing the structural overcapacity of council estates.

6.1 Considerations around top-ups

A top-up is a vertical extension of an existing building by at least one floor. This principle is already applied since 1970 (Cobouw, 2000). Topping up contains both reuse and extension of an existing building.

6.1.1 Potential of reuse

Currently the building sector is responsible for **40 percent of the total energy consumption** in the European Union. This is comparable to that of the transportation sector and more than twice of the industrial sector (Lindberg et al., 2004). Demolition of council estates is presently a dominated strategy to generate new space for houses. Assefa and Ambler (2017) developed distinct reuse scenarios varying from combined demolition and new construction to selective reconstruction [Fig. 6.1]. The horizontal axis indicates seven impact categories. The vertical axis expresses the environmental impact

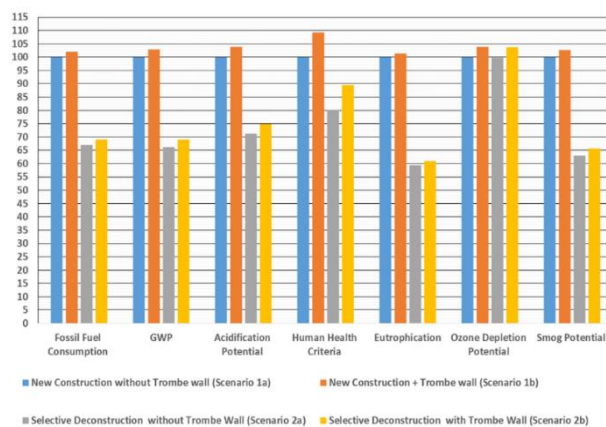


Figure 6.1: Environmental impact of new construction compared to reuse* (Assefa & Ambler, 2017: 151).

*A Trombe wall is an important green architectural feature that aides the ventilation, heating, and cooling of buildings. (Saadatian et al., 2012)



Figure 6.2: Top-up project on council estates in Germany (NOS, 2022).

as percentage of scenario 1a. In six out of the seven categories, the environmental impact of reuse is lower compared to new construction. The research estimates a saving of 3,300 ton CO₂-equivalent per repurposed building. Considering the four cities with council estates [Section 5.3], a saving of 165 megatons of CO₂-equivalent could be obtained. The **reduction of CO₂-emissions** is a dominating factor in the argumentation for reuse. Besides that, other aspects could be mentioned as: keep the vernacular architecture up, reactivate the social fabric and preserve rural landscapes (Martín et al., 2010).

6.1.2 Consequences of a vertical extension

The densification in the Netherlands is addressed in the preliminary research [Section 3.1.3]. The expansion of cities happens in an unsustainable way. Nilsson et al. (2014) performed research in the negative effects of such expansions. This **PLURAL project** defined four main categories:

- Consumption of land, what results in a loss of productive agricultural land;
- Destruction of biotopes and habitats with fragmentation of landscape structure and decline of ecosystem services;
- Less open spaces and longer distances to recreational areas;
- Increased dependency on private car use, traffic congestion, longer commuting times and distances, climate change emissions, noise, and air pollution.

The Brundtland commission defined a **sustainable development** as: *“a development that meets the needs of the present without compromising the ability of future generations to meet their own needs”* (World Commission on Environment and Development, 1987). Pelczynski and Tomkowicz (2019) provides densification of cities as a method that underlines the sustainable development: more space for natural assets and reduction of raw material use. A top-up is a proposed method, for three reasons:

1. Preservation of free terrain, while creating usable space (Pelczynski and Tomkowicz, 2019; Dind et al., 2018).
2. Use of already existing infrastructure (Pelczynski and Tomkowicz, 2019). Presence of infrastructure formed a beneficial point, in the vertical extensions of German council estates (NOS, 2022) [Fig. 6.2]. An extension keeps the travel distances. Note that traffic congestion and commuting time could become a negative effect.
3. Reduction of both construction time and costs. More negative are the large renovation costs, which slows down the process. Use of the profit from the sold dwellings could be a viable solution (Aparicio-Gonzalez et al., 2020).

Two aspects must be considered. First, a vertical extension **increases shading for neighbouring areas** and buildings. This effect is neglectable till 2-storey extensions. Second, the **inconvenience for the co-users** forms one of the most complicated aspects of the process (Pelczynski and Tomkowicz, 2019). The German top-upped flats have been renovated to create added value for the co-uses (NOS, 2022).

6.1.3 Motivation

The needs of the current and future generation are considered by application of a top-up on a council estate. The current generation is provided with more liveable space. The PLURAL project takes into account the future generations. Preservation of agricultural land, biotopes, habitats, landscape, travel distances, and open spaces, while the CO₂ emissions are lower compared to new construction. A vertical extension forms a sustainable solution for the housing demand in the Netherlands.

6.2 Case studies

Section 6.2 will provide a summary of four case studies. The studies will be conducted to enhance the author's structural knowledge of vertical extensions. Interviews with involved structural engineers will be provided in appendix B.1. Detailed information of the distinct cases will be provided in appendix B.2.

6.2.1 Motivation for cases

The archive of Van Rossum B.V. is used to have access to extensive information, while being able to contact the involved structural engineer. Unfortunately, the archive of Van Rossum B.V. does not contain a top-upped council estate. Four projects are studied, provided in table 6.1.

Table 6.1: Studied cases, including motivation.

Project	Structural engineer	Motivational aspects
Parelhof, Heerhugowaard [Fig. 6.3a]	M. Freriks	Timber extension; Known engineer; Archive Van Rossum B.V.
De Boelelaan, Amsterdam [Fig. 6.3b]	M. Tromp	Construction period (gallery flat); Known structural engineer; Archive Van Rossum B.V.; Timber extension.
Duinluststraat, Amsterdam [Fig. 6.3c]	Unknown	Council estate; Modular, timber extension.
Wibautstraat, Amsterdam [Fig. 6.3d]	M. Tromp	Construction period (gallery flat); Known structural engineer; Archive Van Rossum B.V.



Figure 6.3: Case studies; [a] Parelhof (Breddels architecten, 2022); [b] De Boelelaan (Hans van Heeswijk architecten, 2016); [c] Duinluststraat (Hans van Heeswijk architecten, 2003); [d] Wibautstraat (Penta Architecten, 2016).

6.2.2 Summary of the cases

Section 6.2.2 will provide a summary of the Parelhof, Boelelaan, Duinluststraat and Wibautstraat, respectively. Details will be provided in appendix B.2.

6.2.2.1 Parelhof, Heerhugowaard (project under development)

The existing building consists of an in-situ concrete structure, divided in two parts: A and B [Fig. 6.4]. Part A consists of a cross-directional wall-structure. Walls in two directions provide stiffness. Part B contains a concrete column-structure. The monolithic connection between columns and floors forms stiff portals in cross-direction. Longitudinally, stability walls exist.

A recalculation (NEN 8700), and a reconsideration of the material factor resulted in overcapacity of the existing structure. Part A will be 2-layered extended, using a light-weight timber construction. Part B will be extended by using both steel columns and HSB-walls (HSB = Timber Frame Construction; Dutch: Houtskeletbouw). The columns will be constructed above the existing ones. No strengthening will be applied for the increased vertical load. A new floor will be constructed above the existing roof. Part C (new construction) will strengthen the existing structure for stability. The extension at part A will be stabilized by walls, similar for part B (in cross-direction). Longitudinally, part B will be structurally connected to part C. The dilatation between A and B will be horizontally linked, to increase the longitudinal stiffness.

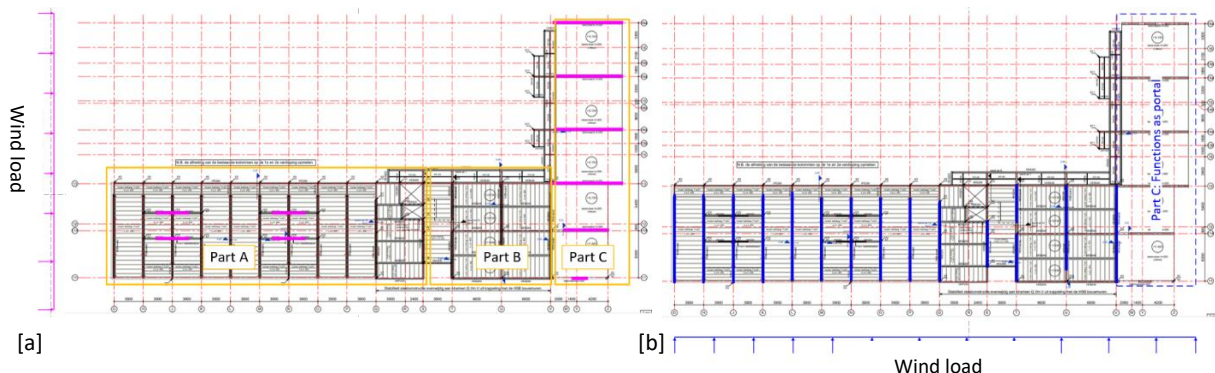


Figure 6.4: Top-up Parelhof. [a] Wind load in longitudinal direction; [b] Wind load in cross direction (Van Rossum B.V.; Internal communication; Adapted by author).

6.2.2.2 De Boelelaan, Amsterdam

The first layers of the existing gallery flat consist of monolithic connected, in-situ columns and floors. The 2nd till 8th floor consists of a cross-orientated, prefabricated walls. These walls provide stiffness in cross-direction. Structural walls between bedrooms provide stiffness in longitudinal direction.

A recalculation (NEN 8700), and a reconsideration of the material factor resulted in overcapacity of the existing structure. The outer parts of the building have been top-upped with a one-storey, lightweight,

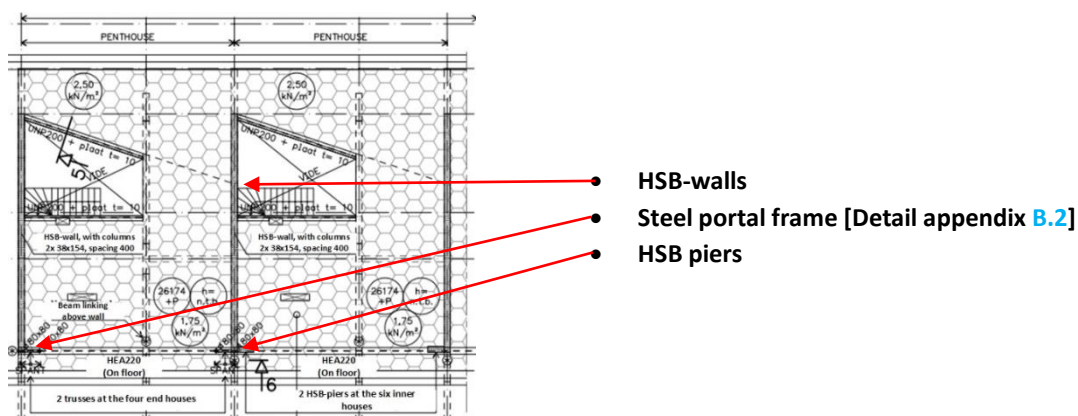


Figure 6.5: Top-up Boelelaan, Amsterdam (Van Rossum B.V.; Internal communication; Translated).

timber structure. Cross-orientated, stiff HSB-walls have been constructed above the existing walls. Longitudinally, HSB piers provide stiffness. The end apartments contain a stiffening steel portal [Fig. 6.5]. The existing roof-slab has been reinforced by a structural, reinforced concrete layer. No strengthening has been applied for the increased horizontal and (other) vertical load.

6.2.2.3 Duinluststraat, Amsterdam

The existing council estate was 4-storey high, containing an additional room. The walls were made of ‘Korrelbeton’ and the floors of ‘Schokbeton’ elements. In cross-direction the stability has been realized by structural walls; in longitudinal direction by the stairwell wall [Fig. 6.6a].

A modular, light-weight timber top-up has been applied [Fig. 6.6b, c]. A steel interlayer has been placed above the structural walls. On top of this interlayer, (varying) units have been placed. No strengthening has been applied for both stability and added vertical load.

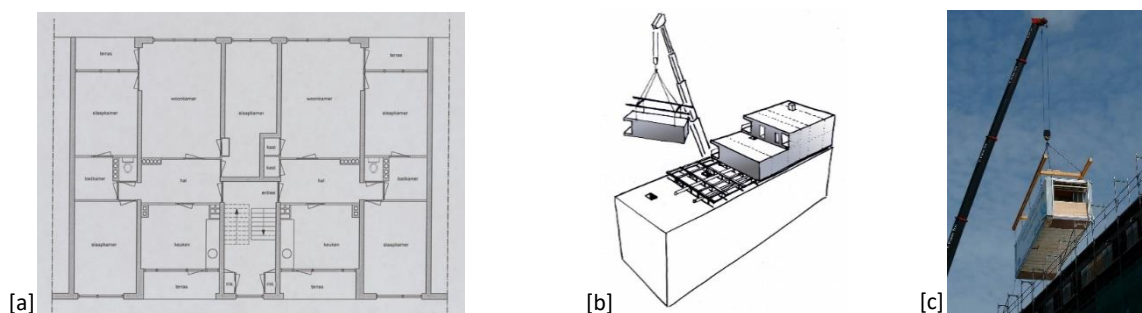


Figure 6.6: Top-up Duinluststraat; [a] Existing flat (Archive Municipality Amsterdam, 2022); [b] Top-up principle; [c] Actual placement (Hans van Heeswijk architecten, 2003).

6.2.2.4 Wibautstraat, Amsterdam

The existing, six-storey high gallery flat was formed out of an in-situ concrete column-beam structure. The building consists of three parts (A, B and C), where part A and C have been extended [Fig. 6.7]. The parts have been separated by a dilatation. The portals form stiffening elements in both wind directions.

A recalculation (NEN 8700), a reconsideration of the material factor and usage, and a demolition of the sixth floor, generated overcapacity to construct three new layers in steel. The new structure consists of a column-beam structure, combined with composite floor slabs. Steel braces provide stiffness to the top-up. Due to horizontal linking of the dilatation, the stiffness increases. The existing structure has been strengthened for stability by staggered placement of steel braces.

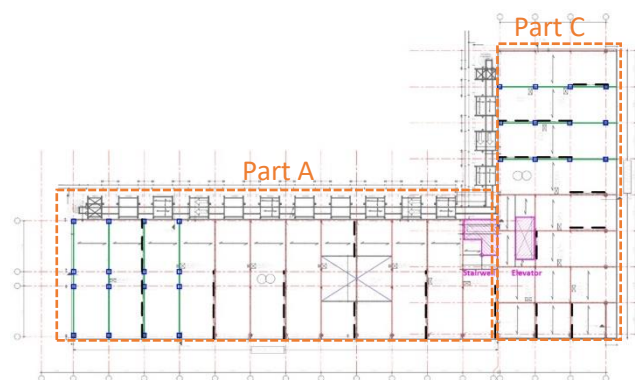


Figure 6.7: Top-up on Wibautstraat. Indication of columns [blue], beams [green] and braces [black] (Van Rossum B.V.; Internal communication; Adapted by author).

6.2.3 Conclusions on case studies

Based on the case studies and interviews, three aspects are concluded.

Minimize vertical weight

The main structural consideration for top-ups is to minimize the added vertical weight. The structural system of the top-up relates to existing structure. The bearing walls continue in the extension. Strengthening for additional vertical loading was occasionally applied. This process is complicated and time consuming, what will increase the costs. The structural engineers addressed that clients then often quit the project. Extending within the structural boundaries is also more sustainable (M. Tromp).

Strengthen for stability

Both the stability of the extension and the existing structure must be (re)considered. An increase in horizontal and relative low increase in vertical loads, often results in stability issues. Strengthening for stability is a commonly applied intervention. Three methods were encountered:

1. Horizontal structural linking of the dilatations (shrinkage and settlements already occurred);
2. Application of braces between column structure;
3. Using a new construction to take up additional horizontal loading.

Recalculate the existing structure

Three methods must be used to calculate the (vertical) overcapacity of the existing structure.

1. Recalculate the existing calculation with reduced safety factors (NEN 8700). The new construction must be calculated according to NEN 1990;
2. Reconsidering historically applied material factors. E.g., a comprehensive safety factor of 1.7 or 1.8 is applied on concrete, while no factor is applied on loading;
3. Reconsider the functions of the building.

6.3 Rooftop extension in timber

The scope of the research will be limited to a **timber** top-up design. This section will provide insight in the possibilities in timber design.

6.3.1 General introduction

Aparicio-Gonzalez et al. (2020) defined three systems for extensions: frame, panel, and modular system [Fig. 6.8]. A **frame system** is composed of unidimensional elements, e.g., pillars. Frames need additional elements for the stability. A **panel system** consists of bi-dimensional elements as plates, stable from themselves. A **modular system** is composed of three-dimensional modules, which are constructed off-side. All systems are applicable in timber (Aparicio-Gonzalez et al., 2020).

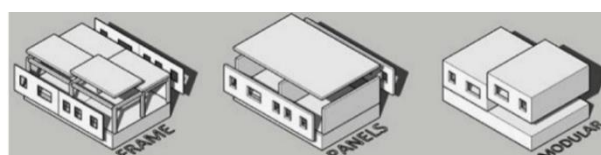
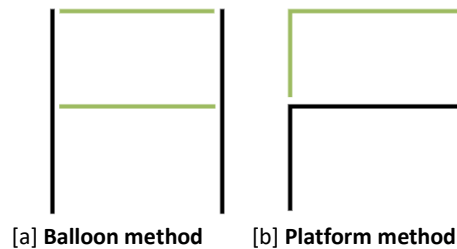


Figure 6.8: Structural systems for timber rooftop extension (Aparicio-Gonzalez et al., 2020).

Švajlenka and Kozlovská (2020) categorized timber structure as: log, column, and panel construction, to investigate on both efficiency and sustainability. Off-side constructed, panel structures are by conclusion the most sustainable variants. Timber construction can be applied in both **balloon and platform method**. Balloon method uses continuous walls from foundation till roof. The floors are connected in between [Fig. 6.9a]. The platform method is a development of the first method, in which the floors are placed on top of the walls below [Fig. 6.9b]. The floor functions as platform for the next layer, what speeds-up the construction process (Van Dessel & Dobbels, 2019).



[a] Balloon method [b] Platform method
Figure 6.9: Timber construction methods (Self-produced).

6.3.2 Comparison HSB and CLT

The case studies favoured HSB (Timber Frame Construction; Dutch: Houtskeletbouw) as structural material for extensions. Literature considers CLT (Cross Laminated Timber) as a competitor. Table 6.2 provides characteristics of both. Standard wall and floor build-ups for both construction methods, as well as structural principles are provided in appendix B.3.

Table 6.2: Comparison HSB and CLT. In case no reference is provided: (Bruggink & Degen, 2022).

	HSB	CLT
Structural	<ul style="list-style-type: none"> ✓ Fast construction due to prefabrication ✓ Adaptability on-site is large • More costs in work, compared to material ✓ Light cranes for hoisting - Limited spans - Finishing with plates • Frame for vertical load; Plates for stiffness (Van Dessel & Dobbels, 2019). 	<ul style="list-style-type: none"> ✓ Fast construction due to prefabrication - Adaptability on-site is small • More costs in material compared to work - Heavy cranes for hoisting ✓ Large spans: more uniform properties in both directions due to lamination (Ashuckian et al., 2019; Brandner et al., 2016.) ✓ Directly finished • Plate for both vertical as horizontal loads (Martinsons, 2016)
Sustainability	<ul style="list-style-type: none"> ✓ Solid wood, fully biobased and compostable - Construction and insulation in same layer ✓ Low wood content, less wood usage • Technical lifespan > 75 years (ArchitectDirect, 2019) 	<ul style="list-style-type: none"> - Glued wood layers (Breneman, 2016): high reusable potential, but not compostable ✓ Construction and insulation in separated layer - High wood content, more wood usage • Technical lifespan > 75 years (Laminated Timber Solutions, 2023)

M. Freriks mentioned that CLT is less applied due to its large self-weight. A weight-comparison is made, based on the Parelhof [Appendix B.3]. A combined unit is defined as a two walls and floors. A CLT unit is approximately 1.7 times as heavy as HSB-construction [Fig. 6.10]. Note that in the design value (additionally finishings; variable loads; safety factors) the difference is much less.

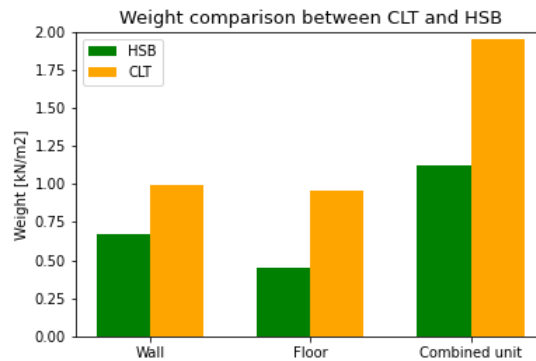


Figure 6.10: Comparison weight CLT-, and HSB-construction. Weights expressed in m² floor area (Self-produced).

6.3.3 Modular construction

An interview with T. Jonker is performed about modular construction [Appendix B.1]. A number of benefits and complications are provided for modular construction [Table 6.3].

Table 6.3: Considerations on modular construction.

Beneficial aspects	Complications
<ul style="list-style-type: none"> ✓ Reduction of CO₂-emissions (Pons & Wadel, 2011) ✓ Strong reduction of landfill waste (Lawson et al., 2012) ✓ Prefabrication allows for application of recycled materials (BC Housing, 2014) ✓ Narrow building sites, higher construction quality, lower construction time (Lawson et al., 2014; Srisangeerthan, 2020) ✓ Reuse of total units (Van Capelleveen, 2022) 	<ul style="list-style-type: none"> - Hoist weight limitation: < 20 tonnes (Liew et al., 2018). - Transportation limitation: L x B x H < 27.5 x 3.5 x 4.15 m (Evofenedex, 2013) - Maximum of eight storeys (Svatoš-Ražnjević et al., 2022; Hamelijnc, 2021) - Lacking guidelines for modular construction (Srisangeerthan, 2020)

Three structural methods exist for structural modules: load-bearing wall, corner-supported and hybrid modules [Fig. 6.11a, b, c]. **Wall modules** transfer all loads through walls and floors. **Corner-supported modules** transfer the vertical load through the floor, edge beams and corner columns. Stability is provided by braces or a (concrete) core (Hamelijnck, 2021; Lacey et al., 2018).

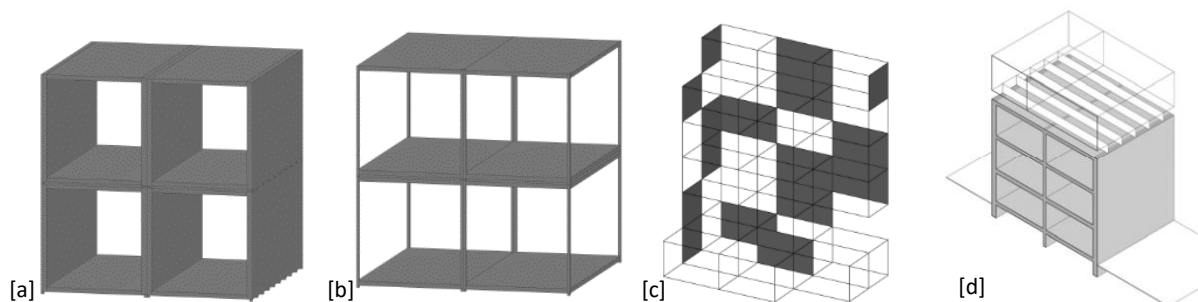


Figure 6.11: Modular construction principles; [a] Load-bearing wall module; [b] Corner supported module; [c] Hybrid module (Hamelijnck, 2021); [d] Structural interlayer (Tulamo et al., 2014)

A **hybrid system** consists of modules which transfer only gravity loading as well as rigid modules containing shear walls (Hamelijnck, 2021). In case a modular extension deviates from the existing structural walls, an **interlayer** must be constructed (Tulamo et al., 2014) [Fig. 6.11d]. Inter-modular connection forms a critical aspect of modular buildings to withstand applied loads. Lacey et al. (2018) grouped three distinct module-connections: inter-module, intra-module, and module-to-foundation. Unfortunately, there is hardly any research in the interconnectivity between modules. Appendix B.4 provides some background information of modular connections.

6.3.4 Fire safety

Rooftop extensions increase the building height. Exceeding the 13-meter boundary tighten the required fire resistance by 30 minutes. Existing (historical) buildings contain acquired rights, which allows to maintain the current fire resistance (60 minutes in most cases). An extension is classified as new construction, resulting in stricter fire regulations. As it is not reasonable to provide the top-up with a higher fire resistance, it is tolerated to maintain the fire resistance of the existing structure (Commission ATGB, 2020; Scholten, 2006). The municipality should provide dispensation according to article 406 of *Bouwbesluit 2003* (Bouwen met Staal, n.d.). The case studies demonstrate the permitted relaxation [Table 6.4]

Table 6.4: Applied fire resistance in the case studies.

Case	Existing structure	Rooftop extension	Highest floor
Parelhof, Heerhugowaard	60 min	60 min	>13 m
De Boelelaan, Amsterdam	60 min	60 min	>13 m
Wibautstraat, Amsterdam	60 min	60 min	>13 m

6.3.5 Strengthening for stability

Increased horizontal loads due to a vertical extension, combined with a light-weight addition potentially causes structural instability in longitudinal direction. Three factors can be defined to increase the horizontal stiffness (Bikulčiūtė, 2021; Papageorgiou, 2016):

1. Ensuring continuity of floors acting as diaphragms in the structure;
2. Increasing the stiffness of vertical elements;
3. Introducing of new stability elements

Factor 1 does not affect traditional reinforced floors, as these are in-situ, continuous floors. For prefabricated floors, a minimum structural screed of 50 mm has to be considered to allow for diaphragm action (De Vree, n.d.-b). Timber floors need a reinforced layer to allow diaphragm action.

Two measures can be considered to increase the stiffness of the stabilizing walls [**Factor 2**]:

- Horizontal linking of the dilations (case studies) [Fig. 6.13];
- Enlarging the wall thickness by placing a (reinforced concrete) wall next to the stabilizing wall. Concrete can be reinforced for the occurring tensile force.

Four measures could be considered, to adjust the existing stability system [**Factor 3**]

- Transition from hinged connections towards a moment-fixed portal frame;
- Construction of new stability wall(s) in reinforced concrete;

- Application of steel bracings (case study, Wibautstraat);
- Construction of a structural elevator (core);

Appendix B.5 explores the structural effects of the distinct measures (6- and 7-storeys). Figure 6.12 proposes four useful methods. Strengthening in cross-direction is not needed. A number of conclusions are made. The environmental impact of the distinct methods will be explored in section 7.2.4.

- The transition from hinged towards monolithic connections is an inefficient method. The occurring tensile force and deflections reduce with a negligible amount. Compared with practical complications, this is considered as an inefficient method.
- Horizontal linking of the dilatations is the most effective method to strengthen in longitudinal direction. It can be less efficient to link all dilatation, as the wind load will be higher. This measure assumes that concrete shrinkage and building settlements already occurred.
- Application of concrete walls and bracings are both useful strengthening methods. A concrete walls is more beneficial due to possibilities of openings. Both methods show an increase in dimensions after six floors.
- A structural core combines both the need for an elevator as well as a higher stiffness. As dimensions are minimized by building regulations, instead of structural necessity, it is an inefficient method (over-dimensioning).
- In nearly all structural interventions [Fig. 6.12a-c], a tensile force must be transferred to the foundation. Tension piles must be constructed below the new stabilizing elements. Appendix B.5.5 provides some background information about the use of tension piles.

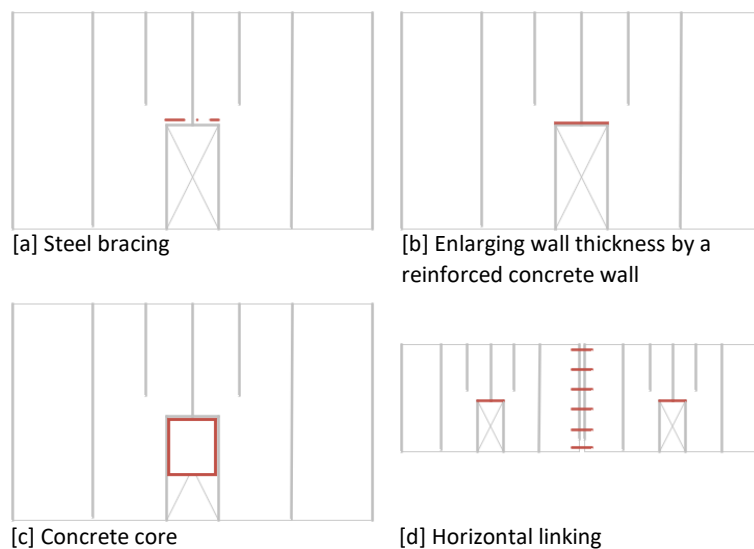


Figure 6.12: Useful strengthening methods for stability (Self-produced).

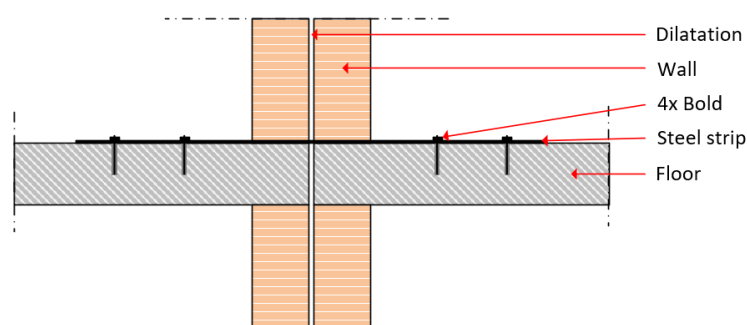


Figure 6.13: Detail horizontal linking (inspired by: Van Rossum B.V.; Internal communication) (Self-produced).

6.3.6 Comparison norms

6.3.6.1 Comparison NEN 1990 and 8700

NEN 1990 defines two normative load combinations [Eq. 6.1; Eq. 6.2].

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \quad (6.1)$$

$$\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_p P + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \quad (6.2)$$

A reduction in safety factors is allowed to recalculate existing buildings. As the council estates can be related to consequence class 2, differences for CC2 will be provided [Table 6.5].

Table 6.5: Comparison load factors of norm NEN 8700 and 1990.

Load combination ULS (CC2)	Existing structure (NEN 8700)	New construction (NEN 1990)
Eq. 6.1_permanent		
• $\gamma_{G,j,sup}$ (unfavourable)	1.20	1.35
• $\gamma_{G,j,inf}$ (favourable)	0.90	0.90
Eq. 6.2_permanent		
• $\xi \gamma_{G,j,sup}$ (unfavourable)	1.15	1.20
• $\gamma_{G,j,inf}$ (favourable)	0.90	0.90
Variable loads		
• Other variable loading normative than wind: $\gamma_{Q,1}$	1.30	1.5
• Wind normative: $\gamma_{Q,1}$	1.40	1.5
• Favourable: $\gamma_{Q,1}$	0	0

6.3.6.2 Historically applied safety

The GBV 1962 formed the code for reinforced concrete. A comprehensive safety factor was applied, which includes uncertainty in material, execution, schematization, and load definition. This factor (γ_{total}) equals 1.7 for concrete (Gijsbers, 2012; Janssen & Staaks, 2001).

The N1055 TGB 1949 and 1955, respectively formed the brickwork building code till 1972. This code used a reduced compression capacity to verify brickwork. Based on the historical design value [N1055] and the characteristic value of the existing code [NPR9096], the comprehensive safety factor is estimated [Table 6.6]. The N1055 has been replaced by the NEN3853, in which a material factor of 2.0 and a load factor of 2.3 were used. The comprehensive safety factor on brickwork can be estimated at $\gamma_{total} = 2.9$ [-].

Table 6.6: Estimation of comprehensive safety factor for brickwork.

Brickwork	N 1055 (Design value) [N/mm ²]	NPR9096 (Characteristic value; M5 mortar) [N/mm ²]	γ_{total} [-]
Rood	1	4.01	4.0
Boerengrauw	1.5	5.22	3.5
Hardgrauw	2	6.29	3.1
Klinker	2.5	7.27	2.9

The current code applies safety factors on loads [Table 6.5] and material factors on characteristic values of materials: 1.15 (reinforcement concrete) and 1.7 (brickwork). The historically applied load

factor (γ_{load}) is calculated based on equation 6.3. This value equals 1.48 [-] for concrete and 1.71 [-] for brickwork structures.

$$\gamma_{load} = \frac{\gamma_{total}}{\gamma_m} \quad (6.3)$$

Appendix B.6 provides an example calculation for a brickwork council estate. A load-factor of 1.48 [-] forms a better indicator for lower brickwork qualities. Historically a γ_{load} of at least 1.4 (dependent on pile type) was applied on a piled foundation (Van Rossum B.V.; Internal communication). As this is lower compared to the brickwork and concrete load factor, the foundation is normative for the vertical top-up capacity.

6.3.6.3 Variable loading

The variable load definition of the N1055 differs from the NEN1990 [Table 6.7].

Table 6.7: Differences in variable load definition.

Type of loading	N 1055 [kg/m ²]	NEN 1990 [kN/m ²]	Difference [%]
Attic floor	200	1.75	-12.5
Residential floor	200	1.75	-12.5
Storage/ ground floor	300	2.00	-33.3
Stairs cases	200	2.00	0.0

Also, the application method of safety on variable load differs from the current code [Table 6.8].

Table 6.8: Application of safety factors on variable loading.

Floor level from top	N 1055	NEN 1990
Roof	100%	Instantaneous values
Top floor	100%	Full loaded*
1	90%	Full loaded
2	80%	Instantaneous values
3	70%	"
4	60%	"
5	50%	"
All lower floors	40%	"

*2 floors full loaded

6.3.6.4 Changed vertical load parameters

The effect of vertical load parameters [Section 5.6.1] change as the foundation is normative. A parameter study has been performed in appendix B.7. Summarizing results are provided below:

1. **Number of storeys:** 4-storey buildings have more overcapacity compared to a 3-storey buildings;
2. **Opening percentage:** No influence on overcapacity;
3. **Weight floor system:** Timber floors result in less overcapacity compared to concrete slabs;
4. **Connected floor length (CFL):** Not changed. Larger CFL has negative effect on overcapacity;
5. **Material structural walls:** Negligible differences.

6.3.6.5 Global benefits from norm comparison

The transition of norms results in a global overcapacity of $\approx 21.0\%$ (compared to NEN8700 loads) [Eq. 6.4].

$$\text{Global benefit [\%]} = \left| \left(\frac{q_{ed, \text{new norm}} - q_{ed, \text{historical norm}}}{q_{ed, \text{historical norm}}} \right) \right| \cdot 100 \text{ [\%]} \quad (6.4)$$

6.4 Top-up flowchart

Two flowcharts are developed to determine the top-up potential of a council estate. Start 1 [Fig. 6.14] provides the maximum addable layers, based on the vertical load (VL) ($\gamma_{load}=1.4$). Start 2 [Fig. 6.15] has to be used as a verification for the stability (S) to reach the found addable layers VL. Appendix B.8 contains an estimation of top-up potential for the distinct flats from the archive inspection. Appendix X.4 and X.5 provide the excel sheets to produce the flowcharts.

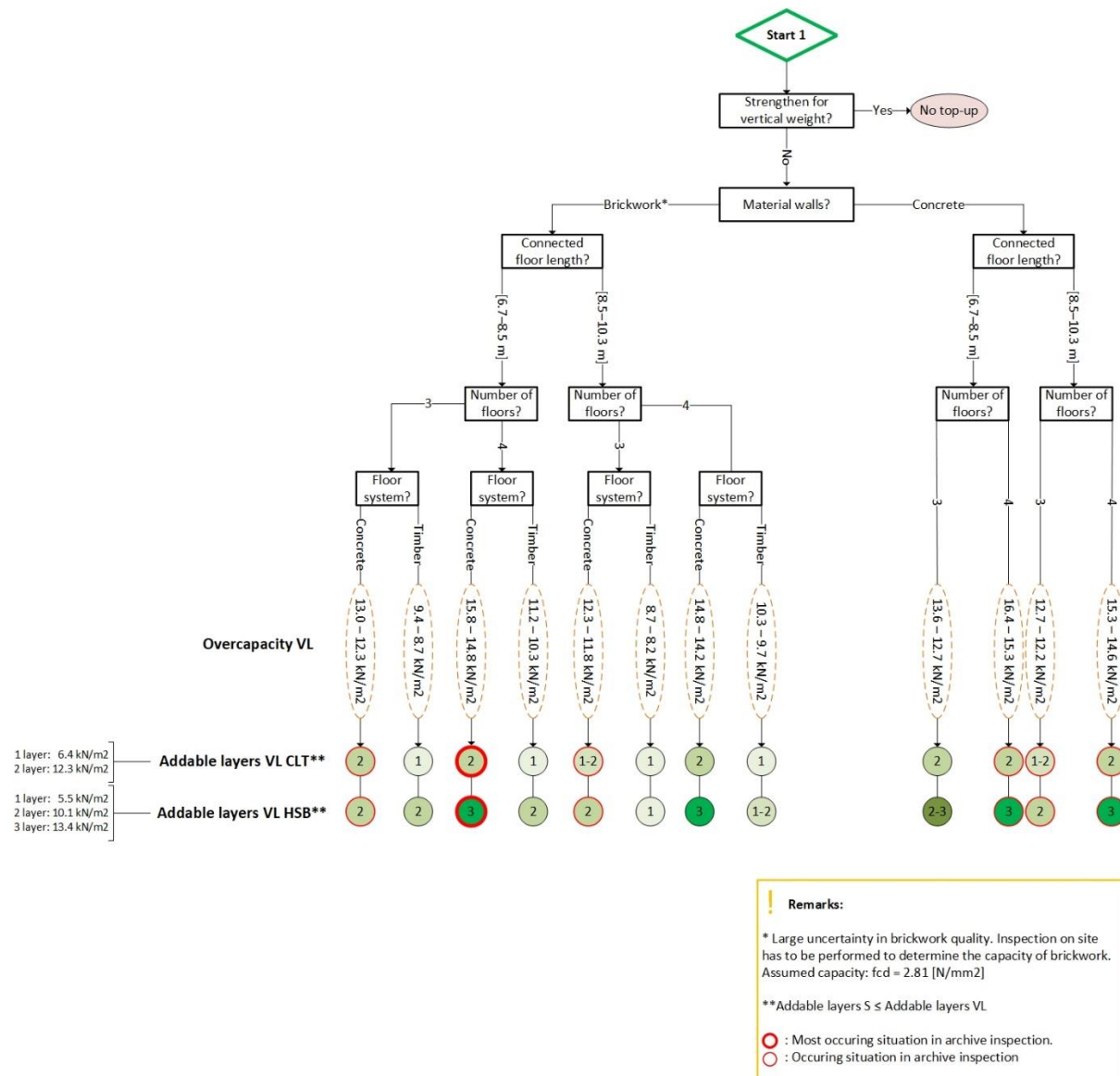
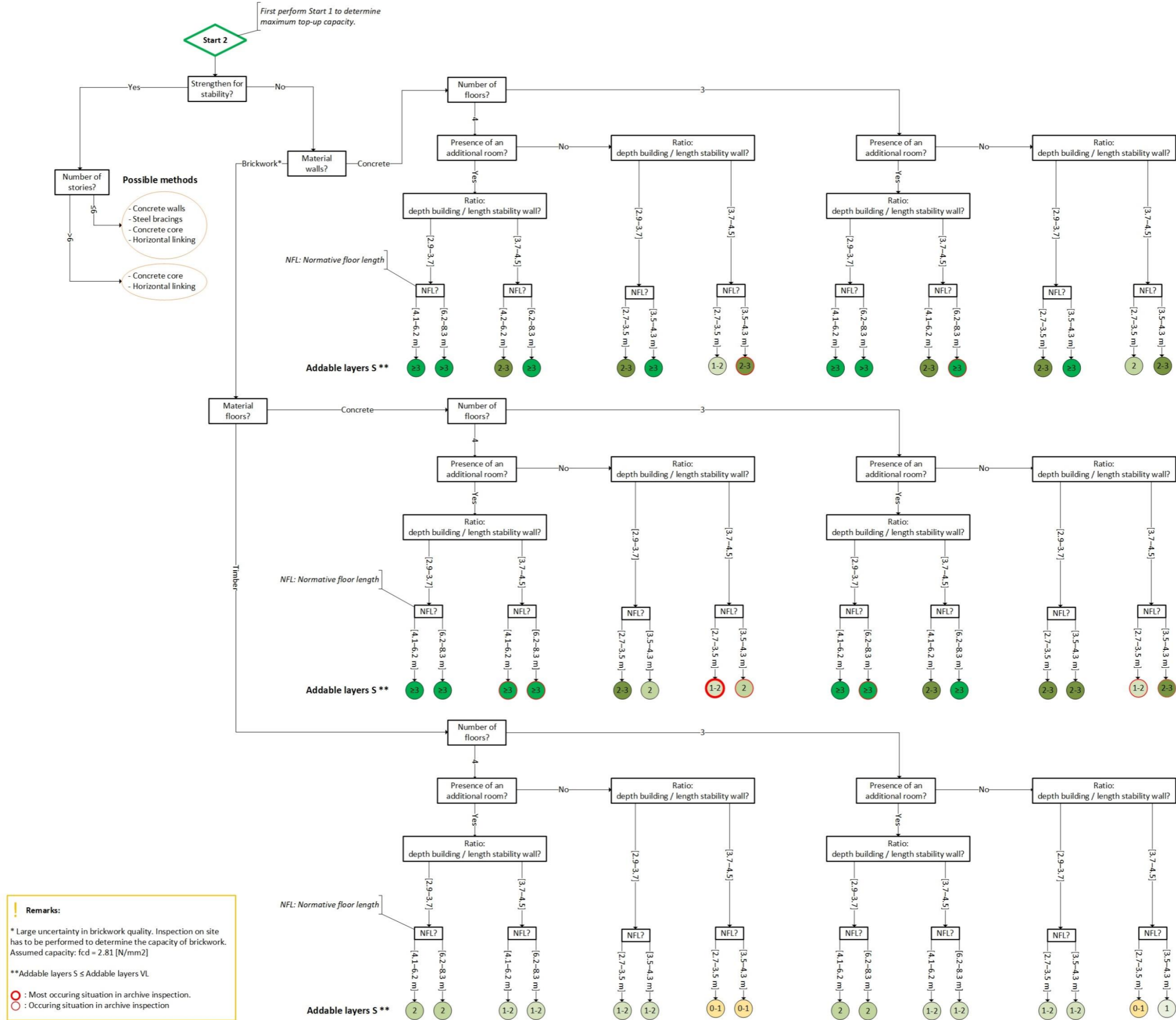


Figure 6.14: Flowchart VL for vertical extension on council estate, considering vertical load transfer (VL) (Self-produced).



Remarks:

- * Large uncertainty in brickwork quality. Inspection on site has to be performed to determine the capacity of brickwork. Assumed capacity: $f_{cd} = 2.81 \text{ [N/mm}^2\text{]}$
- **Addable layers $S \leq$ Addable layers VL
- : Most occurring situation in archive inspection.
- : Occuring situation in archive inspection

Figure 6.15: Flowchart S for vertical extension on council estate, considering stability (S) (Self-produced)

7

Environmental impact

Chapter 7 will generate a theoretical background of environmental aspects related to rooftop extensions. It will provide the general LCA procedure, the used environmental data, preliminary LCA calculations, and the impact of a concrete building.

7.1 Standard procedure LCA

7.1.1 General aspects

The procedure for a LCA (Life Cycle assessment) calculation is formally defined in the European standards ISO 14040 and ISO 14044. These regulations are more specified in the NEN-EN 15804(+A2) (Sustainability of construction works - Environmental product declarations). Five specific life cycle stages are available for a LCA calculation [Fig. 7.1].

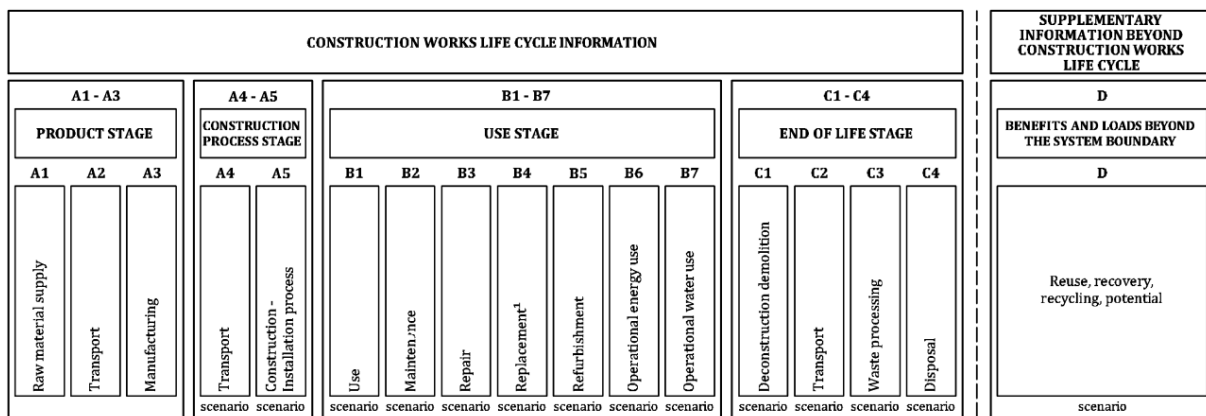


Figure 7.1: Life cycle stages considered in LCA according to standard NEN-EN 15804:2019: **Production stage (half)products** (A1-A3); **Construction stage** (A4-A5); **Use stage** (B1-B7); **End-of-life stage** (C1-C4) and **Re-use and recycle options beyond the building life cycle** (D).

Four steps have to be performed in an LCA calculation, provided below (Jonkers & Ottele, 2022). A more elaborate explanation of the distinct steps is provided in appendix C.1.

1. Definition of goal and scope (including functional unit);
2. Making a Life Cycle Inventory (LCI) analysis;
3. Performing a Life Cycle Impact Assessment (LCIA);
4. Life cycle interpretation.

7.1.2 Changed norm

Till January 2023, 11 impact categories had to be included in a Dutch LCA calculation [Step 3]. These categories contained their own Unit Equivalent (UE) and shadow costs. **Shadow costs**, represent the costs required to bring the environmental impact of a product or process to an acceptable 'sustainable' level (Jonkers & Ottele, 2022). An updated European standard is published: EN15804+A2. This norm increased the compulsory impact categories to 19. Unfortunately, the Unit Equivalents are mostly changed. Due to the transition period of norms, the Dutch government has not yet linked the new UE's to shadow costs. The Global Warming Potential Total (GWP) has an **unchanged UE**, which will be used as indicator for the environmental impact [**Carbon-footprint method**] (Jonkers & Ottele, 2022; Pandey et al., 2011). Appendix C.2 provides an overview of both old and new impact categories.

7.1.3 The environmental impact

The environmental impact will be calculated according to equation 7.1.

$$\text{Environmental impact [kg CO}_2 \text{ eq./y/m}^2 \text{ new floor area]} = \frac{V \cdot UE}{RSL \cdot A} \quad (7.1)$$

- V** = Amount of structural material [m^x]
UE = Unit Equivalent [kg CO₂ eq. /m^x]
RSL = Reference Service Life [years]
A = New realized floor area [m²]

7.2 Exploration

7.2.1 Data generation

The international Environmental Product Declaration (EPD) system is used to generate environmental data from suppliers [Appendix C.3] (EPD International, 2023). NEN-EN 15804+A2 is used as directing code; the NEN-EN 15804+A1 in case no new data is available. A linear relationship between emission and functional unit is assumed in case no other data is available. Data is collected for **stage A1 till A3**. Costs and benefits from stage C and D are typically added when the end-use of a building is certainly known (Jonkers, Internal communication). Table 7.1 provides the environmental data that is used.

Table 7.1: Environmental database (EPD International, 2023).

	Product	Functional unit	Stage A1-A3	Norm NEN-EN
Structural	Sawn wood	kg CO ₂ -eq./m ³ wood	-1.46E+03	15804+A2
	Plywood	kg CO ₂ -eq./m ³ wood	-7.34E+02	15804+A2
	CLT	kg CO ₂ -eq./m ³ wood	-7.14E+02	15804:A1
	Reinforcement FeB500	kg CO ₂ -eq./ 1000 kg steel	1.12E+03	15804+A2
	Reinforced concrete - C30/37; XC1 - 3 - C30/37; XC4, XF1 - C45/55; XC1 - C50/60; XC1	kg CO ₂ -eq./m ³ mixed concrete	2.10E+02 2.17E+02 2.50E+02 2.60E+02	*
	Steel: circular section	kg CO ₂ -eq./ 1000 kg steel	2.98E+02	15804+A2
	Steel: all other profiles	kg CO ₂ -eq./ 1000 kg steel	7.19E+02	15804+A2
Non-structural	Mineral wool	kg CO ₂ -eq./ m ² insulation/ 100 mm	1.21E+00	15804:A1
	Fermacell	kg CO ₂ -eq./ m ² fermacell/ 22 mm	2.01E+00	15804:A1
	Gypsum plasterboard (Fire-resistant; Type F)	kg CO ₂ -eq./ m ² plasterboard/18 mm	3.11E+00	15804+A2

*See appendix C.4 for calculation

7.2.2 Comparison HSB and CLT

Two construction methods [Section 6.3] are environmentally compared: HSB and CLT. Parelhof Heerhugowaard is taken as reference for the HSB-construction. The HSB walls are replaced by 120 mm thick CLT wall and floor panels to transfer the structure into CLT [Appendix B.3]. Appendix X.7 provides the LCA calculation.

The **goal** is to provide the author preliminary data about the environmental impact of both construction methods, to make an educated choice for top-up variants.

The **functional unit** is the annual carbon footprint of the distinct construction methods per square meter new floor area, limited to the product stage: A1-A3. Non-structurally, only plasterboard, insulation, and fermacell floor finishing are be considered. GWP total will be solely taken as impact category. The RSL range from 23 till 42 years, to address the residual life of council estates.

Figure 7.2 provides the results of the LCA. The carbon footprint of wood-based structural materials is negative (CO₂-capture), while for non-structural materials it is positive (emission). The footprint of CLT (-57.85 till -105.65 *10⁻¹) is more negative compared to HSB construction (-31.47 till -57.47 *10⁻³). The non-structural impact is approximately 4.5% (CLT) till 8.9% (HSB) of the structural footprint.

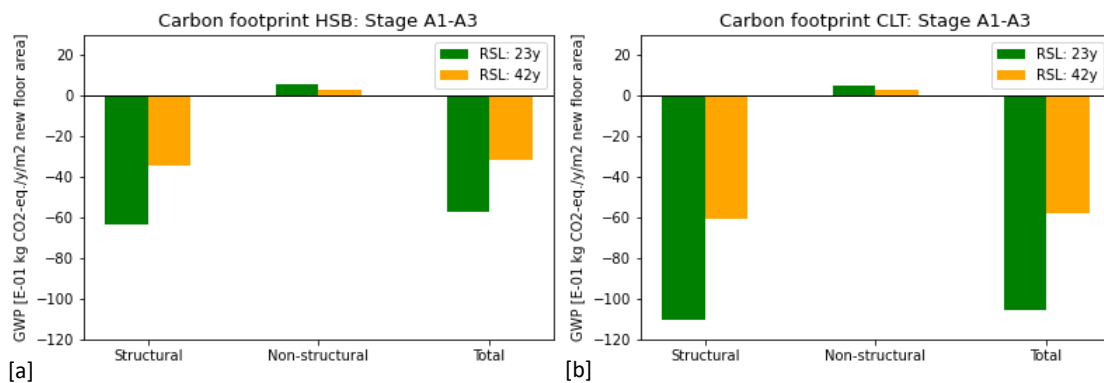


Figure 7.2: Carbon footprint of [a] HSB-construction, and [b] CLT-construction (Self-produced).

The two provided graphs [Fig. 7.2] could develop two misconceptions:

- More material use within a timber structure is beneficial for the environmental impact**

As wood has a CO₂-capture, the GWP is negative. Resultingly, it seems that more wood usage is beneficial. It must be considered that a more logging results in negative effects e.g., future biomass production, nutrients, habitat structure, hydrology and more (Ranius et al., 2018).

- Timber structures should be built with a low RSL**

In case the UE is negative [Fig. 7.2: Structural], the environmental impact gets more negative if the RSL lowers (Eq. 7.1). First, only stage A1 till A3 were considered. In the end-of-life, the captured CO₂ will emit, what can result in an overall positive UE. Second, a short RSL results in a higher construction repetition, which will increase the material use and costs.

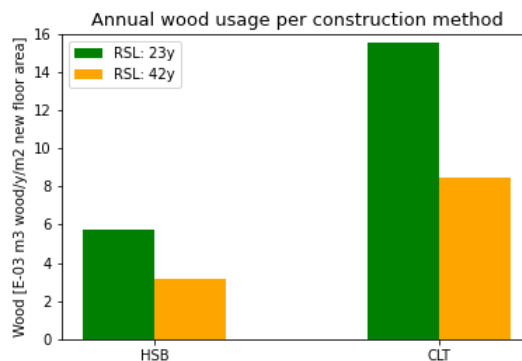


Figure 7.3: Wood usage per construction method (Self-produced).

Figure 7.3 indicates the annual material use for both HSB and CLT construction. It can be concluded that material usage for CLT is much higher compared to HSB. The annual wood use increases with a lower RSL. Based on figure 7.2 and 7.3, the author concludes three aspects:

- Both CLT as HSB are sustainable construction methods, as the carbon footprint is negative. That makes these methods a ‘carbon-sink’ (Younis & Dodoo, 2022);
- HSB is more sustainable compared to CLT, as the material use per year is lower (in this case);
- A longer service life is preferred to reduce material usage per year.

7.2.3 End-of-life scenarios

Four distinct end-scenarios exist for timber (EPD International, 2023; Younis & Dodoo, 2022):

1. **Reuse** – The wood is assumed to be removed from a building and (partly) reused with no further processing. Full reuse is considered as modular construction;
2. **Recycling** – The wood shredded and effectively downcycled into wood chips;
3. **Incineration with(out) energy recovery** – The timber construction material is combusted. The recovered thermal energy replaces the energy from natural gas (in case of energy recovery);
4. **Landfill** – It is assumed that the wood is disposed; emissions from landfill will occur.

Younis & Dodoo (2022) provide GWP-data for CLT construction, including the distinct end-of-life scenarios. This data is restructured to a net-UE GWP as percentage of the product stage (A1-A3) [Table 7.2]. Limited literature is available for HSB. Values for both reuse and recycling will be lower, due to more interwoven (non)-structural elements (Bruggink & Degen, 2022). Reuse and recycling are the most preferred options at end-of-life [Fig. 7.4]. Reaching end-of-second life, the wood has to be preferably incinerated (with energy recovery).

Table 7.2: Different (estimated) end-of-life scenarios for CLT and HSB (Younis & Dodoo, 2022).

Scenario		CLT net-UE GWP [% of product stage]	HSB net-UE GWP [% of product stage]
1a	Reuse (modular)*	96.6	≤ 96.6
1b	Reuse (Partly)*	92.8	≤ 92.8
2	Recycling*	90.1	≤ 90.1
3a	Incineration (no recovery)	-11.5	≈ -11.5
3b	Incineration (energy recovery)	45.6	23.0 - 85.0**
4	Landfill	4.8	≈ 4.8

* The captured CO₂ will be released at the end of the second life.

** 23.0% (Plywood); 85% (Sawn timber) (EPD International, 2023).

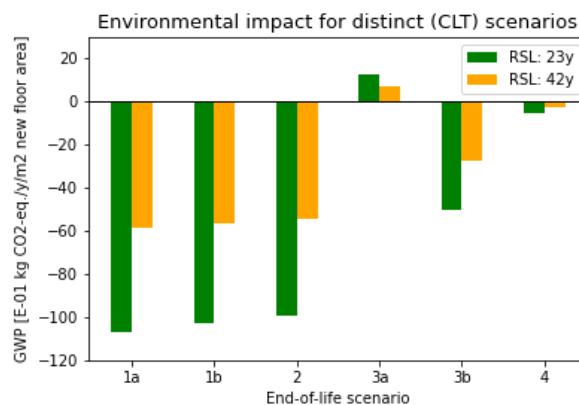


Figure 7.4: End-of-life scenarios for CLT-construction in section 7.2.2 (Self-produced).

7.2.4 Strengthening methods for stability

Distinct strengthening methods [Section 6.3.5] are environmentally compared. Appendix X.6 will be used for the structural design.

The **goal** is to direct the author preliminarily in a sustainable consideration for a strengthening method. The **functional unit** is the annual carbon footprint of the distinct strengthening methods per square meter new floor area, limited to the product stage: A1-A3. Only structural materials are considered. The calculated impact holds for a 4-storey traditional constructed council estate, containing wooden floors, having no additional room [NFL=3.47; Ratio depth to stab. wall length=4.2]. The additional effect of the foundation is approximated, based on estimated pile dimensions [Appendix X.6].

The volumes or weights of both steel and concrete are provided in appendix X.8. The impact of both reinforced concrete and steel is based on table 7.1. Figure 7.5 provides the results. Based on the graphs, six aspects are concluded:

1. All strengthening methods (excluding horizontal linking) have a positive carbon footprint (10.86 till $109.17 \cdot 10^{-2}$ kg CO₂-eq./y/new m²) [Fig. 7.5], smaller than the negative footprint of both HSB- and CLT-construction (-31.47 till $-105.65 \cdot 10^{-1}$ kg CO₂-eq./y/m² new fl. area)[Fig. 7.2].
2. Horizontal linking of buildings is the most sustainable strengthening method. This method has a zero-carbon footprint [Fig. 7.5a]. The method is only applicable for a dilatated building.
3. Thickening the stabilizing wall with concrete has less impact compared to placement of a steel truss [Fig. 7.5a-d]. This difference is nihil (-1.28 till $-2.68 \cdot 10^{-2}$ kg CO₂-eq./y/m² new fl. area).
4. A core is an unsustainable strengthening method for council estates [Fig. 7.5f, g]. It is dimensioned based on minimum dimensions (Bouwbesluit 2012). The core is structurally over-dimensioned, what results in material waste and a large environmental impact.

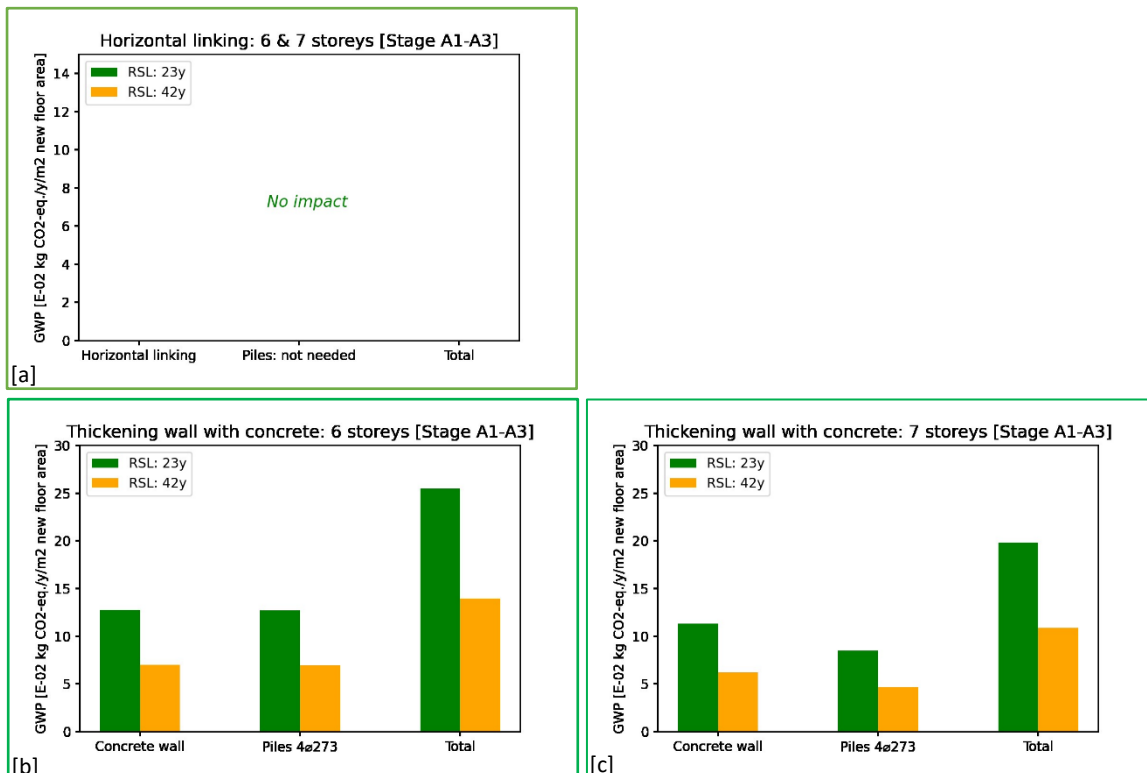


Figure 7.5: Environmental impact of distinct strengthening methods (Self-produced).

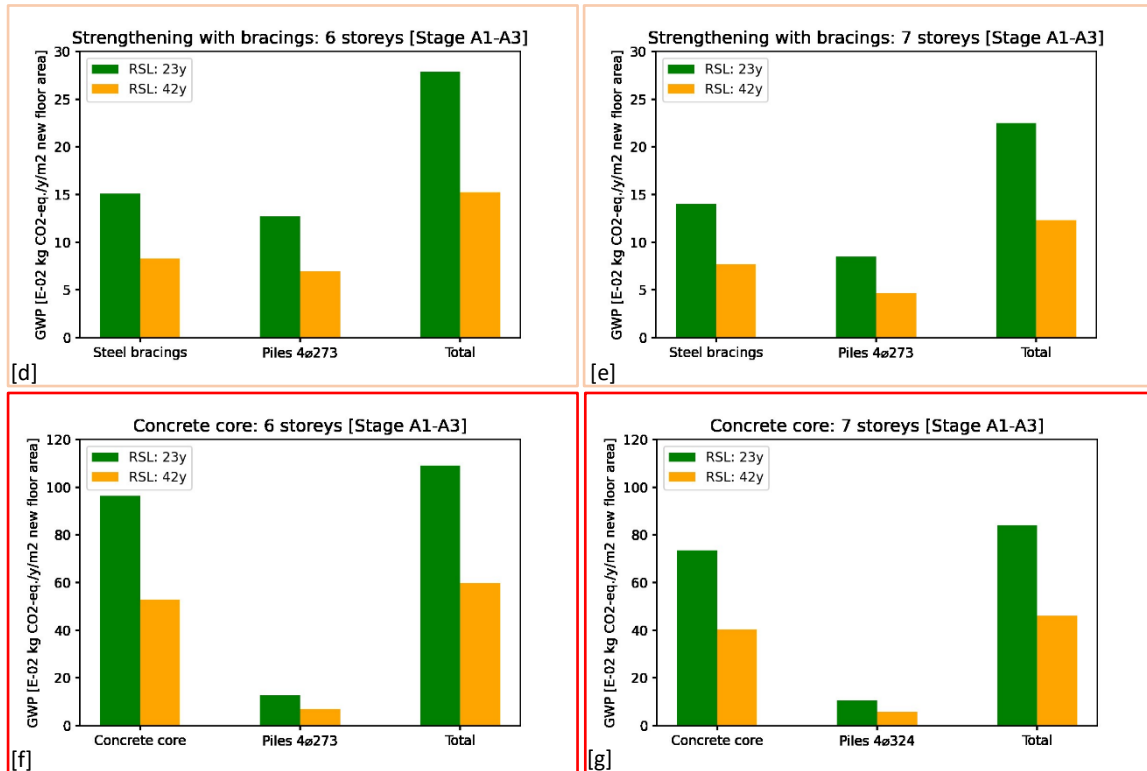


Figure 7.5 (continue): Environmental impact of distinct strengthening methods (Self-produced).

- The impact of a strengthening method reduces, in case more storeys are top-upped [Fig. 7.5b-g]. This due to a faster increase in floor area, compared to the rise in material use. Note that this only hold till the inspected 7 storeys.
- Reinforcing buildings with a larger RSL, will reduce the impact of the applied strengthening method [Fig. 7.5b-g].

The conclusions are based on a specific council estate. Due to the spatial variety in flats, four points are pointed out.

- The zero-impact of horizontal linking is independent of the spatial dimensions of a building;
- The carbon footprint of a concrete core is similar for all variants. The dimensions of the core are minimized by spatial building regulations, instead of structural limitations;
- As the impact of steel bracings and a concrete wall is nearly equal, it is unpredictable what the effect will be for other variants;
- All impacts have been compared based on a similar council estate, considering 2-linked porches. Only structural materials are compared. That makes it a **fair LCA-calculation**.

7.3 New constructed (concrete) building

The impact of new construction (instead of a rooftop extension) is determined based on the Swietenhof in Amsterdam. This is a 6-storey, residential building, designed in concrete (under development). To extrapolate the results to a 4-, 5- and 7-storey concrete building, distinct floors are removed or added. Appendix C.5 provides the details about the building. The calculation is provided in appendix X.9.

The **goal** is to investigate the environmental impact of a new constructed (concrete) building, to address the benefits to vertically extend a council estate.

The **functional unit** is the annual carbon footprint of the new constructed building per square meter new floor area, limited to the product stage: A1-A3. The RSL is set to 75 years. Only structural materials are considered.

Figure 7.6 provides the **results**. The impact of the actual calculated building varies between 31.32 and 46.98 [10^{-1} kg CO₂-eq./y/m² new floor area]. These are positive values, indicating CO₂-emissions. In case less floor area needs to be realized, the environmental impact per square meter increases. Mark that these observations hold in the range 4 till 7 storeys.

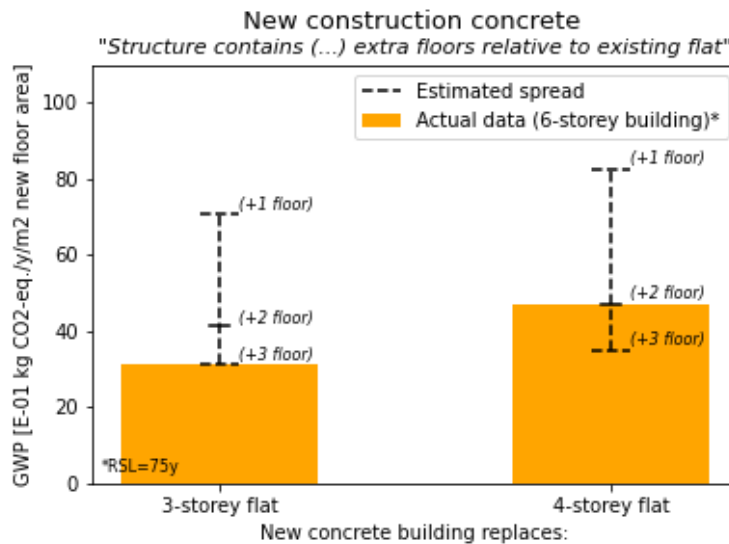


Figure 7.6: Environmental impact of a new concrete building. The graph presents the impact for the actual reference building [Orange]. Additionally, it provides the estimated impact as function of relative added floors (compared to existing flat) [Dotted lines] (Self-produced).

A number of aspects needs to be **discussed**.

- The used building has a slanted roof-structure, what reduces the amount of concrete in the top-layer. Still the building is representative for new-construction, as slanted roofs are not uncommon.
- The reference service life is set to 75 years, as that is representative for residential buildings [Section 5.7]. Note that the provided results for HSB- and CLT-construction are based on a shorter RSL, resulting in a higher 'impact' per year.
- The impact of new added layers is estimated, based on the assumption that the amount of concrete stays similar per floor. This is an allowed assumption, as usually a standardized wall thickness is used [M. Freriks, Van Rossum B.V.; Internal communication].

7.4 Conclusions on the environmental impact

A number of aspects are concluded, based on chapter 7.

- **Changing norms** – Transition from the EN 15804:A1 to the EN 15804+A2 results in a change of impact categories (and units). The GWP-total has to be used as indicator for the environmental impact, as it is a continuing category and unit.
- **Functional unit** – The carbon footprint must be expressed in kg CO₂-eq./y/m² new floor area. For both HSB and CLT, this cannot be used as single indicator, due to the negative footprint of wood. Material usage should be used as additional indicator.
- **Reference service life** – A long RSL has positive effects on material use and environmental impact of strengthening methods.
- **Material usage** – The material use of HSB is much lower compared to CLT (Similar RSL).
- **Carbon sink** – Wood captures CO₂. As the emissions due to strengthening methods are lower compared to the CO₂-capture of wood, the top-up can function as carbon-sink.
- **Strengthening method** – Dilatated buildings must be linked horizontally to strengthen for stability; undilatated buildings must be strengthened by thickening the stability wall with concrete.
- **End-of-life scenario** – Reuse or recycling increase the RSL, while keeping the positive CO₂-capture. Reaching the end-of-second life, the most sustainable method is: incineration with energy recovery.

8

Variant design

Chapter 8 will provide the structural design of the distinct top-up variants. First, ‘an effective method’ will be defined. Second, the distinct variants will be described and evaluated.

8.1 ‘An effective method’

Based on the theoretical background, five criteria are defined to concretize **an effective method** for a vertical rooftop extension on council estates:

- 1a. The top-up is independent of spatial placement of the existing structural walls (allows for variation within council estates) [Chapter 5];
- 1b. Or, the top-up is designed based on all buildings, using maximum spans or a parametric script.
2. Consists of at least two additional floors (in case the vertical load capacity allows; Chapter 6): solves (in the most optimistic scenario) about 70% of housing shortage [Appendix D.1];
3. Can be performed without strengthening for additional vertical load [Chapter 6];
4. The GWP of the top-up (structural elements) [Stage A1-A3] is negative [Chapter 7];
5. Minimizes material use per year for one square meter of new floor area [Chapter 7].

8.2 Defined variants

Two structural methods are considered: CLT- and HSB-construction [Chapter 6]. HSB is preferred for its low self-weight [Fig. 6.10], while CLT for its robustness [Interview T. Jonker]. Both methods have a CO₂-capture. Resultingly, material usage per year forms an additional parameter for sustainability [Section 7.2]. The performed research results in two variants that needs to be further investigated:

1. **Modular CLT-construction** – A modular CLT-construction could potentially have a low material-footprint per year (considering a large lifespan). Especially for the early-built flats [Residual lifespan council estate = 23 years], a modular CLT-construction can be more sustainable.
2. **Non-modular HSB-construction** – The material usage and self-weight of an HSB-construction is lower compared to CLT (considering a similar lifespan) [Section 7.2]. That motivates to use HSB-construction for a non-modular top-up.

The distinct variants are explained and drawn in appendix D.2. Below a brief description.

8.2.1 Modular CLT-construction

The units are designed based on **transportation limits**, **minimal spatial dimensions**, and **supplier criteria**. The outer dimensions L x B x H are 9.45 x 3.50 x 2.95 m. In nearly all cases, four modular units can be placed above one unit council estate. Two variants are designed:

1. Single person unit: floor area $\geq 24 \text{ m}^2$ [Fig. 8.1a]
2. Linked double person unit: floor area $\geq 50 \text{ m}^2$ [Fig. 8.1b]

The **stabilizing wall** is in the middle of a unit, connected to a side wall. A **steel transfer structure** is needed due to discontinuation of the bearing lines. The transfer structure transfers a horizontal load of the extension to the existing stability wall. The transfer structure has to be elevated ($\approx 5 \text{ cm}$) to allow deflection of the beams.

The CLT walls are one single plate. The **elevated floors** ($\approx 10 \text{ cm}$) consist of several plates. This elevation provides space for e.g., installation and pipes. The units include part of the **gallery**. The gallery can be accessed by a non-structural floor or elevator.

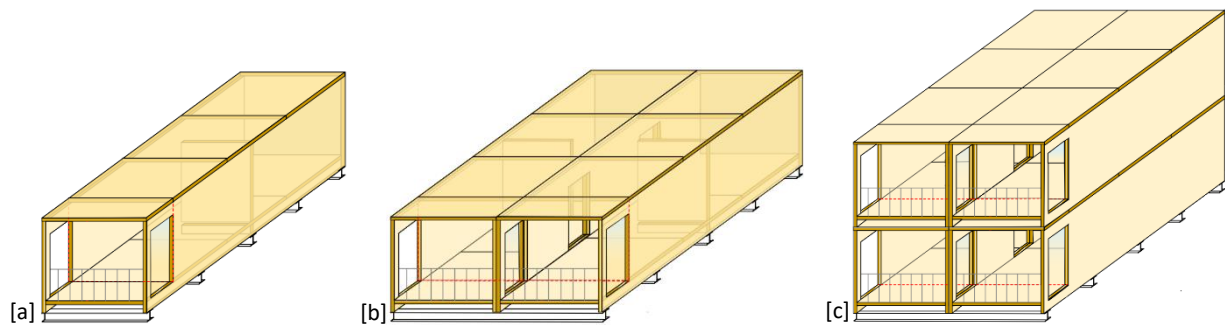


Figure 8.1: Design of modular CLT units. [a] Single unit ($>24 \text{ m}^2$); [b] Double person unit ($>50 \text{ m}^2$); [c] Example of stacked unit. More drawings in appendix D.2.1 (Self-produced).

8.2.2 Non-modular HSB-construction

The HSB-variant is constructed according to the **platform method** [Section 6.3.1]. The bearing lines of the existing structure continue, by placing the structural walls above the existing ones. The existing **roof structure remains**, to minimize disturbance to residents. A **gallery** (1.20 m) is constructed to access the apartments. The new floors can be reached by the **continued stairwell** at end-porches or **non-structural elevator**. Strengthening for stability is done by **horizontal linking** of the dilatations to minimize the environmental impact. Figure 8.2 provides a structural drawing.

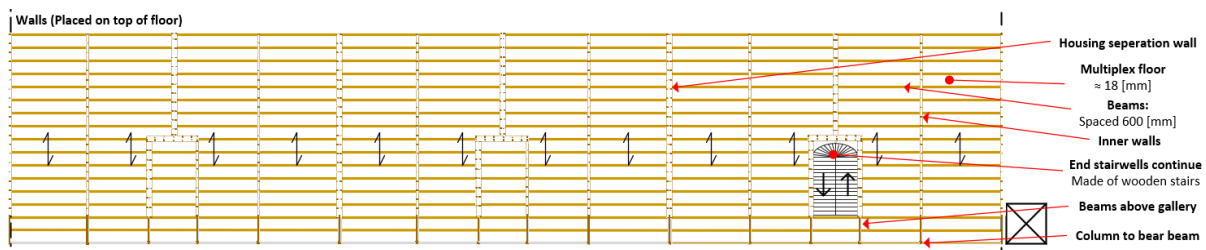


Figure 8.2: Structural design HSB-variant. More drawings in appendix D.2.2 (Self-produced).

8.3 Calculation software

The calculation principles of HSB- and CLT-construction are evaluated. The author concludes that Excel and Matrixframe are sufficient for the calculations. Validations must be done by hand.

8.4 Structural verification of the variants

The variants will be structurally verified. The **NEN-EN 1990, 1991, and 1995, and CLT handbook** will be used as guiding norms (Borgström & Fröbel, 2019). This section will provide the summarizing results.

8.4.1 Modular CLT-construction

A description of the CLT-calculations is provided in appendix D.3. Appendix X.10.1 till X.10.9 provide excel sheets (including hand validations).

8.4.1.1 Wall panels

A 3-layered wall panel (30-40-30) is applied with a total thickness of 100 mm. This is set as minimum wall thickness to allow for installation slots, connections, lay-up length, and building physic related aspects. The wall is calculated for two floors, but still has a lot of overcapacity.

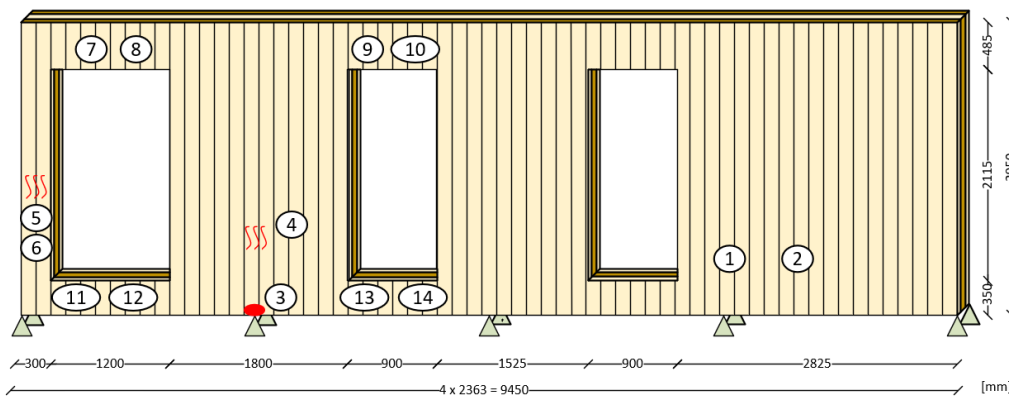


Figure 8.3: Dimensions of CLT Wall. Provided numbers explained in table 8.1 (Self-produced).

Table 8.1: Applied verifications for CLT wall.

Verification	UC	Verification	UC
1 Bending wall	0.02	8 Moment lintel above gallery	0.14
2 Shear force wall	0.20	9 Shear force lintel above door	0.27
3 Stress concentration support	0.38	10 Moment lintel above door	0.12
4 Buckling at support	0.31	11 Shear force lintel below gallery	0.35
5 Buckling/ normal stress column gallery*	0.53	12 Moment lintel below gallery	0.32
6 Deflection column gallery*	0.65	13 Shear force lintel below door	0.33
7 Shear force lintel above gallery	0.25	14 Moment lintel below door	0.20

*Note that the wind area for column is taken conservative (includes half of gallery), what leads to higher values.

8.4.1.2 Steel interlayer

The steel interlayer is constructed of five steel HEB260-S235 beams. Figure 8.4 shows the normative load combination. The combination maximizes vertical load. The axial ($N_{ed} < 0.25 N_{pl}$) and shear force ($V_{ed} < 0.5V_{pl}$) could be neglected. The performed verifications are provided in table 8.2.

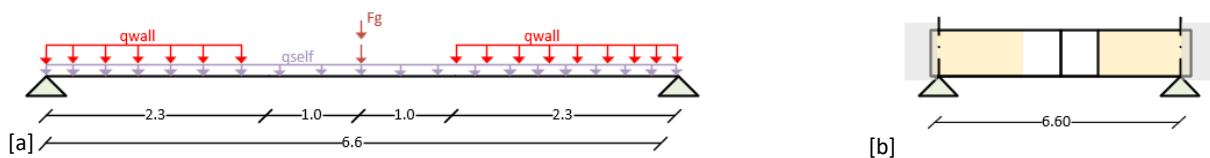


Figure 8.4: Steel interlayer validation; [a] Normative load combination for steel interlayer (q_{wall} = load stability wall; q_{self} = self-weight steel beam; F_g = normal force from wall); [b] Placement of units on steel layer (Units: m) (Self-produced).

Table 8.2: Verifications steel interlayer.

Verification steel	UC
Cross-section check bending	0.80
Lateral torsional buckling	0.95
Deflection	0.83

8.4.1.3 Floor and roof panels

The 3-layered roof panel (20-40-20) is verified for strength (ULS = Ultimate Limit State) and deflections (SLS = Serviceable Limit State). These panels have the capability to accommodate solar panels in the future. The 3-layered floor panels (40-40-40) are also checked for vibrations, as this is an important aspect for wooden floors (Huang et al., 2020). A 5-layered floor panel (20-30-20-30-20) would have higher unity checks. Unfortunately, material use equalizes the 3-layered variant, while glue increases.

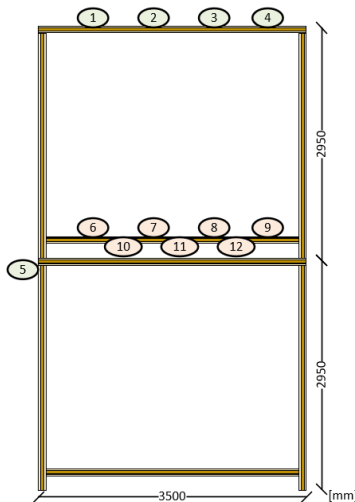


Figure 8.5: Section of unit. Provided numbers explained in table 8.3 and 8.4 (Self-produced).

Table 8.3: Verifications for roof panels.

Verification roof panel		UC
1	Bending	0.27
2	Shear	0.03
3	Rolling shear	0.18
4	Deflection	0.82
5	Compression perpendicular to the grain	0.05

Table 8.4: Verifications for floor panels.

Verification floor panel		UC
6	Bending	0.18
7	Shear	0.04
8	Rolling shear	0.21
9	Deflection	0.40
10	Vibration – Fundamental frequency	0.52
11	Vibration – Point load deflection	0.65
12	Vibration – Impulse velocity response	0.14

8.4.1.4 Stability wall

The units make use of the principle of active and passive walls [Fig. 8.6]. The tension force (due to wind) locates therefore always at the intersection between stability and sidewall. To allow this principle to be valid, the units must be placed in pairs of two. Two situations are considered.

1. **Placement on a council estate [UC1];** One block is considered (two porches), meaning that four walls will resist against the horizontal wind load (8 units).
2. **Placement at ground level [UC2];** At least four units must be placed to have no tension.

A similar wall as the side walls is applied (3-layered; 30-40-30). The wall length is maximized (2.5 m), to minimize the tension force. Table 8.5 and 8.6 provide the results.

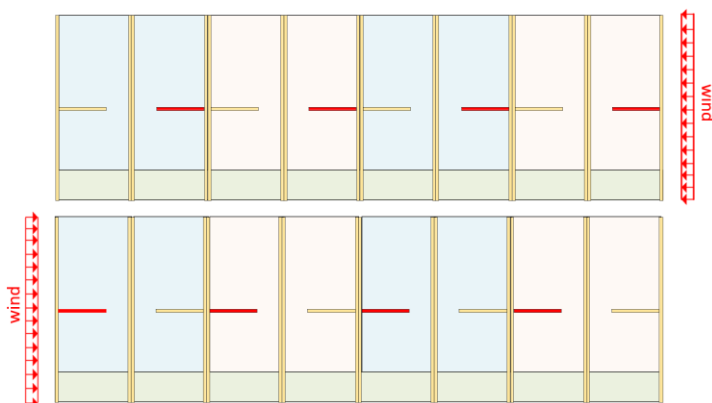


Figure 8.6: Principle of active and passive walls (Self-produced).

Table 8.5: Verification stability on flat.

Verification stability Placement on council estate (8 Units considered)		UC1
Deflection		0.06
Tension		No
Compression \perp grain*		0.64

Table 8.6: Verification stability ground.

Verification stability Placement at ground level (4 Units considered)		UC2
Deflection		0.07
Tension		No
Compression \perp grain*		0.83

*Floor panel below stability wall

8.4.1.5 Weight of the CLT top-up

The weight of the top-up equals 6.4 kN/m² for 1, and 12.3 kN/m² for 2 added layer(s). This value considers an exceeding of 5% of the over-capacity (Van Rossum B.V.; Internal communication).

8.4.2 Non-modular HSB-construction

A number of verifications are performed on the HSB design. A detailed description of these calculations can be found in appendix D.4. Appendix X.11.1 till X.11.7 provide excel sheets (including validations).

8.4.2.1 Walls

The wall panels consist of a timber frame (C24), clad with 18 mm plywood. Both, a **single wall** [Fig. 8.7b] and a **housing separation wall** [Fig. 8.7c] are designed for 1 till 3 storeys. The columns are constructed in SLS wood (e.g., 38x89; 38x120; 38x140), supported in the weak axis for buckling. The beams are subjected to compression perpendicular (\perp) to the grain, which is a normative for more than one storey. The columns that transfer load of the façade are placed outside the beam, to prevent compression \perp to the grain. Table 8.7 provides only normative results.

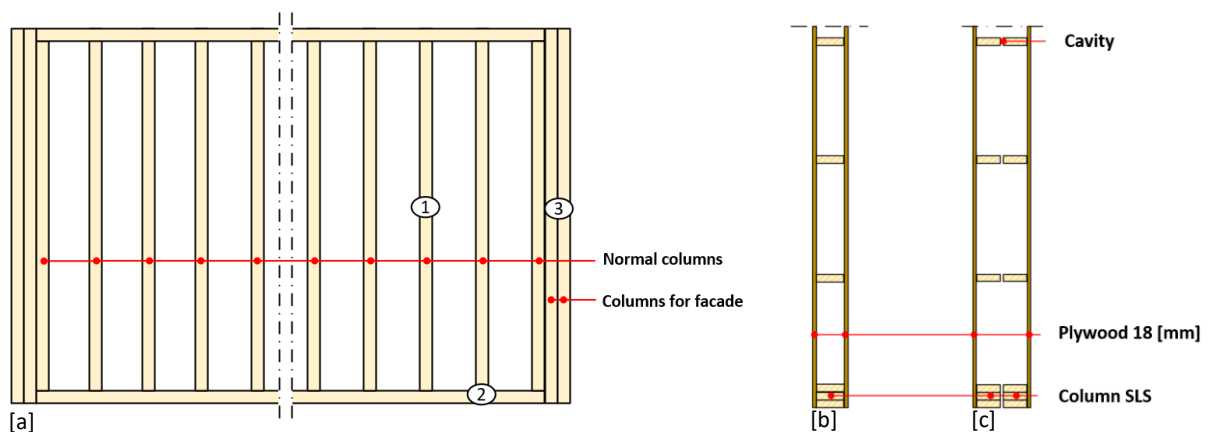


Figure 8.7: Structural design of HSB wall; [a] Side view; [b] Top section single wall; [c] Top section housing separation wall (Self-produced).

Table 8.7: Element sizes of wall, including (normative) unity check(s) (Number refers to Fig. 8.7a).

Storeys	CFL [m]	Column size [mm]	(1) UC_N ; buckling	(2) $UC_{beam\perp}$	(3) $UC_{column\ for\ façade\ (*)}$
1	Single wall (No wind; Combination 1)				
	6.70	SLS 38x89 – spaced 600	0.78	0.53	0.67 (2)
	8.49	SLS 38x89 – spaced 600	0.95	0.65	0.84 (2)
	10.28	SLS 38x120 – spaced 600	0.50	0.57	0.44 (2)
	Housing separation wall				
	6.70	SLS 38x89 – spaced 600	0.38	0.26	0.93 (1)
	8.49	SLS 38x89 – spaced 600	0.47	0.32	0.58 (2)
10.28	SLS 38x89 – spaced 600	0.56	0.38	0.70 (2)	
2	Single wall (No wind; Combination 1)				
	6.70	SLS 38x140 – spaced 600	0.61	0.89	0.94 (1)
	8.49	SLS 38x140 – spaced 400	0.51	0.74	0.55 (2)
	10.28	SLS 38x140 – spaced 400	0.60	0.88	0.66 (2)
	Housing separation wall				
	6.70	SLS 38x120 – spaced 600	0.45	0.51	0.97 (1)
	8.49	SLS 38x120 – spaced 600	0.55	0.64	0.61 (2)
10.28	SLS 38x120 – spaced 600	0.66	0.76	0.74 (2)	
3	Single wall (No wind; Combination 1)				
	6.70	SLS 38x140 – spaced 400	0.58	0.85	0.63 (2)
	8.49	SLS 38x170 – spaced 400	0.47	0.87	0.51 (2)
	10.28	SLS 38x184 – spaced 400	0.48	0.95	0.53 (2)
	Housing separation wall				
	6.70	SLS 38x120 – spaced 600	0.63	0.73	0.70 (2)
	8.49	SLS 38x120 – spaced 600	0.78	0.90	0.88 (2)
10.28	SLS 38x140 – spaced 600	0.63	0.92	0.72 (2)	

*Number provides the needed columns at the end of the wall, to transfer the load of the façade.

8.4.2.2 Floor and roof structure

Timber beams (C24) span between the HSB walls (Spacing_{max}=600 mm), clad with plywood. Four beam spans are defined, based on the archive inspection [Appendix A.1]. The largest span exceeds structural limit of a standard HSB floor. Steel beams (HEA220, S235) span in major, while timber beams in minor direction. The roof is verified for deflection [SLS] and strength [ULS]. The floor structure is additionally verified for human-induced vibrations [SLS]. The results are provided in table 8.8.

Floor span range in archive: $\left\{ \begin{array}{ll} 3.9 - 4.6 \text{ m} & \text{(Most occurring range in)} \\ 4.7 - 5.4 \text{ m} & \text{(Range present in archive)} \\ 5.5 - 5.7 \text{ m} & \text{(Limit of timber floor structure; Not in archive)} \\ 5.8 - 6.6 \text{ m} & \text{(Placement of additional steel beams; 1x present in archive)} \end{array} \right.$

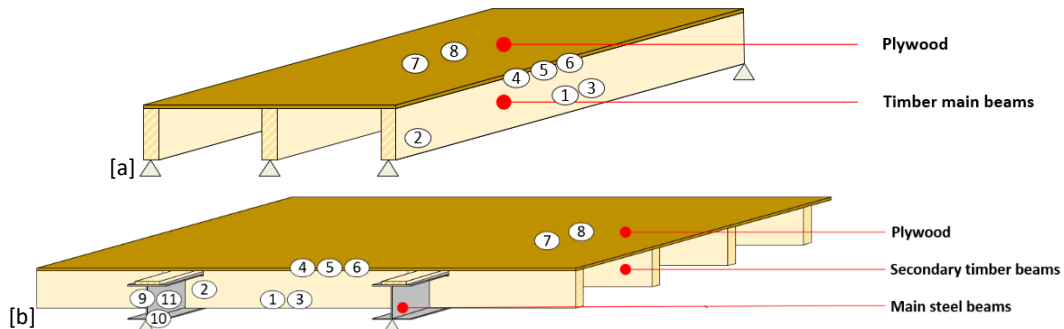


Figure 8.8: 3D overview of floor and roof system; [a] Basic system for both floor and roof; [b] Adapted floor system for span 5.8 - 6.6 m (Self-produced).

Table 8.8: Unity checks for floor and roof. Numbers refer to Fig. 8.8. (W) = wood C24; (S) = Steel beam HEA220 - S235.

Property for system			Floor system				Roof system		
Span range [m]			3.9-4.6	4.7-5.4	5.5-5.7	5.8-6.6	3.9-4.6	4.7-5.4	5.5-6.6
Timber beam size [mm]			70x270	95x295	95x295*	70x245	70x170	70x195	70x245
Spacing [mm]			400	500	400	400	500	400	400
Thickness plywood [mm]			18	21	21	18	18	18	18
Nr.	Element	Verification	Unity check (UC) ↓						
1	Beam (W)	Bending	0.37	0.4	0.36	0.34	0.67	0.58	0.56
2	Beam (W)	Shear	0.13	0.13	0.11	0.12	0.15	0.13	0.12
3	Beam (W)	Deflection	0.36	0.42	0.40	0.31	0.98	0.87	0.83
4	System	Fundamental frequency	0.45	0.55	0.57	0.40	-	-	-
5	System	Point load deflection	0.97	0.93	0.95	0.78	-	-	-
6	System	Impulse velocity response	0.75	0.64	0.59	0.66	-	-	-
7	Plywood	Bending	0.14	0.16	0.1	0.10	0.12	0.08	0.08
8	Plywood	Deflection	0.18	0.23	0.12	0.12	0.19	0.10	0.11
9	Beam (S)	Bending	-	-	-	0.65	-	-	-
10	Beam (S)	Shear	-	-	-	0.19	-	-	-
11	Beam (S)	Deflection	-	-	-	0.93	-	-	-

*Place 2 beams below façade

8.4.2.3 Gallery beam

The gallery beams span between the gallery column and HSB wall. These beams could be constructed smaller than the connected floor or roof beams. To consider a feasible connection, a **practical size** is chosen: size of floor or roof beams. The most unfavourable situation is verified [Table 8.9].

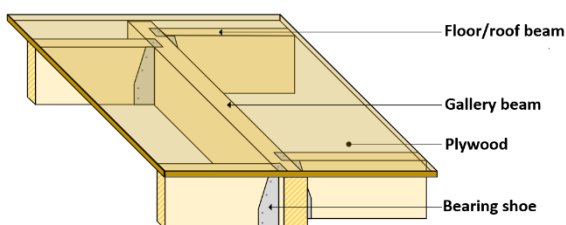


Figure 8.9: 3D image of gallery beam (including connection) (Self-produced).

Table 8.9: Worst case results for gallery beams.

Verification	UC _{floor level}	UC _{roof level}
Bending	0.35	0.50
Shear	0.47	0.42
Compr. ⊥ grain*	0.80	0.45
Deflection	0.10	0.21

*Beam on top of HSB wall

8.4.2.4 Gallery columns

The buckling resistance in both directions is equal (supported at floors), resulting in a square column design. The columns are verified in the ULS for buckling (including axial load). Figure 8.10 provides the used schematization of the columns. The results are provided in table 8.10.

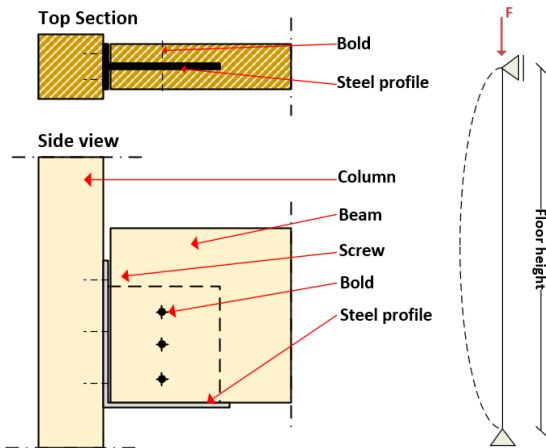


Table 8.10: Results for gallery columns.

Storeys	CFL [m]	Column size [mm]	UC _N ; buckling [-]
1	6.70-8.49	70x70	0.65
	8.49-10.28	70x70	0.79
2	6.70-8.49	95x95	0.56
	8.49-10.28	95x95	0.67
3	6.70-8.49	95x95	0.78
	8.49-10.28	95x95	0.95

Figure 8.10: Gallery column; [a] Connection between gallery beam and column; [b] Structural schematization column (Self-produced).

8.4.2.5 Stability wall

The stability properties depend on the existing structure. This report evaluates the stability for a 3-layered top-up (Ratio depth to length stab. wall = 4.5 [-]; No add. room). To reduce the tension force in the stability walls, a horizontal linking of two blocks is assumed. It is most probable that the existing structure needs to be linked, to provide enough horizontal resistance [Fig. 6.15].

Deflection of an HSB-wall is usually not verified. Verified are **the presence of tension, maximal compression, compression ⊥ to the grain, and needed nails** in the stability wall [Table 8.11]. Note that for future projects, the stability of the top-up must be recalculated. Stability in cross-direction is provided in appendix D.4.6.3.

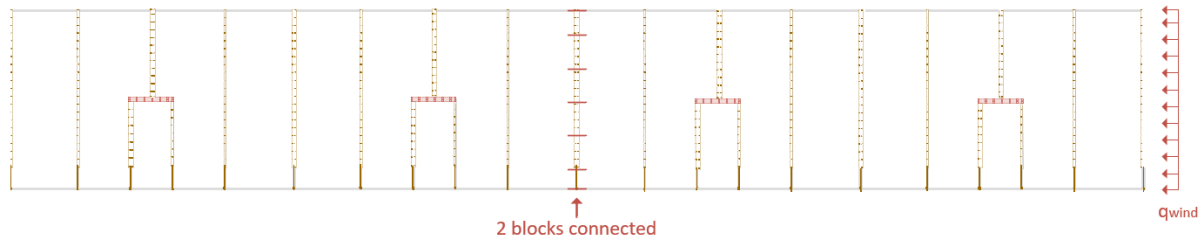


Figure 8.11: Collaboration of walls to resist the horizontal wind load (Self-produced).

Table 8.11: Results stability calculation.

Verification	Aspect
Presence of tension?	Yes -> 2x tension anchor WHT440
Maximal compression	UC = 0.54 [-]
Compression ⊥ to the grain	UC = 0.62 [-]
Needed nails	∅3.1x50 – spaced 100 mm

8.4.2.6 Weight of the HSB top-up

The weight of the top-up equals 5.5 kN/m² for one, 10.1 kN/m² for two, and 13.4 kN/m² for three added layers. This value considers an exceeding of 5% of the over-capacity (Van Rossum B.V.; Internal communication).

8.5 Environmental impact of the variants

Both variants will be environmentally evaluated. This section will provide the results.

8.5.1 Modular CLT-construction

The environmental calculations are performed with excel. Appendix X.10.10 provides the LCA sheets.

8.5.1.1 Goal and functional unit

The **goal** is to provide the author data about the environmental impact of the designed CLT construction, being part of an educated choice for an effective top-up method for council estates.

Two **functional units** are defined, considering both a technical and functional lifespan of 75 years:

1. The annual carbon footprint of the CLT structure per square meter new floor area, limited to the product stage: A1-A3. Besides CLT and steel, only plasterboard for walls is considered in the calculation (structurally provides fire-safety). GWP total is solely taken as impact category.
2. The structural material usage of the CLT structure per square meter new floor area. Material usage is defined as the sum of CLT, steel and plasterboard.

8.5.1.2 Results

The results are provided in figure 8.12 and 8.13. The CLT construction has a GWP lower than 0.0 [kg CO₂-eq./y/m² new floor area], which is needed for an efficient method. The total impact equals -30.23 (1-layer) to -32.58 (2-layers) [10⁻¹ kg CO₂-eq./y/m² new floor area]. This difference is caused by the steel interlayer. The emission of the plaster board (≈-2.5%) is negligible compared to the CO₂-capture of wood (+109.7 till 118.3%). The total material use varies between 5.52 (2-layers) to 5.56 (1-layer) [10⁻³ m³/y/m² new floor area]. The order of contributors to the material use is approximately: walls (≈50.5%), floors (≈29.0%), roofs (≈19.5%), and steel layer (≈1.0%) [Fig. 8.13]. The GWP results closely equalize the values in chapter 7 [Fig. 7.2]: $105.65 \cdot \frac{23}{75} = 32.39$ [10⁻¹ kg CO₂-eq./y/m² floor area].

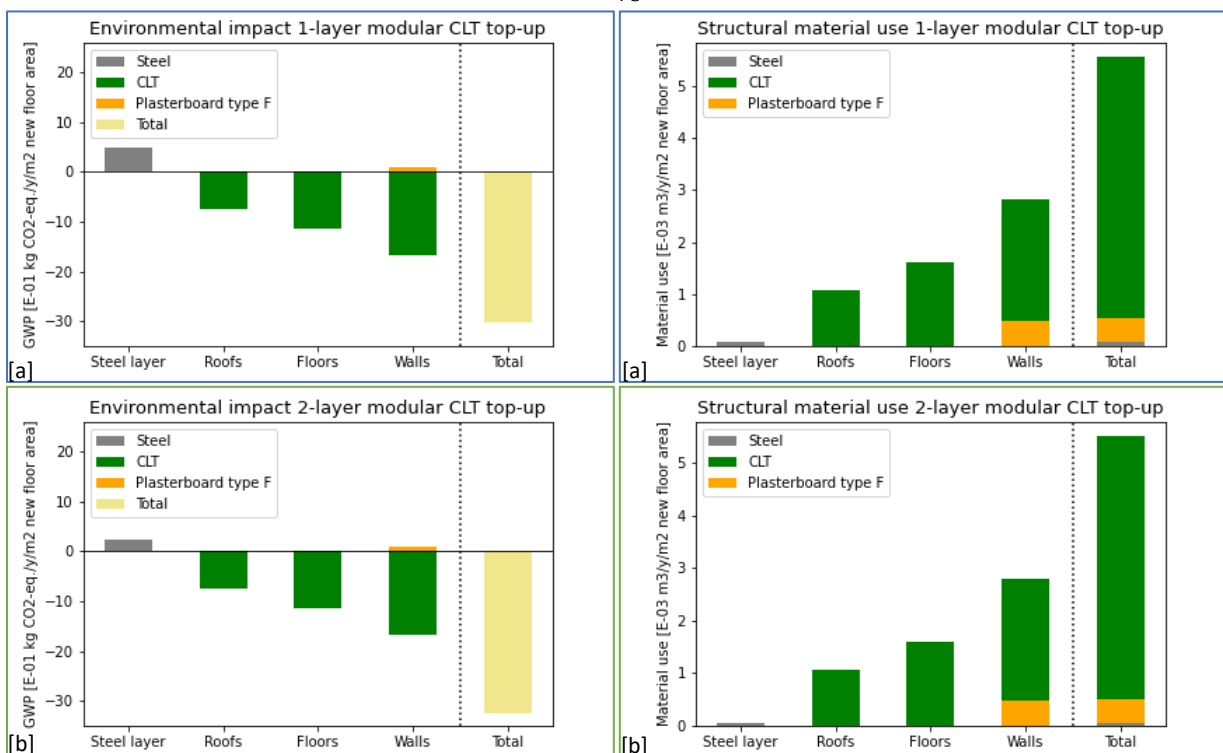


Figure 8.12: GWP CLT construction (RSL=75y);
[a] 1-layer top-up; [b] 2-layer top-up (Self-produced).

Figure 8.13: Material use CLT construction (RSL=75y)
[a] 1-layer top-up; [b] 2-layer top-up (Self-produced).

8.5.2 Non-modular HSB-construction

The environmental calculations are performed with excel. Appendix X.11.8 provides the LCA sheets.

8.5.2.1 Goal and functional unit

The **goal** is to provide the author data about the environmental impact of the designed HSB construction, being part of an educated choice for an effective top-up method for council estates.

Two **functional units** are defined, considering a residual functional lifespan of 23 till 42 years:

1. The annual carbon footprint of the HSB structure per square meter new floor area, limited to the product stage: A1-A3. Besides HSB-structure and steel, only plasterboard for walls is considered (structurally provides fire-safety). GWP total is solely taken as impact category.
2. The structural material usage of the HSB structure per square meter new floor area. Material usage is defined as the sum of sawn wood, plywood, steel, and plasterboard.

8.5.2.2 Decisions in method

Figures 8.14 and 8.15 provide the GWP and material use for a 2-layered top-up, per element. Figure 8.16 provides the GWP, and material use for 1 till 3 added layers. A total overview of all graphs can be found in appendix D.5. Two decisions are made:

- The sum of the GWP of floors and roof forms the largest contributor ($\approx 80\%$) to the total impact [Fig. 8.14]. Resultingly the subdivision in floor span ranges is considered.
- Variation of wall sizes (CFL varies per storey) has limited impact on the total impact. The proportion of $GWP_{\text{sawn wood; wall}}$ tot the GWP_{wall} is $\approx 60.0\%$. Considering volumes this proportion is only 22.0% [Fig. 8.15]. As only the proportion sawn wood changes due to variation in CFL, this variation is excluded from the analysis. It would have tripled the number of graphs.

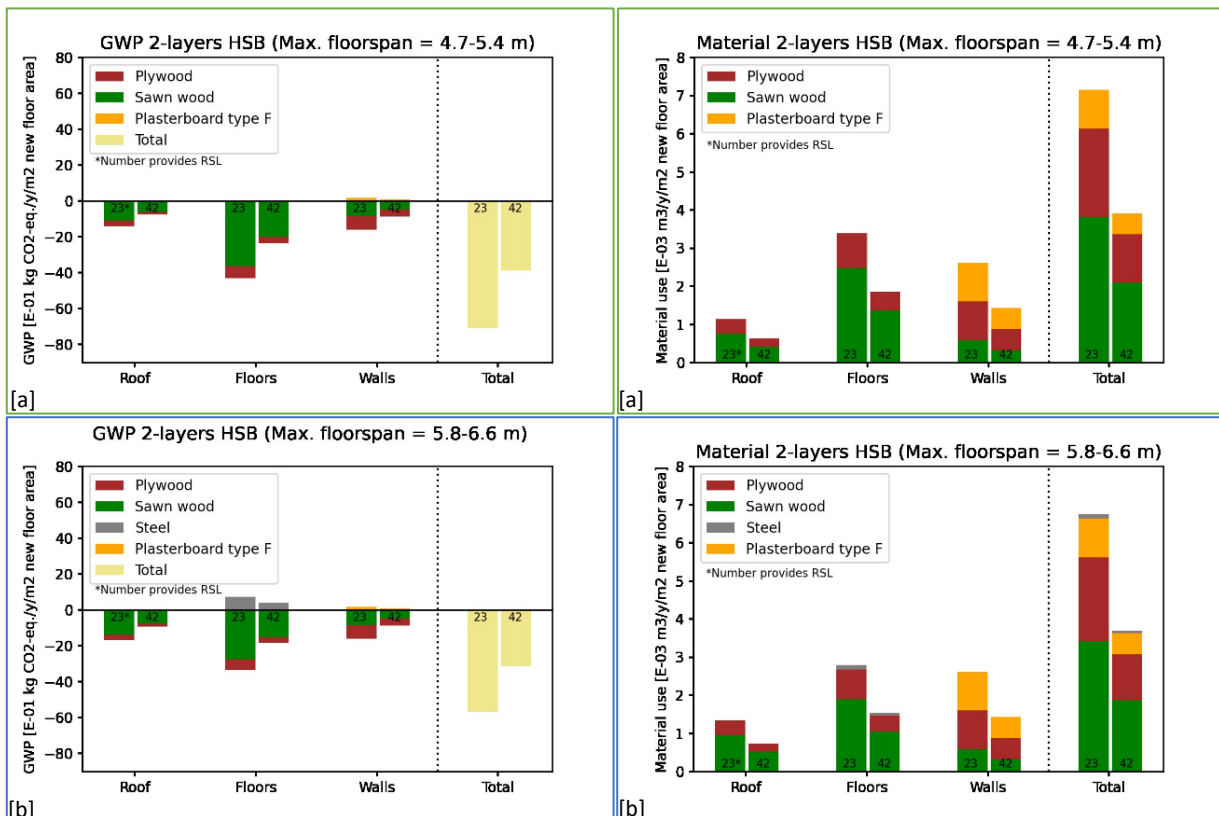


Figure 8.14: GWP HSB construction [Stage A1-A3]. Two spans considered (no add. room); [a] 4.7 -5.4 m; [b] 5.8 – 6.6 m (Self-produced).

Figure 8.15: Material use HSB construction. Two spans considered (no add. room); [a] 4.7 -5.4 m; [b] 5.8 – 6.6 m (Self-produced).

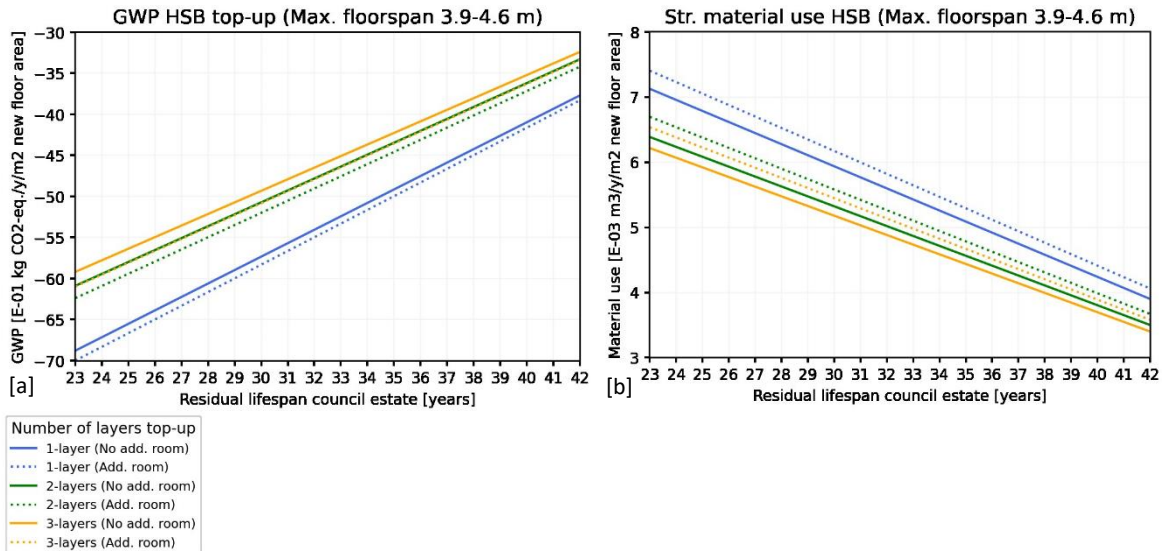


Figure 8.16: Environmental impact HSB top-up (Max. floorspan 3.9 - 4.6 m); [a] Global warming potential [A1-A3]; [b] Structural material use (Self-produced).

8.5.2.3 Results

The GWP and material use depends on the number of storeys, floorspan, residual lifespan of the council estate, and presence of an additional room [Fig. 8.16]. Appendix D.5 contains all combinations to find the impact and material use for a distinct council estate. Extreme values for GWP and material use are provided in table 8.12. The HSB constructions have a negative GWP, which is needed for an efficient method [Section 8.1]. The results are in the order of magnitude of chapter 7 [Fig. 7.2; Fig. 7.3]

The carbon footprint becomes less negative in case the residual lifespan increases, while the material use reduces. It is beneficial to add more layers to the building [Fig. 8.16; Table 8.12]. An increased number of layers reduces the material use. This effect is the much larger from 1 to 2 storeys ($\approx 10.0\%$), compared from 2 to 3 storeys ($\approx 2.0\%$). Presence of additional room causes up to 5.3% additional material (+3.1% GWP).

The parameters GWP and material use are **not directly interchangeable** [Fig. 8.14, 8.15]. The impact of plasterboard on the GWP is negligible ($\approx -2.0\%$), while it fulfils a significant part in the material use ($\approx 12.8\%$). Steel behaves conversely to plasterboard, having a significant impact on the GWP (-12.8%), and a low contribution to the material use (1.9%). That explains the deviating results of the largest floorspan (5.7 – 6.6 m) [Fig. 8.14b; 8.15b]. That variant contains steel main beams, with timber secondary beams [Fig. 8.8]. The material use of floors becomes lower, while the GWP becomes less negative.

Table 8.12: Extreme values for HSB. Unit GWP: kg CO₂-eq./y/m² floor area; Unit material use: 10⁻³ m³/y/m² floor area.

	RSL = 23y	RSL = 42y		RSL = 23y	RSL = 42y
1 layer top-up	-68.8	-37.7	≤ GWP ≤	-97.7	-53.6
2 layers top-up	-60.9	-33.3	≤ GWP ≤	-84.2	-46.1
3 layers top-up	-59.2	-32.4	≤ GWP ≤	-80.8	-44.2
1 layer top-up	7.1	3.9	≤ Material use ≤	9.4	5.1
2 layers top-up	6.4	3.5	≤ Material use ≤	8.3	4.5
3 layers top-up	6.2	3.4	≤ Material use ≤	8.0	4.4

9

Results

Chapter 9 will provide an overview of the results, as well as a description of the produced method.

9.1 Results of thesis

The first part of the thesis is aimed for researching and learning about council estates [Chap. 5], vertical extensions [Chap. 6], and environmental impact [Chap. 7]. Each component is analysed in a chapter. The most important aspects around council estates are provided in section 5.8 and figure 5.2, addressing the variety within the existing structure. The key aspects of chapter 6 are summarized in flowcharts [Fig. 6.14; 6.15]. Essential elements of chapter 7 are provided in section 7.4, emphasising that minimalization of material use must be a criterion for an efficient method [Section 8.1].

The second part of the thesis is focused on the structural design of two variants and the determination of the corresponding environmental impact. The results of the modular CLT design are given in sections 8.4.1 and 8.5.1, resulting in a value for GWP and material use. The results of the non-modular HSB design are described in sections 8.4.2 and 8.5.2, resulting in linear graphs for GWP and material use [Appendix D.5]. The weights of the designed extensions are linked back to the flow chart V [Fig. 6.14].

9.2 Minimization of material use

An efficient method must fulfil five requirements [Section 8.1]. Both methods comply with the first four points: (1) Allows for variation in structural plans; (2) Adds (if structurally possible) at least two floors; (3) Applies no strengthening for additional vertical loads; (4) GWP of top-up is negative; In conclusion, the assessment of construction methods must be done by minimalization of material use (5). Figure 9.1a-c indicates the tilting point between construction methods. Appendix E.1 provides three graphs for council estates having an additional room. The graph '2 layers HSB – 1-layer CLT' is not provided, as the effective method forces application of two layers. The graphs indicate two aspects:

- More added layers increase the effective period to apply HSB. This is a consistent result, as the material use per m² floor area reduces by more floors [Table 8.12].
- For council estates with a low residual lifespan (≤ 25 years) a modular CLT unit is more efficient.

9.3 Designed top-up method

An effective top-up method for council estates depends on the flat characteristics. A 3-step procedure is developed to determine an effective top-up method for a distinct flat, provided below. An example is provided in Appendix E.2.

Step 1) Determine structural properties of the council estate

The following parameters have to be determined:

- Flowchart VL: material walls; floor system; number of floors; CFL; [Section 6.3.6.4]
- Flowchart S: presence of an add. room; ratio depth flat to length stab. wall; NFL; [Section 5.6.2]
- Material use: construction year council estate; floorspan.

Step 2) Determine the structural overcapacity

- Determine the maximum number of layers that can be added for the vertical load (for both construction methods: CLT and HSB), using the flowchart VL [Fig. 6.14].
- Verify if the addable layers for vertical load is smaller or equal to the addable layers for stability, using the flowchart S [Fig. 6.15]. If not, the structure must be strengthened by using horizontal linking [Section 6.3.5; 7.2.4].

Step 3) Determine the method with minimal material use

- Residual lifespan = (Construction year + 100) – (current year, e.q., 2023)
- Use the graph from figure 9.1 or E.1 to determine the method with minimal material use.

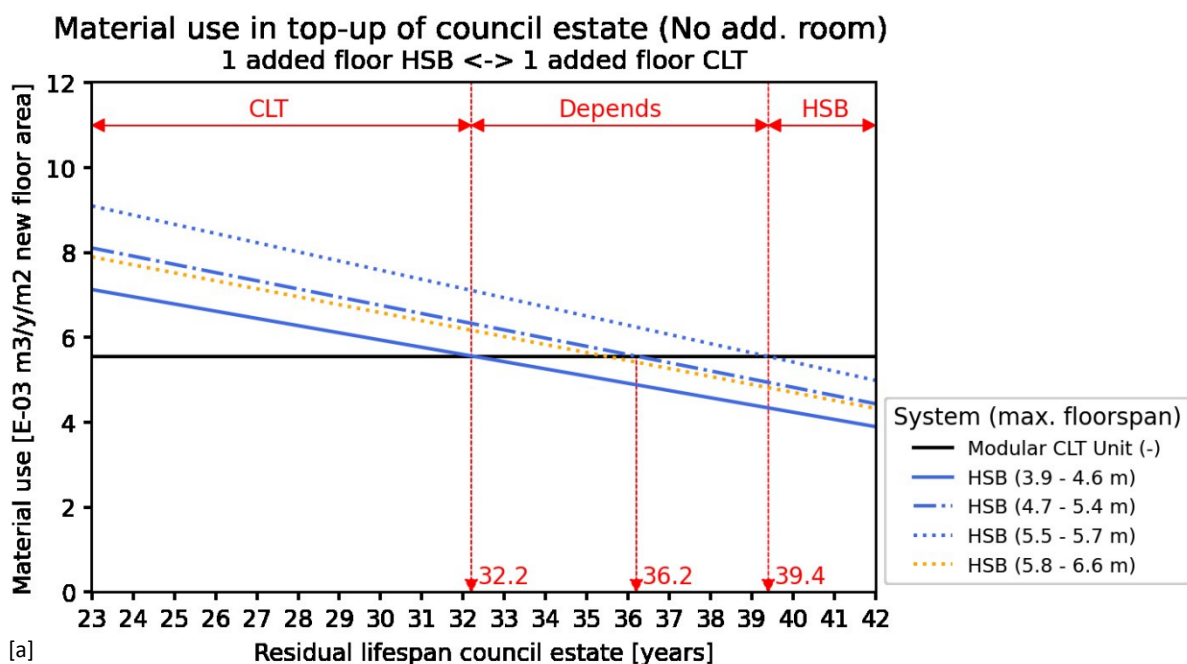


Figure 9.1: Minimalization of material use for vertical extension on council estate (no additional room); [a] 1-layer HSB - 1-layer CLT; [b] 2 layers HSB - 2 layers CLT; [c] 3 layers HSB - 2 layers CLT (Self-produced).

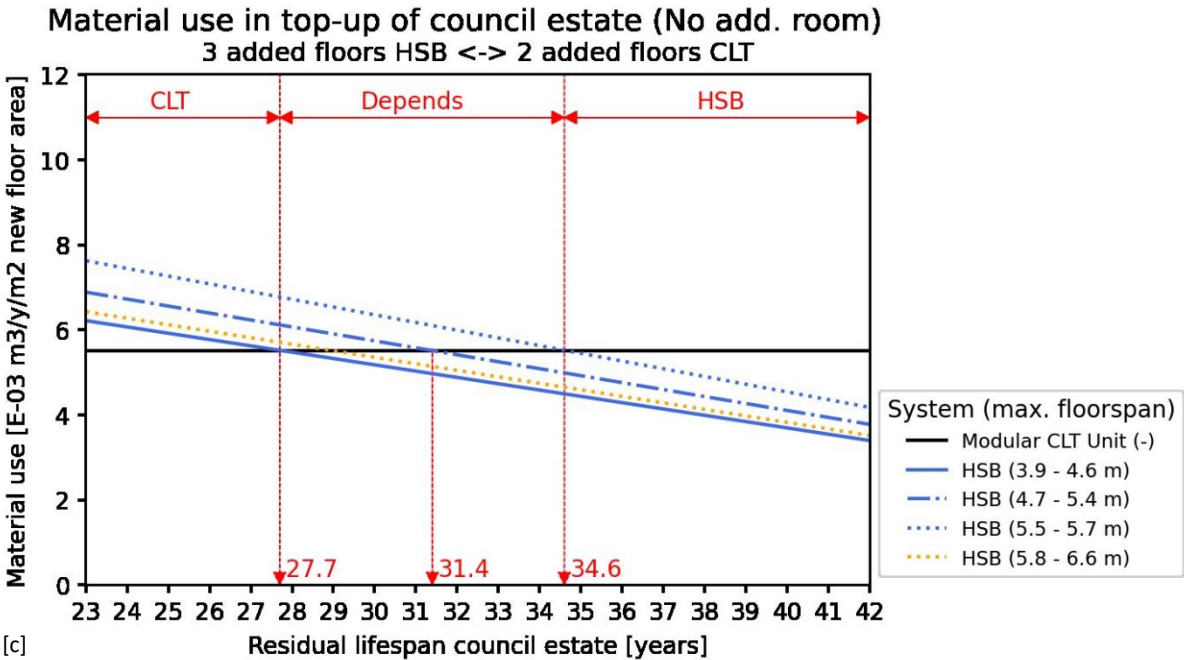
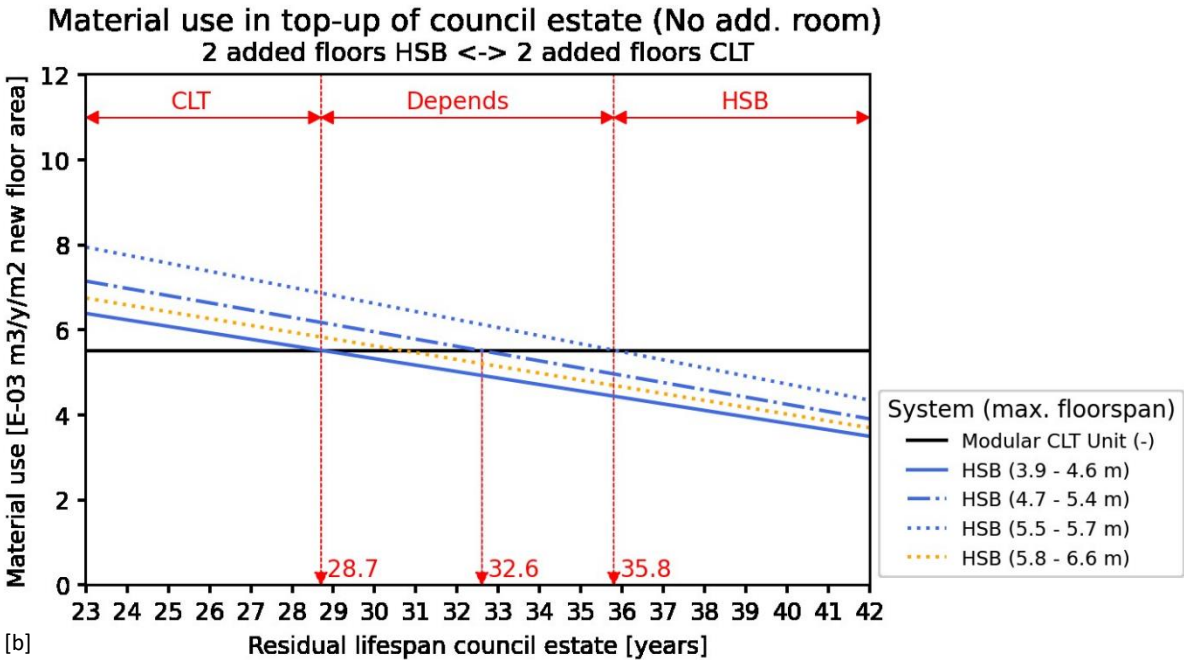


Figure 9.1 (Continue): Minimalization of material use for vertical extension on council estate (no additional room); [a] 1-layer HSB - 1-layer CLT; [b] 2 layers HSB - 2 layers CLT; [c] 3 layers HSB - 2 layers CLT (Self-produced).

10

Discussion

This thesis aims to develop a top-up method for council estates to contribute to sustainable solutions for the housing demand in the Netherlands. Moreover, this research will broaden the knowledge of structural (timber) top-up methods and their corresponding environmental impact. Therefore, the research question is:

What is an effective method to optimize timber top-up designs for council estates, considering their structural limits and environmental impact?

To answer this question, the five sub-questions are answered, and their assumptions are discussed. Further, the consequences of the chosen scope and the application of the method in practice are reflected upon.

10.1 Answers to the research questions

10.1.1 What defines a council estate?

A council estate is a multi-storey building, where residents enter their houses by making use of a common route through a porch. The majority of these flats were constructed during the interwar (1916 – 1926) and post-war periods (1946 – 1965). The local housing planning caused structural variation during the interwar period, while insufficient money, and a lack of skilled workers and construction materials caused variation in construction methods during the post-war period [Fig. 5.2]. Therefore, a single structural lay-out does not exist. Four aspects need a further reflection.

1. For this research, it was concluded that the large variety of **inter-war buildings** makes them less suitable for a universal method. As the buildings were constructed as blocks following street plan perimeters, irregular floorplans occur. It is not very likely that rectangular CLT units can be applied on those inter-war buildings. The author assumes that for buildings built in the interwar era, which resemble the post-war flats, the method could still be applicable. However, a further study has to investigate if the method is (partly) applicable for the inter-war buildings.
2. An **archive inspection** was conducted to determine the characteristics of the post-war flats. This study covered an enquiry into a sample size of thirteen different council estates in Amsterdam. Although no reason was found to assume these thirteen cases deviate from

buildings elsewhere, a multi-location investigation could have provided a more accurate picture. The sample covered a too small group to determine parameter boundaries. The number of floors is underlined by literature and is therefore assumed to be accurate. The used range for the spatial dimensions is considerably large, which most likely will cover most of the council estates in the Netherlands. In conclusion, the study provided a useful and illustrative overview of the spread in council estates. A large archive study to the spatial dimensions of council estates will improve the method by optimizing the parameter ranges.

3. The study to **non-traditional systems** pointed out that these often have their own builder-specific flooring systems. The produced flow charts are limited to timber and concrete floors. That does not necessarily mean that for those specific flooring systems the method is unusable. Timber floors refer to a low and concrete to a high self-weight. Therefore, the structural engineer can estimate which floor resembles the non-traditional system the most. Based on that, a preliminary decision for a top-up method can be made.
4. The thesis emphasised the **uncertainty in brickwork** quality [Section 5.5.2]. Experts of Nebest and Van Rossum commented that quality must be determined on site. This research assumed a design compressive strength of 2.81 N/mm^2 , which was chosen as a presumably realistic value for many buildings. It must be validated if this holds for the specific building. The maximum compression in the stability wall can become critical if the brickwork quality is lower. Therefore, flowchart S must then be revised. It has to be mentioned that the used compressive strength is quite conservative (Jafari, 2021). Therefore, exceeding of the compressive capacity will most likely not occur. Further research to the brickwork quality of council estates is needed, to make flowchart S proper.

10.1.2 What are possible strategies to top-up a council estate?

Four main aspects influence the possible top-up strategies:

- It is common to continue the bearing walls of the existing structure in a top-up for material efficiency.
- Strengthening for additional vertical load is complicated and expensive. Therefore, the vertical weight of the top-up is minimized, and needed strengthening therefore is not applied.
- Horizontal linking of the dilatations is environmentally the most preferred option to strengthen for stability.
- The negative GWP makes an environmental comparison between timber structures complicated. Minimalization of material is therefore used as additional criterion.

Based on these points, two top-up strategies are considered: a modular CLT and a non-modular HSB variant. Four points are further discussed.

1. The method does not provide a stability solution for **council estates which have no dilatation**. Those are not present in the archive inspection, but it is not strange to expect that these exist. Thickening the stability wall with concrete will be the solution with the lowest environmental impact for those cases. The net GWP will be negative [Section 7.2.4], indicating that the method is still applicable. The structural engineer has to determine the concrete dimensions.
2. A **negative side effect of horizontal linking** is the possible increase of contact noise between two neighbouring apartments. The other apartments in the building block have similar structural systems, so it may be assumed that this disturbance is in those post-war buildings.
3. In retrospect, the use of the **carbon footprint method** can be debated. The negative GWP complicates the comparison of two distinct timber structures. Therefore in addition,

minimalization of material was used as criterion. If all nineteen LCA impact categories were considered, the shadow costs would presumably not have been negative. In that case, the environmental impact could have been minimized. Due to transition of norms, a full LCA-calculation was not possible. Minimalization of material use forms a suitable alternative, as excessive wood logging has also negative effects on the environment. If the new unit equivalents are linked to shadow costs, it would be interesting to redo the research with a complete LCA-calculation to see if similar outcomes will be the result.

4. This research is limited to a modular CLT unit (Cross Laminated Timber) and non-modular HSB-construction (HSB = Timber Frame Construction; Dutch: Houtskeletbouw). **Partly reuse of distinct HSB elements** is not considered, to narrow the study. It could have potential as the technical lifespan of some elements will increase. Exclusion of this variant does not make the method ineffective, as the needed requirements are fulfilled. A further study can expand the method with this variant.

10.1.3 How does the building code count for vertical extensions?

Transition of norms provides council estates overcapacity. This can be calculated by comparing the historically and currently applied safety strategy and loads on the building. The historical codes applied (or can be derived) a comprehensive safety factor, which included both uncertainty in material and load definition. The variable load was historically calculated about 12.5% higher than present loads and was applied with different safety strategy than currently done. The existing code (NEN 8700) allows recalculation of the existing structure with reduced safety factors. Considering all differences results in a global overcapacity of about 21% (compared to the NEN8700). Two aspects are further discussed.

1. The case studies indicated that an existing structure may be calculated using NEN8700: Renovation level. It recently turned out that for top-ups, in which no structural parts of the existing building are modified, it is allowed to **calculate with the NEN8701**: Rejection level (mail correspondence M. Hermens [NEN commission], and Van Rossum B.V.). The small difference will most presumably not increase the overcapacity with one floor, and therefore not change flowchart VL. A further study should verify if additional overcapacity results from NEN8701.
2. The **comprehensive safety factor of brickwork** is derived by comparing the historical design value (N1055) and the current characteristic value (NPR9096). The four types of brickwork investigated, showed a spread in load factor. The small calculation example of Appendix B.6 demonstrated that the derived factor is not that accurate. Therefore, the safety on brickwork contains uncertainty. The normative load factor is based on the foundation. Therefore, the uncertainty in safety, applied on the brickwork walls, has no impact on the flowchart VL.

10.1.4 What are the structural possibilities for the distinct top-up methods?

The vertical overcapacity ranges from 8.2 to 16.4 kN/m² [Fig. 6.14]. The weight of the HSB top-up equals 5.5, 10.1, and 13.4 kN/m² for 1 to 3 storeys, respectively. The weight of the CLT top-up equals 6.4, and 12.3 kN/m², for 1 and 2 storeys, respectively. The horizontal overcapacity ranges from 0 to 3 (or more) addable floors [Fig. 6.15]. If the horizontal resistance is insufficient, the distinct building blocks must be horizontally linked, what will increase the resistance sufficiently. One aspect needs further reflection.

- **Fire safety** forms a crucial aspect in timber buildings. Despite that, it was decided to exclude fire safety of the structural calculations to narrow down the study. Therefore, the completeness of the research can be debated. E.g., clients most presumably prefer CLT over HSB because of the number of openings and cavities in HSB which could accelerate fire spread. At least three fire-safety measures can be taken: apply fire-resistant cladding, increase timber cross-sections and prevent hollow spaces which can act as vertical or horizontal fire distributors. The second solution is not possible for HSB and will increase the self-weight of CLT-elements. Therefore, fire safety cladding forms a better solution. Although no analysis has been conducted yet, the performed calculation considered 18 mm fire retarded plasterboard for the walls. Also, the floors are calculated with a finishing. It might be that the weight of the top-up increases a bit due to fire safety measures. A recalculation according to the NEN8701 ([Sub question 3](#)), combined with fire safety measures will most presumably not change the results of flowchart VL. However, a further study has to point out if this is the case. The change of intersection points in the material minimalization graphs [[Fig. 9.1](#)], due to an (increased) fire safety cladding, will be negligible. This, as (part of the) fire retarded plasterboard has already been considered.

10.1.5 What is the environmental impact of the realized variants?

The residual lifespan of council estates varies between 23 and 42 years based on an assumed 100 year service life and the year of construction. The technical lifespan of a modular CLT construction can be fully developed, resulting in a negative value for lifecycle stage A1 till A3 of -32.58 (2-storey) or -30.23 (1-storeys) [10^{-1} kg CO₂-eq./y/m² new floor area]. The technical lifespan of the non-modular HSB top-up cannot be fully developed, as it will be demolished when the residual lifespan of the flat is reached. Hereby assumed that HSB cannot be reused, in contrast to CLT. The result are linear graphs indicating the GWP as function of the residual lifespan of a council estate [[Appendix D.5](#)]. A large residual lifespan, and number of addable layers increase the potential to apply a HSB top-up. Two points need to be further discussed.

1. The research limited the environmental impact to lifecycle stage A1-A3. During this stage, timber behaves as a **carbon sink**. More wood use seems to be beneficial for the CO₂ reduction in the atmosphere. However, it strongly depends on the end-of-life scenario [[Fig. 7.4](#)]. Wood incineration forms the most unsustainable end-of-life scenario, as all captured CO₂ releases. Most sustainable is to reuse or downcycle the construction elements to reach their technical end-of-life, and then apply landfill. The applicability of this latter scenario is debatable. Therefore, in practice a sub-optimal solution will presumably be applied e.g., wood incineration with energy recovery. In conclusion, the carbon sink behaviour of the extensions strongly depends on the applied end-of-life scenario. The author posits to reuse or downcycle the construction (elements), whereafter landfill must be applied, as a progressive end-of-life scenario for both timber construction methods, to achieve an overall negative GWP.
2. The **impact of the non-modular HSB extension** is calculated based on the plan of the Helena Mercierstraat. The variety within council estates complicates a detailed predication of the environmental impact and material use of a non-modular HSB top-up. Therefore, the variation between flats is considered by expressing the impact per m² floor area. This is detailed enough for a preliminary optimisation on material use. The intersection points in the provided material minimalization graphs must therefore not be read as strict separation points, but more as an indication where the tipping point roughly is located.

10.1.6 Answer to the main research question

Based on the answered sub-questions, an answer to the **research question** is formulated:

The flat characteristics determine which construction method is the most efficient top-up method for a distinct council estate. These characteristics impact the structural overcapacity (structural limits) [Fig. 6.14; 6.15], as well as the material use of the extension [Table 8.7; 8.8]. An effective top-up method considers five aspects [Section 8.1]:

1. Allows for spatial variation in structural plans;
2. Adds (if structurally possible) at least two floors, to consider the large housing demand;
3. Applies no strengthening for additional vertical loads, to minimize complexity and costs;
4. Functions as carbon sink [A1-A3]: Global Warming Potential of the top-up is negative;
5. Minimizes material use per year.

A modular CLT unit, and a non-modular HSB extension fulfil both aspect 1 till 4, implying that minimalization of material use determines which method is more effective [Fig. 9.1; E.1]. Consequently, **a 3-step method** is developed, which considers the flat characteristics, structural overcapacity and minimalization of material use [Section 9.3]. Large-scale application will solve seventy percent of the current housing demand in the most optimistic scenario. To conclude, the provided method optimizes timber top-up designs for council estates, considering a spread in structural limits. The optimization produces CO₂-negative extensions, having a minimized material use per year and new m² floor area.

10.2 Effect of research scope on outcomes

The research was narrowed down by the chosen scope consisting of four delineations. These will be evaluated here to assess their potential impact on the method and results.

10.2.1 Timber top-up method

The research is limited to a timber top-up method because of its sustainable potential, mechanical properties, time constraints, and the author's interest. Adjustment of this delineation (e.g., a steel top-up) will not undo the entire research. On purpose, the vertical overcapacity is provided in kN/m². That allows structural engineers to estimate the number of addable layers when using other materials for the top-up. Also, flowchart S will still be valid, albeit that the maximum compression must be reconsidered in further design stages. The material minimalization graphs will logically not be useable as these are specific for timber construction methods.

10.2.2 Specific for council estates

Council estates form the target group of the study. The large and relatively homogeneous group make them applicable for a general top-up method. Due to the project specificity of vertical extensions, it is not possible to change the target group, while having still a functioning method. However, that makes the research not irrelevant for different target groups. A strategy is provided to determine the vertical overcapacity based on a norm comparison. Additionally, insight is provided in timber top-up strategies. Finally, the use of material minimalization has been suggested to compare timber constructions. In conclusion, the principles provided in the research, can be applied on different target groups.

10.2.3 No strengthening for additional vertical load

The interviews addressed the increase in costs and complexity due to strengthening for additional vertical loads. Resultingly, this is barely applied in practice, and consequently not in the designed method. If it was, structurally more layers could be added to the council estates. Unfortunately, more layers will cause other problems as shadowing effects on neighbouring buildings, pressure on urban infrastructure, and wind discomforts. Therefore, more than two or three added layers will most presumably not be added in practice. In conclusion, this boundary condition keeps the structural design of the top-up method realistic.

10.2.4 No building physic related aspects

The author graduates for the specialization structural design. Therefore, building physic related aspects are excluded from the research. As already stated, the building physic aspects (e.g., daylight access, and shadowing effects) could limit the number of layers that can be top-upped. This will presumably not be normative for the few added layers in the designed method. For final design stages, a building physic expert must inspect these aspects. Acoustic and thermal resistance of walls are not considered and need to be further investigated (vibrations are considered for floors). The additional weights are already included in the structural calculations. Therefore, these aspects will not influence the designed method.

10.3 Reflection to the practice

Important is to reflect to what extent the method will be useful for the practice. On three aspects a reflection is given.

First, it is important is to reflect to what extent the minimalization graphs of material use will be used in practice. A building process is most often dominated by the financial costs of the project. This includes material and labour costs. CLT-constructions have higher material than labour costs. This holds in particular for modular units, as the building process of prefabricated units happens quickly. In contrast, non-modular HSB-construction results in higher labour compared to material costs [Table 6.2]. Therefore, it can be debated if minimalization of material use is the best chosen optimisation to make a distinction between CLT and HSB for practice. This study focussed on the environmental impact of the top-up. An optimisation to environmental impact will not automatically indicate an optimisation to costs. Unsustainable wood logging, what can be the result of excessive material use, results in negative ecological effects (Section 7.2.2). Therefore, this research decided to minimize material use, as a progressive, sustainable solution. The construction sector also benefits from an optimisation to costs. A further study can expand the research by calculating the financial costs of both variants.

Second, the applicability of the assumed functional lifespan needs to be reflected. The residual lifespan of council estates is based on a functional lifespan of 100 years. This assumption only holds in combination with a thorough renovation of the existing structure. In case no renovation will occur, the residual lifespan will be (much) lower. A renovation complicates the top-up process more than it may seem at first hand. It causes disturbance for the co-users of the building, as well as a slow-down of the building process. Most contractors prefer a fast construction process, to minimize disturbance and labour costs. The reduction of inconvenience for co-users, as well as the low residual lifespan of council

estates motivates to apply a modular CLT-construction. This is in line with the material minimalization graphs considering a low residual lifespan. Therefore, CLT units are probably much more frequently used in practice compared to the non-modular HSB-construction.

Third, the usefulness of the constructed flowcharts is reflected. In the authors opinion those schemes will be very useful for structural engineers, especially as a preliminary tool to top-up council estates. Their ease of use, while still being quite detailed make them useful schemes to estimate the number of layers to top-up. The presentation of the vertical overcapacity in kN/m^2 allows application of other structural materials than timber. The detailed design must obviously be developed further. Nevertheless, such flowcharts will provide structural engineers a fast indication about number of layers to top-up during the sittings with e.g., architects.

11

Conclusions

This chapter will provide the conclusions of the research as well as recommendations.

11.1 Conclusions

From the research, four main conclusions are drawn:

1. It is generally not possible to tell a priori which of the two considered timber top-up methods (modular CLT- or non-modular HSB-construction) is the most effective for a particular case, considering the structural limits and environmental impact. The residual lifespan of a building determines which method is the most effective. As an exception, the extreme residual lifespans of council estates have a predictable top-up method. A modular CLT extension forms the most fitted solution for buildings with a residual lifespan less than 25 years, while a non-modular HSB extension is most fitted for buildings with a residual lifespan more than 40 years.
2. An optimisation of timber structures, solely based on the carbon footprint method (lifecycle stage A1-A3) results in unrealistic solutions. Therefore, an additional criterion (e.g., material use or costs) must be considered in such optimisations.
3. Timber vertical extensions form a sustainable replacement for combined demolition and reconstruction of buildings. New concrete buildings cause CO₂-emissions, while timber extensions generally result in CO₂-capture.
4. A vertical extension of council estates has the potential to contribute to solve current housing demand in the Netherlands. In the most optimistic scenario, seventy percent of the housing demand could be solved. Despite this scenario being probably overly optimistic, the method does facilitate the realisation of a substantial number of houses.

11.2 Recommendations for further research

Based on the discussion, recommendations for further research are proposed.

11.2.1 Brickwork quality of post-war council estates

This thesis discussed the uncertainty in brickwork quality. A lower brickwork quality than assumed can result in an exceeding of the compressive capacity in the stability. Although a conservative compressive capacity is assumed, it still forms an uncertain aspect in the study (Jafari, 2021). Therefore, an extensive site-investigation is needed to understand the brickwork quality of council estates, and thereby improve flowchart S or develop more specific flowcharts for stability.

11.2.2 Spatial dimensions of council estates

The parameter range is based on a sample size of thirteen council estates. The size of this sample is too small to reliably determine parameter ranges. A large archive study to the spatial dimensions of council estates will improve the method by fine-tuning the parameter ranges.

11.2.3 End-of-life scenarios for HSB structures

A study to (reasonable) end-of-life scenarios for timber structures should give an estimation for a full environmental profile more substantiation. Section 7.2.3 provides research containing GWP data for distinct CLT end-of-life scenarios. No literature is available which contains such a data for HSB. Further research in the end-of-life scenarios for HSB enables future LCA-calculations to apply a full-life analysis.

11.2.4 Fire safety analysis of a CLT and HSB structure

Fire safety is excluded from the structural calculations. The discussion addressed the difference in risk on fire-spread between those structural methods. Therefore, more research is needed to investigate the potential differences in fire resistance between CLT- and HSB-constructions. This will assist clients more in weighing up the construction methods and will improve the structural design and fire-safety of the variants.

11.2.5 Additional overcapacity due to NEN8701 calculation

It recently turned out that for top-up projects, in which no existing structural parts are modified, it is allowed to recalculate the existing structure with the NEN8701. This norm contains lower safety factors resulting in more overcapacity. A recalculation with the NEN8701 will make flowchart VL more accurate.

11.2.6 Optimisation to costs

The reflection to the practice addressed the need to optimize for costs instead of material use. Therefore, additional research can expand the research by adding an optimisation for costs. This will increase the use of the designed method.

11.3 Recommendations for practice

Five recommendations are done to use the designed method.

1. Use the method primarily as a preliminary design tool. The method enables a fast decision-making process, which is in particular useful when e.g., architects propose a number of layers to top-up. A further detailed design has to be performed by a structural engineer.
2. Perform a site inspection, to find out the used brickwork quality. If this is lower than the assumed design value of 2.81 N/mm^2 , a recalculation must be performed for the compression capacity in the stability wall. In any case, a (basic) stability verification is recommended as the stability situation could differ a bit from the used standard floorplan.
3. Verify if the actual council estate corresponds with the used assumptions in the structural calculations. Important aspects to reconsider are e.g., number of porches in a block, presence of a dilatation, used weights, wall sizes, contributing width, type of foundation, and storey height.
4. Interpret the intersection points in the material minimalization graphs not as strict separation between CLT or HSB, but more as indication where the tilting point by approximation occurs.
5. Apply a modular CLT unit in case the existing building will not thoroughly be renovated. In those cases, the residual lifespan of the building will be much lower compared to the technical lifespan of the extension. A modular unit that is fully reused in another building is then environmentally beneficial.

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A

Council estate

Appendix A will provide background information to chapter 5.

Content appendix A

- A.1** Schematization of council estates and archive inspection
- A.2** Additional information of non-traditional variants
- A.3** Additional information to exploratory calculations
- A.4** Extreme variants, based on the parameter study

A.1 Schematization and archive inspection

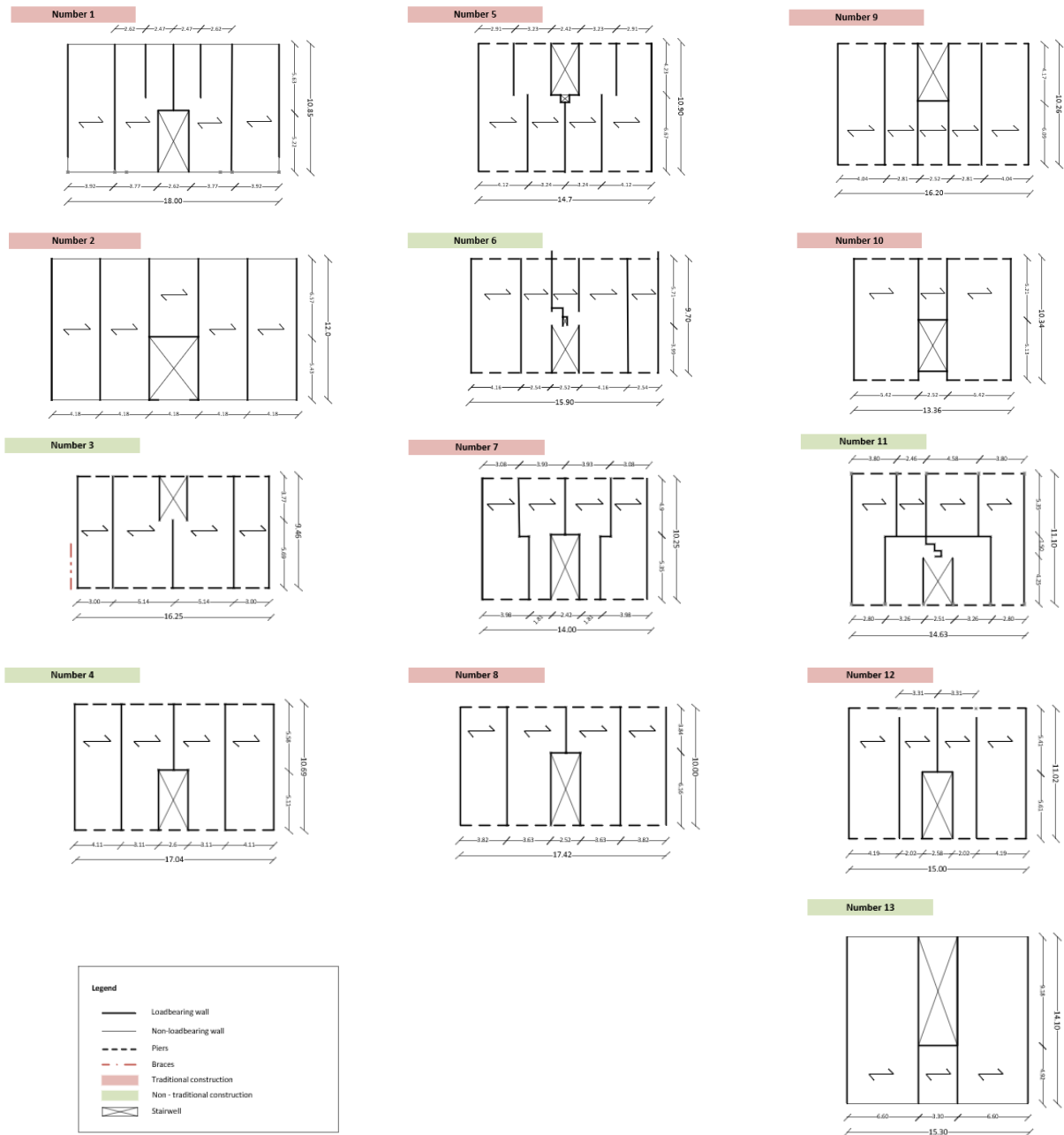


Figure A.1: Processed structural plans of archive research (Self-produced).

Archive inspection	1	2	3	4	5
Inspection number					
General aspects					
Location	Helena Mercierstraat, Amsterdam, Block 20	Willem van Weldammelaan, Amsterdam	Comeniusstraat, Amsterdam	Maassluisstraat, Amsterdam	Anske Lammingsstraat, Amsterdam
Construction year	1960	1961	1956	1959	1954
Construction method	Trad. Brickwork incl reinforced floors and cavity.	Trad. Brickwork incl. reinforced floors and cavity.	Non-traditional, Airey system	Non-traditional, Muwi system	Trad. Brickwork inc. Reinforced floor exc. cavity
Continuous flat [Yes/No]	Continuous (2 between dilatation)	Continuous (2 between dilatation)	Continuous (No dilatation)	Continuous (2 between dilatation)	Continuous (2 between dilatation)
Dimensions building					
Width building (2 Apartments) [m]	18.00 [m]	21.00 [m]	16.25 [m]	17.04 [m]	14.70 [m]
Depth building	10.85 [m]	12.00 [m]	9.46 [m] (Excl. Non-loadbearing facade)	10.69 [m]	10.90 [m]
Height building [m]	13.95 [m]	11.70 [m]	14.18 [m] excl. roof structure	14.08 [m]	12.50 [m]
Number of floors	4	3	4	4	2 with a attic on top
Gross floor height [m]	2.88 [m]	2.875 [m] - Top floor: 2.70 [m]	3.00 [m]	2.84 [m]	2.88 [m]
Standard floorplan [A, C, E, G]	E	E	E	E	E
Grid in X-direction (No exchange rooms: distance till middle wall)	3.92 - 2.62 - 2.47 - 2.47 - 2.62 - 3.92 [m]	4.18 [m]	3.00 - 5.14 - 5.14 - 3.00 [m]	4.11 - 4.41 - 4.41 - 4.11 [m]	4.12 - 3.24 - 3.24 - 4.12 [m]
Bay 1 (Dimension X-direction)	3.92 [m]	4.18 [m]	3.00 [m]	4.11 [m]	4.12 [m]
Bay 2 (Dimension X-direction)	5.09 [m]	4.18 [m]	5.13 [m]	4.41 [m]	3.24 [m]
Bay 3 (Dimension X-direction)	-	-	-	-	-
Stairwell					
Orientation Symmetrical (S) /Asymmetrical (A)	Symmetrical	Symmetrical	Symmetrical	Symmetrical	Symmetrical
Additional room [Yes/No]	No	Yes	No	No	No
Width stairwell	2.62 [m]	4.18 [m]	2.37 [m]	2.60 [m]	2.42 [m]
Depth stairwell	5.22 [m]	5.43 [m]	3.77 [m]	5.15 [m]	4
Floors					
Span direction [X/Y]	X	X	X	X	X
Material	Reinforced concrete	Hollow core slabs	Timber floor (2.5 cm); steel lattice girders	Prefab concrete floor system - Muwi system	Reinforced concrete floor (in-situ)
Thickness [mm]	171 (incl. finishing) [mm]	120 [mm]	450 [mm] Bottom 2 floors: Reinforced concrete (110 mm)	220 [mm]	170 (incl. finishing) [mm]
Loadbearing walls (Perpendicular to facade)					
1. Exterior walls (of apartment)					
Material exterior walls	Brickwork	Brickwork	Sintel (B2) Concrete blocks	Muwi-blocks	Brickwork
Thickness exterior walls [mm]	220 [mm]	200 [mm]	205 [mm]	210 [mm]	220 [mm]
2. Stairwell					
Material walls stairwell	Brickwork	Brickwork	Sintel Concrete blocks (Also holds for middle wall)	Muwi-blocks	Brickwork
Thickness walls stairwell [mm]	220 [mm]	220 [mm]	205 [mm]	210 [mm]	220 [mm]
3. Internal walls					
Distance from exterior wall	3.92 (+) 2.62 [m]	4.18 [m]	3.00 [m]	4.11 [m]	Bottom: 4.12 [m]; Top: 2.91 [m] (+) 3.23 [m]
Material interior walls	Brickwork	Brickwork	Brickwork	Muwi-blocks	Brickwork
Thickness interior walls [mm]	220 [mm]	200 [mm]	152 [mm]	210 [mm]	Bottom: 220 [mm]; Top: 110 [mm]
4. End facade					
Material	Brickwork	Brickwork	Brickwork	Muwi-block + Brickwork	Brickwork
Thickness	420 [mm]	380 [mm]	105 [mm]	360 [mm]	390 [mm]
Loadbearing walls (Parallel to facade)					
1. Stairwell					
Material wall stairwell	Brickwork	Brickwork	Sintel Concrete blocks	Muwi-blocks	Brickwork
Thickness wall stairwell [mm]	220 [mm]	270 [mm] (includes cavity)	205 [mm]	210 [mm]	220 [mm]
2. Interior wall					
Present [Yes/No]	No	No	No	No	Only on ground floor
Location Y direction [m]	-	-	-	-	5.37 (+) 1.76 [m]
Material	-	-	-	-	Brickwork
Thickness [mm]	-	-	-	-	110 [mm]
3. Facade					
Material	Brickwork	Non loadbearing	Brickwork - reinforced concrete pillar - concrete plate	Muwi-block + Brickwork	Brickwork
Thickness	220 [mm], largely not loadbearing	280 [mm]	105 - 125 - 40 [mm]	360 [mm]	280 [mm]
Foundation					
Type of foundation	Piled foundation with foundation strips	Piled foundation with foundation strips	Piled foundation, with reinforced concrete floor	Piled foundation, with reinforced concrete floor	Reinforced concrete strips with concrete piles
Material of foundation	Reinforced concrete	Reinforced concrete	Wooden piles	Reinforced concrete	Reinforced concrete
Cellar/Souterrain/No space below ground	No space below ground	No space below ground	No space below ground	No space below ground	No space below ground
Stability					
X-direction	Stability wall (Middle)	Stability walls (Middle)	Moment resisting connections / stability walls	Walls/ Moment resisting connections	Walls/ Moment resisting connections
Y-direction	Stability walls	Stability walls	Stability walls/ Bracings at the end	Stability walls	Stability walls
Roof structure					
Type [flat/sloped/pent roof]	Flat roof	Flat roof	Sloped roof	Flat roof	Sloped Roof
Structural system	Wooden roof structure	Wooden beams with stony material on top of it. Finishing: stuc on stone mesh	Wooden beams with 'Mastiek schroten'	Reinf. Concr. (20.5 cm) + hollow. slab (7cm)	Wooden roof structure
Balconies					
Present [Yes/No]	Yes	No	Yes	Yes	Yes
Structural system	Cantilevering in-situ floor	-	Prefab concrete floors, connected to lattice girder, additional supported by a reinforced concrete lintel	Cantilevering concrete prefab slabs	Cantilevering reinforced floors
Special aspects					
		[Small cubicles placed on stairwell] [Material definition of exterior walls hatching is unclear, but seems brickwork. Dimensions look like concrete]			Includes a stoney chimney Contains a staircase lift
Some drawings					

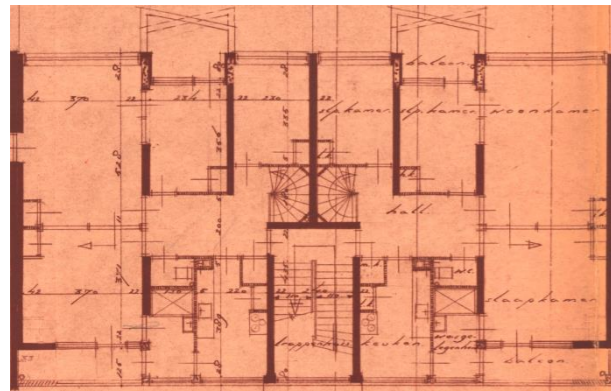
Figure A.2: Archive inspection number 1-5

Archive inspection	6	7	8	9	10
Inspection number					
General aspects					
Location	Petrus Blokhof, Amsterdam	Grasrijk, Amsterdam	Loevenstein, Amsterdam	Jan Voerstraat, Amsterdam	Saafingestraat, Amsterdam
Construction year	1954	1957	1961	1961	1961
Construction method	Non-traditional, comb. Bricks and BB-blocks	Trad. brickwork incl. reinforced floors excl. BB-blocks	Trad. brickwork incl. reinforced floors and cavity	Trad. brickwork incl. reinforced floors excl. Cavity	Trad. Brickwork with reinforced floors, excl. Cavity
Continuous flat [Yes/No]	Continuous (No dilatation)	Continuous (4 between dilatation)	Continuous (2 between dilatation)	Continuous (2 between dilatation)	Continuous (3 between dilatation)
Dimensions building					
Width building (2 Apartments) [m]	15.90 [m]	14.00 [m]	17.42 [m]	16.20 [m]	13.36 [m]
Depth building	9.70 [m]	10.25 [m]	10.00 [m]	10.26 [m]	10.34 [m]
Height building [m]	14 [m] excl. roof structure	17.30 [m] incl. roof structure	11.20 [m]	11.67 [m]	13.98 [m]
Number of floors	4 with an attic on top	4, with an attic on top	3	4	4
Gross floor height [m]	2.84 [m]	2.90 [m]	2.80 [m]	2.88 [m]	2.80 [m]
Standard floorplan [A, C, E, G]	E	G	E	E	G
Grid in X-direction (No exchange rooms: distance till middle wall)	4.16 - 2.54 - 2.52 - 4.16 - 2.54	3.08 - 3.93 - 3.93 - 3.08	3.82 - 4.89 - 4.89 - 3.82	4.04-2.81-2.52-2.81-4.04	5.42 - 2.52 - 5.42 [m]
Bay 1 (Dimension X-direction)	4.15 (Note right apartment is not mirrored)	3.08 [m]	3.82 [m]	4.04 [m]	5.42 [m]
Bay 2 (Dimension X-direction)	2.54 (Note right apartment is not mirrored)	3.93 [m]	4.89 [m]	2.81 [m]	-
Bay 3 (Dimension X-direction)	-	-	-	-	-
Stairwell					
Orientation Symmetrical (S) / Asymmetrical (A)	Symmetrical (Room division not!)	Symmetrical	Symmetrical	Symmetrical	Symmetrical
Additional room [Yes/No]	Yes	No	No	Yes	Yes
Width stairwell	2.52 [m]	2.42 [m]	2.52 [m]	2.52 [m]	2.52 [m]
Depth stairwell	3.99 [m]	5.35 [m]	6.16 [m]	4.77 [m]	5.13 [m]
Floors					
Span direction [X/Y]	X	X	X	X	X
Material	Reinforced concrete floor (in-situ)	Reinforced concrete floor (in-situ)	Reinforced concrete floor (in-situ)	Reinforced concrete floor (in-situ)	Reinforced concrete floor (in-situ)
Thickness [mm]	150 [mm]	100 [mm]	First floor: 220 [mm]; Other: 110 [mm]	110 [mm]	150 [mm]
Loadbearing walls (Perpendicular to facade)					
1. Exterior walls (of apartment)					
Material exterior walls	Partly brickwork / Partly BB blocks	Brickwork	Brickwork	Brickwork	Brickwork
Thickness exterior walls [mm]	Brickwork: 325 / BB Blocks: 215 [mm]	220 [mm]	220 [mm]	220 [mm]	220 [mm]
2. Stairwell					
Material walls stairwell	Brickwork	Brickwork	Brickwork	Brickwork	Brickwork
Thickness walls stairwell [mm]	215 [mm]	220 [mm]	220 [mm]	220 [mm]	220 [mm]
3. Internal walls					
Distance from exterior wall	4.16 [m]	Bottom: 3.98 [m]; Top: 3.08 (+) 3.93 [m]	Bottom: 3.82 [m]; Top: 3.82 (+) 4.89 [m]	4.04 [m]	5.42 [m]
Material interior walls	BB Blocks	brickwork	brickwork	Brickwork	Brickwork
Thickness interior walls [mm]	205 [mm]	110 [mm]	220 [mm]	110 [mm]	220 [mm]
4. End facade					
Material	Brickwork	brickwork	Brickwork	Brickwork	Brickwork
Thickness	390 [mm]	420 [mm]	290 [mm]	410 [mm]	350 [mm]
Loadbearing walls (Parallel to facade)					
1. Stairwell					
Material wall stairwell	Brickwork (Only last porch have it)	brickwork	Brickwork	Brickwork	Brickwork
Thickness wall stairwell [mm]	215 [mm]	220 [mm]	220 [mm]	220 [mm]	220 [mm]
2. Interior wall					
Present [Yes/No]	-	Yes	No	No	No
Location Y direction [m]	-	5.35 [m]	-	-	-
Material	-	brickwork	-	-	-
Thickness [mm]	-	220 [mm]	-	-	-
3. Facade					
Material	Brickwork	brickwork	Brickwork	Brickwork	Brickwork
Thickness	280 [mm]	280 [mm]	340 [mm]	280 [mm]	280 [mm]
Foundation					
Type of foundation	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles
Material of foundation	Reinforced concrete	Reinforced concrete	Reinforced concrete	Reinforced concrete	Reinforced concrete
Cellar/Souterrain/No space below ground	No space below ground	No space below ground	No space below ground	Souterrain	Souterrain
Stability					
X-direction	Stability walls	Stability walls	Stability walls	Stability walls	Stability walls (penants)
Y-direction	Stability walls	Stability walls	Stability walls	Stability walls	Stability walls
Roof structure					
Type [flat/sloped/peet roof]	Sloped roof	Sloped roof	Flat roof	Flat roof	Flat roof
Structural system	Wooden roof structure	Wooden roof structure	Reinf. Concrete (15 cm) + hollowc. slab (7 cm)	Wooden beams + Hollow core slab	Reinf. Concr. (12.5cm) + hollowc. slab (7cm)
Balconies					
Present [Yes/No]	Yes	Yes	Yes, very small (30 cm)	Yes	Yes
Structural system	Cantilevering reinforced floors + longer penant	Cantilevering reinforced floors	Prefab concrete , connected to in-situ floor	In-situ reinforced floor, connected to floor	In-situ reinforced floor, connected to floor
Special aspects					
	Some thicker penants made of brickwork Contains a staircase lift	Contains chimney	Contains chimney	Contains chimneys	Contains chimney

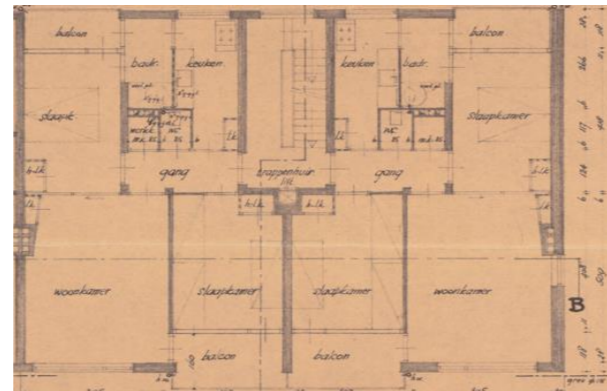
Figure A.3: Archive inspection number 6-10.

Archive inspection				
Inspection number	11	12	13	
General aspects				
Location	Schoonboomstraat, Amsterdam	Honselersdijkstraat, Amsterdam	Steenderenstraat, Amsterdam	
Construction year	1962	1959	1981	
Construction method	Non-trad. Airey system	Trad. Brickwork with reinforced floor, excl. Cavity	Reinforced concrete (in-situ) with prefab floors	
Continuous flat [Yes/No]	Continuous (No dilatation)	Continuous (2 between dilatation)	Continuous (2 between dilatation)	
Dimensions building				
Width building (2 Apartments) [m]	14.63 [m]	15.00 [m]	15.30 [m]	
Depth building	11.10 [m]	11.02 [m]	14.10 [m]	
Height building [m]	14.00 [m]	17.40 [m], incl. roof structure	10.80 [m]	
Number of floors	4	4 with an attic on the top	3	
Gross floor height [m]	2.80 [m]; Top floor 2.65 [m]	2.85 [m]	2.70 [m]	
Standard floorplan [A, C, E, G]	E	E	C	
Grid in X-direction (No exchange rooms: distance till middle wall)	3.80 - 2.46 - 4.58 - 3.80 [m]	4.19 - 3.31 - 3.31 - 4.19 [m]	6.00 - 3.30 - 6.00 [m]	
Bay 1 (Dimension X-direction)	3.80 [m]	4.19 [m]	6.00 [m]	
Bay 2 (Dimension X-direction)	2.46 [m]	3.31 [m]	3.30 [m]	
Bay 3 (Dimension X-direction)	-	-	-	
Stairwell				
Orientation Symmetrical (S) / Asymmetrical (A)	Symmetrical	Symmetrical	Symmetrical	
Additional room [Yes/No]	Yes	No	Yes	
Width stairwell	2.51 [m]	2.58 [m]	3.30 [m]	
Depth stairwell	4.25 [m]	5.61 [m]	9.18 [m]	
Floors				
Span direction [X/Y]				
Material	Reinforced concrete floor (in-situ)	Reinforced concrete floor (in-situ)	Prefab system floor concrete, prestressed	
Thickness [mm]	200 [mm]	145 [mm]	245 [mm]	
Loadbearing walls (Perpendicular to facade)				
1. Exterior walls (of apartment)				
Material exterior walls	B2 blocks (Bottom totally brickwork)	Brickwork	Reinforced concrete	
Thickness exterior walls [mm]	205 [mm]	220 [mm]	200 [mm]	
2. Stairwell				
Material walls stairwell	Sintel (B2) blocks	Brickwork	Reinforced concrete	
Thickness walls stairwell [mm]	205 [mm]	220 [mm]	200 [mm]	
3. Internal walls				
Distance from exterior wall	Bottom: 2.80; Top: 3.80 (+) 2.46 [m]	4.19 [m]	Not present	
Material interior walls	Sand lime stone	Brickwork	-	
Thickness interior walls [mm]	Top layers: 105; Bottom 215 [mm]	220 [mm]	-	
4. End facade				
Material	k - Isolation plate - Reinforced concrete pillar - Concrete plates	Brickwork	Reinforced concrete	
Thickness	70 - 30 - 125 - 40 [mm]	420 [mm]	200 [mm]	
Loadbearing walls (Parallel to facade)				
1. Stairwell				
Material wall stairwell	Brickwork	Brickwork	Reinforced concrete	
Thickness wall stairwell [mm]	215 [mm]	220 [mm]	200 [mm]	
2. Interior wall				
Present [Yes/No]	Yes	No	No	
Location Y direction [m]	5.75 [m]	-	-	
Material	Brickwork	-	-	
Thickness [mm]	105 [mm]	-	-	
3. Facade				
Material	k - Isolation plate - Reinforced concrete pillar - Concrete plates	Brickwork	Non-loadbearing stone + cavity + concrete	
Thickness	70 - 30 - 125 - 40 [mm]	340 [mm]	-	
Foundation				
Type of foundation	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles	Reinforced concrete strips with concrete piles	
Material of foundation	Reinforced concrete	Reinforced concrete	Reinforced concrete	
Cellar/Souterrain/No space below ground	No space below ground	No space below ground	No space below ground	
Stability				
X-direction	Moment connections columns-floors and stability wall	Stability walls	Stability walls	
Y-direction	Stability walls	Stability walls	Stability walls	
Roof structure				
Type [flat/sloped/pent roof]	Flat roof	Sloped roof	Flat roof	
Structural system	Reinf. Concr. + hollowc. slab (7.5 cm)	Wooden roof structure	Prefab floor+ cladding	
Balconies				
Present [Yes/No]	Yes	Yes, Small ones	Yes	
Structural system	In-situ reinforced floor, connected to floor	In-situ reinforced floor, connected to floor	Prefab plate, connected to floor, with cavity	
Special aspects				
	Contains columns in facade made of reinf. Concrete: 250 x 250 [mm]	Contains chimney		

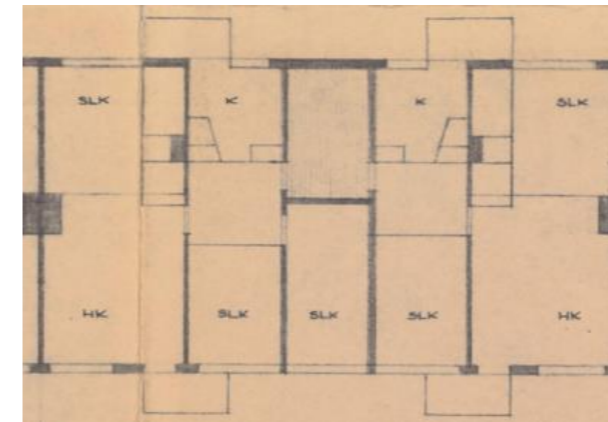
Figure A.4: Archive inspection number 11-13.



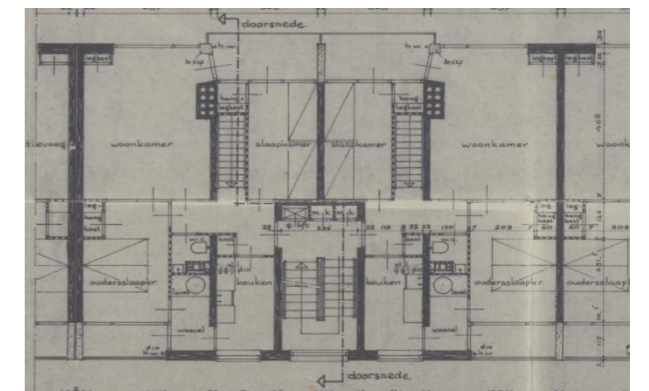
Helena Mercierstraat (1)



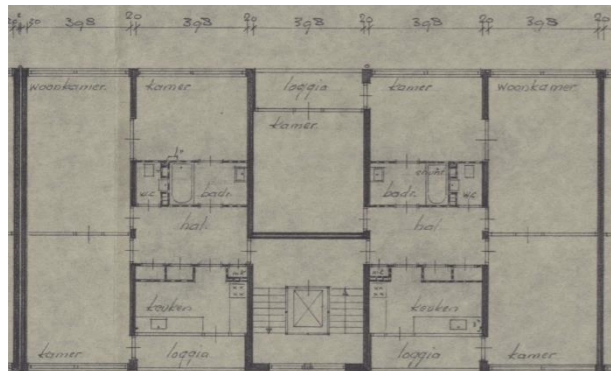
Anske Iammingastraat (5)



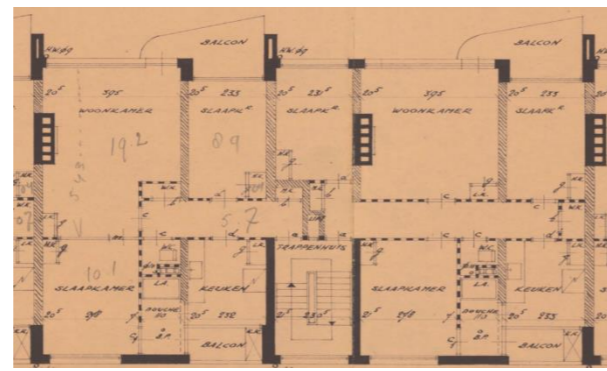
Jan Voerstraat (9)



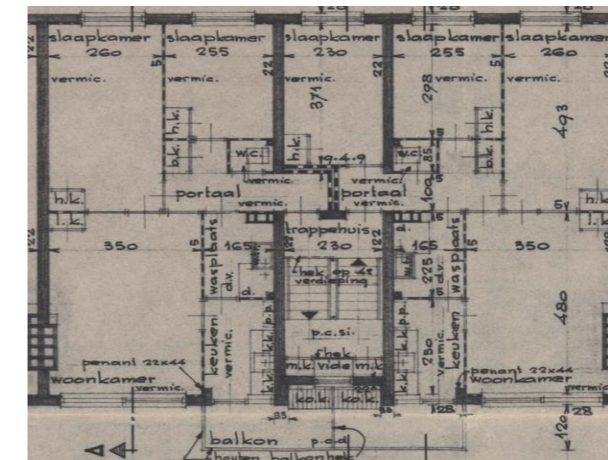
Honselersdijk (12)



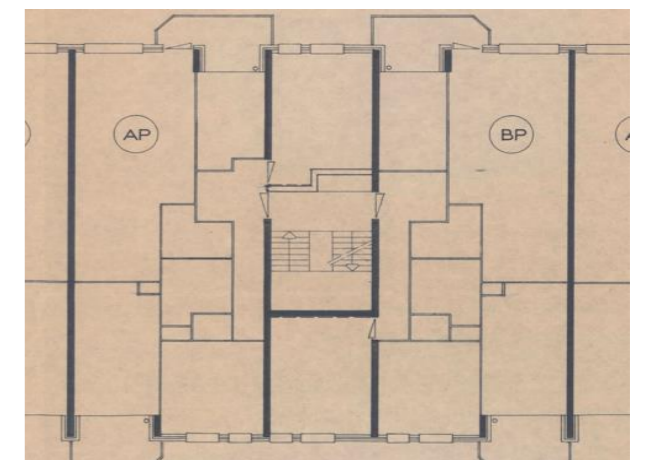
Willem van Weldammelaan (2)



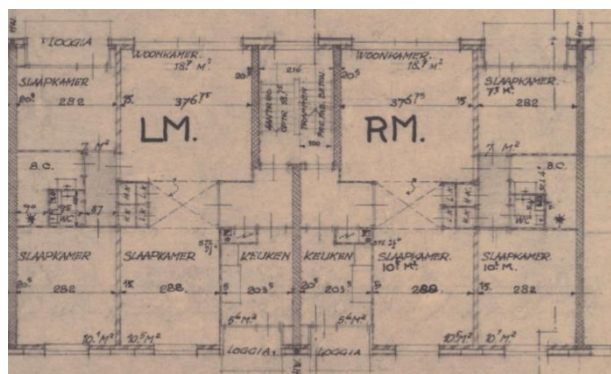
Petrus Blokhof (6)



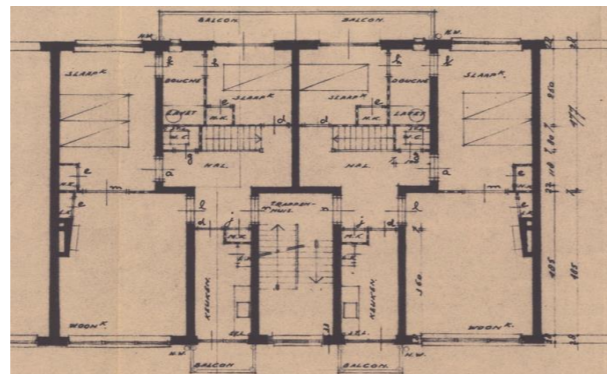
Saftingestraat (10)



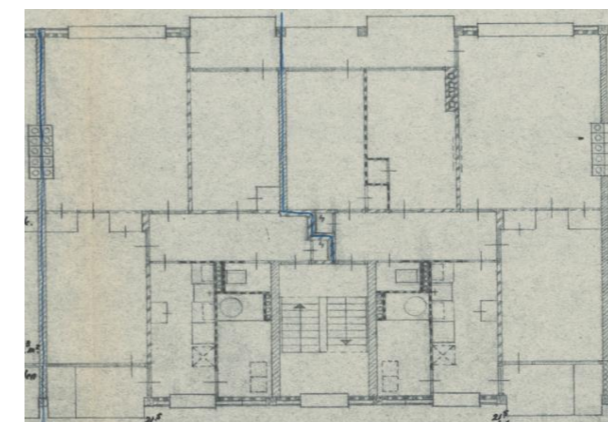
Steenderenstraat (13)



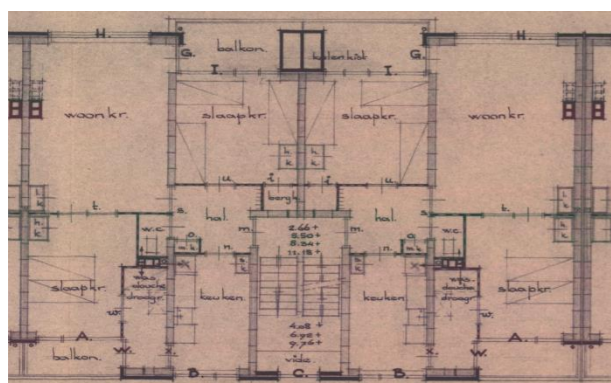
Comeniusstraat (3)



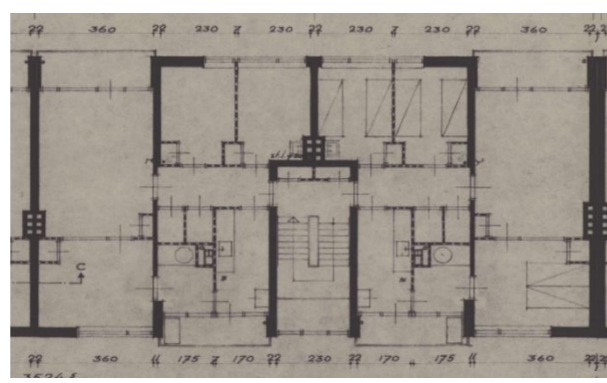
Grasrijk (7)



Schoonboomstraat (11)



Maassluisstraat (4)



Loevenstein (8)

Figure A.5: Overview floorplans inspected council estates (Archive municipality Amsterdam, 2022).

A.2 Explanation of non-traditional variants

This section will provide some background information about the structural methods of the non-traditional variants. Non-traditional construction methods show more variety compared to traditional construction. Often the ground floor was constructed of brickwork, while the layers above were made of various products (Muwi-, Sintel-, or BB-blocks). The archive inspection contains, two Airey and one Muwi, precast stack, and cast in-situ variant. Remarkable is the variation in wall thickness, material composition and stability system. As a result, the distinct variants will be described independently.

Muwi system

This system makes use of Muwi blocks to create the walls. The Maassluisstraat has loadbearing outer façades, constructed out of Muwi-blocks. The interior walls were also constructed with these blocks. The 210 mm hollow blocks are dry stacked, afterwards filled with mortar. The walls in cross direction forms the stiffness in that direction. In longitudinal direction, the façade walls and the stairwell wall provide stiffness.

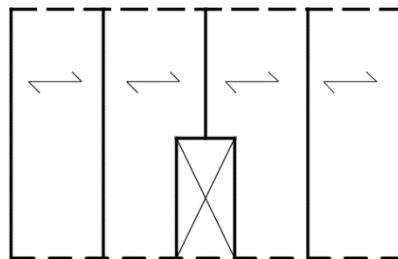


Figure A.6: Schematic representation of Maassluisstraat (Self-produced).

Airey systems

Platform 31 (2013) states that approximately five thousand of Airey houses were built in Amsterdam. Remarkable is the build-up of the outer walls. Both inspected houses have a build-up from inner to outer of brickwork, reinforced concrete styles, finished with small concrete plates. Platform 31 (2013) states that these façades are load bearing. The thickness of the reinforced concrete styles has a cross-section of 62.5 x 125 mm. The small concrete plates have dimensions of 625 x 375 x 40 mm. The brickwork thickness varies within the two variants between 70 and 105 mm.

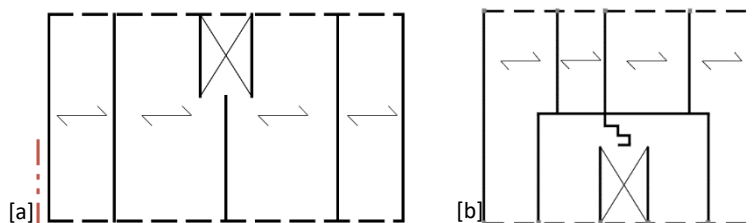


Figure A.7: Schematic representation of [a] Comeniusstraat; [b] Schoonboomstraat (Self-produced).

The Airey system makes use of Sintel (B2) blocks for the stairwell and apartment separation walls. The Schoonboomstraat additional internal loadbearing walls were constructed of 105 mm thick brickwork, instead of Sintel blocks. The cross stability has been realized by the cross-orientated walls. Additional bracings were placed at the Comeniusstraat in the end façades.

The Schoonboomstraat has additional reinforced columns in the façade. These form moment resisting connections with the floors, which provides stiffness in longitudinal direction (also expected for Comeniusstraat). Additionally longitudinal-orientated loadbearing (façade) walls provide also stiffness. Figure A.7 shows a structural overview of the variants.

Precast stack building: BB-Blocks

The precast stack building (Petrus Blokhof) has a lot of similarities with the traditional building. The stability in both directions has been realized by stability walls. The outer walls were partly constructed of brickwork (325 mm) and partly of concrete BB-blocks (215 mm). The walls around the stairwell are realized in 215 mm thick brickwork. Note that no wall in longitudinal direction at the stairwell exist. The interior walls were constructed of 205 mm thick BB-blocks. The stability in longitudinal direction has been realized by the 280 mm thick façade wall.

In the end-porches, the stairwell has a 215 mm thick brickwork longitudinal-orientated wall. The end façade walls in cross direction are 390 mm thick brickwork walls. Note that the building also has a small elevator. This is only for luggage. It is too small to function as a stabilizing element. Figure A.8 provides an overview of the precast stack building.

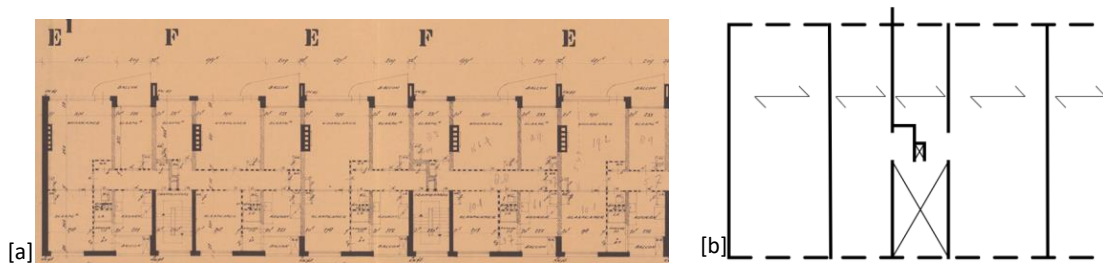


Figure A.8: [a] Petrus Blokhof floorplan (Archive municipality Amsterdam, 2022);
[b] Schematic representation of Petrus Blokhof (Self-produced).

Cast in-situ

The cast-in-situ variant (Steenderenstraat) has been using a similar strategy for the stability as the traditional method. In both directions stability walls were constructed. The floors are supported on top of the walls. All loadbearing walls have a thickness of 220 mm. In cross direction, the walls are located at both ends of the unit and at the stairwell (includes additional room). No additional interior walls exist.

A.3 Exploratory calculations on vertical load transfer

Appendix A.3 will provide some additional information about the exploratory calculations. The exploratory calculations for vertical load will be performed on two brickwork, a concrete and a Muwi system.

A.3.1 Defined loads

The applied loading is provided in section 5.5.1.

A.3.2 Traditional brickwork

All buildings in the archive were constructed in 220 mm thick loadbearing walls. The thickness of these walls was historically based on rules of thumb. In practise, no brickwork with a compressive strength lower than 5 N/mm² was used. Resultingly, load bearing walls had minimum thicknesses. Houses till three storeys must have a minimum wall thickness of 19 centimetres, while the cellar walls should be 29 centimetres (Belgische baksteenfederatie, 2008).

Saaftingestraat is chosen as normative structure for a brickwork variant (inspection number 10). This building includes four storeys with reinforced concrete floors and roof. The connected floor length equals 7.94 m. The roof build-up of these brickwork buildings has been traditionally constructed in timber or concrete. Variant 10 is normative due to its reinforced concrete slabs [Fig. A.9]. The permanent load definition for both floor and roof structure is provided in table A.1. The safety factors are applied according to normal design conditions, e.g., equation 6.1 and 6.2 (NEN 1990).

Table A.1: Permanent load definition for traditional construction; roof [left] and floor [right].

Roof build-up	Thickness/ Weight	Load [kN/m ²]	Floor build-up	Thickness / Weight	Load [kN/m ²]
Gravel	30 mm	0.48	Topping	40 kg/m ²	0.4
Schewil insulation plate	70 mm	0.2	Reinforced concrete floor	150 mm	3.6
Reinforced concrete	125 mm	3	Ceiling	20 kg/m ²	0.2
Ceiling	20 kg/m ²	0.2	Non-loadbearing walls	-	0.5
	Total	3.88		Total	4.7

Within the distinct variants, a lot of spread can be observed. Especially, the non-traditionally built council estates have a larger floor span compared to the traditionally ones. Traditionally constructed flats with larger spans, compared to variant 10, exist in the archive. These variants have less floors, what reduces the total vertical load. As it is incorrect to verify a variant with different dimensions, initially variant 10 is verified with applied dimensions. Afterwards larger spans are applied to generate insight in the effect the span size.

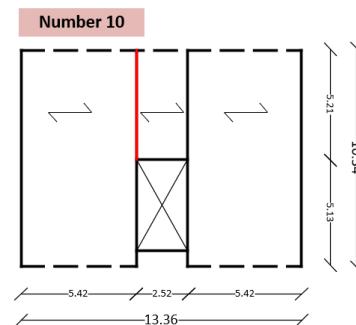


Figure A.9: Normative variant, constructed out of brickwork (Self-produced).

The vertical load causes a compressive load on the walls. Oorschot (2021) state that most of the walls were constructed out of sand-lime brick or traditional brickwork. The compression capacity of traditional brickwork is much lower compared to sand-lime brick. The type of mortar and the quality of the brickwork determines the strength of the brickwork (NPR 9096). The lowest characteristic value for the combined compressive strength equals 2.55 N/mm², while the strongest brickwork has a compressive strength of 11.58 N/mm². This characteristic strength should be divided by 1.7, to result in the design strength of brickwork.

Four distinct variants are considered for traditional construction.

- Variant 3: Brickwork applied on Airey variant (extreme spans);
- Variant 7: Traditional variant. Higher floors have lower wall thickness;
- Variant 8: Originally constructed in 3 storeys. Calculated according to 4 storeys;
- Variant 10: Normative brickwork variant.

Figure A.9 marks the normative wall for variant 10 on the ground floor. The normal stress equals 1.12 N/mm², resulting in a unity check of 0.75 (low quality brickwork). Figure A.10 provides an overview of the verified variants in brickwork. It appears that with the structural floorplan of the Airey variant, the walls still have enough capacity. Variant 7 is evaluated with the thinner walls on the second floor. This was not normative (UC = 0.69). For variant 10, it is possible to add another layer out of brickwork. The unity check becomes in that case 0.92.

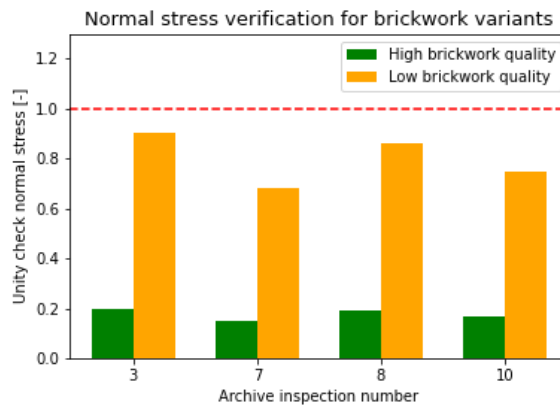


Figure A.10: Verified variants in brickwork (Self-produced).

A.3.3 Non-traditional Muwi

The archive sample contains only one Muwi variant. This variant (inspection number 4) has not the normative span of the distinct non-traditional variants. The Muwi-system is applied on the normative plan of variant 3 (extreme span). Properties about roof structure and flooring system are applied according to variant 4. Detailed information is provided in the archive about the weight of the roof and floor build-up [Table A.2] The walls were constructed out of 210 mm thick Muwi blocks. CFL of variant 3 to the walls equals 10.28 m.

Table A.2: Permanent weights of roof [left] and floor [right] (Muwi system).

Roof build-up	Weight [kN/m ²]	Floor build-up	Weight [kN/m ²]
Prefab system floor	1.88	Prefab system floor	1.88
Roof finishing	0.92	Roof finishing	0.40
Ceiling	0.22	Ceiling	0.22
Total	3.02	Total	2.50

The compressive strength of Muwi-blocks is estimated based on hollow concrete blocks. A weight of 2400 kg/m^3 is used for the weight of the Muwi-blocks. The actual weight will be a bit lower (conservative). The characteristic compressive strength equals 6 N/mm^2 (Makobeton, 2014). Applying a material factor of 1.5, results in a design compressive strength of 4 N/mm^2 . The acting compressive stress equals 1.1 N/mm^2 . That results in a unity check of 0.28.

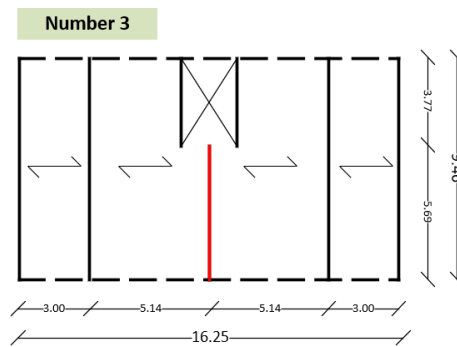


Figure A.11: Normative wall, variant 3 (Self-produced).

A.3.4 Non-traditional concrete

One variant was constructed in concrete (inspection number 13: Steenderenstraat). The floor system consists of a system floor with a self-weight of 220 kg/m^2 . As council estates were often constructed with an in-situ concrete floor, this is also verified. The CFL of variant 13 equals 9.90 m. The walls are 200 mm thick, constructed in concrete. The in-situ floor build-up is according to table A.1 [right]. The prefabricated floor lowers the permanent floor weight from 4.7 kN/m^2 to 3.3 kN/m^2 . The in-situ variant has a traditional roof build-up [Table A.1], while the prefab variant uses a Muwi-system [Table A.2].

Both, the actual floorplan (insp. number 13) as well as the extreme floorplan (insp. number 3) are evaluated. Considering low-strength concrete (C12/15), the design compressive capacity equals 8.00 N/mm^2 . In figure A.12, the resulting unity checks are provided. The concrete variants are strongly over-designed considering the vertical load transfer. Even with low-strength concrete, the unity checks are still lower than 0.5.

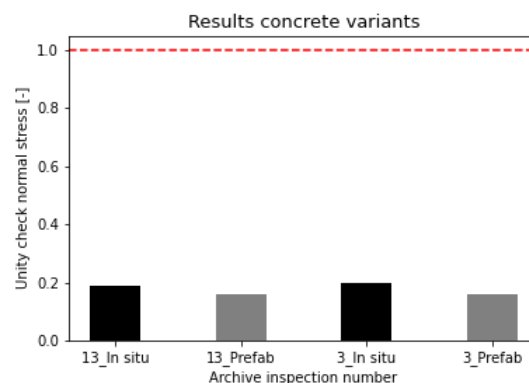


Figure A.12: Results of concrete variants. Concrete quality: C12/15 (Self-produced).

A.4 Extreme variants

Section 5.6 provides a parameter study. Based on this study, the most (un)favourable variants will be defined, both for vertical load and stability. Table A.3 will provide an overview of the most favourable and unfavourable situation for vertical load transfer. Table A.4 will provide it for the stability.

Table A.3: (Un)favourable situation vertical load transfer.

Parameter	Most favourable	Most unfavourable
Number of storeys	3	4
Materialization floors	Timber	Concrete
Connected floor length	6.70 m	10.28 m
Wall opening percentage	15%	31%
Material walls	Concrete	Brickwork

Table A.4: (Un)favourable situation stability.

Parameter	Most favourable	Most unfavourable
Number of storeys	3	4
Materialization floors	Concrete	Timber
Additional room	Yes	No
Normative floor length	8.25 m	2.67 m
Depth / stab. Wall length	2.87 [-]	4.50 [-]

The results of table A.3 and A.4 are visualized in figure A.13.

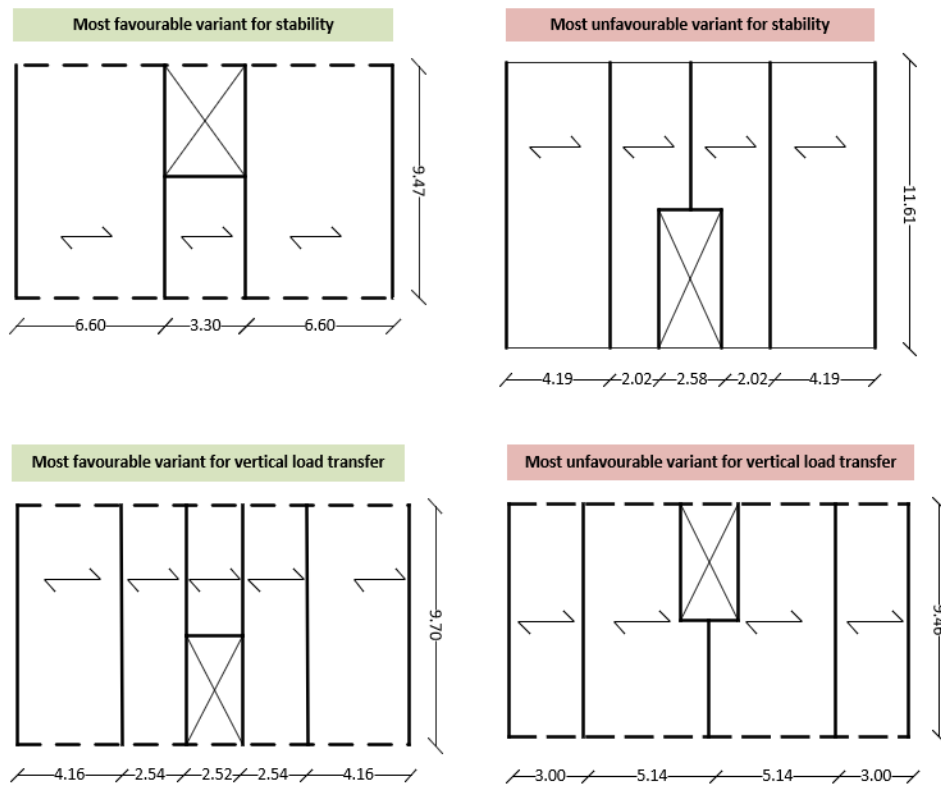


Figure A.13: (Un)favourable fictive variants (Self-produced).

B

Top-up

Appendix B will provide background information to chapter 6.

Content appendix B

- B.1** Interviews
- B.2** Case studies
- B.3** HSB- and CLT construction
- B.4** Connections for modules
- B.5** Strengthening methods for stability
- B.6** Example calculation for vertical overcapacity
- B.7** Parameter study vertical overcapacity (Foundation normative)
- B.8** Top-up possibilities for archive buildings

B.1 Interviews

This chapter will contain two interviews about the case studies [M. Freriks; M. Tromp], and one about modular construction [T. Jonker].

B.1.1 Interview 1 – M. Freriks

General information

Company: Van Rossum raadgevend ingenieurs B.V. (Almere)
 Function: Project leader/ Structural engineer
 Building: Parelhof Heerhugowaard (final design in process)

The interview with M. Freriks is a brief explanation of the design of the Parelhof. The author had a number of conversations with Freriks about the structural design of the top-up. The questions are answered by Mark as a summarization of the performed conversations. This is used for the case study.

Q1: The Parelhof will be partly constructed in timber frame construction and partly in combination steel and timber. What is the structural reasoning behind this applied system?

Answer: That is related to the existing structure of the building. Part of the structure, houses, consists of concrete structural walls, spaced 3.9 m. Such a span is easily constructed by a timber beam layer. The other part, historically office space, has as construction a columns structure, spaced 6.6 m. Thereby are the floors point supported. A steel structure is needed to transfer the forces directly to the underlying columns.

Q2: Were the number of layers that would be top-upped discussed during the design stage? Were there structural limits?

Answer: No, the design of the architect initially consists of two layers top-up. Therefore, we performed a feasibility study to check if it was possible. Two layers was the maximum that could be top-upped, with minimal structural adaptations of the existing structure.

Q3: How is the horizontal load on the stability walls, perpendicular to the structural walls, transferred to the foundation?

Answer: Therefore, a distinction is made between block B and C.

At block B:

In the top-up, stability walls will be constructed. These transfer the wind force to the underlying structure. The existing construction will be linked via block C to the new structure.

At block C:

The new structure at C will take care for the stability of the top-up and the underlying structure.

Q4: How will the vertical load transfer be realized?

Answer: The vertical load will be transferred to the load-bearing points in the underlying structure.

Q5: Do you have strengthened the existing structure? In case yes, which principles do you have used?

Answer: No, that is not needed. With NEN 8700, it is demonstrated that the forces of the vertical load transfer, including top-up, due to lower safety factors of the NEN 8700, are below the capacity of the structure. For stability, see my reaction of question 3.

Q6: Were there some remarkable points you run into, during the project?

Answer: No, there were no remarkable points.

Q7: Has sustainability played a role in the structural design?

Answer: No, the starting point was to top up as light as possible.

Q8: Could you as a structural engineer in general indicate where the difficulties and bottlenecks are in topping up?

Answer: The verification of the stability of the existing structure and the verification of the additional vertical load to the foundation.

B.1.2 Interview 2 – M. Tromp**General information**

Company: Van Rossum raadgevend ingenieurs B.V. (Amsterdam)
 Function: Project leader/ Structural engineer
 Building: De Boelelaan Amsterdam; Wibautstraat Amsterdam

M. Tromp has performed the structural design of both inspected gallery flats. The interview is separated in these two distinct projects. Finally, a more general question is asked. Tromp added some additional structural descriptions and drawings to the interview, which are used in the case studies.

Part 1: Project specific questions about de Boelelaan: ‘De Boel’**Q1: Could you describe the existing structure?**

Answer: See added description and drawings. The substructure consists of concrete floors, beams, and columns. The residential floors contain hollow core slabs and prefab wall elements/blocks. The stability was realized by walls and stairwells. The vertical load is transferred via the structural housing separation walls towards the beams and columns. Or walls go directly to the foundation.

Q2: What material did you use for the top-up?

Answer: The top-up was constructed in light-weight timber. Where needed was steel applied.

Q3: Has there been discussion about the number of layers that would be top-upped? In case yes, was the structure normative in the decision?

Answer: Yes, structural was the capacity of the existing structure normative, without strengthening measures for foundation.

Q4.1: How did you realize the stability of the top-up?

Answer: In both wind directions timber HSB-walls were applied with in both end houses additional steel portals.

Q4.2 Were there some special structural aspects you applied for stability?

Answer: For stability, the houses are horizontal linked to divide the wind load in longitudinal direction.

Q5: How did you realize the vertical load transfer?

Answer: See also added drawing. The HSB-walls and floors are positioned on a steel frame, which transfers the load to the existing load bearing walls.

Q6: Was reinforcing of the current structure needed (for the vertical load)? In case yes, which principles have you applied?

Answer: Only the roof structure has been strengthened with a compression layer of light-weight concrete, to transfer the current roof structure towards a terrace- and residential floor.

Q7: Did sustainability aspects influence the structural design? In case yes, how?

Answer: Not on purpose. It is very sustainable to realize a top-up within the structural boundaries of the existing structure. A reinforcement of the foundation/substructure was not needed.

Q8: What was the largest challenge in the project?

Answer: The esthetical requirements of the architect. Further was part of the building still in use, and not accessible for inspection. That was needed to make the design definitive.

Part 2: [Project specific questions about Wibautstraat: 'Jan Bommerhuis'](#)

Q9: Could you describe the existing structure?

Answer: See also the added structural description. As material, concrete floors, beams/columns and walls are used. For stability: on each grid were concrete portals present and also walls of elevators and stairwells. The vertical load is especially transferred by the column-beam structure, on which the existing concrete floors were located.

Q10: What material did you use for the top-up?

Answer: The top-up was constructed in composite floors of steel and concrete, located on a steel structure. The houses on all layers were provided with balconies of micro-concrete, carried by a steel structure.

Q11: Has there been discussion about the number of layers that would be top-upped? In case yes, was the structure normative in the decision?

Answer: Yes, structurally the capacity of the existing structure was normative without strengthening the foundation. Of course, also costs and revenues are related to this. The existing roof build-up has been demolished and replaced for two additional residential layers.

Q12.1: How did you realize the stability of the top-up?

Answer: In both directions steel trusses have been applied.

Q12.2: Where there some special structural aspects you applied for stability?

Answer: As the building became higher, the wind load increased drastically. The existing portal construction are insufficiently strong and stiff to take up the additional wind loads. For this reason, steel trusses were placed on various grids between the concrete framework on several floors.

Q13: How did you realize the vertical load transfer?

Answer: See also the added drawings. The new columns transfer as much as possible towards the existing columns and walls. In case it was not possible, new columns are placed on a steel truss to transfer the loads subsequently towards the existing columns.

Q14: Was reinforcing of the current structure needed? In case yes, which principles have you applied?

Answer: The existing foundation was not strengthened. As indicated above, the stability has been improved with steel trusses. Additionally current floor dilatations, needed at the time for shrinkage, now closed. Resultingly more walls and frames work together to take up the wind load.

Q15: Did sustainability aspects influence the structural design? In case yes, how?

Answer: We tried to minimize the application of unnecessary material. It is sustainable to realize a top-up within the structural boundaries of the existing building. A reinforcement of the foundation/ substructure was not needed. The new concrete floors were, if I am correct, constructed with a percentage reused concrete granulate.

Q16: What was the largest challenge in the project?

Answer: A part of the building was still in use and not accessible for inspection, needed for the finishing of the design. Especially the quality of the concrete portals was difficult to estimate, and thereby the execution of the wind trusses with glue anchors. The latter part of the construction was a large challenge to design, but also to execute due to the present reinforcement in the construction.

Another large challenge was the load transfer of the steel columns (of the new balconies) on the new steel frame above the existing prefab roof structure of the low-rise. Holes needed to be realized in the existing light-weight concrete roof for the correct transfer on the underlying existing columns. That was a point what was difficult to design based on the available archival drawings. After measurements and various viewings, a suitable solution was conceived for this.

Part 3: General**Q17: Could you as constructor indicate what the difficulties/ bottlenecks are, considering topping up in general.**

Answer: When structural information of the existing construction is not available, or just partly, conservative assumptions must be made. That provides limitations for the possibilities. So, it is important that a complete as possible archive is available. Requesting archival data by municipalities is one of the first steps we perform for such a design. This is followed by puzzling: sometimes a building has already undergone previous renovations and must be examined which pieces are still applicable.

The existing weights are determined globally from drawings and calculations, to define the overcapacity.

Top-ups are often performed in light-weight constructions as wood, steel, and steel-concrete composite floors, which transfer to the existing loadbearing structure of walls or columns.

Constructively, the capacity of the existing structure forms a limitation on a certain moment. Thereby is in the reinforcement calculation often more capacity present compared to for example a pile foundation. This is the result of the applied safety factors. In topping-up, 'weight-neutral construction' has the preference: removing old finishings or interior walls and a new top-up in return. With finally under the line no increase of loads on the foundation. In that case, new verifications of the piles, reinforcement and settlements are not needed.

In case of old, wooden pile foundations, a foundation investigation is always needed to determine the residual life of the foundation. Wooden pile foundations can be limited in overcapacity due to deterioration of settlements.

It becomes also difficult when the existing building consists of a relative light-weight structure (steel/timber). The vertical and horizontal load increase are relatively higher compared to a heavy or high building (brickwork/ concrete).

At certain renovations, a load increase on the foundation cannot be avoided, and a verification with ground penetration tests or reinforcing of piles or substructure is an option. However, these are costly and elaborate adaptation. Not every client thinks the investment is worth it. Probably also due to current renters.

Shortly, for most buildings a top-up with or without reinforcement is possible dependent of the existing construction, costs, and design.

B.1.3 Interview 3 – T. Jonker

General information

Company: Van Rossum raadgevend ingenieurs B.V. (Almere)
 Function: Structural engineer
 Research: T. Jonker is developing an efficient method to examine structurally modular construction.

Q1: What are the advantages of a modular top-up?

Answer: The biggest advantage of modular construction is cost savings by reducing construction time. This is mainly because the modules can be made simultaneously and also in parallel during other preparatory work on site. Failure costs are also reduced because the modular units are made in a controlled environment.

Q2: You mentioned that there is a wide variation in the definition of modular construction. Could you indicate what mainly differs?

Answer: In addition to differences in material use (from concrete, to steel and wood, or a combination of these), there are a number of basic modular variants.

1. *Self-stabilizing modules. In this case, the modules themselves provide stability. These buildings consist entirely of modules.*
2. *Externally stabilized. This requires a stability device such as a concrete core, against which the modules are placed.*

Q3: Are modules always made in standard dimension, or are they also applied in varying dimensions (in one project)?

Answer: Here again, there are two design principles.

1. *A design driven from the modules. Where first the modules are worked out, then a building or several buildings are designed with these modules. The focus is on the production of the modules, how to make them as efficient as possible. Often these modules have fixed sizes.*
2. *Modules driven by the design. Here a building is designed and during this process it is decided to build the building in a modular way. In this case, the modules are made based on the design and often specifically for the particular project. Here the modules may have different dimensions among themselves.*

Q4: Could you describe for me the basic principle of a module?

Answer: Basically, we speak of a module when it is a 3D-volume, which is prefabricated at another location, and consequently transported to the construction site. Generally, a module consists of a floor, walls, and a ceiling. Finishes range from raw casco, to completely finished with the electrical outlets and toilet paper holders mounted.

Q5: Could you indicate what the main variants are in terms of modules in wood?

Answer: With wood modules, a full wall supported module is often chosen over a column module (although the latter is possible). A full-wall supported module consists of load-bearing walls, a floor and usually a ceiling. Here, stability walls can also be included in the module, but not necessarily. The load-bearing components usually consist of CLT, with HSB-walls in between.

Q6: How is stability controlled for single units?

Answer: As stated earlier, there are two ways to stabilize. With an external stabilizing core, the modules will have to be coupled horizontally, so they can form a disk to transfer the horizontal forces to the core.

With self-stabilizing modules, longitudinal stability is obtained from the load-bearing long walls. In the cross-direction, a "short" stability wall is then added.

Q7: We talked about a possible intermediate layer on the council estates, on which the modules could be placed. Could you indicate where the complexities are in the case of such a sub-layer?

Answer: The tricky thing about an intermediate layer is that you always get a stagger in bearing lines. You will have to calculate the intermediate layer based on this. Furthermore, a self-stabilizing module will create a tension/pressure force for stability. You will have to look carefully where this will occur and how you can absorb it in the intermediate layer.

If a module system with an external stability device is chosen, you will have to consider how to link the diaphragm action of the modules to the stability core. An important difference between the two principles is that with the latter, you concentrate the tensile/pressure forces in the stability device, whereas with the former variant you spread this out throughout the building.

Q8: During my research, I came across two types of wood construction: CLT and HSB. Which is particularly used for modular units and why?

Answer: In general, wood modules are made of CLT or Glulam-timber. The advantage of CLT is its good shear strength and easy processing compared to HSB. For one or two storeys, HSB modules are sometimes used, because they are lighter compared to a CLT module.

But a CLT module is much stiffer, so it can stabilize better and is less fragile during transport.

Q9: I showed you the variation in bay-width among the council estates. What would you recommend for a modular vertical extension?

Answer: Since you are looking for some kind of universal solution, I would recommend using reasonable fixed modules with a similar construction but with a small width/length variation. In this way you will be able to accommodate width differences but still keep the advantage of a standard module. The difference can be accommodated in a kind of intermediate layer, which can be made of steel, so that the modules can be connected to this intermediate layer.

An alternative can be to place modules in the width direction. In this way you can span from building wall to building wall with the modules.

The maximum width of a module should not exceed 3.5m. Transportation less wide than 3.5 m is considerably easier and cheaper.

Q10: Finally, can you indicate where, in general, are the peculiarities in module design?

Answer: There are two more components of importance in modular construction that I have not yet mentioned it.

1. Transportation and lifting. When working out a module, as always, the final situation must be taken into account, but must also be calculated for a lifting/transport situation. The force transfers between the two are often very different.

2. The connections between the modules themselves as well as between the modules and the substructure or stabilizing element are all crucial. The tricky thing about these connections is that they can only be connected at the construction site while the module is already finished as far as possible.

These last two aspects may be just outside the scope of your research, but I do recommend that if you are working out a modular solution, you should have an idea of this.

B.2 Case studies

Appendix B.2 will contain a description of the four performed case studies: Parelhof, Boelelaan, Duinluststraat, and Wibautstraat, respectively.

B.2.1 Parelhof Heerhugowaard

The transition of the Parelhof is currently under construction. The ground floor of the building contains restaurants. The other floors are residential area. The structural design is under development by Van Rossum B.V., commissioned by Segesta Vastgoed B.V. The existing building was constructed around 1980 (Breddels architecten, 2022). Information about this case is provided by the archive of Van Rossum B.V. and the involved structural engineer: **M. Freriks**.



Figure B.1: Impression of the designed top-up on the Parelhof (Breddels architecten, 2022).

B.2.1.1 Existing structure

Figure B.2 shows a floorplan of the existing structure. The structure contains a maximum of three storeys with a total height of 12.17 m. In situ, reinforced concrete was used as a structural material for the floors, walls, and columns. The structural system consists of two distinct parts. The left part (A) has a wall structure (grid size is 3900 mm). The right part (B) contains a column structure spaced 6600 mm. The floors have a thickness, varying from 150 till 200 mm. The walls/columns and floors have been monolithic connected. The foundation was realized by a reinforced concrete slab and foundation piles. The roof structure was not designed for residential use. Resultingly, it could not be used as storey floor.

Part A and B have been separated by a dilatation. Part A resist the cross-directional horizontal load by a wall structure. On the ground floor, the wall structure transfers into portal frames of columns and beams. Parallel to the façade, the wind load is resisted by three walls (located at the elevator).

Part B resists the wind load in cross-direction by a combination of a wall (left side), and a portal frame (right side). A stability wall provides stiffness in longitudinal direction.

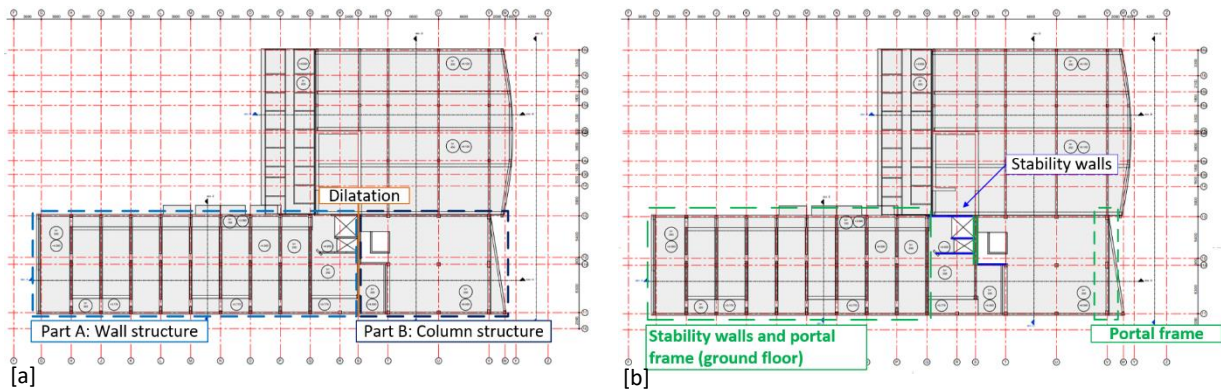


Figure B.2: Structural drawing of the existing structure (Parelhof). [a] Vertical load-bearing system; [b] Stability system (Van Rossum B.V.; Internal communication; Adapted by author).

B.2.1.2 The rooftop extension

The extension will be mainly constructed in timber. The starting point of the project is **to top-up as light as possible**. This will be done by making use of a timber frame structure. Sustainability does not have had impact on the structural design. From design stage on it is the purpose to top-up two layers. The substructure will **not be strengthened for additional vertical weight**. Two additional layers is the structural limit, without reinforcing for additional vertical load. An extra floor will be constructed above the roof structure, as the capacity of the roof was too low. The space between the walls and extended walls will be filled with sand-limestone. The existing structure was calculated according to NEN 8700. The historical code used a **comprehensive safety factor of 1.7**. A structural section is provided in figure B.3. Figure B.4 contains a floorplan (marked stability aspects).

1. Vertical load transfer

The top-up at part A (from gridline G till Q) will be made of timber. HSB-walls will be placed above the existing walls. Between these walls, timber beams of 71 x 221 mm (spaced 600 mm) will be constructed. A finishing layer of fermacell will be applied. Between grid Q and S (at elevator), the span equals 6300 mm. Steel HEB200 profiles will be spanned between the walls to allow for the larger span. Secondary beams will be placed on top of these beams, perpendicular to the façade.

The load at part B must be transferred to the concrete columns. A steel column-beam structure will be used for that top-up. HEB200 beams will span between the columns in all directions. The secondary timber beams of 71 x 221 mm will be constructed (spaced 600 mm).

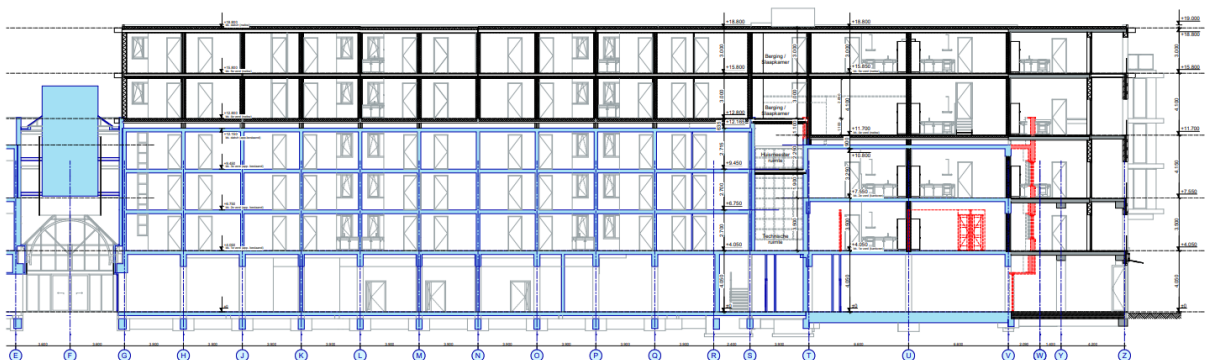


Figure B.3: Structural section Parelhof (Van Rossum B.V.; Internal communication).

2. Stability

Part C will be a new construction. This building will function structurally as stiffening element, parallel to the façade. The extensions of the rooftop will increase the wind load. The original code allowed to reduce the wind load with fifty percent (for this building composition); the current code not. A larger stiffness parallel to the façade is needed (will be realized by new part C). To allow the whole existing building benefits from this stiffening, **the dilatation will be horizontally linked**. The stability walls in part C will be located parallel to the frond façade. Perpendicular to this façade, the in-situ walls and floor will form portals. This portal frame will take over the function of the existing portal frame. Part C will strengthen the existing structure.

The top-up at part A will contain its own stability system. The structural walls of the existing structure will continue in the top-up, providing stability perpendicular to the façade. Parallel to the façade, four stability walls will be constructed. To transfer the wind moment to the substructure, a HEA200 profile will be placed below the stability walls. This profile will be moment fixed to HEA160 columns between existing walls and HSB-walls [Fig. B.5]. That will transfer of the low wind load from the top-up to the structure.

The top-up at part B will transfer the wind load perpendicular to the façade by stability walls. Note that all steel columns will be connected with steel beams. That will form a type of a table structure. Parallel to the façade, the walls of part C will be used for the stability.

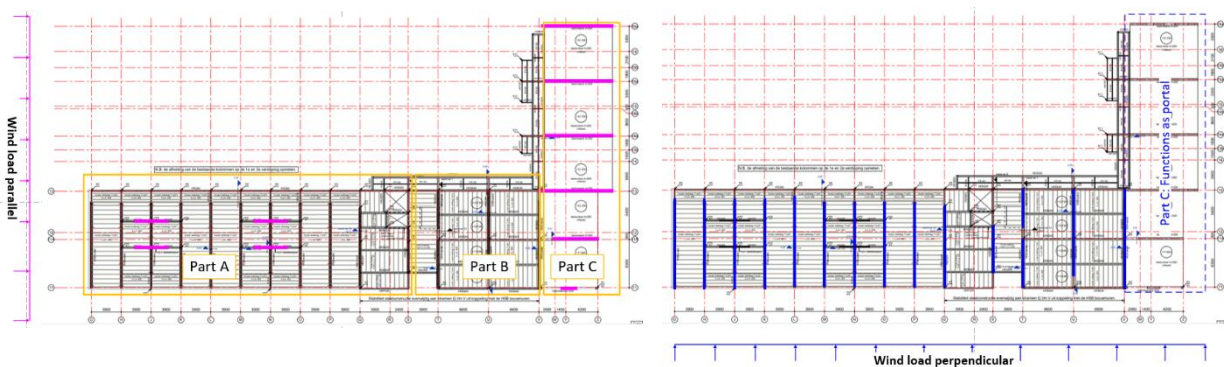


Figure B.4: Stability system of the top-up. Wind load parallel to the facade (left) and perpendicular (right) (Van Rossum B.V.; Internal communication; Adapted by author).

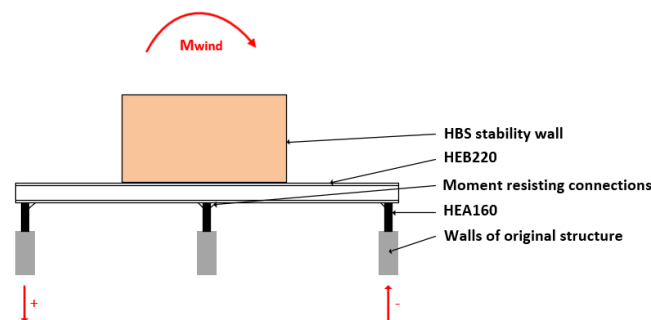


Figure B.5: Principle of stability walls for wind parallel to the facade in part A (Self-produced)

B.2.2 De Boelelaan Amsterdam

De Boelelaan is a **gallery flat**, constructed in 1965. The construction principle of a gallery flat differs from the council estate. After the WWII, an elevator boundary limited to build higher than four storeys (Singelen & Van Vlaenderen, 2011). Around 1965, construction companies started construction with elevators, what resulted in gallery flats. These flats used concrete as structural material. A gallery provided access to the houses.

The structural design was performed by Van Rossum B.V., commissioned by Vesteda Investment Management. The rooftop transition consists of two parts. The two outer parts of the building have been top-upped with one layer. Between these two parts, a sedum roof garden has been constructed [Fig. B.6]. The information is provided by Van Rossum B.V. and the structural engineer: **M. Tromp**.



Figure B.6: Rooftop extension 'De Boelenlaan' (Hans van Heeswijk architecten, 2016).

B.2.2.1 Existing situation

The existing structure was founded on concrete piles (shaft diameter of 320 mm). These piles were founded on the first sand layer (bearing capacity = 60 ton). The ground floor consists of a 150 mm thick, in-situ floor, carried by 35 x 90 cm concrete beams.

The ground floor, and first two storeys, have a concrete column-structure (45 x 45 cm). 50 cm thick, in-situ concrete beams were monolithic connected on top of the columns. The first storey functioned as office space; the second storey as residential area. The thickness of these concrete floors is 14 and 12 cm thick, respectively. At the block ends and the stairwell, concrete walls were constructed.

The structural skeleton above the second floor is prefabricated. 170 mm thick hollow core slabs form both floors and roof. 200 mm thick walls were constructed. Above the sixth floor, these walls were made of light-weight concrete. In the middle grid of the apartments, three concrete columns, with a concrete beam on top were constructed. This beam continuous to outside, where the prefabricated gallery is placed on it.

The cross-directional stability above the second floor, has been realized by housing separation walls. In longitudinal direction, it has been realized by the walls between two bedrooms. In the lower part of the building, stability has been realized by the outer walls, the stairwell, and the walls between blocks. Figure B.7 provides an overview.

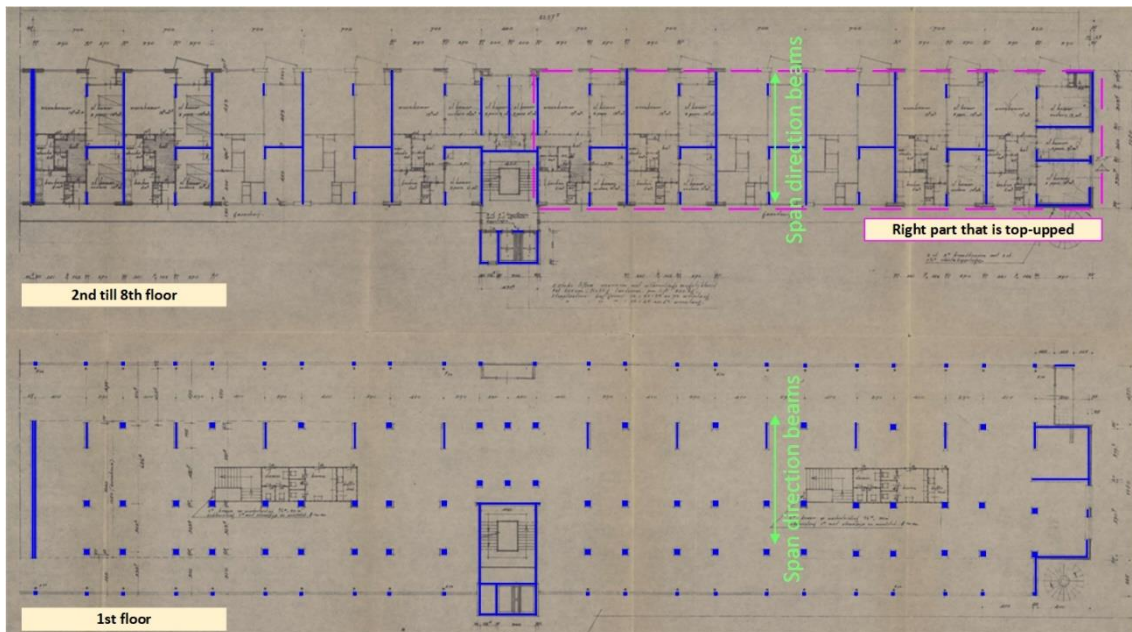


Figure B.7: Existing structural system of De Boelelaan. Blue: structural elements; Rose: part that will be top-upped (Archive municipality Amsterdam, 2022; Adapted by author).

B.2.2.2 The rooftop extension

The rooftop has been extended with one layer (in timber). The existing structure was recalculated according to NEN8700. The top-up is a new structure, so calculated according to NEN1990.

1. Vertical load transfer

A **recalculation of the variable loading** (compared with archive) reduced the total load with 265 kg/m². That provided enough rest capacity to add one layer of timber frame construction. **No strengthening methods for the vertical loads** have been applied.

Timber HSB-walls have been used for the vertical load transfer. The walls consist of a timber columns 38x154 mm which were placed in pairs (spaced 400 mm). Stiffness was provided by timber-based panels. The walls were placed above the existing ones. Note that above the column-beam locations, no HSB-walls were placed. The existing roof slab has been strengthened by a reinforced compression layer, to function as residential floor [Fig. B.9]. An opening has been made in the roof slab to provide access to the new houses. The opening has been surrounded with UNP200 profiles, to strengthen the opening. Figure B.8 provides a floorplan of the added top-up.

The roof has been constructed out of wooden glulam beams, 58x360 mm (spaced 400 mm), finished with 18 mm wood-based panels. These beams must span 7000 mm (Kerto construction method).

2. Stability

The stability in cross-direction (of the extension) has been realized by the HSB-walls. Longitudinally, a distinction was made between the end and inner houses. At the end façades, a HEA200 beam were constructed on top of the concrete floor. At the location of the existing concrete walls, a vertical truss has been placed. That forms a stiff portal frame, what takes up most of the horizontal wind load [Fig.

B.9]. The other houses contain 600 mm HSB-piers inside the façade. Note that also on these locations a HEA200 beam were constructed on the ground. The structure is sufficiently strong for additional horizontal loading, so **no strengthening methods has been applied**.

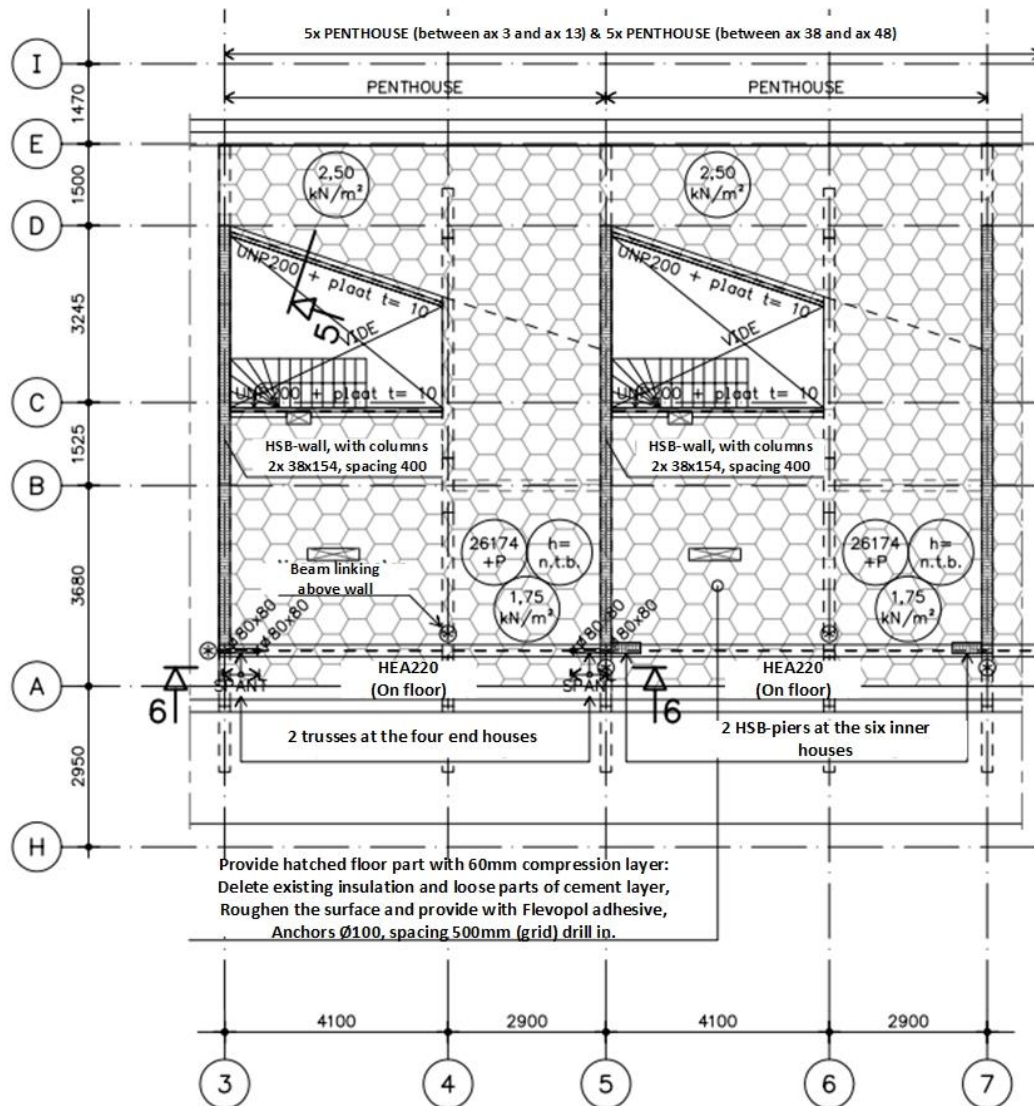


Figure B.8: Applied top-up floorplan of De Boelelaan (Van Rossum B.V.; Internal communication; Translated).

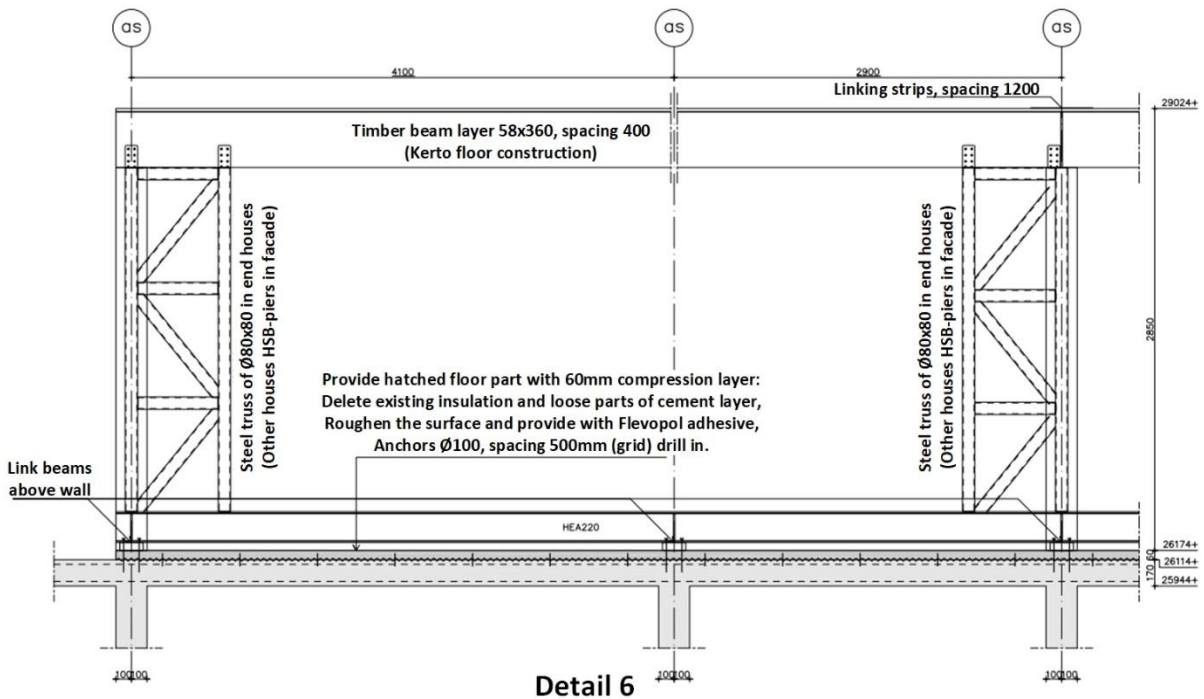


Figure B.9: Detail about stability in longitudinal direction for timber end houses (Van Rossum B.V.; internal communication; translated).

3. Strengthening method applied

No strengthening was needed, due to the extension. A large opening was constructed around one column [1st floor]. The buckling length of this column increased strongly. A strengthening of the column has been applied to account for the doubled span. The surface of the existing columns was roughened and smeared with an adhesive: Flevopol. Thereafter, the column size was increased with an additional layer of reinforced concrete [Fig. B.10]. The column has been increased from 450x450 mm to 650x650 mm. Note that temporary supports were applied, during construction.

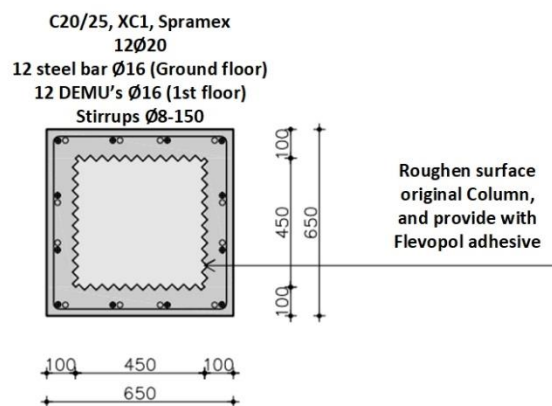


Figure B.10: Detail about strengthening of a concrete column (Van Rossum B.V.; Internal communication; Translated).

B.2.3 Duinluststraat Amsterdam

W.F. van Bodegraven constructed in 1965 a complex of five blocks council estates in Amsterdam-Noord (Het Nieuwe Instituut, 2004). A large reconstruction of the neighbourhood started (in 2000), to solve the housing demand. Hans van Heeswijk architects designed a top up, commissioned by the General Housing Association of Amsterdam (Hans van Heeswijk, 2003). The execution was performed by Moes Bouwbedrijf. The project covered an extension on two blocks. The archive of Amsterdam contains hardly any information. Despite lacking information, the project still provides inspiration for the top-up possibilities on council estates. Most information is provided by the brochure of Hans van Heeswijk architects. Figure B.11 provides an overview of a top-upped building.



Figure B.11: Top-up realized on a council estate (Duinluststraat) (Hans van Heeswijk architecten, 2003).

B.2.3.1 Existing situation

The council estate was constructed according to a non-traditional method [Section 5.3]. 200 mm thick walls were constructed out of 'Korrelbeton'. The floors were made of prefabricated 'Schokbeton' elements. The building contained four storeys. The porch has been located symmetrically, having an additional room [Fig. B.12]. Reinforced concrete floors were applied in the building. The cross-directional stability has been realized by the structural walls. The wall at the end of the stairwell provides stiffness parallel to the façade. The building was founded on reinforced concrete piles. No cellar is present below the council estate (Archive municipality Amsterdam, 2022).



Figure B.12: Floorplan of existing situation Duinluststraat (Archive municipality Amsterdam, 2022).

B.2.3.2 The rooftop extension

The rooftop has been extended with two layers (ten houses per layer). The houses were constructed as **modular timber units**. The current structure is strong enough to bear two additional layers (Hans van Heeswijk architecten, 2003). The units have been placed on top of a **steel frame**. The steel frame has been located above the structural walls of the council estate. The braces of the frame have been placed exactly above the existing stabilizing walls [Fig. B.15]. The frame allows placement of the units on the existing roof slab. The open space below the frame can be used for installations. The existing floorplan consists of three distinct spans between the structural walls [Fig. B.12]. The steel frame has the same spacing, resulting in three sizes of units [Fig. B.13].

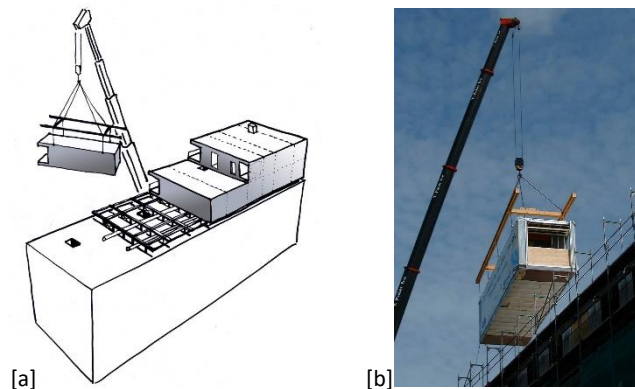


Figure B.13: [a] Top-up principle, and [b] actual construction (Hans van Heeswijk architecten, 2003).

At the end of the building a new tower was constructed. As the elevator boundary was exceeded, the new top-up should have to be accessible by an elevator. The stairwell provides access to the top-up. The non-structural elevator of the new construction can also be used by the top-up residents. At the other façade end, one stairwell has been continued in the top-up. The stairwells provide access to a gallery, which has been connected to the units. Figure B.14 provides a floorplan of the top-upped layer.

1. Vertical load bearing system

The timber units were constructed from HSB-walls. These walls consist of small columns, clad with wood-based panels. The floor and roof consist of a beam structure with wood-based panels on top. The roof transfers the load through the walls and steel frame to the existing walls. The existing structure could bear this additional load (Hans van Heeswijk architecten, 2003).

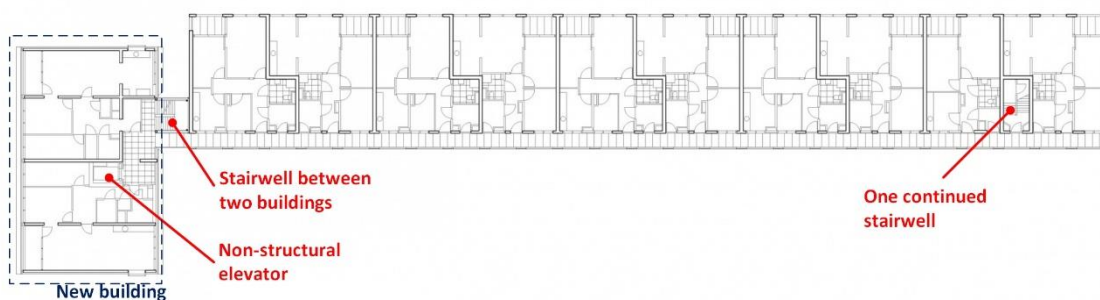


Figure B.14: Floorplan of top-up and new constructed building (Hans van Heeswijk architecten, 2003; Adapted by author).

2. Stability

Although the two buildings have been connected by a (non-structural) stairwell, they still should be considered individually. It can be assumed that the stability walls in the existing structure have enough overcapacity to resist the additional horizontal wind load.

The same stability principle as the existing structure has been applied to the top-up. In cross-direction, the timber walls were placed above the structural walls. In longitudinal direction, walls were placed above the existing ones. The steel frame has been stabilized with braces [Fig. B.15].

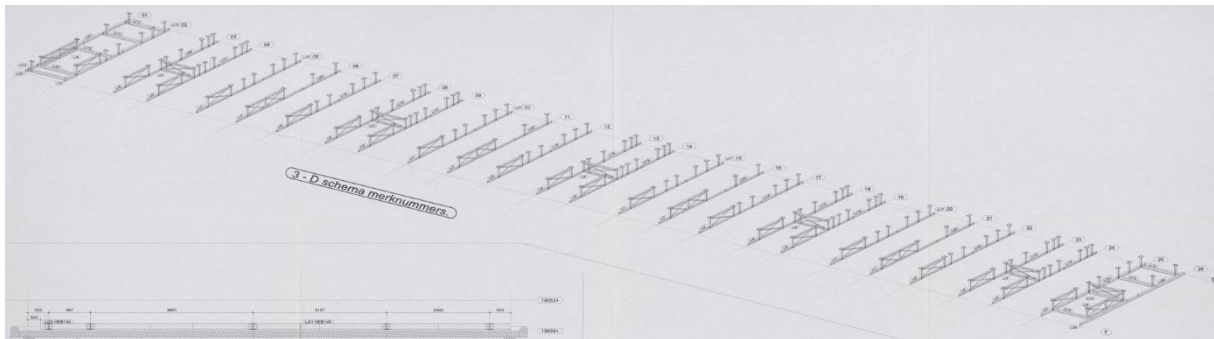


Figure B.15: Steel frame on top of the existing council estate (Archive municipality Amsterdam, 2022).

B.2.4 Wibautstraat Amsterdam

The Wibautstraat was constructed in 1961. It has been classified as gallery flat. The building had functioned as faculty for the university of Amsterdam as well as office building. On the ground floor a supermarket is located. The building (except the supermarket) has been transferred into 162 residential houses, with a parking space in the cellar (Penta Architecten, 2016). The structural design was performed by Van Rossum B.V., commissioned by Boelens de Gruyter. The architectural design was performed by Penta architects. Figure B.16 provides the result of the transformation. Information is provided by the archive of Van Rossum B.V., and the corresponding structural engineer **M. Tromp**.



Figure B.16: Wibautstraat Amsterdam (Penta Architecten, 2016).

B.2.4.1 Existing situation

The existing building contained a cellar, ground floor and six storeys. It was constructed as one monolithic concrete skeleton. The structure can be subdivided into three parts. Part A was constructed at the side of the Wibautstraat. Part B is shopping area behind it. Part C is the constructed part on the Oetgensdwarstraat. The parts have been separated by a dilatation. Part B only contains one floor. Stairwells were used to reach the galleries. Part C contained a non-structural elevator. The elevator did not reach the cellar.

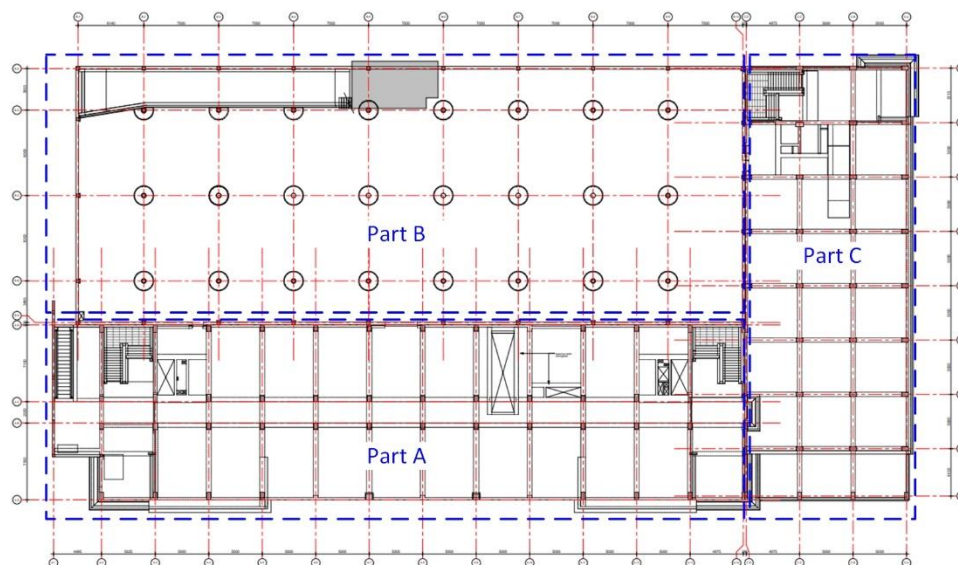


Figure B.17: Existing ground floor Wibautstraat (Van Rossum B.V.; Internal communication; Adapted by author).

The construction was founded on concrete piles (bearing capacity of 800 ton). 200 till 250 mm thick cellar-walls were constructed in reinforced concrete. The cellar floor is 180 mm thick (part A and C) or 400 mm (part B), placed on concrete beams.

The 120 mm thick ground and storey floors were constructed of reinforced, in-situ concrete. A reinforced column structure was applied, with beams in two directions. The beams have been monolithic connected to the columns. The floors resultingly span in two directions. Part B forms a structural exception. A mushroom ground floor was applied with a thickness of 40 cm. Typical in-situ mushroom columns were placed on it. The roof system of part B contains a 11 cm thick bims concrete channel-cassette slab. As the ground floor of part B originally functioned as parking space, a large unsupported opening was constructed. The corresponding beam size is large.

The monolithic connection between the concrete columns and beams has been formed portal frames in both wind directions. These portals provide stiffness in both directions.

B.2.4.2 The rooftop extension

The new added floors have been constructed in steel. A composite floor was used as flooring and roof system. The stairwells at the two end facades were removed. A new outside structure contains a gallery to enter the houses. The new top-up can be reached by the existing elevator or stairwell.

1. Vertical load transfer

The purpose of the extension was to extend two storeys. To allow for the increase in vertical load, the **sixth floor was demolished**. Additionally, the concrete beams below the sixth floor were replaced by steel beams. The rebuild of the sixth floor has been performed like the top-up of the other two storeys.

The reinforced concrete skeleton has been robust dimensioned. A **comprehensive safety factor of 1.8** was used for the safety. The instantaneous values were reduced with ten percent, what provided a large overcapacity to the existing structure, with respect to the current norm for existing buildings (NEN 8700). Due to a change from office towards residential use, the **variable loading was recalculated**. That provided additionally a reduction in vertical loading.

Steel columns were placed above the existing ones. Beams span between the columns, perpendicular to the façade. The composite floor spans between the beams. A steel frame was constructed for the balconies. This frame rests on columns, which are carried by a new steel beams in part B. These transfer



Figure B.18: Sideview of the Wibautstraat (Van Rossum B.V.; Internal communication).

the load towards the columns on the ground floor. The principle is visible in figure B.20. Based on the new structure and usage of the (sixth) floor, no strengthening of the existing structure has been needed for the additional vertical load. Figure B.19 provides a plan of the new seventh floor.

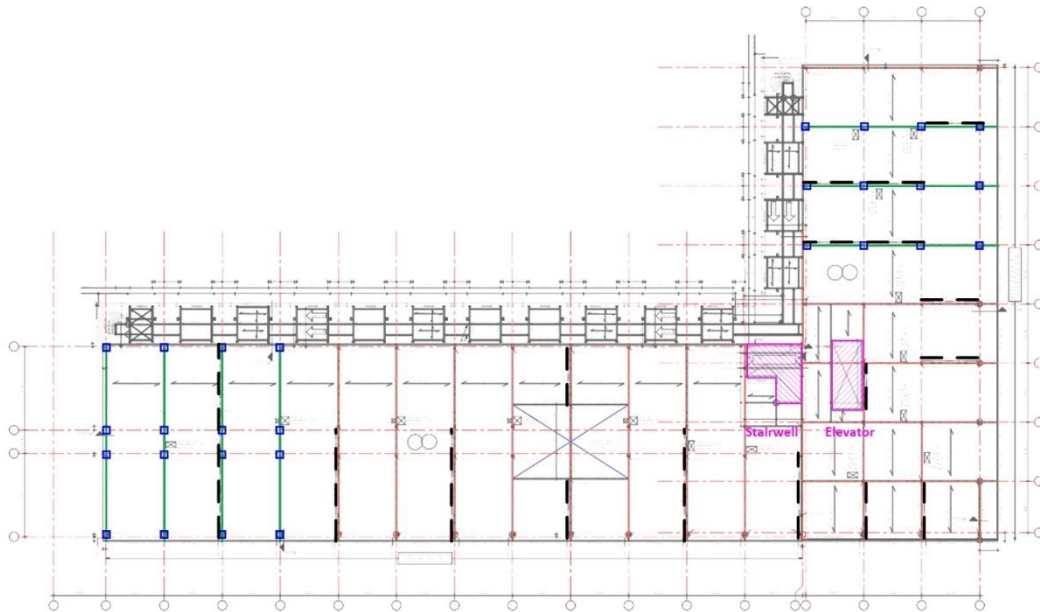


Figure B.19: Floorplan seventh floor extension (Wibautstraat). Green: example span of the beams; Blue: example location of columns; Dotted black: location of wind trusses. (Van Rossum B.V.; Internal communication; Adapted by author).

2. Stability

The existing system was stable due to concrete portals in two directions. The new situation still provides stability by means of these portals. The increased building height results in a larger wind load, causing higher moments in the portals. The **dilatations** between the buildings have been **structurally closed by steel plates**. That divides the wind load over the distinct portals (it is assumed that the concrete shrinkage of the construction and the pile settlements already happened). The new combination of an increased wind load and reduced variable load on the floors was verified. Above the first floor the stability was not sufficient. Stability crosses were placed between the portals. These have been placed staggered between portals to divide the load over the columns. The principle was applied in both part A and C [Fig. B.19; Fig. B.20]. The trusses have been orientated perpendicular when considering part, A and C. As these parts were structurally connected, the stiffness was provided in both directions.

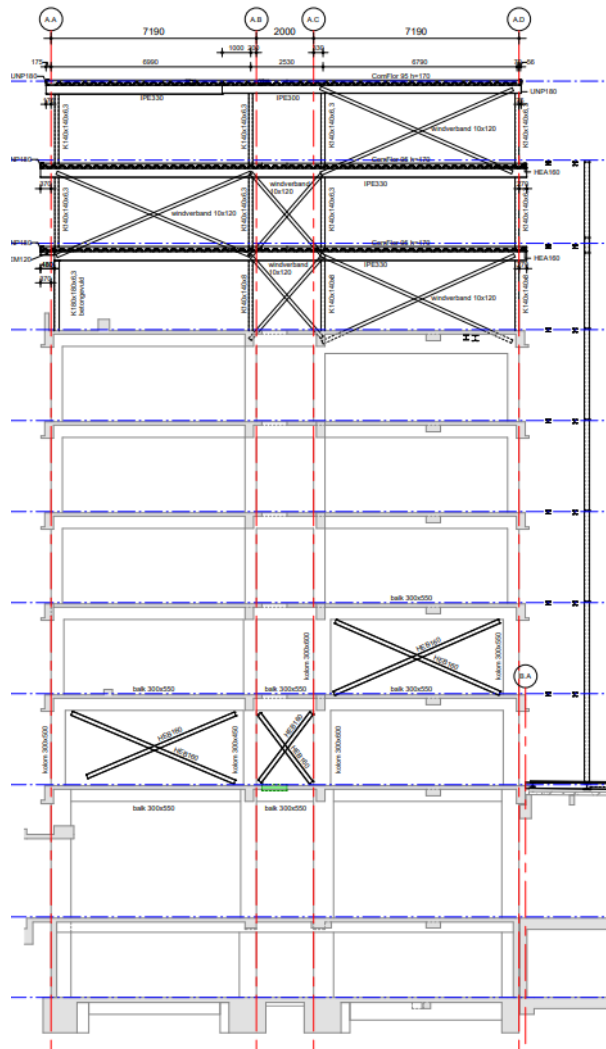


Figure B.20: Example placements of stability crosses on one axis of part A (Van Rossum B.V.; Internal communication).

B.3 HSB- and CLT-construction

Appendix B.3 will provide background information about HSB- and CLT construction.

B.3.1 HSB-construction

B.3.1.1 Principles

The load bearing structure consists of wooden columns and beams, clad with wood-based panels as Orientated Strand Board, Glued laminated wood, plywood, or Laminated Veneer Lumber (Van Dessel & Dobbels, 2019). The combination of columns, beams and panels is often applied as a prefab wall.

B.3.1.2 Walls and floors

The walls are constructed out of a wooden frame, clad at least on one side, with a wood-based panel [Fig. B.21]. The walls can be connected by means of linking beams, placed on top of the walls. The inner walls are finished with gypsum board. The open spaces are filled with insulation material, for both acoustic resistance and insulation. The outer walls are clad with airtight foil (on the inside). The façade allows for a traditional cavity construction, sandwich panels or a wooden finishing (Van Dessel & Dobbels, 2019).

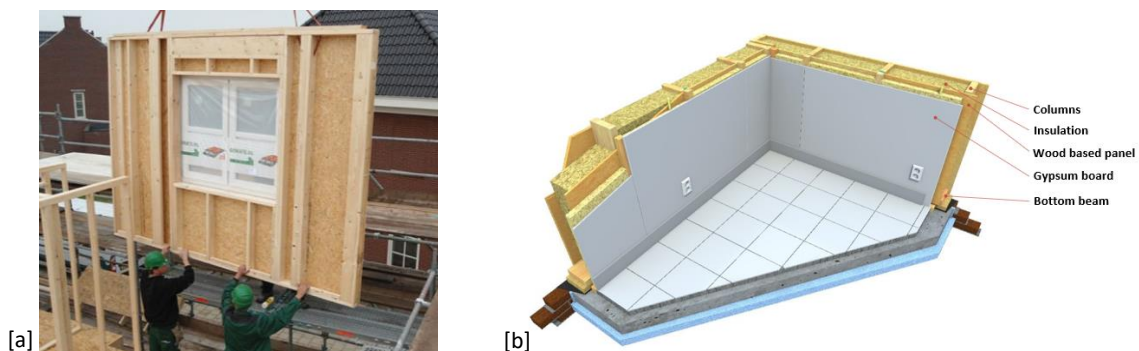


Figure B.21: [a] HSB-wall construction (De Kroon Prefab, 2022); [b] Wall build-up (Passiehuismarkt, 2018).

Floors are constructed of wooden beams, clad with a wood-based panel [Fig. B.22]. To prevent lateral torsional buckling of the beams, small spools are placed between the beams. Insulation can be placed between the structural beams. To reduce the contact noise, a floating screed can be applied. Another method is an acoustically disconnected ceiling (Van Dessel & Dobbels, 2019).



Figure B.22: Typical HSB floor (Alsanit Bouw, n.d.).

B.3.1.3 Structural load transfer

The vertical load from the floors is transferred through the timber plate and beams towards the walls. The walls transfer the vertical load by the columns (spaced 40 till 60 cm). The wood-based panels stiffen the frames, allowing development of diaphragm action (Van Dessel & Dobbels, 2019).

B.3.2 CLT-construction

B.3.2.1 Characteristics of CLT

Cross Laminated Timber (CLT) are engineered wood products, which can form two-dimensional panels. Due to one-directional fibre alignment of wood, it tends to shrink and swell in longitudinal direction. The crosswise orientation of the wood-fibres minimizes the shrinkage and swelling behaviour of the elements in-plane. The laminar structure allows its application of full-size wall and floor element which can bear loads in- and out-of-plane. These two-dimensional elements can span in two directions with precision and accuracy (Ashuckian et al., 2019; Brandner et al., 2016; Martinsons, 2016). An element assembly is strictly separated in construction, insulation, installation, and cladding. That allows for different degrees of prefabrication in fabric, a step-by-step finishing on-site, and an easier execution and repair of alternate work (Brandner et al., 2016).

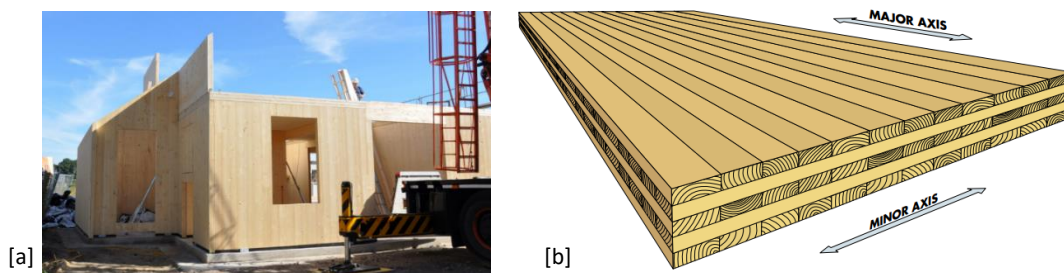


Figure B.23: [a] CLT-construction (Bruggink & Degen, 2022); [b] lay-up (Breneman, 2016).

B.3.2.2 Structural considerations

The two-dimensional panels transfer both horizontal and vertical loading to the foundation (Ashuckian et al., 2019). The panels are classified as quasi-rigid (Brandner et al., 2016). Because of the size and strength of CLT panels, diaphragm behaviour is significantly influenced by the strength, flexibility, and ductility of the connections. The exact CLT diaphragm performance is an area of ongoing research (Breneman, 2016). Human-induced vibrations are more prominent on timber floor systems, due to the low bending strength of timber (Huang et al., 2020).

B.3.2.3 Typical floor and wall build-up

Floor panels can span till 7.7 m. A typical floor build-up consists of a panel, finished with a plaster layer [Fig. B.24a]. Approximately 50 mm insulation is added. To provide a higher acoustic resistance, it is possible to add a floor on top of the CLT layer (Martinson, 2016).

For the outside walls, distinct possibilities exist. Usually, façade cladding is applied with 170 mm insulation between the CLT and cladding. A 0.2 mm thick construction foil is applied on the outside of the wall [Fig. B.24b]. Internal walls are usually two-sided cladded with 15 mm plasterboard. For these walls, no additional insulation is applied (Martinson, 2016).

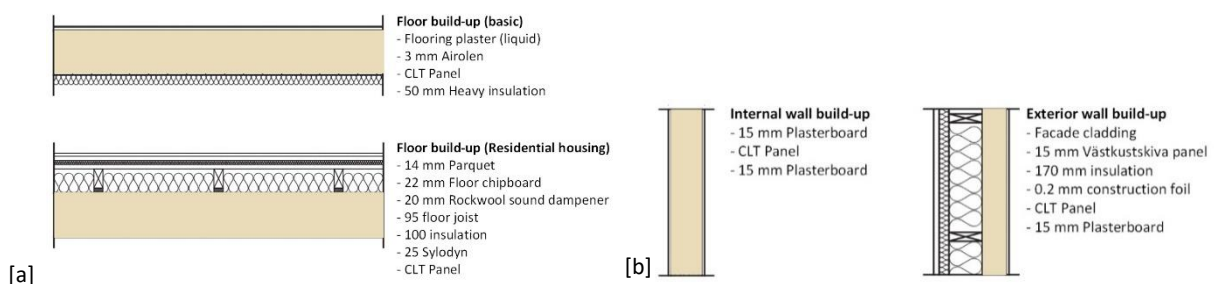


Figure B.24: [a] Typical CLT-floor build-ups; [b] Typical CLT-wall build-ups (Martinson, 2016).

B.3.3 Weight calculation

A weight calculation is performed by the author to compare both HSB and CLT. The Parelhof is used as reference. The HSB design already exists. The CLT calculation is performed based on the design tables of Martinsons' guide to CLT (Martinsons, 2016).

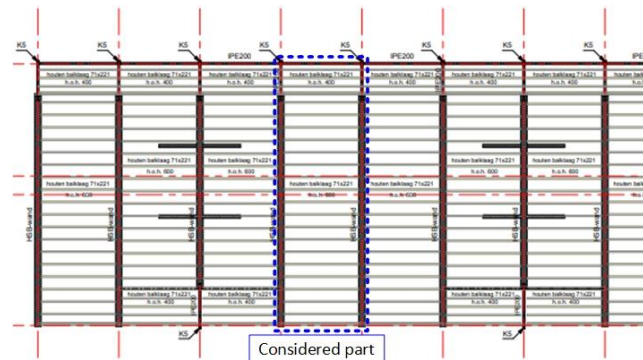


Figure B.25: Considered part in comparison (Parelhof)
(Van Rossum B.V.; Internal communication).

B.3.3.1 HSB-construction

The walls are constructed of columns containing a cross-section of 38x89 (spaced per pair 600 mm). The sides are cladded with 15 mm underlayment. On top of these panels, an additional layer of 12.5 mm gypsum board is placed. The open spaces inside the wall are filled with insulation.

The floor is constructed in timber beams (C24), with a cross-section of 71x221 mm, spaced 400 mm. On top, a layer of 18 mm underlayment is placed. Ceiling and finishes are not considered in the comparison.

B.3.3.2 CLT-construction

For a residential house, a wall thickness of 120 mm is used. Both sides of the wall are cladded with 12.5 mm thick gypsum board. The same CLT-floor floor panel is applied for a span of 3.9 m (Martinsons, 2016).

B.3.3.3 Comparison weight

Figure B.26 provides a graph with the results. The CLT-walls are approximately 1.47x as heavy as normal HSB-walls. The CLT floors are 2.14 times as heavy. A combined unit of two floors and walls is approximately 1.74 times as heavy as normal HSB-construction. Note that this comparison does not include safety factors and variable loading.

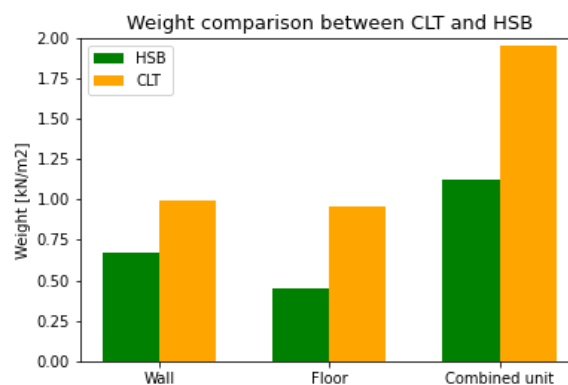


Figure B.26: Comparison weight CLT and HSB. Weight expressed per m2 floor area (Self-produced).

B.4 Connections for modules

Interconnection of frame members and modules forms a critical aspect of modular buildings to withstand applied loads. Despite that, there is hardly any research in the interconnectivity between modules. Lacey et al. (2018) grouped three distinct connections: inter-module, intra-module, and module to foundation.

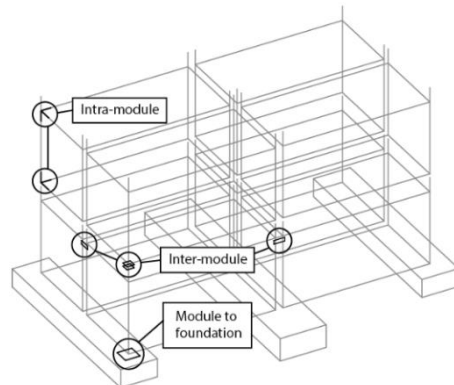


Figure B.27: Possible connections in modular construction (Lacey et al., 2018).

Intra-module connections include connections within a module. These are representative of traditional connection details. Both moment-resisting as bolted connections can be used. Moment resisting connections can provide stability for low-rise buildings (Lacey et al., 2018).

Inter-module connections include horizontal connections between adjacent modules and vertical connections between two stacked modules. Usually, a gap is provided between the floor and the ceiling beams for external access to inter-module connections and services (Lacey et al., 2018). These inter-module connections must resist axial and/or shear forces. Figure B.28 provides a range of possible inter-module connections for timber modules.

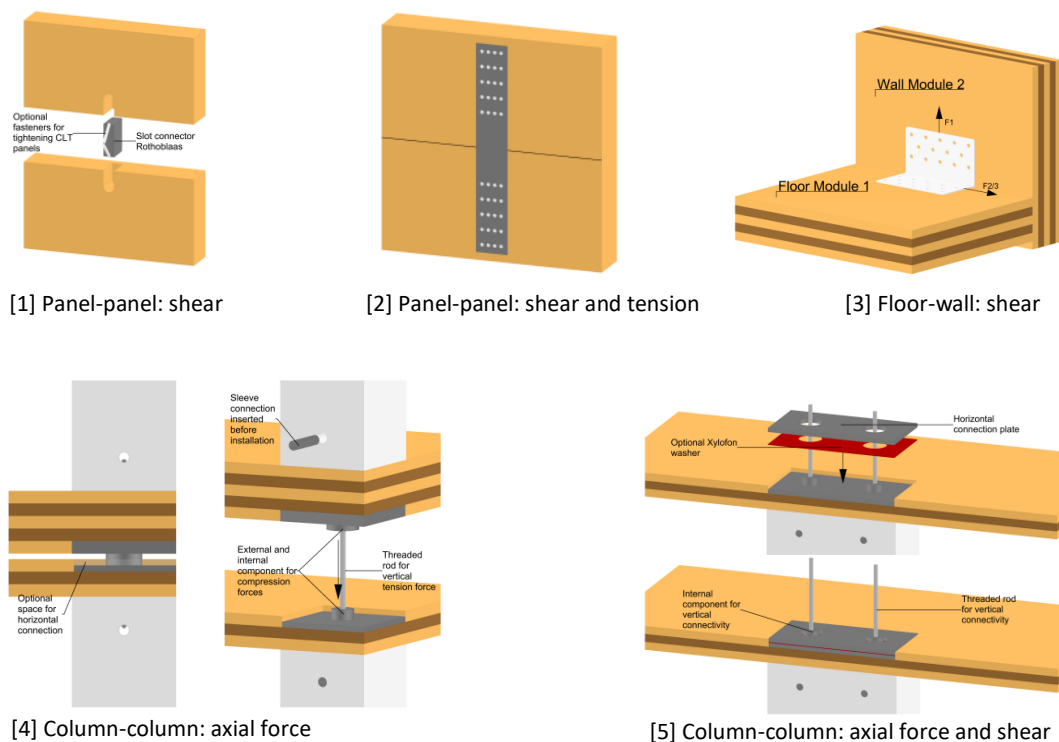


Figure B.28: Inter-module connections between timber modules (Hamelijnc, 2021).

B.5 Strengthening methods for stability

Appendix B.5 will provide an investigation in strengthening possibilities for the existing structure. It will contain: Transition to monolithic connections; Horizontal linking of dilatations; Steel bracings; Reinforced concrete walls and construction of a concrete core. Appendix X.6 will provide used excel sheets of calculations.

B.5.1 Introduction of monolithic connections

Papageorgiou (2016) proposes to transform the simple connections between floors and walls into monolithic connections. A simple scheme is made to investigate the effect in case a stability wall is present. The results show a reduction of 5 kN in tension force [Fig. B.29] and 0.2 mm in deflection [Fig. B.30]. Next to the practical complications, it is structurally not of major impact. It can be concluded that this is an inefficient method to strengthen the council estates for stability.

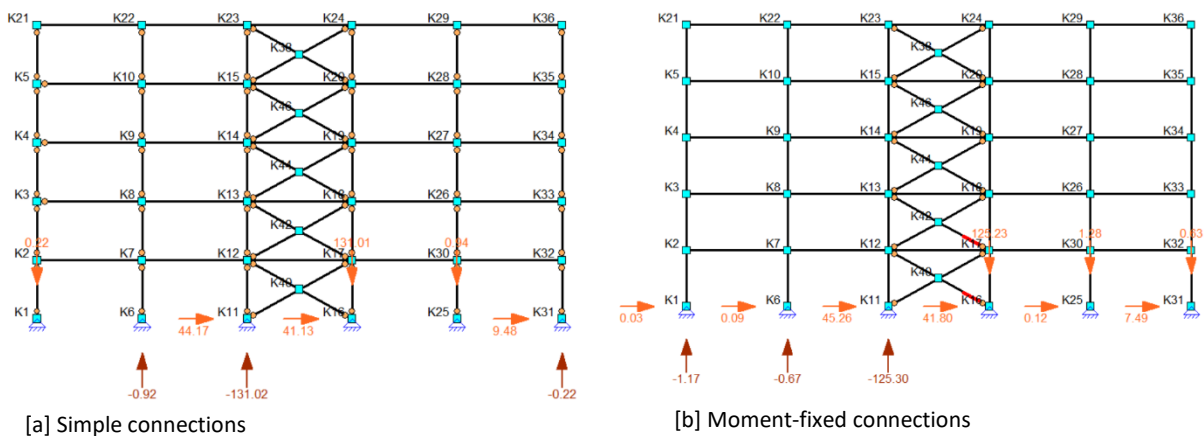


Figure B.29: Effect of change towards monolithic system on reaction forces (unit: kN).

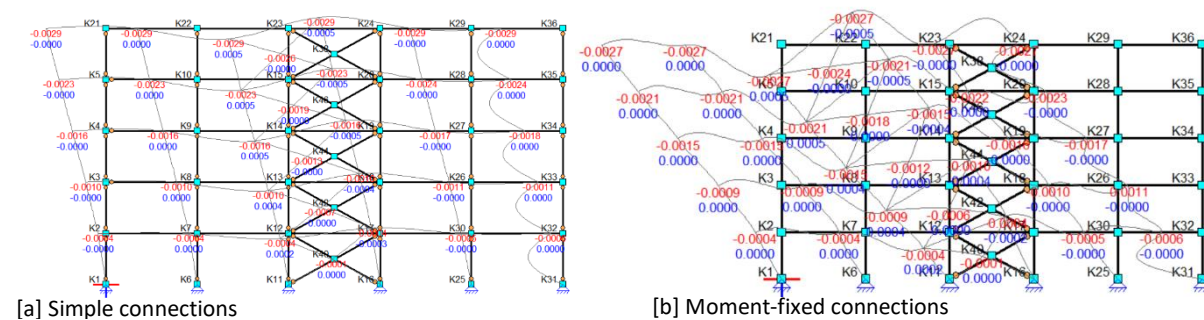


Figure B.30: Effect of change towards monolithic system on displacements (Units: m).

B.5.2 Horizontal linking of dilatation

By horizontal linking, the longitudinal stability walls can collaborate. Resultingly, a higher stiffness can be obtained. Both linking of two blocks (4 porches) and three blocks (6 porches) are considered. It is assumed that the connection of two blocks still is a dilatated building, meaning that only suction is considered ($C_f=0.8$). 3 linked blocks are considered as a not-dilatated building, meaning both suction and pressure occur ($C_f=1.5$). Further properties and calculations are according to the Helena Mercierstraat [Section 5.5.3]. It can be concluded that it is efficient to link buildings horizontally. Additionally, it is more efficient to link 2 blocks instead of the total building [Table B.1].

Table B.1: Effect of horizontal linking of dilatations.

Storeys (height)	q_p [kN/m ²]	UC_2 blocks	UC_3 blocks
6 (19.6m)	0.90	0.41	0.51
7 (22.4m)	0.93	0.55	0.69

B.5.3 Steel bracings

Application of steel bracings is a proposed solution to strengthen the existing structure. Tension occurs in brickwork, resulting in crack-formation. Steel bracings form a new stabilizing element. It is assumed that the total wind-load will be distributed to the new truss. B.31 shows proposed location. The exploration of this strengthening method is performed for a council estate, excluding an additional room, including wooden floors (NFL = 3.47; Ratio depth building / length stability wall = 4.2).

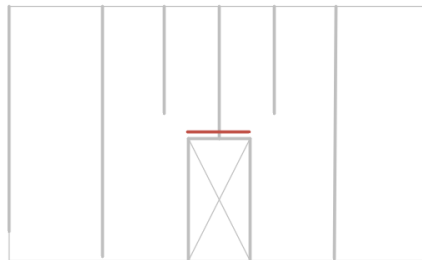


Figure B.31: Positioning of bracing (Self-produced).

Normally, the width of the trusses is estimated based on a rule of thumb: $width = 2 * height / 8$. Unfortunately, the width of the longitudinal-orientated trusses is limited to 2.6 m (Wall length). Figure B.32 provides the used schematizations.

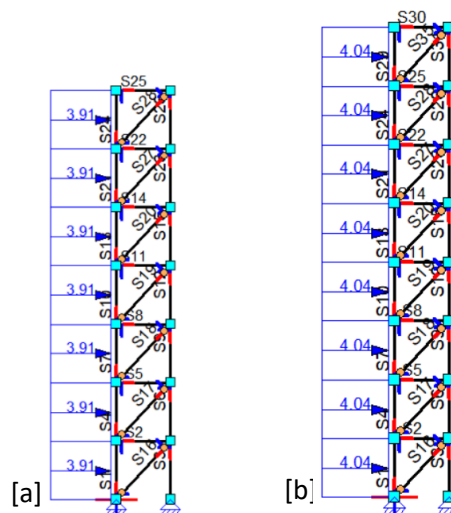


Figure B.32: Mechanical schemes and loads for braces. Note that only the SLS forces are presented; [a] 6-storey building; [b] 7-storey building.

The wind load is related to the height of the building. As two porches are linked between dilatation, the wind load in longitudinal direction is equally divided over two braces [division factor = 2]. Table B.2 provides an overview of the used wind loads. For the peak pressure, Area II, urban is used.

Table B.2: Used wind loads.

Factor	Longitudinal direction	Cross direction
$C_s C_d$	1	1
C_f	+0.8	+1.5
$q_p(19.6 \text{ m})$	0.90 kN/m ²	0.90 kN/m ²
$q_p(22.4 \text{ m})$	0.93 kN/m ²	0.93 kN/m ²
Width	10.85 m	18.00 m
Division factor	2	-
$Q_{wind,19.6m,SLS}$	3.91 kN/m	24.3 kN/m
$Q_{wind,22.4m,SLS}$	4.04 kN/m	25.1 kN/m

The trusses are verified in both ULS and SLS. Equation B.1 and B.2 provides the used verifications. Note that for the ULS a safety factor of 1.4 is used for the variable loading and 0.9 for permanent. A relative low unity check is used for the deflections, due to additional deflections caused by the foundation.

$$\text{SLS: } w < \frac{1}{500} h_{building} \quad (\text{B.1})$$

$$\text{ULS: } \frac{N}{A} < f_{yd} \quad (\text{B.2})$$

Table B.3 and B.4 provides the results of the dimensioning of the trusses for both 6- and 7-storey buildings.

Table B.3: Results in longitudinal direction (Steel bracings).

Height [m]	Ø Tie [mm]	Other profiles	f_{yd} [N/mm ²]	N_{tie} [kN]	$N_{columns}$ [kN]	w [mm]	UC _{tie} (ULS)	UC _{columns} (ULS)	UC _w (SLS)
19.6	40	HEA200	235	146.3	404.1	27.4	0.50	0.32	0.70
22.4	40	HEA240	235	166.2	545.2	30.4	0.56	0.30	0.68

Using bracings in longitudinal direction is not an optimal method. Especially for more than six floors, large elements should be used. The tensile force must be transferred to the foundation. For this, tensile foundation piles should be used. The tensile capacity of piles equals approximately 100 kN (M. Freriks, Van Rossum B.V., Internal communication). Based on table B.4, it can be concluded that two piles are needed to transfer the tensile force (4 piles in total).

Table B.4: Needed tension piles for bracings.

Height [m]	Tensile force wind [kN]	Normal force structure [kN]	Tension to foundation [kN]	Needed piles	Piles applied/wall
19.6	404.1	325.1	79.0	1	4
22.4	545.2	330.8	214.4	2	4

B.5.4 Concrete reinforced walls

Thickening the brickwork walls by concrete forms another strengthening method (location similar to Fig. B.31). Concrete walls can contain openings, what makes them more flexible compared to steel bracings. The exploration of this strengthening method is performed for a council estate, excluding an additional room, including wooden floors (NFL = 3.47; Ratio depth building / length stability wall = 4.2). The reinforcement inside the concrete wall must take up the occurring ULS tensile force (Eq. B.3) [Appendix X.5]. This tensile force is reduced with the self-weight of the concrete wall (Eq. B.4). The reinforcement is calculated according to equation B.5.

$$F_{tension,brickwork} = F_{tension,concr.} \quad (\text{B.3})$$

$$F_{tension,reinf} = F_{tension,concr.} - (b_{wall,c}L_{wall,c}H_{wall,c} \cdot \gamma_c) \quad (\text{B.4})$$

$$A_{reinf.} = \frac{F_{tension}}{435 \cdot 2} \cdot \frac{2}{L_{wall,c}} \quad [\text{mm}^2/\text{m}/\text{row}] \quad (\text{B.5})$$

The stiffnesses of both walls are compared to verify the deflections. It is assumed that the wind load distributes over both walls based on the stiffness. On both sides of the wall, a 'flange-width' of 350 mm is assumed, contributing to the stiffness (Eq. B.6). The E-modulus of brickwork strongly depends on the quality of the brickwork. The stiffness can be estimated as $1000 \cdot F_k$ (Van der Cruyssen, 2019). The stiffness of the brickwork is assumed to be 4770 N/mm^2 . A cracket stiffness of $11,000 \text{ N/mm}^2$ is used for concrete. The wind load is divided based on equation Eq. B.6 till B.9. It appeared that both walls take up half of the wind load.

$$I_{wall,br} = \left(\frac{1}{12} t_{wall,b} L_{wall,b}^3 \right) + \left(2xt_{wall,b} \cdot \left(\frac{L_{wall,b}}{2} \right)^2 \right) + \left(\frac{2}{12} xt_{wall,b}^3 \right) \quad (\text{B.6})$$

$$I_{wall,c} = \frac{1}{12} t_{wall,c} L_{wall,c}^3 \quad (\text{B.7})$$

$$EI_{tot} = (E_b I_{wall,b}) + (E_c I_{wall,c}) \quad (\text{B.8})$$

$$q_{wind,c} = \frac{E_c I_{wall,c}}{EI_{tot}} \cdot q_{wind} \quad (\text{B.9})$$

The deflection is calculated according to equation B.10 and verified with equation B.11 in the SLS. Figure B.33 provides an overview of the used schematization for the deflection. A doubling of the deflection is assumed, to consider the foundation.

$$w = \left(\frac{q_{wind,c} H_{building}^4}{8 E_c I_{wall,c}} \right) \cdot 2 \quad (\text{B.10})$$

$$w < \frac{1}{500} h_{building} \quad (\text{B.11})$$

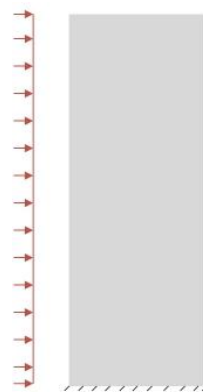


Figure B.33: Schematization wall (Self-produced).

Table B.5 provides an overview of the dimensions of the concrete wall. The minimal wall thickness is set to 150 mm, to provide enough space for reinforcement.

Table B.5: Results concrete wall in longitudinal direction.

Height [m]	L _{wall} [m]	t _{wall} [m]	w [mm]	UC SLS	UC _{brickwork}	F _{reinf.} [kN]	A _{reinf.} [mm ² /m/row]	Applied reinf. 2 rows
19.6	2600	150	27.7	0.71	0.71	59.8	52.9	Ø5-300mm
22.4	2600	200	42.0	0.94	0.73	182.3	161.2	Ø8-300mm

The number of tensile piles is estimated based on a tensile capacity of 100 kN (Van Rossum B.V., Internal communication). In all situations, four piles are needed to thicken the brickwork wall with concrete [Table B.6].

Table B.6: Estimation number of tension piles needed for wall.

Height [m]	Tension force [kN]	Piles needed [-]	Minimum applied piles/wall [-]
19.6	59.8	0.6	4
22.4	182.3	1.8	4

B.5.5 Concrete reinforced elevator

As final option, the use of a structural elevator is considered [Fig. B.34]. As the elevator boundary will be exceeded, the presence of an elevator is obligatory. It is an option to make this elevator structural. In the calculations it is assumed that every porch contains a structural elevator. It is assumed that the elevator replaces the existing stability element in longitudinal direction. The exploration of this strengthening method is performed for a council estate, excluding an additional room, including wooden floors (NFL = 3.47; Ratio depth building / length stability wall = 4.2).

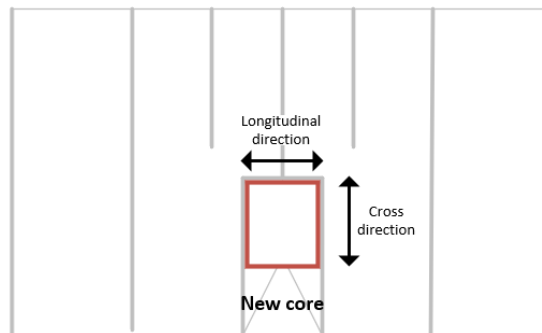


Figure B.34: Construction of a core (Self-produced).

A cracked stiffness for concrete is used in the calculation: 11,000 N/mm². The allowed deflection is limited to equation B.1. Note that the occurring deflection is multiplied with a factor 2 to take into account the foundation (Eq. B.13). Equation B.12 till B.16 are used for the calculation of the internal forces in longitudinal direction. The normal force is equally divided over the elevator area to calculate the normal stress. Additionally, the stress due to its self-weight is considered (Eq. B.16).

$$I_{longit. direct.} = \frac{1}{12} L_{cross} L_{longitudinal}^3 - \frac{1}{12} (L_{cross} - t_{wall})(L_{longitudinal} - t_{wall})^3 \tag{B.12}$$

$$w_{longit.direct.} = \left(\frac{q_{longit.} H_{building}^4}{8 E_c I_{longit.direct.}} \right) \cdot 2 \tag{B.13}$$

$$M_{longit.direct.} = \left(\frac{1}{2} \right) q_{longit.} H_{building}^2 \tag{B.14}$$

$$\sigma_{M, longit.} = \frac{M_{longit.direct.} \cdot L_{longitudinal}}{2 I_{longit.direct.}} \tag{B.15}$$

$$\sigma_{N,tot} = \frac{F_{normal,appendix X}}{A_{elevator}} + \gamma_c H_{building} \quad (\text{B.16})$$

In cross-direction, the stiffness of the elevator is compared with the stiffness of the existing walls (Eq. B.17 till Eq. B.20). A brickwork stiffness of 4770 N/mm² is assumed. The elevator only contributes in cross-directional stiffness for 5.0%.

$$I_{cross\ direct,c.} = \frac{1}{12} L_{longitudinal} L_{cross}^3 - \frac{1}{12} (L_{longitudinal} - t_{wall})(L_{cross} - t_{wall})^3 \quad (\text{B.17})$$

$$I_{cross,walls} = \sum_{i=1}^N \frac{1}{12} \cdot t_{wall} L_{wall}^3 \quad (\text{B.18})$$

$$(EI)_{tot} = (E_c I_{cross\ direct,c.}) + E_b \sum_{i=1}^N \frac{1}{12} \cdot t_{wall} L_{wall}^3 \quad (\text{B.19})$$

$$q_{cross,elevator} = \frac{E_c I_{cross\ direct,c.}}{(EI)_{tot}} \cdot q_{cross} \quad (\text{B.20})$$

$$w_{cross\ direct.} = \left(\frac{q_{cross,elevator} H_{building}^4}{8 E_c I_{cross\ direct,c.}} \right) \cdot 2 \quad (\text{B.21})$$

$$M_{cross\ direct.} = \left(\frac{1}{2} \right) q_{cross,elevator} H_{building}^2 \quad (\text{B.22})$$

$$\sigma_{M,longit.} = \frac{M_{cross\ direct.} L_{cross}}{2 I_{cross\ direct.}} \quad (\text{B.23})$$

Equation B.24 till B.26 are used for the calculation of the needed reinforcement (FeB 500) in longitudinal direction. The reinforcement has a design capacity of $f_d=435$ N/mm². In cross-direction, the compressive force is higher than the tensile force. Only minimum reinforcement is needed.

$$\sigma_{tension,longit} = \sigma_{M,longit} - \sigma_{N,tot} \quad (\text{B.24})$$

$$F_{tension,longit} = \sigma_{tension,longit} \cdot L_{cross} \cdot t_{wall} \quad (\text{B.25})$$

$$A_{reinf.;longit.} = \frac{F_{tension,longit}}{435 \cdot L_{cross} \cdot 2} \quad [\text{mm}^2/\text{m}] \quad (\text{B.26})$$

The main disadvantage of the elevator are minimum required dimensions. The minimum floor area equals 1.05 x 2.05 m with a wall thickness of 250 mm for residential housing [>6 storeys] (Bouwbesluit 2012; Article 4.28). Removal of the structural walls will be a complicated process, so the elevator should fit inside the stairwell. In longitudinal direction, the wall is structurally connected to the cross-orientated stairwell-walls. In cross-direction, the minimum dimension of 2.05 + 0.50 = 2.55 m is used. The results of the performed calculations are provided in table B.7 and B.8. It can be concluded that this is a useful method in longitudinal direction. In cross-direction, a core is an over-dimensioning.

Table B.7: Verification of deflections (structural elevator).

Height [m]	L _{longit.} [mm]	L _{cross} [mm]	t _{wall} [mm]	w _{longit.} [m]	w _{cross.} [m]	UC _{longit,SLS} [-]	UC _{cross,SLS} [-]
19.6	2160	2550	250	13.8	3.6	0.35	0.09
25.2	2160	2550	250	24.3	6.3	0.54	0.14

Table B.8: Applied reinforcement (structural elevator).

Height [m]	$\sigma_{t,cross}$ [N/mm ²]	A _{reinf.,cross} [mm ² /m/row]	Reinf. cross (longit.-wall) 2 rows	Tension to foundation [kN]	Applied piles (total)
19.6	0.14	39.3	Ø5-300mm	87.2	4
25.2	0.37	105.7	Ø6-250mm	235.0	4

B.5.6 (Tension) piles

The construction of steel bracings, a concrete wall or core results in tensile forces, which must be transferred to the foundation. The main complication is the construction of foundation piles in an existing structure. The new constructed piles cannot be placed below or very close to an existing wall. Resulting, piles are often placed next to the wall (Van Rossum B.V., Internal communication). Figure B.35 provides a schematization for both a wall and core.

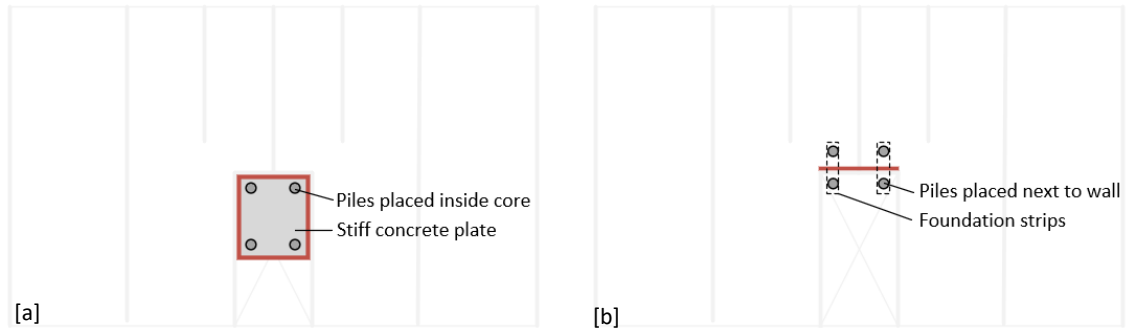


Figure B.35: Schematization of pile placement. [a] Core; [b] Wall/Bracings (Self-produced).

‘De Waalpaal’ is a company, which is specialized in the construction of (tensile) foundation piles in limited spaces. The company has four distinct systems, which can be applied to in limited space [Table B.9]. Most systems consist of steel tubular pipes with concrete inside.

Table B.9: Possible pile systems for tensile piles in limited space (De Waalpaal, 2023).

System	Placement system	Vibrations
[a] Internally driven, tubular steel pile system	Ground displacement, internally piling system	Limited
[b] Waal screw-injection pile	Mixed-in-place system. Steel screwing pipe segments, with a grout-injection at the bottom.	No
[c] Casing draw pile (including grout injection)	Ground displacement, screwing of the steel pile segments (filled with reinforced concrete).	No
[d] Waal-compact-pile	Mixed-in-place system. Steel screwing pipe segments, with a grout-injection.	No



Figure B.36: Examples of pile systems; [a] Internally driven, tubular steel pile; [c] Casing draw pile; (De Waalpaal, 2023).

Most renovation projects in Amsterdam are founded on the second sand layer, which is around NAP-20 m (Keijer & Seignette, 2019). The compression capacity of a pile is around 10 N/mm². The tensile capacity around 100 kN (Van Rossum B.V., Internal communication).

B.6 Example calculation for vertical overcapacity

Appendix B.6 will provide a calculation example of a brickwork council estate. The roof-structure will be neglected in the example but considered in the final calculations. In this example, a psi-value of 0.5 will be used. In the actual excel, this will be changed to 0.4.

1. Calculation according to historical situation

Assumed connected floor length: 10.28 m

Weight wall = thickness x height x weight = 0.22 m * 2.8 m * 19 kN/m³ = 11.70 kN/m

Table B.10: Historical situation (N1055).

Historical situation (N1055)	
4 th floor	
Permanent load floors + structural walls*	$(4.70 * 5.14) + 11.70 = 35.86$ kN/m
Variable load (100%)	$2.00 * 5.14 = 10.28$ kN/m
3 rd floor	
Permanent load floors	$(4.70 * 5.14) + 11.70 = 35.86$ kN/m
Variable load (90%)	$0.9 * 2.00 * 5.14 = 9.25$ kN/m
2 nd floor	
Permanent load floors	$(4.70 * 5.14) + 11.70 = 35.86$ kN/m
Variable load (80%)	$0.8 * 2.00 * 5.14 = 8.22$ kN/m
1 st floor	
Permanent load floors	$(4.70 * 5.14) + 11.70 = 35.86$ kN/m
Variable load (70%)	$0.7 * 2.00 * 5.14 = 7.20$ kN/m
Ground floor	
Permanent load floors	$(5.20 * 5.14) + 11.70 = 38.43$ kN/m
Variable load (60%)	$0.6 * 3.00 * 5.14 = 9.25$ kN/m
Total load	226.07 kN/m
Design load = Total load * γ_{load}	$226.07 * 1.71^* = 386.58$ kN/m

**If this value is chosen to the value of concrete (1.48), a load of $226.07 * 1.48 = 334.58$ kN/m, what results in a unity check of 1.0 for the lowest brickwork quality. This forms a better indication compared to the calculated 1.71.*

2. Calculation according to NEN 8700

Table B.11: Recalculation NEN8700.

Recalculation (NEN 8700)			
		[kN/m]	
4 th floor (Full loaded)			
	Permanent	35.86	Eq. 6.1
	Variable	$1.75 * 5.14 = 9.00$	Eq. 6.2
3 rd floor			
	Permanent	35.86	48.88
	Variable	$1.75 * 5.14 = 9.00$	47.09
2 nd floor			
	Permanent	35.86	48.88
	Variable	$1.75 * 5.14 = 9.00$	47.09
1 st floor			
	Permanent	35.86	48.88
	Variable	$1.75 * 5.14 = 9.00$	47.09
Ground floor (Full loaded)			
	Permanent	38.43	51.97
	Variable	$1.75 * 5.14 = 9.00$	55.89
Used equations		Total_Ed	247.49
6.1: $\sum_{j \geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,i} \psi_{0,i} Q_{k,i} = 1.2G + 1.3 * 0.5 * Q$			250.1
6.2: $\sum_{j \geq 1} \xi_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} = 1.15G + 1.3Q_{k,1} + \sum_{i > 1} 1.3 * 0.5 * Q_{k,i}$			

Three capacities can be considered:

- Theoretical top-up capacity ($\gamma_{\text{total}} = 1.71$) → Over-capacity = $386.58 - 250.10 = 136.48$ kN/m
- Based on foundation ($\gamma_{\text{total}} = 1.48$) → Over-capacity = $334.58 - 250.10 = 84.48$ kN/m
- Considering foundation ($\gamma_{\text{total}} = 1.4$) → Over-capacity = $316.50 - 250.10 = 66.40$ kN/m

The foundation is normative.

B.7 Parameter study vertical overcapacity (Foundation normative)

The effect of the vertical load parameters [Section 5.6] change in case the capacity of the foundation is normative. The wall opening percentage does no longer have influence. 4-storey buildings have more top-up capacity compared to 3-storey buildings [Fig. B.37b, c]. Buildings with timber floors have less top-up potential compared to ones with concrete [Fig. B.37a]. The influence of connected floor length is unchanged: larger CFL has negative effect on top-up potential [Fig. B.37a, b]. Concrete structural walls have a slightly higher top-up potential compared to brickwork walls, but the difference is negligible [Fig. B.37c].

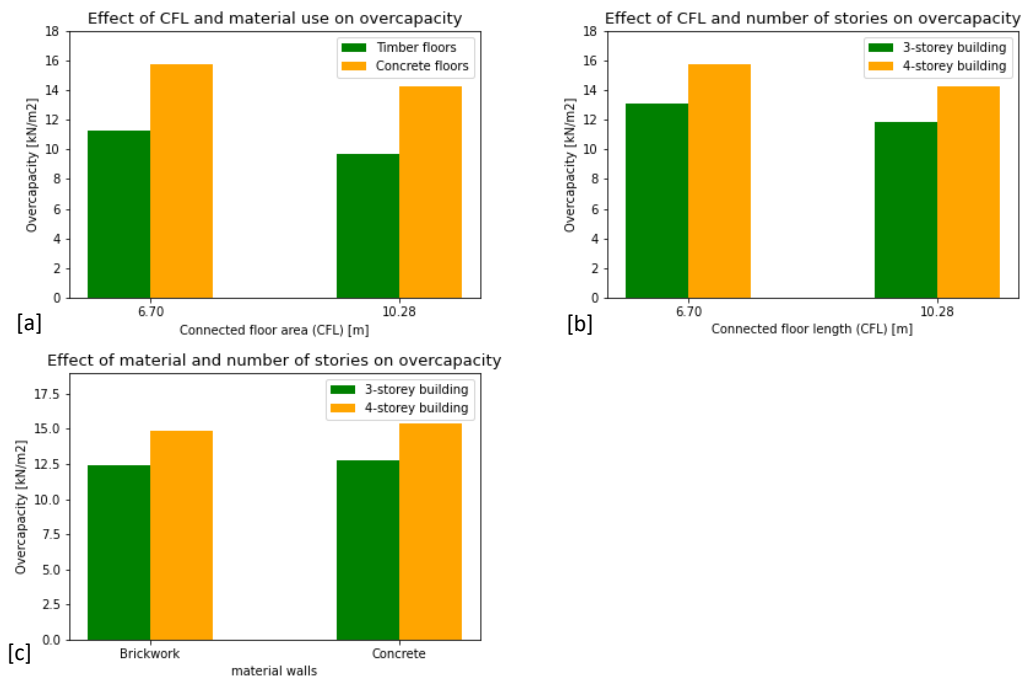


Figure B.37: Parameters vertical load transfer, in case foundation is normative (Self-produced).

B.8 Top-up possibilities for archive buildings

Table B.12 provides an overview of estimated top-up possibilities for the distinct flats in the archive. These values are based on the flowchart, provided in section 6.4.

Table B.12: Estimation of addable layers of archive variants.

Variant	Addable layers VL CLT	Addable layers VL HSB	Addable layers S	Addable layers
1	2	3	1-2	1-2
2	2	2	3	2
4	2	3	2-3	2-3
5	2	2	1-2	1-2
7	2	3	1-2	1-2
8	1-2	2	2-3	1-2
9	2	3	3	2-3
10	2	3	3	2-3
12	2	3	1-2	1-2
13	1-2	2	3	1-2

C

Environmental impact

Appendix C will provide background information to chapter 7.

Content appendix C

- C.1** LCA-calculation method
- C.2** Overview changed impact categories
- C.3** Produced impact database
- C.4** Environmental impact of concrete
- C.5** Impact calculation new constructed concrete building

C.1 LCA-calculation method

Four steps are involved in the LCA calculation method (Jonkers & Ottele, 2022) [Fig. C.1a]:

1. Definition of goal and scope (including functional unit);
2. Making a Life Cycle Inventory (LCI) analysis;
3. Performing a Life Cycle Impact Assessment (LCIA);
4. Life cycle interpretation.

Step 1: Definition of goal and scope

The first aspect is the definition of the goal of the LCA calculation. This consists of three aspects:

- Description of the **intended application**;
- Explanation for the **reason why** the LCA is performed;
- Description of the **audience for which** the LCA is intended.

The second aspect of step 1 is the definition of the scope. The scope consists of six aspects:

- Definition of the **functional unit** (product performance criteria and intended life span);
- Establishing of the **system boundaries** (Which life cycle stages included; cut-off criteria);
- Establishment of the **followed LCA methodology**: 'scoring' and evaluation method used as well as the number and types of environmental impact categories considered;
- The **type and sources** of the product life cycle inventory data used;
- The **level of quality** of the data used;
- Establishment of the **need and quality of the review** of the analysis.

Step 2: Making a Life Cycle Inventory analysis

A list of all environmental relevant inputs (raw materials and processes) and outputs (emissions and wastes) should be made. A process tree can be used to visualize the results [Fig. C.1b]. A process tree contains: a system boundary; involved life cycle stages; input and involved processes, and outputs.

Step 3: Performing a Life Cycle Impact Assessment (LCIA)

The data that is collected in step 2 is assigned to the impact categories and their Unit Equivalent. Consequently, this value is monetarized by making use of defined shadow costs per Unit Equivalents. This allows comparison of distinct scenarios. An environmental profile should be made to clarify the results.

Step 4: Life cycle interpretation

Step 4 contains discussions and conclusions on the found results. Additionally, limitations of the used methods can be discussed.

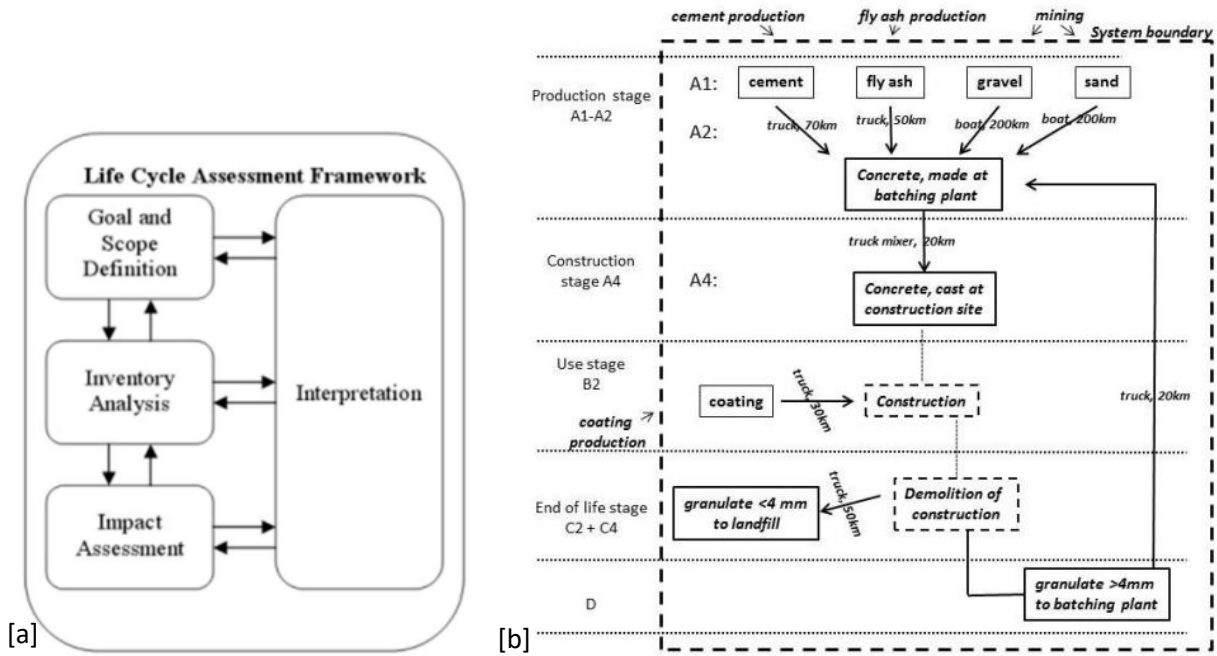


Figure C.1: [a] Procedure of LCA. [b] Example of a process tree (Jonkers & Ottele, 2022).

C.2 Overview changed impact categories

Table C.1 provides an overview in the change of environmental impact categories.

Table C.1: Change in European norm: more and changed impact categories (Jonkers & Ottele, 2022).

Number	NEN-EN 15804:A1		NEN-EN 15804+A2	
	Environmental impact category unit	Equivalent	Environmental impact category unit	Equivalent
1	Climate change -GWP100 j.	kg CO2-eq.	Climate change - total	kg CO2-eq.
2	Depletion of abiotic raw materials (excluding fossil energy carriers) - ADP	kg Sb-eq.	Climate change - fossil	kg CO2-eq.
3	Depletion of fossil energy carriers - ADP	kg Sb-eq.	Climate change - biogenic	kg CO2-eq.
4	Ozone layer depletion - ODP	kg CFK-11-eq.	Climate change – land use and change to land use	Kg CO2-eq.
5	Photochemical oxidant formation – POCP	kg C2H4-eq.	Ozone layer depletion	Kg CFC11-eq.
6	Acidification - AP	kg SO2-eq.	Acidification	Mol H+-eq.
7	Eutrophication - EP	kg P04-eq.	Freshwater eutrophication	kg p04-eq.
8	Human toxicity - HTP	kg 1,4-DCB-eq.	Seawater eutrophication	kg N-eq.
9	Ecotoxicological effects, aquatic (freshwater) - FAETP	kg 1,4-DCB-eq.	Land eutrophication	mol N-eq.
10	Ecotoxicological effects, aquatic (marine) - MAETP	kg 1,4-DCB-eq.	Photochemical ozone formation	Kg NMVOC-eq
11	Ecotoxicological effects, terrestrial - TETP	kg 1,4-DCB-eq.	Depletion of abiotic raw materials, minerals, and metals	Kg Sb-eq.
12			Depletion of abiotic raw materials and fossil fuels	MJ, net cal. val.
13			Water use	m3 world eq. deprived
14	Only 11 categories exist in NEN-EN 15804		Fine particulate emissions	Illness incidence
15			Ionizing radiation	kBq U235-eq
16			Ecotoxicity (freshwater)	CTUe
17			Human toxicity, carcinogenic	CTUh
18			Human toxicity, non-carcinogenic	CTUh
19			Land-use related impact/soil quality	Dimensionless

C.3 Produced impact database

Appendix C.3 contains the excel sheets of the data collection (EPD international, 2023).

Timber products						
Product	Distinct suppliers	Norm	Country	Density [kg/m3]	Functional unit	A1-A3 [kg CO2-eq/unit]
Sawn timber	Planet sawn timber	15804+A2	Latvia	460	m3	-1,46E+03
	C24 Graded, untreated kiln-dried and planed carcassing	15804+A2	United Kingdom	480	m3	-6,97E+02
	Imported C16+ Graded, kiln dried and planed CLS	15804+A2	United Kingdom	480	m3	-6,73E+02
	TR26 graded, untreated and treated, kiln dried and planed roof truss timber	15804+A2	United Kingdom	480	m3	-6,76E+02
	Super Yellow™ B5534 Graded, kiln-dried and treated roofing battens	15804+A2	United Kingdom	480	m3	-6,93E+02
	Swedish sawn dried timber of spruce or pine	15804+A2	Sweden	489	m3	-7,44E+02
	Australian Sawn White Cypress Timber	15804-A1	Australia	830	m3	-1,07E+03
	Softwood Timber	15804-A1	Australia	551	m3	-7,64E+02
	Sawn and Planed Timber (CLT)	15804-A1	New Zealand	480	m3	-7,14E+02
	Solid, Finger-Jointed and Laminated Timber Products including timber preservation options	15804-A1	New Zealand		m3	
	Sawn, kiln dried radiata pine			488		-7,47E+02
	Surfaced, kiln dried timber			486		-7,28E+02
	Finger jointed timber			475		-6,97E+02
	Glue laminated timber			491		-6,68E+02
	Wood panels	EPD Plywood from pine	15804+A2	Sweden	470	m3
Construction Panels (t>25 mm)		15804+A2	Latvia	460	m3	-5,60E+02
WISA spruce plywood, uncoated		15804+A2	Finland	470	m3	-7,34E+02
Cross-laminated timber (CLT)		15804-A1	New Zealand	480	m3	-7,14E+02
Xlam CLT panel		15804-A1	Australia	480	m3	-2,93E+02
CLT (Cross Laminated Timber)		15804-A1	Finland	470	m3	-7,07E+02

Using values	Name product	Country	A1-A3	Unit
Sawn timber	Planet sawn timber	Latvia	-1,46E+03	kg CO2-eq/m3
Plywood (cladding)	WISA spruce plywood, uncoated	Finland	-7,34E+02	kg CO2-eq/m3
CLT (Panels)	Cross-laminated timber (CLT)	New Zealand	-7,14E+02	kg CO2-eq/m3

Steel products						
Product	Distinct suppliers	Norm	Country	Functional unit	A1-A3 [kg CO2-eq/unit]	
Steel circular pipes	PROFILED TUBES FROM HOT AND COLD ROLLED STRIP, HEAT TREATED AND DRAWN	Hot rolled profiled tube	15804+A2	Italy	1000 kg	2,39E+03
		Hot rolled and pickled profiled tube			1000 kg	2,28E+03
		Cold rolled profiled tube (full hard)			1000 kg	2,65E+03
		Profiled and heat treated tube from hot rolled strip			1000 kg	2,57E+03
		Profiled and heat treated tube from hot rolled and pickled strip			1000 kg	2,10E+03
		Profiled tube, heat treated and straightened by hot rolled and pickled strip			1000 kg	2,15E+03
		Steel profile (For ties e.g.)	15804+A2	Chile	1000 kg	2,98E+02
		Steel pipes and profiles	15804+A2	Turkey	1000 kg	3,05E+03
		Steel Beams (all types of profiles)	15804+A2	Sweden	1000 kg	7,19E+02
	Steel beams/columns	Steel - Welded Beams and Columns	15804+A2	Australia	1000 kg	2,84E+03
		Steel Beam (I, H and U profiles)	15804+A2	Sweden	1000 kg	7,52E+02
		Steel beams (I, H and U profiles)	15804-A1	Sweden	1000 kg	1,02E+03
		Steel beams and angles	15804-A1	Italy	1000 kg	9,81E+02
		Steel Wide Flange Beam manufactured from steel scrap	15804-A1	Mexico	1000 kg	8,55E+02

Using values	A1-A3	Unit
Ties	2,98E+02	kg CO2 eq/ 1000 kg
Profiles	7,19E+02	kg CO2 eq/ 1000 kg

Concrete and reinforcement products								
Product	Distinct suppliers	Exposure class	Norm	Country	Functional unit	A1-A3 [kg CO2-eq/unit]		
Concrete	Ready Mix Concrete Lafarge - Attica	C16/20-16mm	X0	Greece	m3	1,94E+02		
		C16/20-31.5mm	X0			1,75E+02		
		C20/25-16mm	XC1, XC2			2,14E+02		
		C20/25-31.5mm	XC1, XC2			1,90E+02		
		C25/30-16mm	XC1, XC2, XC3			2,21E+02		
		C25/30-31.5mm	XC1, XC2, XC3			2,05E+02		
		C25/30-31.5mm-SEASIDE	XS1, XS2			2,17E+02		
		C25/30-31.5mm-WATERPROOF	XC1, XC2, XC3			2,15E+02		
		C30/37-16mm	XC1, XC2, XC3, XC4			2,57E+02		
		C30/37-31.5mm	XC4, XD1, XF1, XF4, XA1			2,39E+02		
		C30/37-31.5mm-SEASIDE	XS1, XS2, XS3			2,49E+02		
		C30/37-31.5mm-WATERPROOF	XC1, XC2, XC4, XS1, XS2, XS3			2,42E+02		
		C35/45-31.5mm	XC4, XS3, XD3, XM1			2,99E+02		
		C40/50-31.5mm	XC4, XS3, XD3, XM2			3,39E+02		
		PILES (C20/25, C25/30, C30/37)	-			2,76E+02		
	KOULAS GEOHEAT L.T.D. Ready mixed concrete	C20/25	XC1	15804+A2	Sweden	m3	2,35E+02	
		C25/30	XC3	15804+A2			2,59E+02	
		C30/37	XS1, XS2	15804+A2			2,85E+02	
	C30/37_30S24C1000000_Interbeton	C30/37	XS2,S4	15804+A2	Greece	m3	2,49E+02	
	C40/50	C40/50	XD3	15804+A2	Greece	m3	2,93E+02	
	UK manufactured generic ready-mixed concrete	C30/37	XC1, XC2, XC3	15804-A1	German	m3	2,46E+02	
	Goudse betonmortel centrale	C20/25	XC2	15804-A1	Netherlands	m3	1,01E+02	
15804-A1				Netherlands			m3	1,06E+02
15804-A1				Netherlands			m3	1,10E+02
15804-A1				Netherlands			m3	1,11E+02
Literature (Nováková et al., 2023)	C30/37	XF4, XD3	15804-A1	Netherlands	m3	1,17E+02		
REINFORCEMENT STEEL PRODUCTS	C30/37	X0, XC1-4, FX1, XD1-3, XA1-4; XS1-3	-	Finland	m3	1,15E+02		
			-			1,50E+02		
			-			1,60E+02		
			-					
			-					
Reinforcing steel	Steel for concrete reinforcements		15804+A2	Latvia	1000 kg	2,12E+03		
			15804+A2			Spain	1000 kg	3,94E+03

Using values	A1-A3	Unit
C30/37, XC1-3	1,10E+02	kg CO2-eq/m3
Reinforcement steel	2,12E+03	kg CO2-eq/ 1000 kg

Figure C.2: Environmental impact database.

Product	Distinct suppliers	Non-structural products					Functional unit	A1-A3 [kg CO2-eq/unit]	
		Norm	Country	Thickness [mm]	Density [kg/m ³]				
Fermacell	Gypsum Fibre board Fermacell GmbH	15804:A1	Germany	12.5	14.75		m2	1,14E+00	
Gypsum plasterboard_fire resistance	Fire resistant plasterboard	15804+A2	Sweden	12.5	10.7		m2	1,89E+00	
	FIBRANGypsum plasterboards	15804+A2	Italy	12.5	9.3		m2	1,87E+00	
	Fire plasterboard	15804+A2	Portugal	12.5	10.2		m2	1,86E+00	
	Red Corex	15804+A2	Turkey	12.5	10.3		m2	1,81E+00	
				15	12,4		m2	2,59E+00	
	ISOPLAC plasterboards	15804+A2	Spain	12.5	9.3		m2	1,87E+00	
Insulation		Norm	Country	Thickness [mm]	Density [kg/m ³]	Thermal conductivity (W/(mK))	Functional unit	A1-A3 [kg CO2-eq/unit]	
	NaturBoard VENTI GVB, NaturBoard VENTI	15804:A1	Belgium	50	50	0,035	m2	2,61E+00	
	ODE Starflex & Evomineral 031 Glasswool Insulation Materials	15804:A1	Belgium	100	15	0,036-0,039	m2	1,60E+00	
		15804:A1	Belgium	100	21	0,036-0,039	m2	2,22E+00	
	Glass mineral wool insulation λ 0.036 – 0.039 W/mK	15804:A1	Belgium	100	11,5	0,04	m2	1,21E+00	
	Mineral Plus, Mineral Wool Products λ 0.035 W/mK	15804:A1	Belgium	100	19,1	0,035	m2	2,60E+00	
	Mineral Plus, Mineral Wool Products λ 0.037 W/mK	15804:A1	Belgium	100	15	0,037	m2	2,33E+00	
	UK plants - Glass mineral wool insulation λ 0.031-0.033 W/mK	15804:A1	Belgium	100	48	0,031-0,033	m2	5,12E+00	
	Glass mineral wool insulation λ 0.034 - 0.035 W/mK	15804:A1	Belgium	100	19,5	0,034-0,035	m2	2,06E+00	
		15804:A1	Belgium	100	26,5	0,034-0,035	m2	2,80E+00	
	Using values							A1-A3	Unit
Fermacell (22 mm)							2,01	CO2-eq/m2	
Gypsum board									
				12.5 mm		1,81	CO2-eq/m2		
				15 mm		2,59	CO2-eq/m2		
Insulation									
<i>(assumption: linear relation thickness - impact)</i> 90 mm							1,21	CO2-eq/m2/100mm	
				100 mm		1,09	CO2-eq/m2		
				100 mm		1,21	CO2-eq/m2		
				170 mm		2,06	CO2-eq/m2		

Figure C.3: Environmental impact database (continue).

C.4 Environmental impact of concrete

In general, three types of cement can be defined (VVMcem., 2019):

- CEM I: Portland cement with a maximum of 5% other contents;
- CEM II: All types of blends of Portland cement (at least 65% Portland cement), with e.g., shale, fly ash, blast furnace slag;
- CEM III: Blast furnace/ Portland cement mixture in 3 grades: A, B and C; where CEM III/A contains the least (40%) and CEM III/C contains the most (90%) blast furnace slag.

Portland cement clinker (binder) is the main contributor in CO₂ emissions of concrete. CEM I contains approximately 100% Portland cement clinker. This in contrast to CEM III (Blast furnace cement) in which the clinker reduced to approximately 30% (Kerkhoven, 2022-b). In the Netherlands mainly CEMIII/B is applied.

Within the International EPD system limited concrete data is available. Also, the Dutch suppliers hardly provide any data about the emissions of the used concrete. The International EPD system only contains a mixture from Greece, which is clearly specified with properties (EPD International, 2023). Also, a Dutch company provides data on their website (Goudse Betonmortel Centrale, 2023)

Table C.2: GWP concrete.

Country	Company	Cement	Class	GWP [kg CO ₂ -eq./m ³]
Greece	HERACLES Group of Companies	CEM II/B	C30/37, XC1,2, 3	2,42E+02
Netherlands	Goudse Betonmortel Centrale	CEM III	C30/37, XC1,2,3	1,10E+02

In a report of CE Delft, it is stated that the average climate impact of concrete (containing CEM III) for residential housing varies between 90 and 140 [kg CO₂-eq./m³] (Bijleveld et al., 2013). This report stimulates the use of CEM III, but states that in 2013 mainly CEM I was used. The author concludes that the value of Goudse Betonmortel Centrale is a reasonable assumption for concrete CEM III.

A reinforcement percentage of 0.6% is assumed in the reinforcement calculation (De Vree, n.d.-a). The GWP equals 2,12E+03 [kg CO₂-eq./1000 kg] (EPD International, 2023). Table C.3 provides the calculation for the impact of reinforced concrete.

Table C.3: Calculation impact reinforced concrete.

Material	Volume [m ³]	Impact [kg CO ₂ -eq./m ³ concrete]
Concrete	1	110
Reinforcement steel (7850 kg/m ³)	$(0.6/100)*1 = 0.006$	$0.006 * 7850 * 2120/1000 = 100$
	Total	210

C.5 Impact calculation new constructed concrete building

Swietenhof Amsterdam is a residential complex, consisting of six floors. The building is defined as reference building to determine the environmental impact of new construction. Figure C.4 provides an impression of the project. Figure C.5 and C.6 provide structural drawings. Appendix X.9 provides excel sheets of the LCA calculation.



Figure C.4: Impression Swietenhof Amsterdam
(Van Rossum B.V., Internal communication).

C.5.1 Structural aspects

Vertical load transfer

The 6-storey building was founded on reinforced concrete piles and foundation beams. The ground floor was constructed out of a ribcassette floor [d=350 mm]. The structure on top consists of a concrete wall structure in cross-direction [d=250 mm]. The cross-directional orientated façade walls, and the ones at the dilatation, have a smaller thickness [200 mm]. The distinct floors can be reached by a prefabricated elevator or stairwell. These floors were made of reinforced concrete [d=230 mm], finished with 70 cm sand-cement. The gallery floors and prefabricated balconies are 150 and 220 mm thick, respectively. The balconies were constructed on top of cantilevering prefabricated beams. The roof has been cladded with prefabricated roof-elements. These consist of 3 beams [20 x 171 mm], cladded on two sides with a 5 mm thick timber-based panel.

Stability

The complex has been divided into two, by means of a dilatation. In cross-direction, the structural concrete walls provide enough stiffness. In longitudinal direction, one part is stabilized by a structural wall [d=250 mm] (behind the elevator). The other part is stabilized by moment-fixed connections between floors and walls.

Concrete used

The impact of the distinct materials is provided in table C.4. The impact of a ribcassette floor is estimated based on the amount of concrete (C50/60; XC1). The impact equalizes: $210 \text{ [kg/m}^2\text{]} * 260 \text{ [kg CO}_2\text{-eq./m}^3\text{]} / 2500 \text{ [kg/m}^3\text{]} = 21.84 \text{ [kg CO}_2\text{-eq./m}^2\text{ floor]}$. 210 kg/m² is the weight of the ribcassette floor per square meter floor area.

C.5.2 Results

Table C.5 provides the results of the LCA, expressed in values. These values are used in figure 7.6. The LCA itself is described in section 7.3.

Table C.4: Used concrete in the building.

Element	Class	Impact kg CO ₂ -eq./m ³ (No reinforcement)	Impact kg CO ₂ -eq./m ³ (0.6% reinforcement)*
Walls/floors	C30/37; XC1	1.10E+02	2.10E+02
Prefab	C45/55; XC1	1.50E+02	2.50E+02
Prefab	C50/60; XC1	1.60E+02	2.60E+02
Foundation	C30/37; XC4, XF1	1.17E+02	2.17E+02

*Appendix C.4 for calculation procedure

Table C.5: Results LCA. Values used in figure 7.6. RSL=75y.

Original flat (→) New building (↓)	3-storeys GWP [10 ⁻¹ kg CO ₂ -eq./y/m ² new floor area]	4-storeys GWP [10 ⁻¹ kg CO ₂ -eq./y/m ² new floor area]
4-storeys	70.90 (+1)	-
5-storeys	41.33 (+2)	82.66 (+1)
6-storeys	31.32 (+3)	46.98 (+2)
7-storeys	-	35.09 (+3)

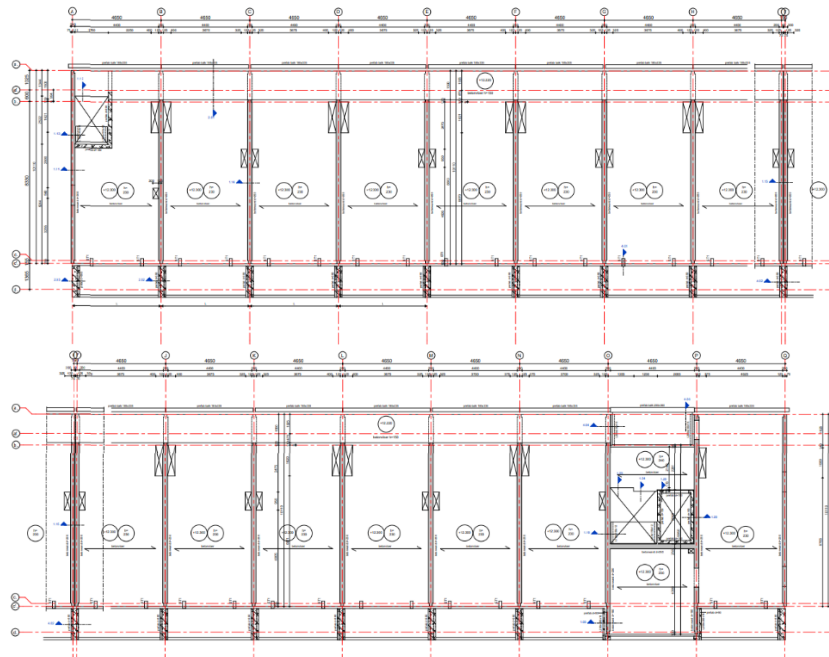


Figure C.5: Floorplan 4th floor (Van Rossum B.V.; Internal communication).

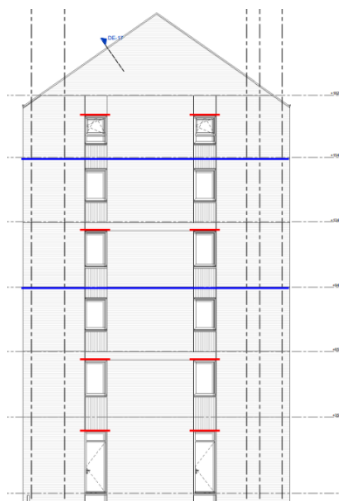


Figure C.6: East facade (Van Rossum B.V.; Internal communication).

D

Variant design

Appendix D will provide background information to chapter 8.

Content appendix D

- D.1** Estimation of number of layers needed to be top-upped
- D.2** Description variants
- D.3** CLT calculation
- D.4** HSB calculation
- D.5** Environmental impact HSB extension

D.1 Estimation of number of layers needed to be top-upped

	A	B	C	D	E	F
1	Estimation council estates in the Netherlands (Post-war)					
2	Housing data					Formule B
3	Housing crisis Netherlands 2023	325000				
4	Number of council houses (TU Delft, 2022)	847000	Houses			
5						
6	Devison council estates (Lijbers et al., 1984)					
7	Percentage 3-storey	40	contains ->	6	Houses	
8	Percentage 4-storey	60	contains ->	8	Houses	100-B7
9						
10	Result					
11	Number of council houses	119992	Flats (defined as single apartment)			$((B7/100)*B4/D7) + ((B8/100)*B4/D8)$
12						
13	Specification traditional/ non-traditional	Percentage [% Houses [-] (Oorschot et al., 2018)				Formule C
14	Traditional mixed-construction	53	63596			$B15/100*B11$
15	Non-traditional construction	47	56396			$B16/100*B11$
16						
17	Percentage top-up	Percentage [% Added houses				
18	Percentage solving housing crisis	70				
19	2 layers added	90	215985			$(B20/100)*B11*2$
20	1 layer added	10	11999	70% crisis ↓		$(B21/100)*B11$
21			227984	>	227500	$SOM(C20:C21)$

Figure D.1: Excel sheet to estimate number of layers to be top-upped.

D.2 Description variants

The research developed two main variants that needs to be further investigated:

1. **Modular CLT-construction** – A modular CLT-construction can potentially have a low material-footprint per year (considering a large lifespan). Especially for the later-built flats [Residual lifespan council estate = 23 years], a modular CLT-construction could be more sustainable.
2. **Non-modular HSB-construction** – The material usage and self-weight of an HSB-construction is lower compared to CLT (considering a similar RSL) [Section 7.2]. That motivates to use HSB-construction for a non-modular top-up.

The distinct variants will be explained below in more detail.

D.2.1 Modular CLT-construction

The CLT variant increases the RSL of the units, to lower the material usage per year. Single units are designed, which can be coupled to form double units. A number of spatial regulations are considered in the unit design:

Bouwbesluit Online 2012

1. Apartment for one person → floor area $\geq 24 \text{ m}^2$
2. Apartment for two persons → Floor area $\geq 50 \text{ m}^2$
3. Internal height $\geq 2600 \text{ mm}$

Transportation [Section 6.3.3]

4. Width $\leq 3.50 \text{ m}$
5. Height $\leq 4.15 \text{ m}$
6. Length $\leq 27.50 \text{ m}$

CLT supplier (Stora Enso, 2022)

7. Standardized widths: 2.25 m/ 2.45 m/ 2.75 m/ 2.95 m/ (*wider panels: special delivery*)
8. Length $\leq 18.00 \text{ m}$

Figure D.2 provides an overview of the designed units. The maximum outer dimensions are $L \times B \times H = 9.45 \times 3.50 \times 2.95$ m. By linking two single person units, a double-person unit can be realized (area > 50 m²). The **stability wall** is localized in the middle of the unit. That allows an open front and back façade for daylight. The designed unit includes part of the gallery. Note that the façade is located at the outside of the balcony, to prevent a thermal bridge. By **disconnecting the gallery floor** and applying a floating screed, the contact noise can be minimized.

With a unit-width of 3500 mm, approximately four units can be placed next to each other above one 'council estate unit' [Fig. D.5]. Resultingly, the loadbearing lines from the existing structure do not continue. A **steel interlayer** needs to be realized. Five steel beams must be placed, elevated (≈ 5 cm) of the roof slab [Fig. D.3; D.4]. The steel interlayer transfers the horizontal wind load of the stabilizing walls to the original stability wall.

The unit walls can be realized from one plate (2.95 x 9.45 m). The span of the floors and roofs equals 3.50 m. As this is larger than the maximum plate width, individual panels will span in between the walls. The floors span between the walls. The roof is placed on top of the wall-panels, to minimize the wall height.

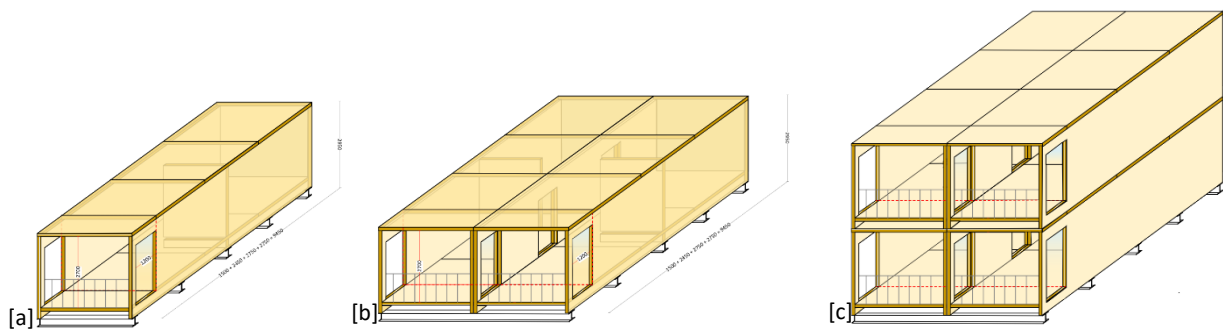


Figure D.2: Design of modular units. [a] Single unit (>24 m²); [b] Double person unit (>50 m²); [c] Example of stacked unit (Self-produced).

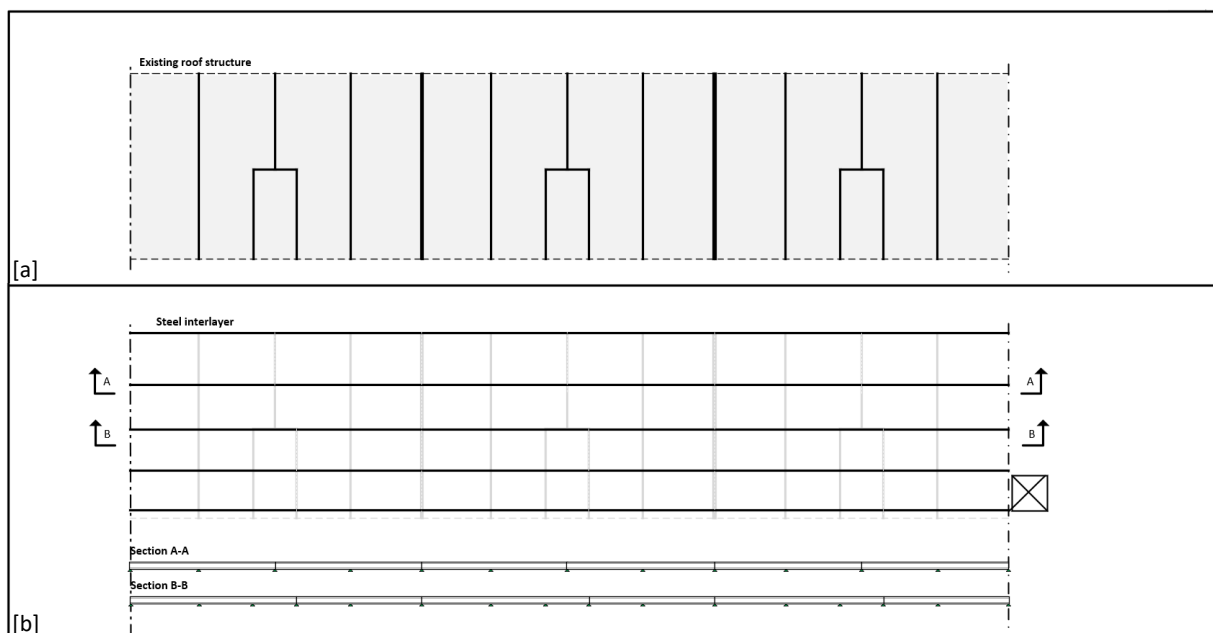


Figure D.3: Steel interlayer; Excluding additional room (Self-produced).

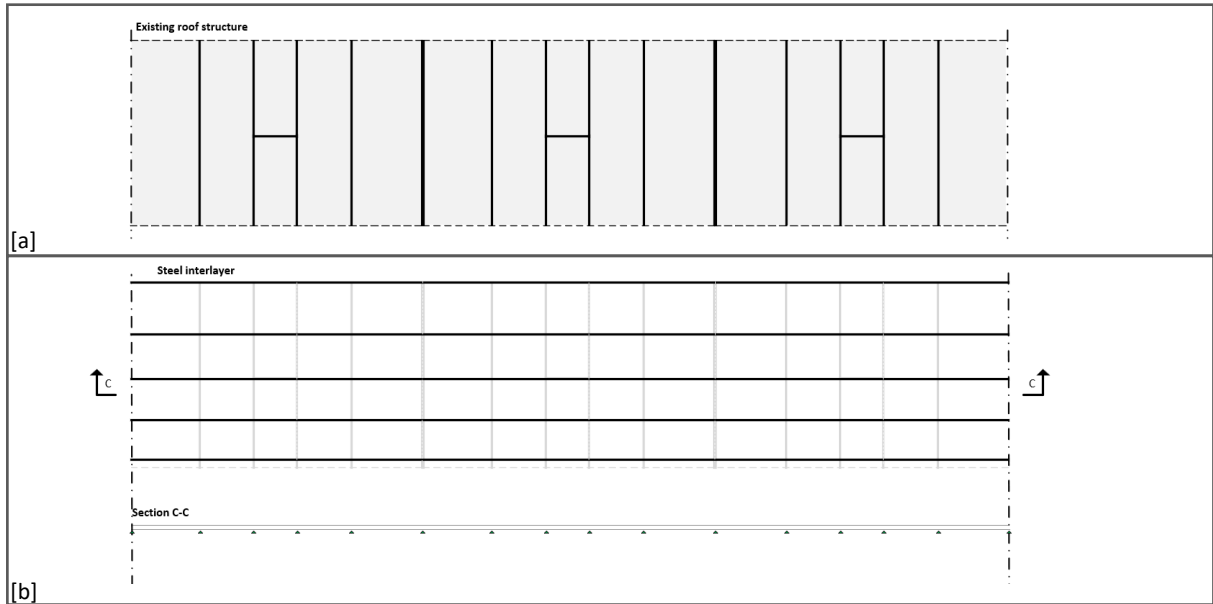


Figure D.4: Steel interlayer; Including additional room (Self-produced).

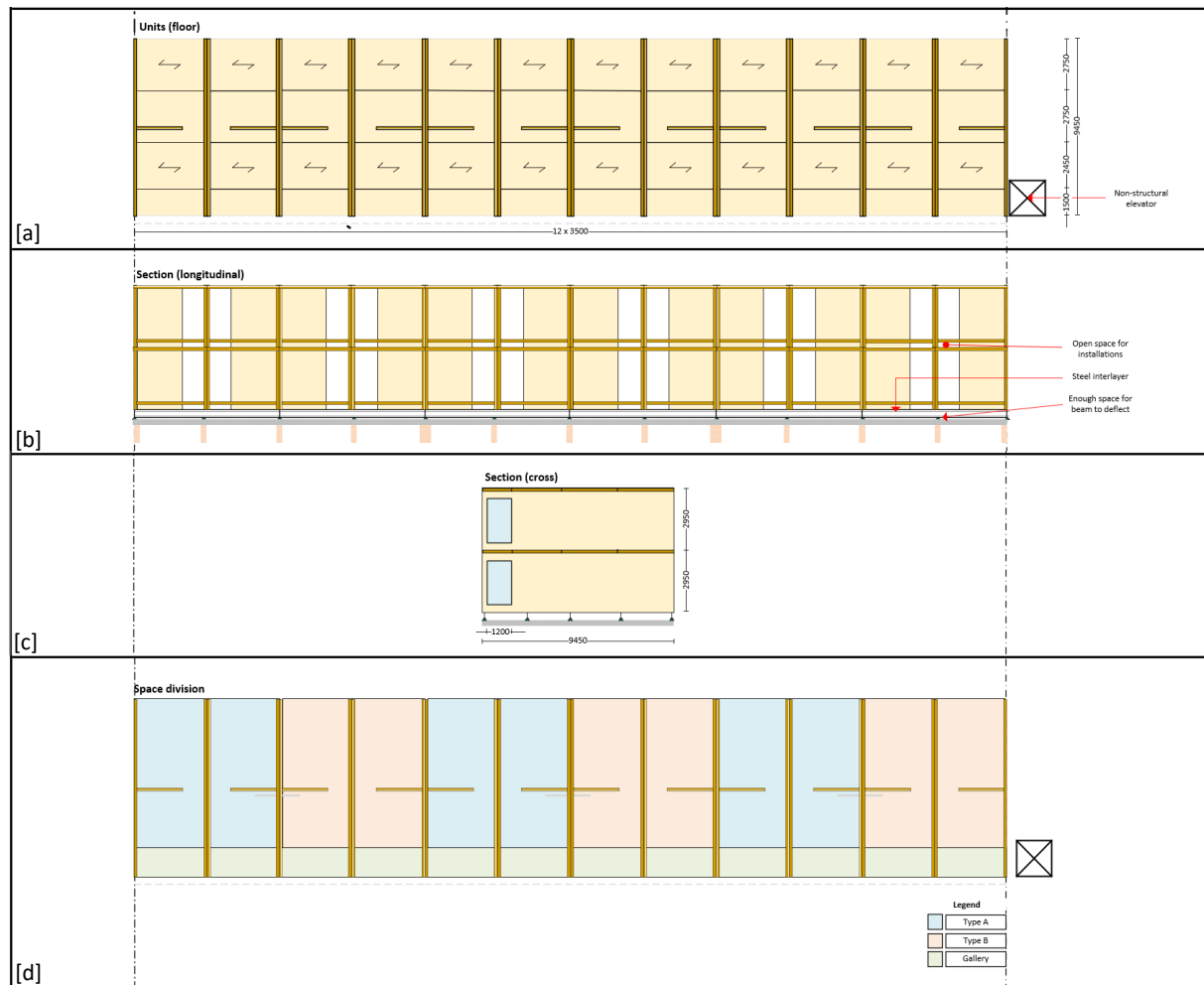


Figure D.5: CLT modular unit's drawings (Self-produced).

D.2.2 Non-modular HSB-construction

The case studies addressed the structural effectiveness to place structural walls of a vertical extension above the existing ones [Section 6.2]. An effective method must then count for extreme spans or must be more specified for distinct spans (parametric approach).

This variant **continues the bearing lines** of the structural walls. Especially for 2-layers, an elevator is obligatory (Bouwbesluit 2012). This **elevator** is placed at the ends of a block, non-structural [Section 7.2.4]. To access the houses, a 1.20 m width gallery is constructed. The end porches continue to the top-up, to provide **access by stairs** [Fig. D.6; D.7].

The **platform method** is used for the design of the top-up [Section 6.3.1]. This is an applied, faster construction method [Fig. D.6e; D.7e]. Between distinct houses, **apartment-separation walls** must be used [Fig. D.6f; D.7f]. Similar holds for the walls around the continuing stairwell. The dilatations should be **horizontally linked** in case stability becomes critical [Section 6.3.5]. That increases the stability enough to top-up 2-3 layers.

The archive inspection indicated roof structures, which are too thin to bear a residential load. Distinct options are considered for the **existing roof-structure** (Tulamo et al., 2014):

1. Reinforcing of original ceiling;
2. Replacement of original ceiling;
3. Original ceiling + additional floor.

Option 1 – This option strongly increases the amount of concrete and weight of the structure.

Option 2 – This option has two disadvantages. First, it lowers the diaphragm action. Second, it disrupts the current residents of the top-floor highly. Especially the latter forms the reason that it is not applied in practise.

Option 3 – This is the chosen option. This method minimally disrupts the existing residents. The new constructed floor is made of wood, which has CO₂ capture [A1-A3]. Unfortunately, there is a bit less overcapacity. In case of a slanted or pitched roof, the whole roof structure must be demolished.

The **floor beams span**, simply-supported, between structural walls. These beams are spaced 600 mm, with a timber-based panel on top of it. A small elevation should be constructed between the existing roof slab and floor beams. That allows the floor beams to deflect.

The **walls** are constructed of columns, clad with wood-based panel. The housing separation walls consist of a double layer of columns, separated by a cavity. The inner walls only have one layer of columns.

The **stabilizing wall** is placed above the existing wall. That allows transfer of the wind force, directly to the existing stabilizing structure.

Figures D.6 and D.7 provide drawings of the top-up design. Also, a space division is provided [Fig. D.6f; D.7f]. The extension contains in three different apartments, all with an area larger than 50 m².

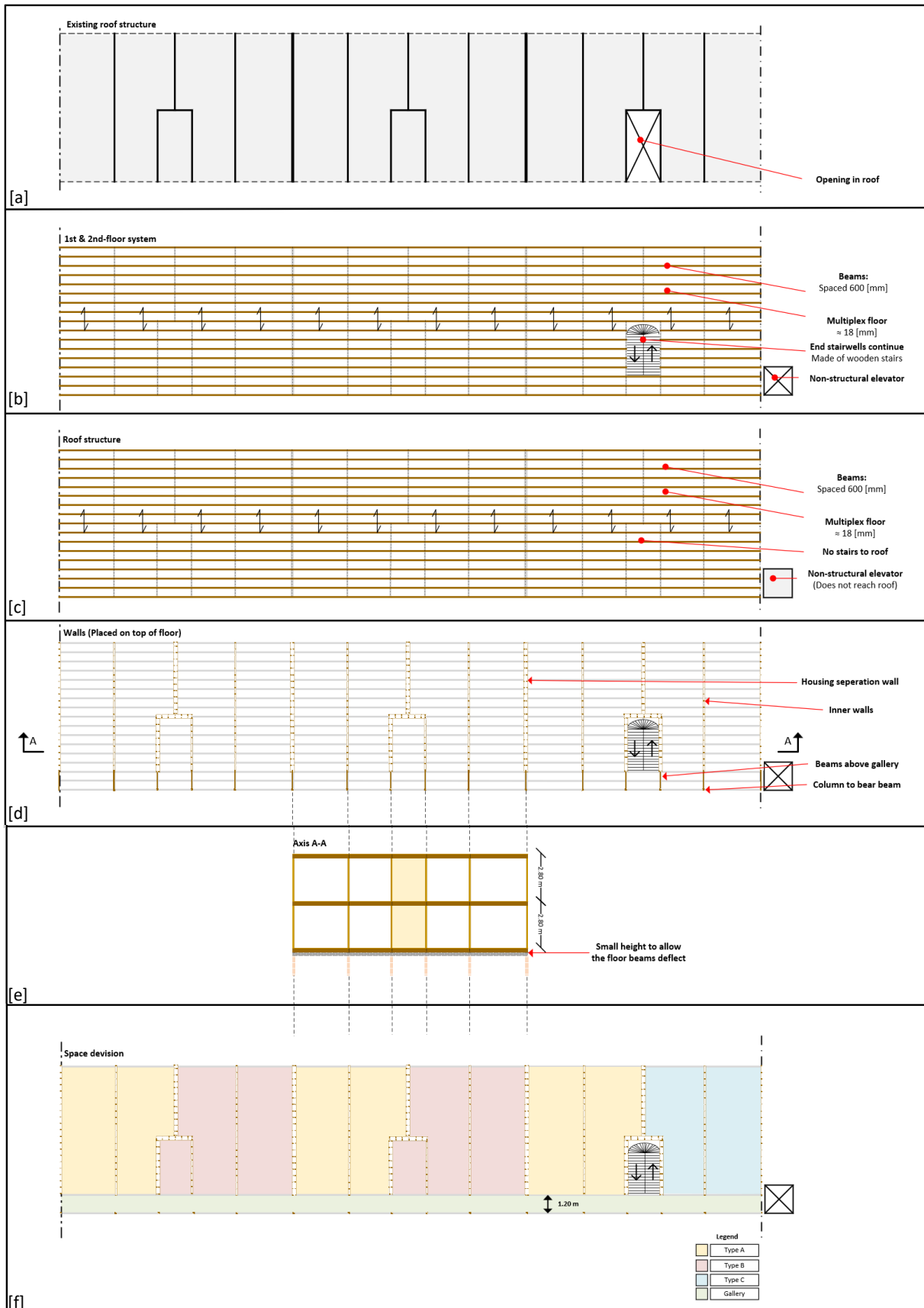


Figure D.6: HSB-construction; council estate excluding additional room (Self-produced).

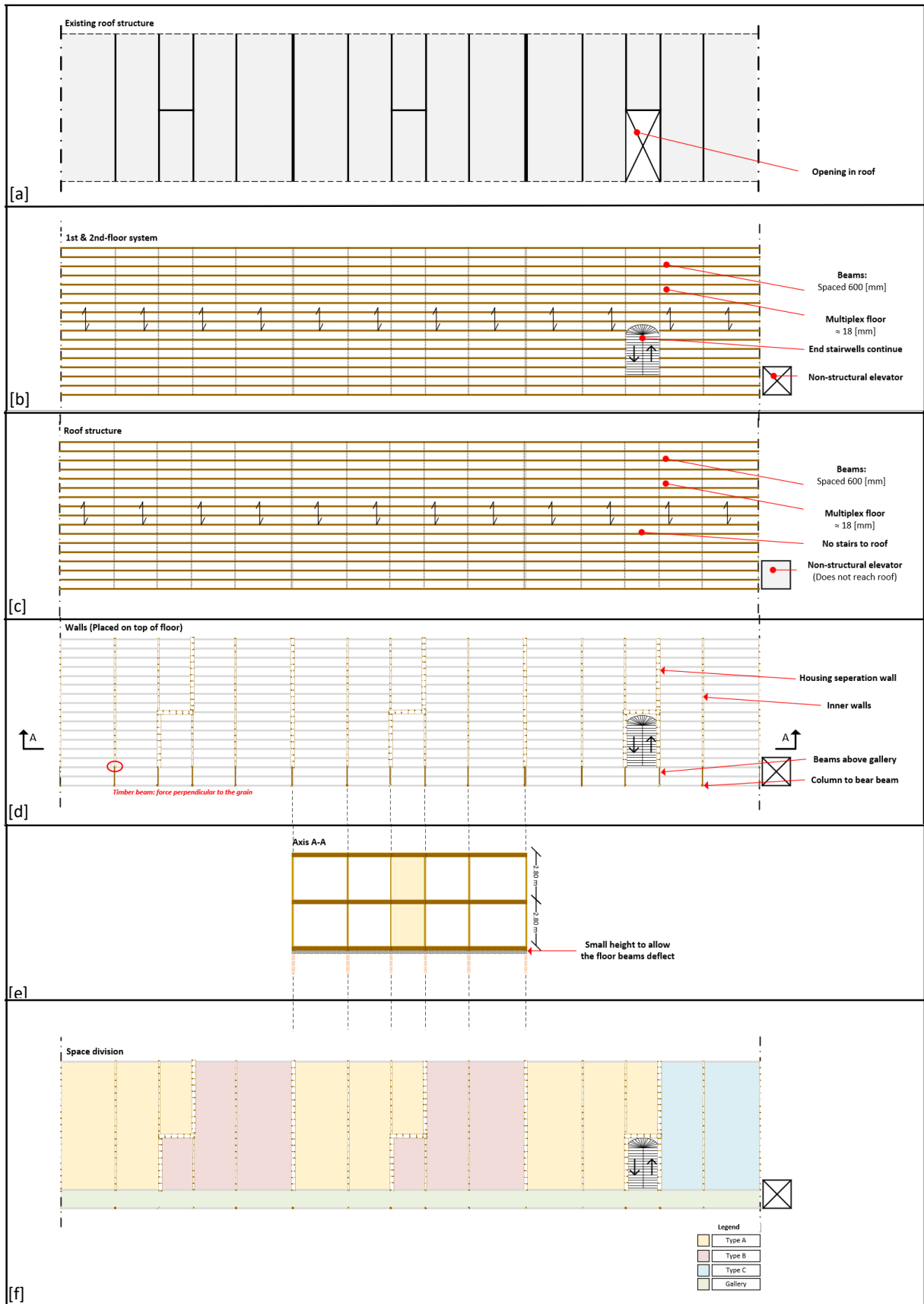


Figure D.7: HSB-construction; council estate, including additional room (Self-produced).

D.3 CLT calculation

Calculation of CLT differs from ‘general timber’, as it consists of crosswise laminated layers. Appendix D.3 will provide the structural principles for the CLT design. The **Swedish CLT-wood handbook** (Borgström & Fröbel, 2019) contains a detailed description of CLT properties, and calculation rules, which will be used.

D.3.1 General section properties CLT

CLT construction consists of crosswise laminated layers, to provide strength in two directions. The major direction is determined by the orientation of the wood-strips of the outer layers [Fig. D.8]. Considering floors, the outer layers are orientated in span direction. In case of walls, these are vertically orientated.

The section properties are determined by the layers which provide mostly the stiffness [Table D.1], e.g., stiffness around the y-axis is caused by the x-directional layers: layer 1, 3, 5 ... [Fig. D.8]. **Steiner** is used to determine the combined moment of inertia: $I_{com} = I_{self} + A \cdot a^2$. **Loading out of plane** results in internal sliding between layers, what causes ‘rolling shear’. The ‘Gamma method’ includes this in the **effective moment of inertia** [Table D.2].

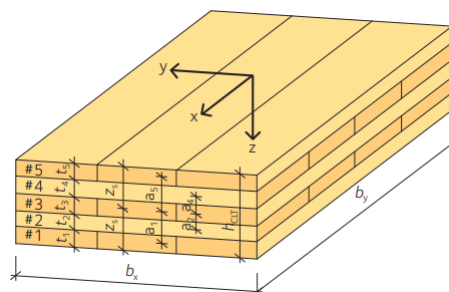


Figure D.8: Definition directions: b_x = minor axis; b_y = major axis (Borgström & Fröbel, 2019: p.40).

To calculate the section properties for multiple panels, an excel sheet is made [Appendix X.10.1]. This sheet is validated with table 3.11, 3.12, 3.14, and 3.15 of the CLT handbook (Borgström & Fröbel, 2019: p.45 - 52)

Table D.1: General section calculations (Borgström & Fröbel, 2019: p.41).

Property	Parallel with the main direction of the load	Perpendicular to the main direction of the load
Gravity centre	$z_s = h_{CLT}/2$	$z_s = h_{CLT}/2$
Gross Area [mm ²]	$A_{x,brutto} = b_x h_{CLT}$	$A_{y,brutto} = b_y h_{CLT}$
Net Area [mm ²]	$A_{x,net} = b_x h_x$ $h_x = t_1 + t_3 + t_5$	$A_{y,net} = b_y h_y$ $h_y = t_2 + t_4 + \dots$
Net, Moment of inertia [mm ⁴]	Rotation around the y-axis $I_{x,net} = \sum \frac{E_{x,i}}{E_{ref}} \cdot \frac{b_x t_i^3}{12} + \sum \frac{E_{x,i}}{E_{ref}} \cdot b_x t_i a_i^2$ $= \frac{b_x t_1^3}{12} + b_x t_1 a_1^2 + \frac{b_x t_3^3}{12} + b_x t_3 a_3^2 + \frac{b_x t_5^3}{12} + b_x t_5 a_5^2 + \dots$ Rotation around the z-axis $I_{z,x,net} = \sum \frac{E_{x,i}}{E_{ref}} \cdot \frac{t_i b_x^3}{12} = \frac{t_1 + t_3 + t_5 + \dots}{12} b_x^3$	Rotation around the x-axis $I_{y,net} = \sum \frac{E_{y,i}}{E_{ref}} \cdot \frac{b_y t_i^3}{12} + \sum \frac{E_{y,i}}{E_{ref}} \cdot b_y t_i a_i^2$ $= \frac{b_y t_2^3}{12} + b_y t_2 a_2^2 + \frac{b_y t_4^3}{12} + b_y t_4 a_4^2 + \dots$ Rotation around the z-axis $I_{z,y,net} = \sum \frac{E_{y,i}}{E_{ref}} \cdot \frac{t_i b_y^3}{12} = \frac{t_2 + t_4 + \dots}{12} b_y^3$
Net, Section modulus [mm ³]	$W_{x,net} = \frac{2 \cdot I_{x,net}}{h_{CLT}}$	$W_{y,net} = \frac{2 \cdot I_{y,net}}{h_{CLT}}$
Static moment of rolling shear [mm ³]	$S_{R,x,net} = b_x t_1 a_1$	-
Static moment of longitudinal shear	3-layers: $S_{x,net} = b_x t_1 a_1$ 5-layers: $S_{x,net} = b_x t_1 a_1 + b_x \cdot \frac{t_3^2}{4 \cdot 2}$	-

Table D.2: Effective Moment of inertia (Loading out of plane) (Borgström & Fröbel, 2019: p.49-52).

Property	3-layered panel	5-layered panel
γ_1	1	$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_{x1} t_1}{l_{ref}^2} \cdot \frac{t_2}{G_{9090,2}}}$
γ_3	$\gamma_3 = \frac{1}{1 + \frac{\pi^2 E_{x3} t_3}{l_{ref}^2} \cdot \frac{t_2}{G_{9090,2}}}$	1
γ_5	-	$\gamma_5 = \frac{1}{1 + \frac{\pi^2 E_{x5} t_5}{l_{ref}^2} \cdot \frac{t_4}{G_{9090,4}}}$
$I_{x,ef}$	$I_{x,ef} = \frac{b_x t_1^3}{12} + b_x t_1 a_1^2 + \frac{b_x t_3^3}{12} + \gamma_3 b_x t_3 a_3^2$ $= b_x (\frac{2 \cdot t_1^3}{12} + (1 + \gamma_3) t_1 a_1^2)$	$I_{x,ef} = \frac{b_x t_1^3}{12} + \gamma_1 b_x t_1 a_1^2 + \frac{b_x t_3^3}{12} + \frac{b_x t_5^3}{12} + \gamma_5 b_x t_5 a_5^2$
Effective radius of inertia	$i_{x,eff} = \sqrt{\frac{I_{x,ef}}{A_{x,net}}}$	-

The Swedish handbook specifies for CLT general wood properties [Table D.3 – D.5].

Table D.3: Strength properties CLT based on strength properties of timber boards. About directions, Fig. D.8 (Borgström & Fröbel, 2019: Table 3.6).

Characteristic strength properties	CLT panels C24 [MPa]	
Bending strength	$f_{m,x,k}$ $f_{m,y,k}$	24 24
Tension strength, in plane	$f_{t,0,x,k}$ $f_{t,0,y,k}$	14.5 14.5
Tension strength, perpendicular to the plane	$f_{t,90,x,k}$ $f_{t,90,y,k}$	0.4 0.4
Compression strength, in plane	$f_{c,0,x,k}$ $f_{c,0,y,k}$	21 21
Compression strength, perpendicular to the plane	$f_{c,90,z,k}$	2.5
Shear strength, longitudinal shear	$f_{v,090,xlay,k}$ $f_{v,090,ylay,k}$	4 4
Shear strength, rolling shear	$f_{v,9090,xlay,k}$ $f_{v,9090,ylay,k}$	1.1 ¹⁾ or 0.7 ²⁾ 1.1 ¹⁾ or 0.7 ²⁾

¹⁾ Used for CLT panels with edge-glued boards or where the board thickness is less than 45 mm and the width to thickness ratio for the boards is equal to or greater than 4.

²⁾ Used for CLT panels where the boards are not edge-glued and where the width to thickness ratio for the boards is less than 4, or where grooves have been cut into the boards.

Table D.4: Characteristic stiffness values for CLT panels based on stiffness properties of timber boards. About directions, Fig. D.8 (Borgström & Fröbel, 2019: Table 3.7).

Characteristic stiffness values	CLT panel C24 [MPa]	
Mean value of modulus of elasticity	$E_{0,x,mean}$ $E_{90,x,mean}$ $E_{0,y,mean}$ $E_{90,y,mean}$	11000 0 ¹⁾ or 400 ²⁾ 11000 0 ¹⁾ or 400 ²⁾
Fifth percentile value of modulus of elasticity	$E_{0,x,05}$ $E_{0,y,05}$	7400 7400
Mean value of modulus of shear	$G_{090,xlay,mean}$ $G_{090,ylay,mean}$	690 690
Mean value of modulus of rolling shear	$G_{9090,xlay,mean}$ $G_{9090,ylay,mean}$	50 50

¹⁾ Used for CLT panels without edge-glued boards.

²⁾ May be used for CLT panels with edge-glued boards

Table D.5: Density of CLT panels (Borgström & Fröbel, 2019: Table 3.8).

Density	CLT panels C24 [kg/m ³]	
Characteristic value	$\rho_{xlam,k}$	350
Mean value	$\rho_{xlam,mean}$	420

The structure will be calculated according to the NEN-EN 1995. This norm specifies material, modification, and deformation factors to calculation timber [Table D.6 till D.8].

Table D.6: Material factor (EN 1995: Table 2.3).

Type of Wood	γ_m
Sawn wood	1.3
Limed, laminated wood	1.25
LVL, multiplex, OSB	1.2
Particle board	1.3
Fiberboard, hard	1.3
Fiberboard, medium	1.3
Fiberboard, MDF	1.3
Fiberboard, soft	1.3
Connections	1.3
Metal bonding Plate	1.25

Table D.7: Values for k_{mod} [NEN-EN 1995: Table 3.1].

Material	Norm	Service-class	Load duration class					
			Permanent	Long-term	Medium-term	Short-term	Instantaneous	
Sawn wood	EN 14081-1	1	0.60	0.70	0.80	0.90	1.10	
		2	0.60	0.70	0.80	0.90	1.10	
		3	0.50	0.55	0.65	0.70	0.90	
Limed, laminated wood	EN 14080	1	0.60	0.70	0.80	0.90	1.10	
		2	0.60	0.70	0.80	0.90	1.10	
		3	0.50	0.55	0.65	0.70	0.90	
LVL	EN 14374, EN 14279	1	0.60	0.70	0.80	0.90	1.10	
		2	0.60	0.70	0.80	0.90	1.10	
		3	0.50	0.55	0.65	0.70	0.90	
Multiplex	EN 636	Type EN 636-1	1	0.60	0.70	0.80	0.90	1.10
		Type EN 636-2	2	0.60	0.70	0.80	0.90	1.10
		Type EN 636-3	3	0.50	0.55	0.65	0.70	0.90
OSB	EN 300	OSB/2	1	0.30	0.45	0.65	0.85	1.10
		OSB/3, OSB/4	1	0.40	0.50	0.70	0.90	1.10
		OSB/3, OSB/4	2	0.30	0.40	0.55	0.70	0.90

Table D.8: Values for k_{def} [NEN-EN 1995: Table 3.2].

Material	Norm	Service class			
		1	2	3	
Sawn wood	EN 14081-1	0.60	0.80	2.00	
Limed, laminated wood	EN 14080	0.60	0.80	2.00	
LVL	EN 14374, EN 14279	0.60	0.80	2.00	
Multiplex	EN 636	Type EN 363-1	0.80	-	-
		Type EN 363-2	0.80	1.00	-
		Type EN 363-3	0.80	1.00	2.50
OSB	EN 300	OSB/2	2.25	-	-
		OSB/3, OSB/4	1.50	2.25	-

D.3.2 Structural design of the roof panels

The roof panels are placed on top of the walls, to close the units [Fig. D.9a]. The panels span 3.5 m, schematized as a simply supported beam [Fig. D.9b]. A three-layered panel (20-40-20) is applied, with the outer layers in the major direction [Fig. D.9c]. Appendix X.10.2 provides the excel sheet of the calculation. The excel is validated by hand.

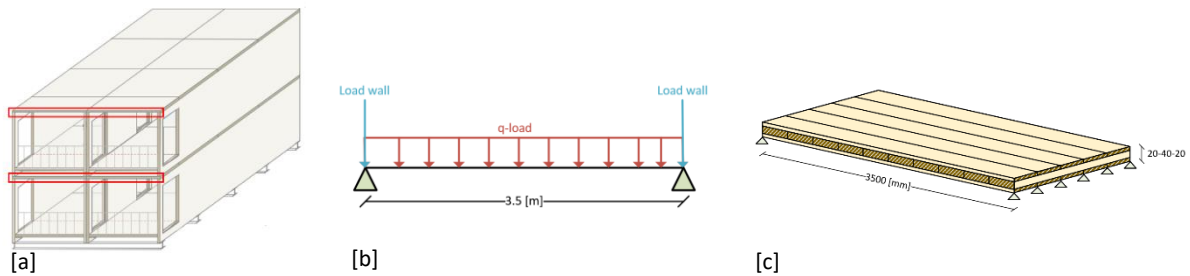


Figure D.9: Schematization roof panel. [a] Location; [b] Structural scheme; [c] 3D roof panel (Self-produced).

Table D.9 provides an overview of the used loads. The roof panel is calculated to allow future placement of solar panels. NEN 1990 is used as guiding norm for the load definition. Equation D.1 and D.2 are used to calculate the design load. Table D.10 provides properties of the chosen panel.

$$\sum_{j \geq 1} 1.35 G_{k,j} + 1.5 \psi_{0,1} Q_{k,1} + \sum_{i > 1} 1.5 \psi_{0,i} Q_{k,i} \quad (D.1)$$

$$\sum_{j \geq 1} 1.2 G_{k,j} + 1.5 Q_{k,1} + \sum_{i > 1} 1.5 \psi_{0,i} Q_{k,i} \quad (D.2)$$

Table D.9: Load definition roof.

Load	Value [kN/m ²]
Permanent	
Self-weight	0,336
Solar panels	0,450
Total permanent load	0,786
Variable ($\psi_0 = 0$)	
Wind (Area II; Urban; 22.4m)	-0.186
Snow	0.560
Imposed load	1.000
<ul style="list-style-type: none">q_{ULS}; Eq D.1	1.06
<ul style="list-style-type: none">q_{ULS}; Eq D.2 (normative)	2.44

Table D.10: Chosen roof panel.

Property	Value	Unit
Panel build-up	20-40-20 (80)	mm
b _x	1000	mm
I _{x,net}	3.733E+07	mm ⁴
I _{x,eff} (I _{ref} = 3.5 m)	3.510E+07	mm ⁴
W _{x,net}	9.333E+05	mm ³
S _{x,net} = S _{x,rolling}	6.000E+05	mm ³
g _{mean}	33.6	Kg/m ²
E _{x,mean}	11000	N/mm ²
f _{V,090,x,lay,k}	4	N/mm ²
f _{V,9090,x,lay,k}	0.7	N/mm ²
f _{c,90,x,lay,k}	2.5	N/mm ²
f _{m,X,k}	24	N/mm ²

D.3.2.1 Verification ULS

The panel is verified for bending, (rolling) shear, and compression perpendicular to the grain.

$$k_{mod} = 0.7 \text{ (long term; Service class 1)}$$

$$\gamma_M = 1.25 \text{ [-]}$$

$$k_h = \min\left(\left(\frac{600}{h}\right)^{0.1}; 1.1\right) = 1.10 \text{ [-]}$$

1. Bending

$$M_d = \frac{q_d L^2}{8} = 3.74 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W_{x,net}} < f_{M,Rd} = \frac{f_{M;k}}{\gamma_m} \cdot k_{mod} \cdot k_h \rightarrow 4.01 < 14.78 \text{ [N/mm}^2\text{]} \text{ (UC} = 0.27\text{)}$$

2. Shear

$$V_d = 0.5q_d L = 4275 \text{ [N]}$$

$$\tau_d = \frac{V_d \cdot S_{x,net}}{I_{x,net} \cdot b_x} \leq f_{v,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.07 < 2.24 \text{ [N/mm}^2\text{]} \text{ (UC=0.03) (Normal shear)}$$

$$\tau_{Rv,d} = \frac{V_d \cdot S_{Rx,net}}{I_{x,net} \cdot b_x} \leq f_{Rv,d} = \frac{f_{v,9090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.07 < 0.39 \text{ [N/mm}^2\text{]} \text{ (UC = 0.18) (Rolling shear)}$$

3. Compression perpendicular to the grain (wall on top of roof panel)

$$q_{c,z,d} = 14.56 \text{ [kN/m]}$$

$$t_{eff} = t_{wall} + 30 = 130 \text{ [mm]} \text{ (Borgström & Fröbel, 2019: p.54-55)}$$

$$K_{c,90} = 1.5 \text{ [-]}$$

$$\sigma_{c,z,d} = \frac{q_{c,z,d}}{t_{eff}} \leq f_{c,90,xlay,d} = k_{c,90} \cdot k_{mod} \cdot \frac{f_{c,90,xlay,k}}{\gamma_M} \rightarrow 0.11 < 2.1 \text{ [N/mm}^2\text{]} \text{ (UC = 0.05)}$$

D.3.2.2 Verification SLS

The panel is verified in the SLS for deflection. Vibrations are not included in the roof panel, as these are inaccessible for public.

$$k_{def} = 0.6 \quad (\text{Service class 1})$$

$$\psi_2 = 0 \text{ [-]}$$

$$u_{fin,G} = u_{inst,G} (1 + k_{def}) = \frac{5}{384} \frac{q_g L^4}{E_{x,mean} I_{x,eff}} (1 + k_{def}) = 6.36 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1} (1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1} L^4}{E_{x,mean} I_{x,eff}} (1 + \psi_2 k_{def}) = 5.06 \text{ [mm]}$$

$$u_{fin,Qi} = u_{inst,Qi} (\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi} L^4}{E_{x,mean} I_{x,eff}} (\psi_0 + \psi_2 k_{def}) = 0.00 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} = 11.43 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 11.43 < 14.00 \text{ [mm]} \text{ (UC = 0.82)}$$

D.3.2.3 Optimization of panel

Table D.11 provides an overview of different panels, to show that the panel is optimized.

Table D.11: Optimization of roof panel.

Panel build-up	UC _{Bending}	UC _{Shear}	UC _{Rolling shear}	UC _{Deflection}
20-20-20	0.42	0.04	0.24	1.60
20-30-20	0.33	0.04	0.20	1.11
20-40-20	0.27	0.03	0.18	0.82

D.3.3 Structural design of the floor panels

The floor panels are placed between two walls [Fig. D.10a]. The panels span 3.5 m, schematized as a simply supported beam [Fig. D.10b]. A three-layered panel (40-40-40) is applied, with the outer layers in the major direction [Fig. D.10c]. Appendix X.10.3 provides the excel sheet of the calculation. The excel is validated by hand.

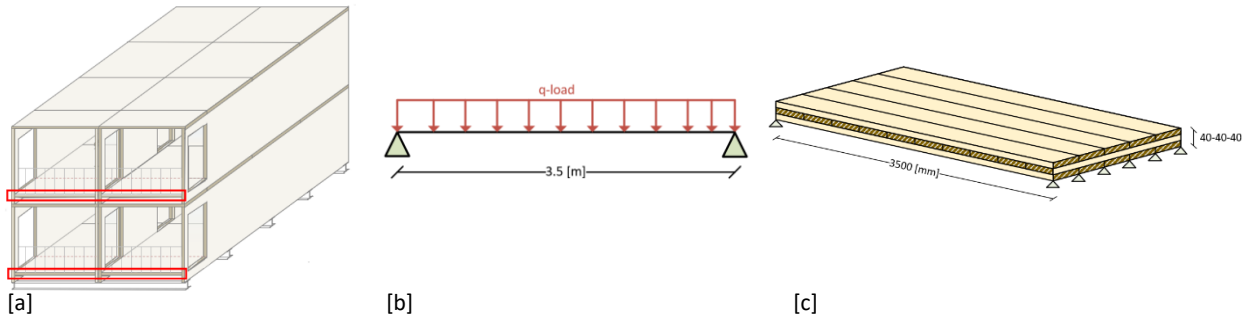


Figure D.10: Schematization floor panel. [a] Location; [b] Structural scheme; [c] 3D floor panel (Self-produced).

Table D.12 provides an overview of the applied loads on the floor. Equations D.1 and D.2 are used to calculate the design load. Table D.13 shows the properties of the chosen panel.

Table D.12: Load definition floor.

Load	Value [kN/m ²]
Permanent	
Self-weight	0.504
Fermacell finishing (22 mm)	0.260
Installations	0.200
Total permanent load	0.964
Variable ($\psi_0 = 0.4$)	
Residential load	1.75
Non-structural walls	0.5
$q_{ULS,Eq D.1}$	2.65
$q_{ULS,Eq D.2}$ (normative)	4.08

Table D.13: Chosen floor panel.

Property	Value	Unit
Panel build-up	40-40-40 (120)	mm
b_x	1000	mm
$I_{x,net}$	1.387E+08	mm ⁴
$I_{x,eff}$ ($l_{ref} = 3.5$ m)	1.245E+08	mm ⁴
$W_{x,net}$	2.311E+06	mm ³
$S_{x,net} = S_{x,rolling}$	1.600E+06	mm ³
g_{mean}	50.4	Kg/m ²
$E_{x,mean}$	11000	N/mm ²
$f_{v,090,xlay,k}$	4	N/mm ²
$f_{v,9090,xlay,k}$	0.7	N/mm ²
$f_{c,90,xlay,k}$	2.5	N/mm ²
$f_{m,x,k}$	24	N/mm ²

D.3.3.1 Verification ULS

The panel is verified for bending and (rolling) shear.

$$k_{mod} = 0.7 \text{ (long term; Service class 1)}$$

$$\gamma_M = 1.25 \text{ [-]}$$

$$k_h = \min\left(\left(\frac{600}{h}\right)^{0.1}; 1.1\right) = 1.10 \text{ [-]}$$

1. Bending

$$M_d = \frac{q_d L^2}{8} = 6.25 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W_{x,net}} < f_{M,Rd} = \frac{f_{M;k}}{\gamma_m} \cdot k_{mod} \cdot k_h \rightarrow 2.70 < 14.78 \text{ [N/mm}^2\text{]} \text{ (UC = 0.18)}$$

2. Shear

$$V_d = 0.5q_d L = 7143 \text{ [N]}$$

$$\tau_d = \frac{V_d \cdot S_{x,net}}{I_{x,net} \cdot b_x} \leq f_{v,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.08 < 2.24 \text{ [N/mm}^2\text{]} \text{ (UC=0.04) (Normal shear)}$$

$$\tau_{Rv,d} = \frac{V_d \cdot S_{Rx,net}}{I_{x,net} \cdot b_x} \leq f_{Rv,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.08 < 0.39 \text{ [N/mm}^2\text{]} \text{ (UC = 0.21) (Rolling shear)}$$

D.3.3.2 Verification SLS

The panel is verified for deflection and vibrations in the SLS. Vibrations result in inconvenience for users, which is an important design aspect for timber floors (Huang et al., 2020).

1. Deflection

$$k_{def} = 0.6 \quad (\text{Service class 1})$$

$$\psi_2 = 0.3 \text{ [-]}$$

$$u_{fin,G} = u_{inst,G}(1 + k_{def}) = \frac{5}{384} \frac{q_g L^4}{E_{x,mean} I_{x,eff}} (1 + k_{def}) = 2.20 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1} L^4}{E_{x,mean} I_{x,eff}} (1 + \psi_2 k_{def}) = 2.95 \text{ [mm]}$$

$$u_{fin,Qi} = u_{inst,Qi}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi} L^4}{E_{x,mean} I_{x,eff}} (\psi_0 + \psi_2 k_{def}) = 0.41 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} = 5.56 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 5.56 < 14.00 \text{ [mm]} \text{ (UC = 0.40)}$$

2. Vibration (Borgström & Fröbel, 2019: p.108-109)

Three aspects are verified for floor panels, related to vibrations: fundamental frequency, point deflection and impulse velocity response.

1. Fundamental frequency f_1 (Lowest frequency > 8 Hz)

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{(E_{x,0,mean} I_{x,ef})L}{m}} \geq 8 \text{ Hz} \rightarrow 15.28 > 8 \text{ [Hz]} \text{ (UC}_{vibr1} = 0.52)$$

L	Span floor panel	= 3.50	[m]
$E_{x,0,mean}$	mean modulus of elasticity	= $1.10 \cdot 10^{10}$	[N/m ²]
$I_{x,ef}$	Effective moment of inertia.	= $1.25 \cdot 10^{-4}$	[m ⁴]
m	floor structures mass per meter width	= 96.4	[kg/m ²]

2. Point deflection (F = 3 kN) should be smaller than 1.0 [mm/kN]

$$w = \frac{FL^3}{48E_{x,0,mean} I_{x,ef}} = 1.96 \text{ [mm/3 kN]}$$

$$w_{per \text{ kN}} = w / 3.0 = 0.65 < 1.0 \text{ [mm/ kN]} \text{ (UC}_{vibr2} = 0.65)$$

3. Impulse velocity response v

$$n_{40} = \left[\left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{B}{L} \right)^4 \left(\frac{E_{mean} I_{x,net}}{E_{mean} I_{y,net}} \right) \right]^{0.25} = 2.51 \text{ [-]}$$

f_1	fundamental frequency	= 15.28	[Hz]
B	Floor width	= 2.50	[m]
L	Floor span	= 3.50	[m]
E_{mean}	mean modulus of elasticity	= $1.10 \cdot 10^{10}$	[N/m ²]
$I_{x,net}$	Moment of inertia	= $1.39 \cdot 10^{-4}$	[m ⁴]
$I_{y,net}$	Moment of inertia	= $5.33 \cdot 10^{-6}$	[m ⁴]

$$v = \frac{4(0.4+0.6n_{40})}{mBL+200} = 0.0073 \text{ [m/s]}$$

- m floor structures mass per meter width = 96.4 [kg/m²]
- B Floor width = 2.50 [m]
- L Floor span = 3.50 [m]
- n_{40} - = 2.51 [-]

$$v \leq b(f_1\xi-1) \rightarrow 0.0073 < 0.05 \text{ [m/s]} \quad (UC_{vibr3} = 0.14)$$

- b parameter related to point deflection = 120 [m/Ns²]
- ξ damping for CLT floor = 0.025 [-]

D.3.3.3 Optimization of floor panel

Table D.14 provides an overview of different panels, to show that the panel is optimized. Chosen is for the 3-layered panel (40-40-40) instead of the 5-layered (20-30-20-30-20), as it minimizes the use of glue (lower impact on environment).

Table D.14: Optimization of floor panel.

Panel build-up	UC _{Bending}	UC _{Shear}	UC _{Rolling shear}	UC _{Deflection}	UC _{vibr1}	UC _{vibr2}	UC _{vibr3}
20-20-20-20-20	0.31	0.04	0.22	0.77	0.71	1.31	0.19
40-20-40	0.25	0.05	0.26	0.61	0.63	1.04	0.31
20-30-20-30-20	0.25	0.03	0.18	0.54	0.61	0.88	0.12
40-40-40	0.18	0.04	0.21	0.40	0.52	0.65	0.14

D.3.4 Structural design of the wall panels

The floor panels are placed on top of five steel beams (steel interlayer). The span between two beams equals 2.36 m. The beam is schematized as a statically indeterminate beam with five supports [Fig. D.11b]. The normative wall contains three wall openings. A three-layered panel (30-40-30) is applied, with the outer layers orientated vertically [Fig. D.11a]. Appendixes X.10.4 till X.10.6 provide the excel sheets of the calculations. The excels are validated by hand.

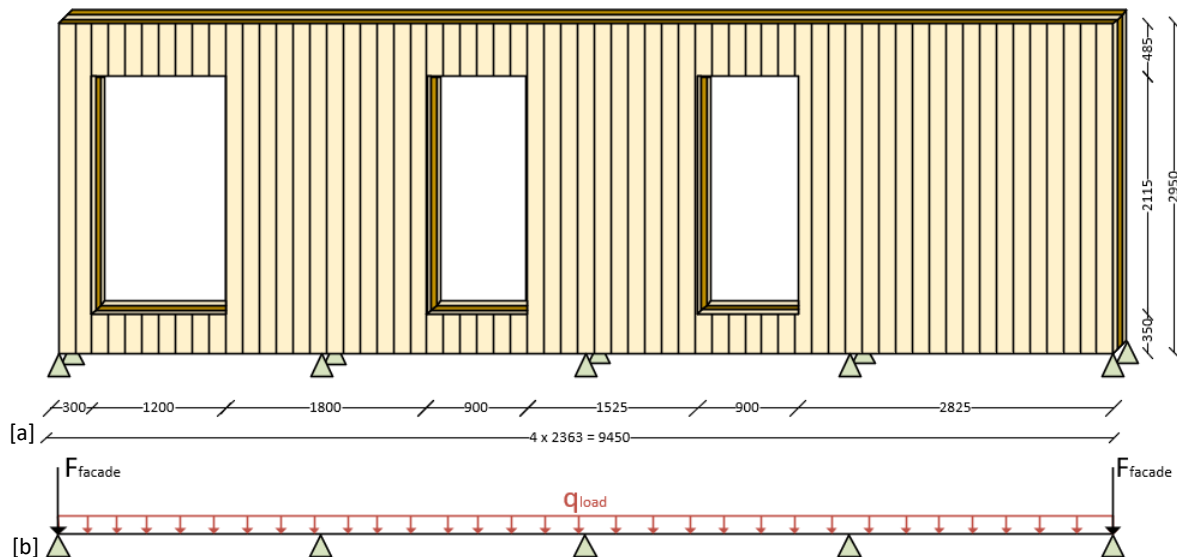


Figure D.11: Structural design of the CLT wall. [a] 3D representation. [b] Structural scheme (Self-produced).

D.3.4.1 ULS verification of wall as 'beam'

The wall is schematized as beam according to figure D.11. The wall is designed for two units above each other. The resulting q_{ULS} equals 25.09 kN/m. The design point load due to the façade equals 6.20 kN. A detailed overview of the load combinations can be found in the excel sheet [Appendix X.10.4]. Table D.15 provides an overview of the properties of the chosen panel. Figure D.12a-d provides respectively the Matrix frame input, support reactions, moment, and shear force line.

Table D.15: Chosen wall panel (1).

Property	Value	Unit
Panel build-up	30-40-30 (100)	mm
Wall height (l_e)	2950	mm
b_x	1000	mm
b_y	2950	mm
$I_{x,net}$	7.800E+07	mm ⁴
$I_{x,eff}$ ($l_{ref} = 2.95$ m)	6.95E+07	mm ⁴
$i_{x,eff}$	34.04	mm
$A_{x,net}$	6.00E+04	mm ²
$W_{x,net}$	1.56E+06	mm ³
$I_{z,y,net}$	8.56E+10	mm ⁴
g_{mean}	42	Kg/m ²
$E_{x,mean}$	11000	N/mm ²
$E_{0.05}$	7400	N/mm ²
$f_{v,090,xlay,k}$	4	N/mm ²
$f_{c,0,x,k}$	21	N/mm ²
$f_{m,x,k}$	24	N/mm ²

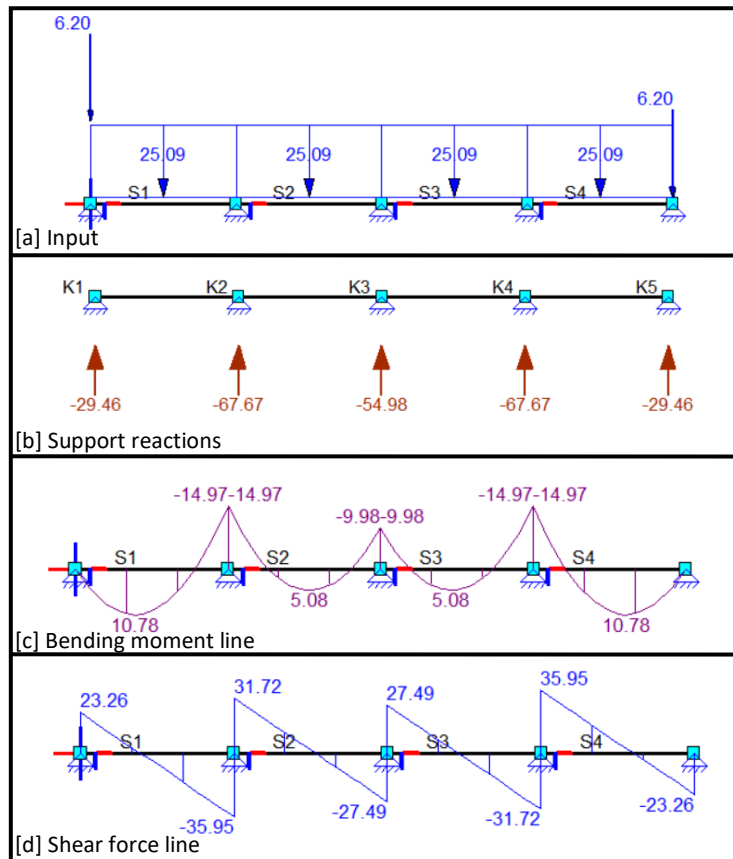


Figure D.12: Results of matrix frame for schematization wall as beam.

The beam is verified in the ULS for: 'buckling at support', 'normal stress concentration at support', bending and shear force.

1. Buckling at support

Support K2 [Fig. D.12b] forms the normative location for buckling at support ($N_{ed} = 67.67$ kN). It is allowed to spread the load by an angle of 30 degrees till a height of $0.25 \cdot L_{wall}$ (Borgström & Fröbel, 2019: p.131) [Fig. D.13b]. The spread area equals 851 mm. The equivalent q-load for 1 m is 93.31 kN/m. The normal stress $\sigma_{c,0,d}$ equals $q_{ULS} / (2 \cdot t_{outer}) = 1.56$ N/mm².

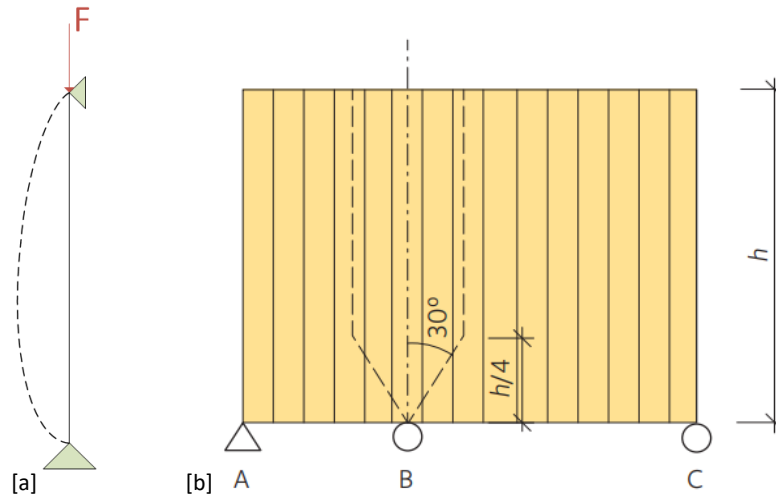


Figure D.13: Buckling of the wall at support. [a] buckling shape. [b] Load spread at support (Borgström & Fröbel, 2019: p.131).

The reduction factor is calculated according to the following formula's:

$$\lambda_y = \frac{l_e}{i_{x,ef}} = 86.66 [-]$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = 1.47 [-]$$

$$k_y = 0.5(1 + 0.1(\lambda_{rel} - 0.3) + \lambda_{rel,y}^2) = 1.64 [-]$$

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel}^2}} = 0.42 [-]$$

Verification:

$k_{mod} = 0.7$ (long term; Service class 1); $\gamma_M = 1.25$

$$\sigma_{c,0,d} \leq k_{c,y} \cdot f_{c,0,d} = k_{c,y} \cdot k_{mod} \cdot \frac{f_{c,0,k}}{\gamma_M} \rightarrow 1.56 < 0.42 \cdot 11.76 = 4.94 \text{ [N/mm}^2\text{]} \quad (\text{UC}=0.31)$$

2. Normal stress at support

The local normal stress is calculated at the width of a steel beam: 250 mm. The area over which the normal force must be distributed equals $250 \cdot 2 \cdot t_{outer\ layer} = 15000$ mm². The normal force at K2 equals 67.67 kN, resulting in a normal stress of 4.51 N/mm².

$$\sigma_{c,0,d} \leq f_{c,0,d} \rightarrow 4.51 < 11.76 \text{ [N/mm}^2\text{]} \quad (\text{UC} = 0.38).$$

3. Bending moment

M_{ed} at k2 equals 14.97 [kNm] [Fig. D.12c]

$k_{mod} = 0.7$ (long term; Service class 1)

$\gamma_M = 1.25$ [-]

$$\sigma_{M;d} = \frac{M_d \cdot 0.5h}{I_{z,y,net}} < f_{M,Rd} = \frac{f_{M;k}}{\gamma_m} \cdot k_{mod} \rightarrow 0.26 < 13.44 \text{ [N/mm}^2\text{]} \quad (\text{UC} = 0.02)$$

4. Shear force

$$V_d = 35.95 \text{ [kN]}$$

$$A_{z,net} = t_{middle} \cdot L_{wall} = 118000 \text{ [mm}^2\text{]}$$

$$k_{mod} = 0.7 \text{ (long term; Service class 1); } \gamma_M = 1.25$$

$$\tau_d = \frac{3V_d}{A_{z,net}} \leq f_{v,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.46 < 2.24 \text{ [N/mm}^2\text{]} \quad (\text{UC} = 0.20)$$

D.3.4.2 Verification buckling (ULS) and deflection (SLS) of 'column' at gallery

The second aspect that will be verified is buckling of the 300 mm width column at the gallery. The sectional properties of the panel are recalculated with a 'b_x' of 300 mm. Table D.16 provides the properties. Appendix X.10.5 contains the full calculation.

Table D.16: Chosen wall panel (2).

Property	Value	Unit
Panel build-up	30-40-30 (100)	mm
Wall height (l _e)	2950	mm
b _x	300	mm
b _y	2950	mm
I _{x,net}	2.340E+07	mm ⁴
I _{x,eff} (l _{ref} = 2.95 m)	2.09E+07	mm ⁴
I _{x,eff}	34.04	mm
A _{x,net}	1.80E+04	mm ²
W _{x,net}	4.68E+05	mm ³
g _{mean}	42	Kg/m ²
E _{x,mean}	11000	N/mm ²
E _{0.05}	7400	N/mm ²
f _{v,090,xlay,k}	4	N/mm ²
f _{c,0,x,k}	21	N/mm ²
f _{m,x,k}	24	N/mm ²

It is assumed that the half load at the gallery is transferred to the column [Fig. D.14a]. For the combination including wind, it is conservatively calculated that half of the wind load (of the gallery area) is transferred to the column (although there is an opening). Figure D.15 provides the two situations, including the normative loads.

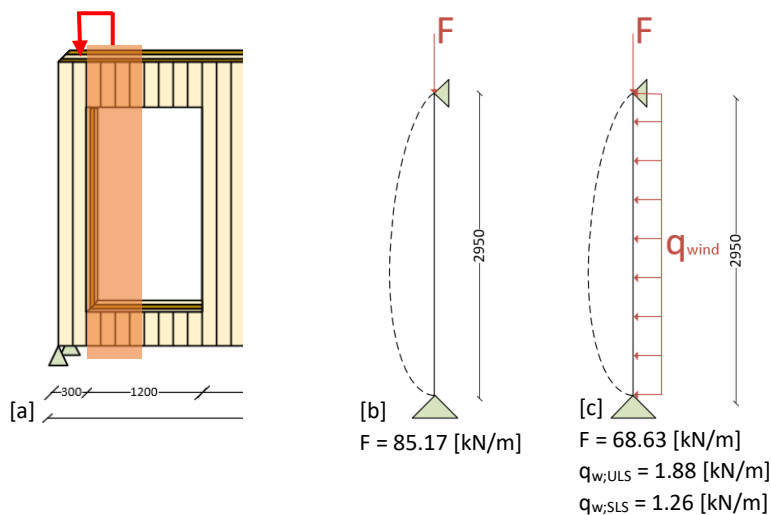


Figure D.14: [a] area transferred to wall. [b] buckling combination 1. [c] Buckling combination 2 (Self-produced).

1. Buckling combination 1 [Fig. D.14b] (ULS)

$K_{c,y} = 0.42$ [-] (already calculated at 'buckling at support' [Appendix D.3.4.1])

$k_{mod} = 0.7$ (long term; Service class 1); $\gamma_M = 1.25$

$$\sigma_{c,0,d} = \frac{F}{2 \cdot t_{outer}} \leq k_{c,y} f_{c,0,d} = k_{c,y} \frac{f_{c,0,k} \cdot k_{mod}}{\gamma_M} \rightarrow 1.42 < 0.42 \cdot 11.76 = 4.93 \text{ [N/mm}^2\text{]} \quad (\text{UC} = 0.29)$$

2. Buckling combination 2 [Fig. D.14c] (ULS)

$K_{c,y} = 0.42$ [-] (already calculated at 'buckling at support' [Appendix D.3.4.1])

$k_{mod} = 0.7$ (long term; Service class 1); $\gamma_M = 1.25$; $k_h = 1.1$ [-]

$$M_{ed} = \frac{1}{8} q_{w,ULS} L^2 = 2.05 \text{ [kNm]}$$

$$\sigma_{M,d} = \frac{M_{ed}}{W_{x,net}} = 4.38 \text{ [N/mm}^2\text{]}$$

$$\sigma_{c,0,d} = \frac{F}{2 \cdot t_{outer}} = 1.14 \text{ [N/mm}^2\text{]}$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} (f_{c,0,k} \frac{k_{mod}}{\gamma_M})} + \frac{\sigma_{M,d}}{f_{M,x,d} \frac{k_h}{\gamma_M}} \leq 1.0 \rightarrow \frac{1.14}{0.42 \cdot 11.76} + \frac{4.38}{14.78} = 0.53 < 1.0 \quad (\text{UC} = 0.53)$$

3. Verification of deflection of columns (SLS)

Deflection occurs due to the wind load, so only combination 2 is verified.

$\psi_2 = 0.3$; $k_{def} = 0.6$.

$$u_{fin,Q1} = u_{inst,Q1} (1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1,SLS} L^4}{E_{x,mean} I_{x,eff}} (1 + \psi_2 k_{def}) = 5.40 \text{ [mm]}$$

$$u_{fin,Q1} < \frac{L}{300} \rightarrow 5.40 < 9.83 \text{ [mm]} \quad (\text{UC} = 0.65)$$

D.3.4.3 Verification of lintels at gallery

The lintels form a crucial aspect of the wall panels, due to a reduced height. The lintels are schematized as one-side moment fixed beams [Fig. D.15a]. For the verification, it is assumed that the upward force F is equally distributed over the four lintels. That results in a shear force of $29460 / 4 = 7365$ [N / Lintel] [Fig. D.12b]. The excel calculation can be found in appendix X.10.6.

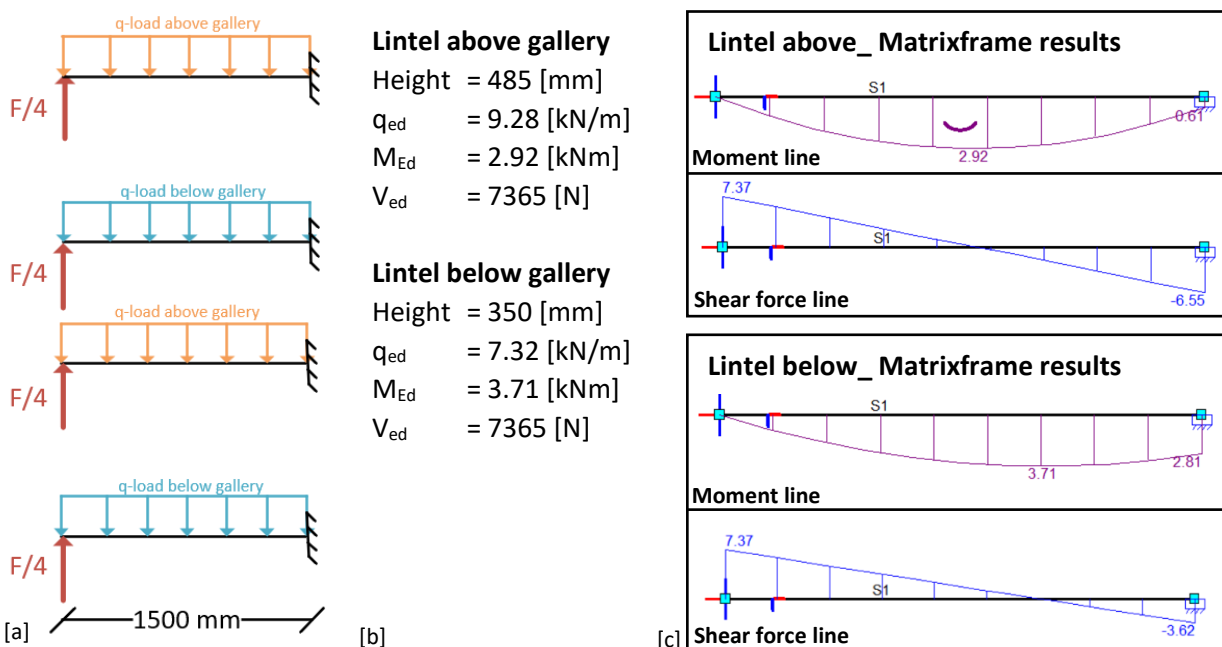


Figure D.15: Structural analysis lintels gallery. [a] Structural schematization; [b] Dimensions and values; [c] Matrixframe results.

1. Bending

$$W_z = \frac{1}{6} t_{hor.layer} \cdot H_{lintel}^2 = 1.57 \text{ E}+06 \text{ [mm}^3\text{]} \text{ (Above gallery)} / 8.17\text{E}+05 \text{ [mm}^3\text{]} \text{ (Below gallery)}$$

Above gallery:

$$k_{mod} = 0.7 \text{ (long term; Service class 1); } \gamma_M = 1.25 \text{ [-]; } k_h = 1.02$$

$$\sigma_M = \frac{M_{ed}}{W_z} < f_{m,d} = \frac{f_{m,k} \cdot k_{mod} \cdot k_h}{\gamma_M} \rightarrow 1.86 < 13.73 \quad (\text{UC} = 0.14)$$

Below gallery:

$$k_{mod} = 0.7 \text{ (long term; Service class 1); } \gamma_M = 1.25 \text{ [-]; } k_h = 1.06$$

$$\sigma_M = \frac{M_{ed}}{W_z} < f_{m,d} = \frac{f_{m,k} \cdot k_{mod} \cdot k_h}{\gamma_M} \rightarrow 4.54 < 14.18 \quad (\text{UC} = 0.32)$$

2. Shear

Above gallery:

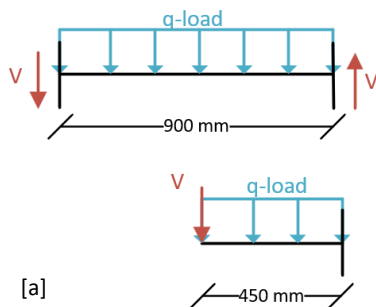
$$\tau_d = \frac{3V_d}{2 \cdot t_{hor;layer} \cdot H_{Lintel}} \leq f_{v,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.57 < 2.24 \quad (\text{UC} = 0.25)$$

Below gallery:

$$\tau_d = \frac{3V_d}{2 \cdot t_{hor;layer} \cdot H_{Lintel}} \leq f_{v,d} = \frac{f_{v,090,xlay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.79 < 2.24 \quad (\text{UC} = 0.35)$$

D.3.4.4 Verification of lintels at doors

The lintels above the doors are less crucial compared to the lintels at the gallery (as the doors are located approximately V=0 location [Fig. D.12d]). For completeness, these lintels are verified. A similar schematization as the lintels at gallery can be applied [Fig. D.16a]. The normative shear force V is approximately 14.4 kN, which is spread over 4 lintels. The excel calculation can be found in appendix X.10.6.



[a]

Lintel above door

Height = 485 [mm]

q_{ed} = 9.28 [kN/m]

M_{Ed} = 2.56 [kNm]

V_{ed} = 7780 [N]

[b]

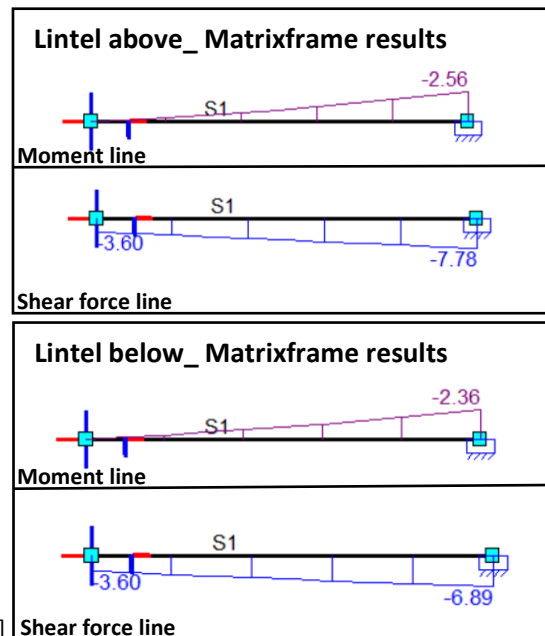
Lintel below door

Height = 350 [mm]

q_{ed} = 7.32 [kN/m]

M_{Ed} = 2.36 [kNm]

V_{ed} = 6890 [N]



[c]

Figure D.16: Structural analysis lintels gallery. [a] Structural schematization; [b] Dimensions and values; [c] Matrixframe results.

1. Bending

$$W_z = \frac{1}{6} t_{hor.layer} \cdot H_{lintel}^2 = 1.57 \text{ E}+06 \text{ [mm}^3\text{]} \text{ (Above gallery)} / 8.17\text{E}+05 \text{ [mm}^3\text{]} \text{ (Below gallery)}$$

Above door:

$$k_{mod} = 0.7 \text{ (long term; Service class 1); } \gamma_M = 1.25 \text{ [-]; } k_h = 1.02$$

$$\sigma_M = \frac{M_{ed}}{W_z} < f_{m,d} = \frac{f_{m,k} \cdot k_{mod} \cdot k_h}{\gamma_M} \rightarrow 1.63 < 13.73 \quad (\text{UC} = 0.12)$$

Below door:

$k_{mod} = 0.7$ (long term; Service class 1); $\gamma_M = 1.25$ [-]; $k_h = 1.06$

$$\sigma_M = \frac{M_{ed}}{W_z} < f_{m,d} = \frac{f_{m,k} \cdot k_{mod} \cdot k_h}{\gamma_M} \rightarrow 2.89 < 14.18 \quad (\text{UC} = 0.20)$$

2. Shear

Above door:

$$\tau_d = \frac{3V_d}{2 \cdot t_{hor;layer} \cdot H_{Lintel}} \leq f_{v,d} = \frac{f_{v,090,clay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.60 < 2.24 \quad (\text{UC} = 0.27)$$

Below door:

$$\tau_d = \frac{3V_d}{2 \cdot t_{hor;layer} \cdot H_{Lintel}} \leq f_{v,d} = \frac{f_{v,090,clay,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.74 < 2.24 \quad (\text{UC} = 0.33)$$

D.3.4.5 Optimisation of wall panel

A wall thickness of 100 mm is applied for all walls. Structurally, thinner walls could be applied. However, the minimum wall thickness is set to 100 mm to take into account: installation slots, connections, lay-up length for roof, and building physic related aspects. An additional benefit is the overcapacity to place in future more than two units above each other.

D.3.5 Structural design of the stability wall

A longitudinal orientated stability walls is in the middle of the CLT unit. This wall is on one side connected with the cross-orientated wall. The distinct units work in pairs of two, to make use of the principle of 'active' and 'passive' walls. This provides always the tension force at the cross-orientated wall side, to minimize tension in the unit [Fig. D.17a]. The wall length is set to 2.5 m. That allows the residents to move from one to another room. By maximizing the wall length: 2.5 m, the tension force is minimized. It is assumed that on 1 block (2 porches), 8 units can be placed, which will work together. Resultingly, 4 walls provide stiffness per wind-direction. If the units will be placed at another location, at least 4 units should be linked, to have no tension in the lowest unit. The excel calculation can be found in appendix X.10.7. A similar wall panel as the other walls is applied. Table D.17 provides the wind load starting points. The panel properties are provided in table D.18.

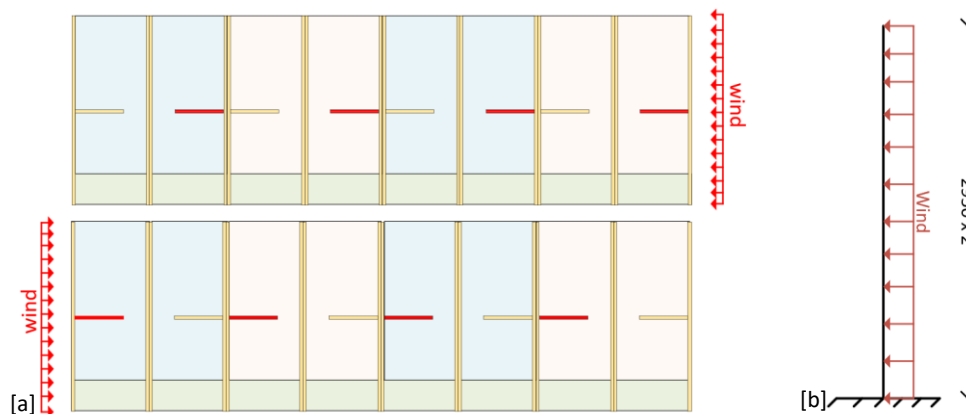


Figure D.17: [a] Principle of active and passive walls; [b] Structural scheme of wall (Self-produced).

Table D.17: Wind load.

Load	Value
Wind load in case applied on flat	
Number of walls	4
Wind properties	II; Urban; 19.6 m
q_{wind} per wall; SLS	3.19 kN/m
q_{wind} per wall; ULS	4.78 kN/m
Wind load at new location	
Number of walls	2
Wind properties	II; Urban; 6.0 m
q_{wind} per wall; SLS	4.11 kN/m
q_{wind} per wall; ULS	6.17 kN/m

Table D.18: Panel properties.

Property	Value	Unit
Panel build-up	30-40-30 (100)	mm
$b_x = l_{stab\ wall}$	2500	mm
Total height	5900	mm
$I_{z,x}$	3.510E+07	mm ⁴
$W_{z,x}$	9.333E+05	mm ³
g_{mean}	42	Kg/m ²
$E_{x,mean}$	11000	N/mm ²

D.3.5.1 Verification of deflection (SLS)

Verification if applied on council estate:

$k_{def} = 0.6$; $\psi_2 = 0.3$; Load duration: short term;

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{1}{8} \frac{q_{w,SLS} L^4}{E_{x,mean} I_{z,d}} (1 + \psi_2 k_{def}) = 0.66 \text{ [mm]}$$

$$u_{fin,Q1} < H/500 \rightarrow 0.69 < 11.8 \text{ [mm]} \quad (UC = 0.06)$$

Verification if applied on new situation:

$k_{def} = 0.6$; $\psi_2 = 0.3$; Load duration: short term;

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{1}{8} \frac{q_{w,SLS} L^4}{E_{x,mean} I_{z,d}} (1 + \psi_2 k_{def}) = 0.85 \text{ [mm]}$$

$$u_{fin,Q1} < H/500 \rightarrow 0.85 < 11.8 \text{ [mm]} \quad (UC = 0.07)$$

D.3.5.2 Verification if tension will occur (ULS)

This load combination multiplies the self-weight with a factor 0.9.

Verification if applied on council estate:

$$M_{ULS,wind} = \frac{1}{2} q L^2 = 83.3 \text{ [kNm]}$$

$$F_{tension} = \frac{M_{ULS,wind}}{L_{wall}} = 33.31 \text{ [kN]}$$

$$N_{stability\ wall} = 3.72 \text{ [kN]}$$

$$q_{N,side\ walls} = 7.07 \text{ [kN/m]}$$

$$\text{Needed side wall length} = \frac{F_{tension} - N_{stability\ wall}}{q_{N,side\ walls}} = 4.19 \text{ [m]} < 9.45 \text{ [m]} \text{ (Unit length)}$$

-> No tension will occur, as the normal force from the side wall is larger than the tension force due to the wind.

Verification if applied on new situation:

$$M_{ULS,wind} = \frac{1}{2} q L^2 = 107.0 \text{ [kNm]}$$

$$F_{tension} = \frac{M_{ULS,wind}}{L_{wall}} = 42.93 \text{ [kN]}$$

$$N_{stability\ wall} = 3.72 \text{ [kN]}$$

$$q_{floors\ and\ walls\ above} = 7.07 \text{ [kN/m]}$$

$$\text{Needed side wall length} = \frac{F_{tension} - N_{stability\ wall}}{q_{N,side\ walls}} = 5.55 \text{ [m]} < 9.45 \text{ [m]} \text{ (Unit length)}$$

-> No tension will occur, as the normal force from the side wall is larger than the tension force due to the wind.

D.3.5.3 Verification compression perpendicular to the grain (ULS)

The floorplate below the stability wall is verified on compression perpendicular to the plain.

Verification if applied on council estate:

$$M_{ULS,wind} = \frac{1}{2}qL^2 = 83.3 \text{ [kNm]}$$

$$\sigma_M = \frac{M_{ULS,wind}}{W} = 1.33 \text{ [N/mm}^2\text{]}$$

$$\sigma_{N,selfweight\ wall} = q_{wall}/(2 \cdot 30 + t_{wall}) = 0.02 \text{ [N/mm}^2\text{]}$$

$$\sigma_{compr.tot} = \sigma_M + \sigma_{N,selfweight\ wall} = 1.35 \text{ [N/mm}^2\text{]}$$

$$K_{c,90} = 1.5; k_{mod} = 0.7; \gamma_M = 1.3$$

$$f_{c,90,xlay,d} = 1.4 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_{compr.tot}}{f_{c,90,xlay,d} \cdot k_{c,90}} = 0.64 \text{ [-]}$$

Verification if applied on new situation:

$$M_{ULS,wind} = \frac{1}{2}qL^2 = 107.0 \text{ [kNm]}$$

$$\sigma_M = \frac{M_{ULS,wind}}{W} = 1.72 \text{ [N/mm}^2\text{]}$$

$$\sigma_{N,selfweight\ wall} = q_{wall}/(2 \cdot 30 + t_{wall}) = 0.02 \text{ [N/mm}^2\text{]}$$

$$\sigma_{compr.tot} = \sigma_M + \sigma_{N,selfweight\ wall} = 1.74 \text{ [N/mm}^2\text{]}$$

$$K_{c,90} = 1.5; k_{mod} = 0.7; \gamma_M = 1.3$$

$$f_{c,90,xlay,d} = 1.4 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_{compr.tot}}{f_{c,90,xlay,d} \cdot k_{c,90}} = 0.83 \text{ [-]}$$

D.3.6 Structural design of the steel interlayer

The structural schemes of the archive inspection are studied, to find the normative shape of the steel interlayer. To limit extreme spans, at least one cut should be made in the beam. That results in single and double scheme's. Two extreme variants are defined:

1. Inspection number 13: Single span beam: 6.60 m.
2. Inspection number 2: Double span beam, with two equal fields.

1. Single span beam

As upward deflection will not occur for a single span beam, the vertical load at midspan must be maximized. For that, two load combinations hold.

4. Combination 1a: $1.2G + 1.5Q_{wind} + 1.5\psi_0Q_{variable\ other}$
5. Combination 1b: $1.2G + 1.5Q_{residential} + 1.5\psi_0Q_{variable\ other}$

Both combinations are put into mechanical scheme's [Fig. D.18]. F_g (normal force due to self-weight in sidewalls) has the highest contribution. In combination 1b, F_g is put into the centre [Fig. D.18b]. In combination 1a it is shifted a bit to the left, to consider the downward wind load (F_w) [Fig. D.18a]. q_{wall} represents the self-weight of the stability wall and q_{self} the self-weight of the steel beam.

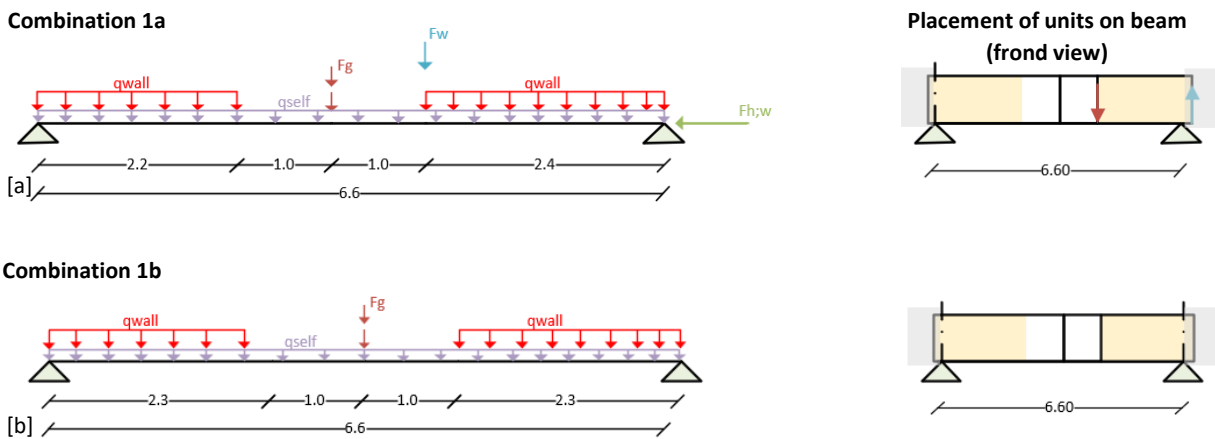


Figure D.18: Structural schemes of [a] combination 1a; [b] combination 1b (Units: [m]) (Self-produced).

2. Double span beam

The double span beam is certainly not normative for the strength, as the span is much lower. Potentially, upwards deflection could occur. For certainty, this variant is also examined. For upward lift, combination 2 is normative. In the orientation of the units, the downward loads are orientated in the centre of the right field, while the upward load in the centre of the left field [Fig. D.19].

- Combination 2: $0.9G + 1.5Q_{wind} + 1.5\psi_0Q_{other\ variable\ loads}$

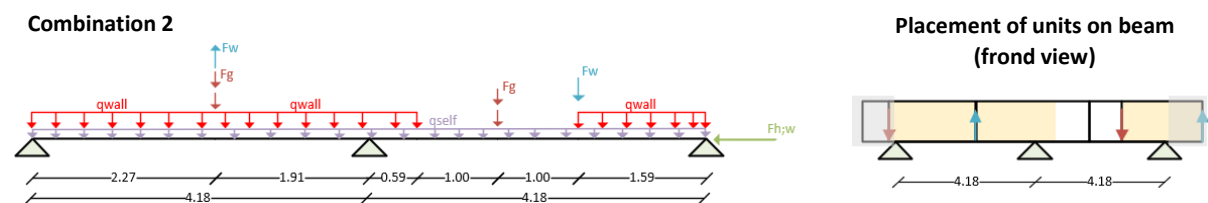


Figure D.19: Structural scheme of combination 2 (Units: m) (Self-produced).

D.3.6.1 Results structural analysis

The provided structural schemes are put into Matrixframe, to get the moments, shear force and deflections [Table D.19]. Details about the load calculation can be found in appendix X.10.8. The appendix is validated by hand. The table includes also the values for a normal beam (without stabilizing elements) [Combination 3]. The Matrix frame in- and output is provided for the normative load combination: 1b [Fig. D.20].

Table D.19: Internal forces of distinct load combinations.

Combination	M [kNm]	V [kN]	N [kN]	W [mm]
1a	229.43	85.33	112.90	21.9
1b	239.88	80.47	-	21.9
2	72.57	87.35	112.90	2.4
3	228.57	75.47	-	20.4

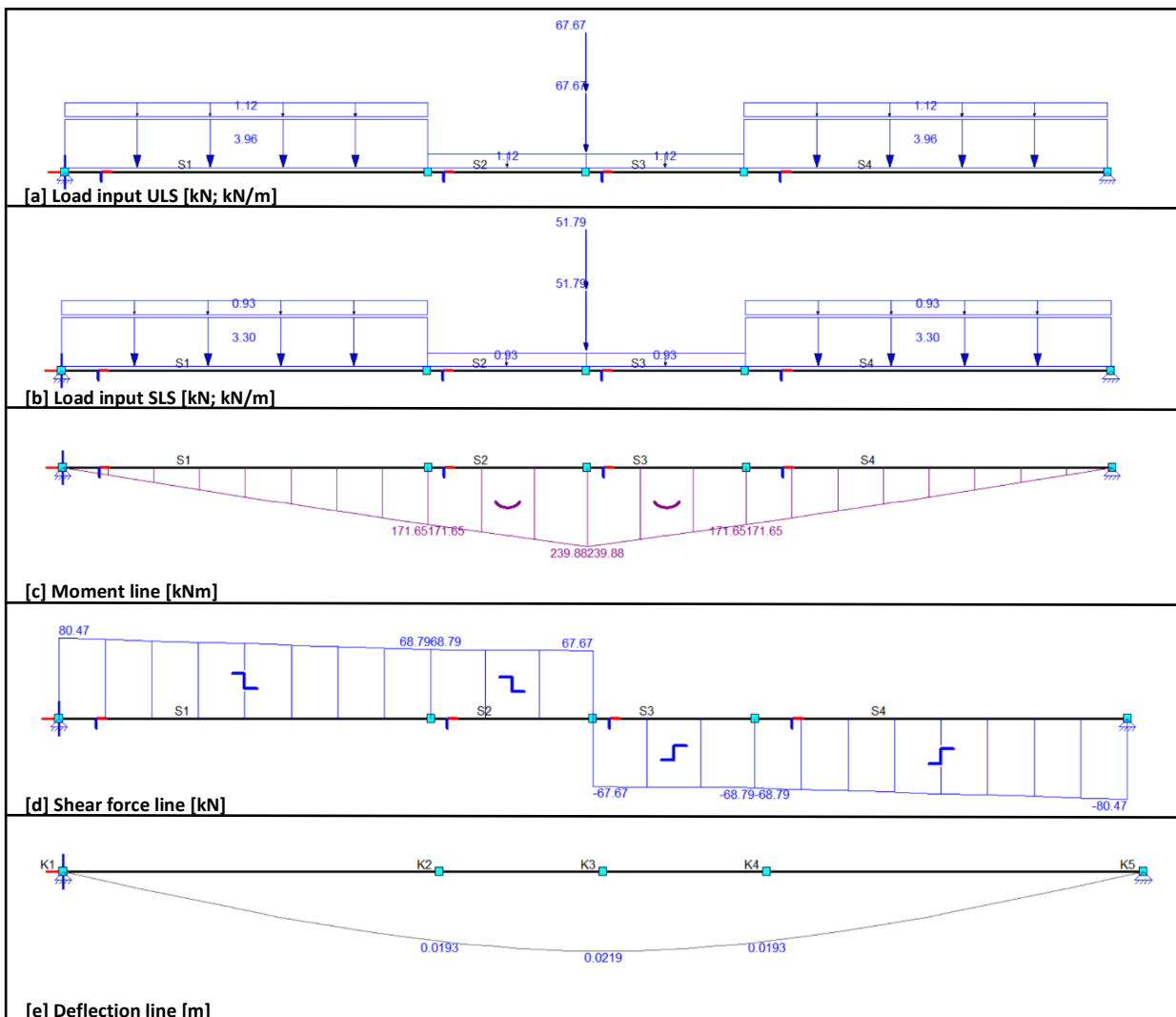


Figure D.20: Normative results Matrixframe (Combination 1a).

The beam is analysis in both ULS and SLS.

D.3.6.2 Chosen profile

Table D.20 provides the properties of the chosen profile: HEB260-S235.

Table D.20: Properties HEB260.

Property	Value	Unit
f_{yd}	235	N/mm ²
E_s	210000	N/mm ²
G	81000	N/mm ²
I_w	7.3628E+11	mm ⁶
I_t	1257000	mm ⁴
h	260	mm
b	260	mm
t_f	17.5	mm
t_w	10	mm
r	24	mm
$W_{pl,y}$	1283000	mm ³
A	11844	mm ²
I_z	51350000	mm ⁴

D.3.6.3 Verification ULS

1. Cross-section classification

Flange:

$$c = b / 2 - t_w / 2 - r = 101 \text{ [mm]}; \quad t = t_f$$

$$c/t = 5.77 < 9\xi$$

-> Class 1

Web:

$$c = h - 2t_f - 2r = 177 \text{ [mm]}; \quad t = t_w$$

$$\alpha = \frac{1}{c_w} \cdot \left(\frac{h}{2} + \frac{N_{ed}}{2t_w f_y} - (t_f + r) \right) = 0.64$$

$$c/t = 17.7 < \frac{396\xi}{13\alpha - 1} = 54.07 \text{ [-]}$$

-> Class 1

The profile is classified as class 1, meaning that a plastic calculation can be performed.

2. Verification of axial force should be considered (Note this is another load combination)

$$a) N_{ed} \leq 0.25N_{pl,Rd} = A \cdot f_{yd} \rightarrow 112.90 < 2783.34 \text{ [kN]}$$

$$b) N_{ed} \leq \frac{0.5h_w t_w f_y}{\gamma_{M0}} \rightarrow 112.90 < 264.38 \text{ [kN]}$$

Both equation hold, so axial force can be neglected.

3. Verification of shear force should be considered

$$A_{v,z} = \text{maximum}(A - 2bt_f + (t_w + 2r)t_f; \eta h_w t_w) = 3759 \text{ [mm}^2\text{]}$$

$$V_{ed} \leq 0.5V_{pl} = 0.5 \cdot \frac{A_v \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}} \rightarrow 80.47 < 255.01 \text{ [kN]}$$

Equation hold, so shear force can be neglected.

4. Cross-section check bending moment

$$M_{ed} < M_{pl,y} = W_{pl,y} f_{yd} \rightarrow 239.88 < 301.51 \text{ [kNm]} \quad (\text{UC} = 0.80)$$

5. Lateral torsional buckling of element

Calculated with no lateral supports, although they are there (At walls of units).

$$L_{kip} = L_g = 6600 \text{ [mm]}$$

$$C_1 = 1.35 \text{ [-]} \text{ (Assume only point load, as this is the main contributor)}$$

$$C_2 = -0.55 \text{ [-]}$$

$$S = \sqrt{\frac{EI_w}{GI_t}} = 1232 \text{ [-]}$$

$$C = \frac{\pi C_1 L_g}{L_{kip}} \cdot \left(\sqrt{1 + \left(\frac{\pi^2 S^2}{L_{kip}^2} (C_2^2 + 1) \right)} \right) + \frac{\pi C_2 S}{L_{kip}} = 3.74 \text{ [-]}$$

$$k_{red} = 1$$

$$M_{cr} = k_{red} \cdot \frac{C}{L_g} \cdot \sqrt{EI_z GI_t} = 593055169.2 \text{ [Nmm]}$$

$$\lambda_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} = 0.71 \text{ [-]}$$

Buckling curve = a

$$\alpha_{LT} = 0.21$$

$$\theta_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2] = 0.81 \text{ [-]}$$

$$\chi_{LT} = \frac{1}{\theta_{LT} + \sqrt{\theta_{LT}^2 - \lambda_{LT}^2}} = 0.84 \text{ [-]}$$

$$M_{pl,Rd} = \chi_{LT} W_{pl,y} f_{yd} = 253.73 \text{ [kNm]}$$

$$M_{ed} < M_{pl,Rd} \rightarrow 239.88 < 253.73 \quad (\text{UC} = 0.95)$$

-> The chosen profile is sufficiently strong.

D.3.6.4 Verification SLS

$$w < 0.004L \rightarrow 21.9 < 26.4 \quad (\text{UC} = 0.83)$$

D.3.6.5 Optimisation profile

The designed profile is based on the archive inspection. Note that for an individual building, always the profile should be verified. The chosen profile is optimized. Table [D.21](#) shows that smaller profiles are not possible.

Table D.21: Optimization of steel interlayer.

Profile	w [mm]	UC _{SLS}
HEA260	31.3	1.2
HEB240	29.1	1.1

D.3.7 Weight of the CLT top-up

The weight of the CLT-construction is calculated for one and two added layers. The final weight is a bit higher than the assumed weight (5.88 kN/m²), due to the steel interlayer, and considered façade weight. Note that a 5% exceeding of the overcapacity is used (Van Rossum B.V.; Internal communication). The calculation of the new weight can be found in appendix [X.10.9](#). The permanent load for 1 unit (105.84 kN) does not exceed the hoisting limit: 200 kN (Liew et al., 2018). Design weights for top-up:

- 1 added floor: 6.4 kN/m²
- 2 added floors: 12.3 kN/m²

D.4 HSB calculation

Appendix D.4 will describe the structural verification of the HSB variant. The guiding norms are the **NEN-EN 1990 and 1995**. The norm specifies material, modification, and deformation factors to calculation timber, provided in appendix D.3.1 [Table D.6 till D.8]. Appendix X.11 will contain the excel sheets of the calculation.

D.4.1 Structural design of the roof structure

Timber beams (C24) span from wall-to-wall, schematized as simply supported beams [Fig. D.21a; D.21b]. A plywood panel is placed on top of the timber panels [Fig. D.21a]. The spacing is set at maximum to 600 mm, which is a standardized size in HSB-construction (Van Rossum B.V.; Internal communication). As the roof is inaccessible, it only must be verified for deflection [SLS] and strength [ULS]. Appendix X.11.1 provides the sheet of the calculation (including validation). The excel sheet is validated by hand.

The archive inspection [Appendix A.1] shows variety in spans between structural walls. The group is consequently subdivided into three span ranges:

1. 3.9 – 4.6 m *(Most occurring range)*
2. 4.7 – 5.4 m
3. 5.5 – 6.6 m *(Contains larger spans)*

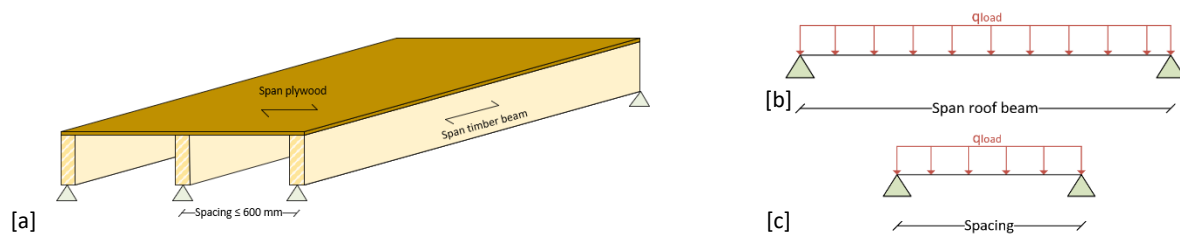


Figure D.21: Roof structure. [a] 3D Structure; [b] Structural scheme beam; [c] Structural scheme plywood (Self-produced).

This appendix provides the calculation for the **range 3.9 - 4.6 m**. Results of the other spans are summarized in appendix D.4.1.3. Table D.22 provides an overview of the used loads. Equation D.1 and D.2 are used to calculate the design load. Table D.23 provides properties of the used wood.

Table D.22: Load definition roof (Span 3.9 – 4.6 m).

Load	Value [kN/m ²]
Permanent	
Beams	0.10
Plywood	0.10
Ceiling	0.20
Roof finishing	0.20
	0.60
Variable	
Wind (Area II; Urban; 22.4m)	-0.19
Snow	0.56
Imposed load	1.00
$q_{ULS}; Eq. D.1$	0.81
$q_{ULS}; Eq. D.2$	2.22

Table D.23: Wood properties roof (Span 3.9-4.6 m).

Property	Value	Unit
Beam (C24)		
Dimensions	70 x 170	mm
Spacing	500	mm
E-modulus	11000	N/mm ²
ρ_{mean}	420	kg/m ³
$f_{m,k}$	24	N/mm ²
$f_{v,k}$	4	N/mm ²
Plywood (C18)		
Thickness	18	mm
E-modulus	9000	N/mm ²
ρ_{mean}	550	kg/m ³
$f_{m,k}$	18	N/mm ²

D.4.1.1 Verification beam

$k_{mod} = 0.7$ (long term; Service class 1)

$\gamma_M = 1.3$ [-]

$k_h = 1.0$ ($h_{beam} \geq 150$ [mm])

$L = 4.6$ [m]

1. Bending [ULS]

$$M_d = \frac{q_d L^2}{8} = 2.93 \text{ [kNm]}$$

$$W = \frac{1}{6} b h^2 = 3.37 \cdot 10^5 \text{ [mm}^3\text{]}$$

$$\sigma_{M;d} = \frac{M_d}{W} < f_{M,Rd} = \frac{f_{M;k}}{\gamma_m} \cdot k_{mod} \cdot k_h \rightarrow 8.70 < 12.92 \text{ [N/mm}^2\text{]} \quad (\mathbf{UC1 = 0.67})$$

2. Shear [ULS]

$$V_d = 0.5 q_d L = 2.55 \text{ [kN]}$$

$$\tau_d = \frac{3V_d}{2bh} \leq f_{v,d} = \frac{f_{v;k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.32 < 2.15 \text{ [N/mm}^2\text{]} \quad (\mathbf{UC2 = 0.15})$$

3. Deflection [SLS]

$k_{def} = 0.6$ (Service class 1)

$\psi_0 = \psi_2 = 0$ [-]

$$I_z = \frac{1}{12} b h^3 = 2.87 \cdot 10^7 \text{ [mm}^4\text{]}$$

$$u_{fin,G} = u_{inst,G} (1 + k_{def}) = \frac{5}{384} \frac{q_g L^4}{EI} (1 + k_{def}) = 8.86 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1} (1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1} L^4}{EI} (1 + \psi_2 k_{def}) = 9.25 \text{ [mm]}$$

$$u_{fin,Qi,s} = u_{inst,Qi,s} (\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi,s} L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.0 \text{ [mm]}$$

$$u_{fin,Qi,w} = u_{inst,Qi,w} (\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi,w} L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.0 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi,s} + u_{fin,Qi,w} = 18.11 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 18.11 < 18.40 \text{ [mm]} \quad (\mathbf{UC3 = 0.98})$$

D.4.1.2 Verification plywood

$k_{mod} = 0.7$ (long term; Service class 1)

$\gamma_M = 1.2$ [-]

$L = 500$ [mm]

$q_{ed} = 2.22$ [kN/m] (Considering 1.00 meter)

1. Bending [ULS]

$$M_d = \frac{q_d L^2}{8} = 0.69 \text{ [kNm]}$$

$$W = \frac{1}{6} b h^2 = 5.40 \cdot 10^4 \text{ [mm}^3\text{]}$$

$$\sigma_{M;d} = \frac{M_d}{W} < f_{M,Rd} = \frac{f_{M;k}}{\gamma_m} \cdot k_{mod} \rightarrow 1.28 < 10.50 \text{ [N/mm}^2\text{]} \quad (\mathbf{UC4 = 0.12})$$

2. Deflection [SLS]

$k_{def} = 0.8$ (Service class 1)

$\psi_0 = \psi_2 = 0$ [-]

$$u_{fin,G} = u_{inst,G} (1 + k_{def}) = \frac{5}{384} \frac{q_g L^4}{EI} (1 + k_{def}) = 0.11 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1} (1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1} L^4}{EI} (1 + \psi_2 k_{def}) = 0.19 \text{ [mm]}$$

$$u_{fin,Qi,s} = u_{inst,Qi,s}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi,s} L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.00 \text{ [mm]}$$

$$u_{fin,Qi,w} = u_{inst,Qi,w}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi,w} L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.00 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi,s} + u_{fin,Qi,w} = 0.39 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 0.39 < 2.00 \text{ [mm]} \quad \mathbf{UC5 = 0.19}$$

D.4.1.3 Summary results

Table D.24 provides an overview of the dimensions and unity checks for the distinct spans. Deflection is the normative design parameter.

Table D.24: Results HSB roof structure.

Roof structure properties				Results beams			Results plywood	
Span	Beam size [mm]	Spacing [mm]	Thickness plywood [mm]	UC1 (M)	UC2 (V)	UC3 (w)	UC4 (M)	UC5 (w)
3.9-4.6	70x170	500	18	0.67	0.15	0.98	0.12	0.19
4.7-5.4	70x195	400	18	0.58	0.13	0.87	0.08	0.10
5.5-6.6	70x245	400	18	0.56	0.12	0.83	0.08	0.11

D.4.2 Structural design of the floor structure

The structural system of the floor is similar to the roof [Fig. D.22a-c]. Since the floor is accessible, it should be additionally verified for vibrations [SLS]. The beam below the façade is further verified for strength and deflection. Appendix X.11.2 provides the sheet of the calculation (including validation). The excel sheet is validated by hand.

The archive inspection [Appendix A.1] shows variety in spans between structural walls. Spans larger than 5.7 m are nearly possible to construct in wood (needed dimensions: 95 x 295 mm – spaced 170 mm). Resultingly, steel beams span between the structural walls (spaced 4.0 m), while the timber beams span perpendicular to these beams [Fig. D.22d, e]. The group is consequently subdivided into four span ranges:

1. 3.9 – 4.6 m *(Most occurring range)*
2. 4.7 – 5.4 m *(Situation still occurs in archive)*
3. 5.5 – 5.7 m *(Limit of timber floor structure > range not in archive)*
4. 5.8 – 6.6 m *(Placement of additional steel beams)*

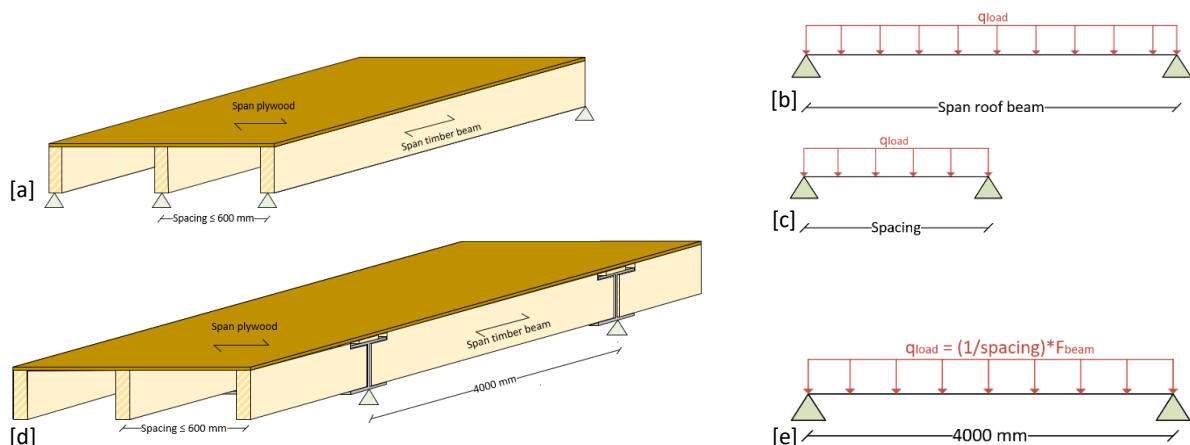


Figure D.22: Floor structure. [a] 3D Structure; [b] Structural scheme beam; [c] Structural scheme plywood (Self-produced); [d] Structural system for spans > 5.8 m; [e] Schematization steel beam (Self-produced).

This appendix provides the calculation for the **range 3.9 - 4.6 m**. Results of the other spans are summarized in appendix D.4.2.6. Table D.25 provides an overview of the used loads. Equation D.1 and D.2 are used to calculate the design load. Table D.26 provides properties of the used wood.

Table D.25: Load definition floor (Span 3.9 – 4.6 m).

Load	Value [kN/m ²]
Permanent	
Beams	0.20
Plywood	0.10
Ceiling	0.20
Fermacell 22 mm	0.26
	0.76
Variable	
Imposed residential loading	1.75
Non-structural walls	0.5
$q_{ULS;Eq D.1}$	2.37
$q_{ULS;Eq D.2}$	3.83
Beam below façade (0.5 kN/m ²) > q_{ULS}	3.22 [kN/m]

Table D.26: Wood properties floor (Span: 3.9 – 4.6 m).

Property	Value	Unit
Beam (C24)		
Dimensions	70 x 270	mm
Spacing	400	mm
E-modulus	11000	N/mm ²
ρ_{mean}	420	kg/m ³
$f_{m,k}$	24	N/mm ²
$f_{v,k}$	4	N/mm ²
Plywood (C18)		
Thickness	18	mm
E-modulus	9000	N/mm ²
ρ_{mean}	550	kg/m ³
$f_{m,k}$	18	N/mm ²

D.4.2.1 Verification beam strength [ULS]

The beam is verified in the ULS for bending and shear. Both beside and below the façade is considered.

$$k_{mod} = 0.7 \text{ (long term; Service class 1)}$$

$$\gamma_M = 1.3 \text{ [-]}$$

$$k_h = 1.0 \text{ (} h_{beam} \geq 150 \text{ [mm])}$$

$$L = 4.6 \text{ [m]}$$

1. Bending

$$W_z = \frac{1}{6} b h^2 = 8.51 \cdot 10^5 \text{ [mm}^3\text{]}$$

1a. Normal beam

$$M_d = \frac{q_d L^2}{8} = 4.06 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W} < f_{M,Rd} = \frac{f_{M,k}}{\gamma_M} \cdot k_{mod} \cdot k_h \rightarrow 4.77 < 12.92 \text{ [N/mm}^2\text{]} \quad \text{(UC1a = 0.37)}$$

1b. Below façade

$$M_d = \frac{q_d L^2}{8} = 8.50 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W} < f_{M,Rd} = \frac{f_{M,k}}{\gamma_M} \cdot k_{mod} \cdot k_h \rightarrow 9.99 < 12.92 \text{ [N/mm}^2\text{]} \quad \text{(UC1b = 0.77)}$$

2. Shear

2a. Normal beam

$$V_d = 0.5 q_d L = 3.53 \text{ [kN]}$$

$$\tau_d = \frac{3V_d}{2bh} \leq f_{v,d} = \frac{f_{v,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.28 < 2.15 \text{ [N/mm}^2\text{]} \quad \text{(UC2a = 0.13)}$$

2b. Below façade

$$V_d = 0.5 q_d L = 7.39 \text{ [kN]}$$

$$\tau_d = \frac{3V_d}{2bh} \leq f_{v,d} = \frac{f_{v,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.59 < 2.15 \text{ [N/mm}^2\text{]} \quad \text{(UC2a = 0.27)}$$

D.4.2.2 Verification beam deflection [SLS]

The beam is verified in the SLS for deflection. Both beside and below the façade is considered.

$$k_{def} = 0.6 \text{ (Service class 1)}$$

$$\psi_0 = 0.4; \psi_2 = 0.3 \text{ [-]}$$

$$I_z = \frac{1}{12}bh^3 = 1.15 \cdot 10^8 \text{ [mm}^4\text{]}$$

$$L = 4.6 \text{ [m]}$$

3a. Normal beam

$$u_{fin,G} = u_{inst,G}(1 + k_{def}) = \frac{5}{384} \frac{qgL^4}{EI} (1 + k_{def}) = 2.24 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{5}{384} \frac{qQ_1 L^4}{EI} (1 + \psi_2 k_{def}) = 3.81 \text{ [mm]}$$

$$u_{fin,Qi} = u_{inst,Qi}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{qQ_i L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.54 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} = 6.59 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 6.59 < 18.40 \text{ [mm]} \quad \text{(UC3a = 0.36)}$$

3b. Below façade

$$u_{fin,G} = u_{inst,G}(1 + k_{def}) = \frac{5}{384} \frac{qgL^4}{EI} (1 + k_{def}) = 12.58 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{5}{384} \frac{qQ_1 L^4}{EI} (1 + \psi_2 k_{def}) = 3.81 \text{ [mm]}$$

$$u_{fin,Qi} = u_{inst,Qi}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{qQ_i L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.54 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} = 16.93 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 16.93 < 18.40 \text{ [mm]} \quad \text{(UC3b = 0.92)}$$

D.4.2.3 Verification vibrations beam [SLS]

Three aspects are verified for floor panels, related to vibrations: fundamental frequency, point deflection and impulse velocity response.

1. Fundamental frequency f_1 (Lowest frequency)

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{(EI)_L}{m}} \geq 8 \text{ Hz} \quad \rightarrow \quad 17.76 > 8 \text{ [Hz]} \quad \text{(UC4 = 0.45)}$$

L	Span	= 4.6	[m]
m	Floor structures mass per meter width	= 55.75	[kg/m ²]
$(EI)_l$	= $E \cdot \frac{1}{12} b_{beam} h_{beam}^3 \cdot 1 \text{ [m]/spacing [m]}$	= $3.16 \cdot 10^6$	[Nm ² /m]

2. Point deflection (F = 3 kN) should be smaller than 1.0 [mm/kN]

$$w = \frac{k_r FL^3}{48(EI)_{single\ beam}} = 2.90 \text{ [mm/3 kN]}$$

$$w_{per\ kN} = 2.90 / 3 = 0.97 < 1 \text{ [mm/kN]} \quad \text{(UC5 = 0.97)}$$

→ K_r : Spread of point load (Van Rossum B.V.; Internal communication)

$k_r = 0.37 + 0.8 \cdot \frac{a}{a_1} - \frac{E_{0,ser;rep} \cdot I}{50000}$	($0.33 < k_r < 1.00$)	= 0.60	[-]
a	Spacing of beams	= 0.40	[m]
a_1	Reference distance	= 1.00	[m]
$E_{0,ser;rep} \cdot I$	Bending stiffness of the wood-based panels	= $4.37 \cdot 10^3$	[Nm ² /m]

3. Impulse velocity response v

$$n_{40} = \left[\left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{B}{L} \right)^4 \left(\frac{(EI)_l}{(EI)_b} \right) \right]^{0.25} = 16.06 \quad [-]$$

f_1	Fundamental frequency	= 17.67	[Hz]
B	Floor width	= 10.00	[m]
L	Floor span	= 4.60	[m]
$(EI)_l$	Bending stiffness per meter	= $3.16 \cdot 10^6$	[Nm ² /m]
$(EI)_b$	Bending stiffness wood based panel	= $4.37 \cdot 10^3$	[Nm ² /m]

$$v = \frac{4(0.4+0.6n_{40})}{mBL+200} = 0.015 \quad [\text{m/s}]$$

m	Floor structures mass per meter width	= 55.75	[kg/m ²]
B	Floor width	= 10.00	[m]
L	Floor span	= 4.6	[m]
n_{40}		= 16.06	[-]

$$v \leq b(f_1 \xi^{-1}) \quad \rightarrow 0.015 < 0.019 \text{ [m/s]} \quad (\text{UC6} = 0.75)$$

b	Standardized value, related to point deflection	= 120	[m/Ns ²]
ξ	damping	= 0.01	[-]

D.4.2.4 Verification plywood

$k_{\text{mod}} = 0.7$ (long term; Service class 1)

$\gamma_M = 1.2$ [-]

$L = 400$ [mm]

$q_{\text{ed}} = 3.83$ [kN/m] (Considering 1.00 meter)

1. Bending [ULS]

$$M_d = \frac{q_d L^2}{8} = 0.77 \text{ [kNm]}$$

$$W = \frac{1}{6} b h^2 = 5.40 \cdot 10^4 \text{ [mm}^3\text{]}$$

$$\sigma_{M;d} = \frac{M_d}{W} < f_{M,Rd} = \frac{f_{M,k}}{\gamma_m} \cdot k_{\text{mod}} \rightarrow 1.42 < 10.50 \text{ [N/mm}^2\text{]} \quad (\text{UC7} = 0.14)$$

2. Deflection [SLS]

$k_{\text{def}} = 0.8$ (Service class 1)

$\psi_0 = \psi_2 = 0$ [-]

$$u_{\text{fin},G} = u_{\text{inst},G} (1 + k_{\text{def}}) = \frac{5}{384} \frac{q_g L^4}{EI} (1 + k_{\text{def}}) = 0.10 \text{ [mm]}$$

$$u_{\text{fin},Q1} = u_{\text{inst},Q1} (1 + \psi_2 k_{\text{def}}) = \frac{5}{384} \frac{q_{Q1} L^4}{EI} (1 + \psi_2 k_{\text{def}}) = 0.17 \text{ [mm]}$$

$$u_{\text{fin},Qi} = u_{\text{inst},Qi} (\psi_0 + \psi_2 k_{\text{def}}) = \frac{5}{384} \frac{q_{Qi,s} L^4}{EI} (\psi_0 + \psi_2 k_{\text{def}}) = 0.02 \text{ [mm]}$$

$$u_{\text{fin}} = u_{\text{fin},G} + u_{\text{fin},Q1} + u_{\text{fin},Qi,s} + u_{\text{fin},Qi,w} = 0.29 \text{ [mm]}$$

$$u_{\text{fin}} < 0.004L \rightarrow 0.29 < 1.60 \text{ [mm]} \quad (\text{UC8} = 0.18)$$

D.4.2.5 Verification steel beam (Span 6.60 m)

This section differs from D.4.2.1-D.4.2.4, as it is related to the floor range **5.8 – 6.6 m**. The structural verification is performed for both ‘normal beams’, as ‘the steel beam below the façade’. Table D.27 provides the loads, while D.28 contains the properties of the steel HEA220 beam.

Table D.27: Load definition on steel element.

Aspect	Value	Unit
Normal beams		
Span	4000	mm
Design load SLS	10.83	kN/m
Design load ULS	15.33	kN/m
Beams below facade		
Span = (4000/2) + (1200/2)	2600	mm
Design load SLS	8.44	kN/m
Design load ULS	11.65	kN/m

Table D.28: Properties HEA220.

Property	Value	Unit
Yield strength f_y	235	N/mm ²
b_{beam}	220	mm
h_{beam}	210	mm
A	6430	mm ²
$W_{y,pl}$	$5.69 \cdot 10^5$	mm ³
I_y	$5.41 \cdot 10^7$	mm ⁴
ρ	7850	kg/m ³
E_s	210000	N/mm ²
t_f	11	mm
t_w	7	mm
r	18	mm

1. Cross-section classification

Flange:

$$c = b / 2 - t_w / 2 - r = 88.5 \text{ [mm]}; \quad t = t_f$$

$$c/t = 8.05 < 9\xi = 9 \text{ [-]}$$

-> Class 1

Web:

$$c = h - 2t_f - 2r = 152 \text{ [mm]}; \quad t = t_w$$

$$c/t = 21.71 < 72\xi = 72 \text{ [-]}$$

-> Class 1

The profile is classified as class 1, meaning that a **plastic calculation** can be performed.

2. Verification bending [ULS]

2a Normal beam

$$M_d = \frac{q_d L^2}{8} = 86.8 \text{ [kNm]}$$

$$\sigma_M = \frac{M_d}{W_{pl,y}} = 152.7 \text{ [N/mm}^2\text{]}$$

$$\sigma_M < f_{yd} \rightarrow 152.7 < 235.0 \text{ [kNm]}$$

(UC = 0.65)

2b Façade beam

$$M_d = \frac{q_d L^2}{8} = 66.7 \text{ [kNm]}$$

$$\sigma_M = \frac{M_d}{W_{pl,y}} = 117.4 \text{ [N/mm}^2\text{]}$$

$$\sigma_M < f_{yd} \rightarrow 117.4 < 235.0 \text{ [kNm]}$$

(UC = 0.50)

3. Verification shear [ULS]

$$A_{v,z} = \text{maximum}(A - 2bt_f + (t_w + 2r)t_f; \eta h_w t_w) = 2067 \text{ [mm}^2\text{]}$$

$$V_{pl} = \frac{A_v \left(\frac{f_y}{\sqrt{3}} \right)}{\gamma_{M0}} = 280.45 \text{ [kN]}$$

3a Normal beam

$$V_{ed} = 0.5q_{ed}L = 52.6 \text{ [kN]} < 280.45$$

(UC = 0.19)

3b Façade beam

$$V_{ed} = 0.5q_{ed}L = 40.4 \text{ [kN]} < 280.45$$

(UC = 0.14)

4. Verification deflection [SLS]

4a Normal beam

$$w = \frac{5}{384} \frac{qL^4}{EI} = 24.65 \text{ [mm]}$$

$$w \leq 0.004L \quad \rightarrow \quad 24.65 < 26.40 \text{ [mm]} \quad (\text{UC} = 0.93)$$

4a Normal beam

$$w = \frac{5}{384} \frac{qL^4}{EI} = 19.45 \text{ [mm]}$$

$$w \leq 0.004L \quad \rightarrow \quad 19.45 < 26.40 \text{ [mm]} \quad (\text{UC} = 0.93)$$

D.4.2.6 Summary results

Table D.29 and D.30 provide an overview of the dimensions and unity checks for the distinct spans.

Table D.29: Results floor structure 'Normal beams'.

Floor structure properties				Beam						Plywood	
Span [m]	Beam [mm]	Spacing [mm]	Plywood [mm]	UC1a (M)	UC2a (V)	UC3a (w)	UC4 (Vibr.1)	UC5 (Vibr.2)	UC6 (Vibr.3)	UC7 (M)	UC8 (w)
3.9-4.6	70x270	400	18	0.37	0.13	0.36	0.45	0.97	0.75	0.14	0.18
4.7-5.4	95x295	500	21	0.40	0.13	0.42	0.55	0.93	0.64	0.16	0.23
5.5-5.7	95x295	400	21	0.36	0.11	0.40	0.57	0.95	0.59	0.10	0.12
5.8-6.6*	70x245	400	18	0.34	0.12	0.31	0.40	0.78	0.66	0.10	0.12

*Results are the cross beams between steel elements (HEA220)

Table D.30: Results beam below façade.

Floor beam		Results		
Span [m]	Beam [mm]	UC1b (M)	UC2b (V)	UC3b (w)
3.9-4.6	70x270	0.77	0.27	0.92
4.7-5.4	95x295	0.74	0.24	0.93
5.5-5.7	95x295 (2x)	0.37	0.12	0.5

D.4.3 Structural design of the HSB walls

Two types of walls are considered: **Single walls** [Fig. D.23a] and **housing separation walls** [Fig. D.23e]. The walls consist of timber columns (C24), clad with 18 mm plywood. The axial load is taken by the columns. The plywood provides stiffness to the walls, as well as buckling resistance (weak axis) to the columns [Fig. D.23d, g]. Typically, SLS-wood is used (sizes: 38x89; 38x120; 38x140; 38x170; 38x184 ... mm), as it has a large dimension in the strong axis, and a small dimension in the weak axis (no buckling in weak axis). The maximal spacing is set to 600 mm (Van Rossum B.V.; Internal communication). A beam is constructed below and above the columns (equivalent size as columns). This must be verified for compression perpendicular to the grain [Fig. D.23h]. The columns that bear weight of the façade are placed outside the beam, as the compression perpendicular to the grain became too large [Fig. D.23h].

The connected floor length (CFL) is needed to determine the load on the walls. Preliminary it is decided that the CFL excludes the span of the housing separation walls. A **magnification factor of 1.4** for the CFL covers all spans for housing separation walls. The walls are only verified for strength [ULS], considering 1 till 3 layers top-up. The excel sheets of the calculation can be found in appendix X.11.3 (including validation).

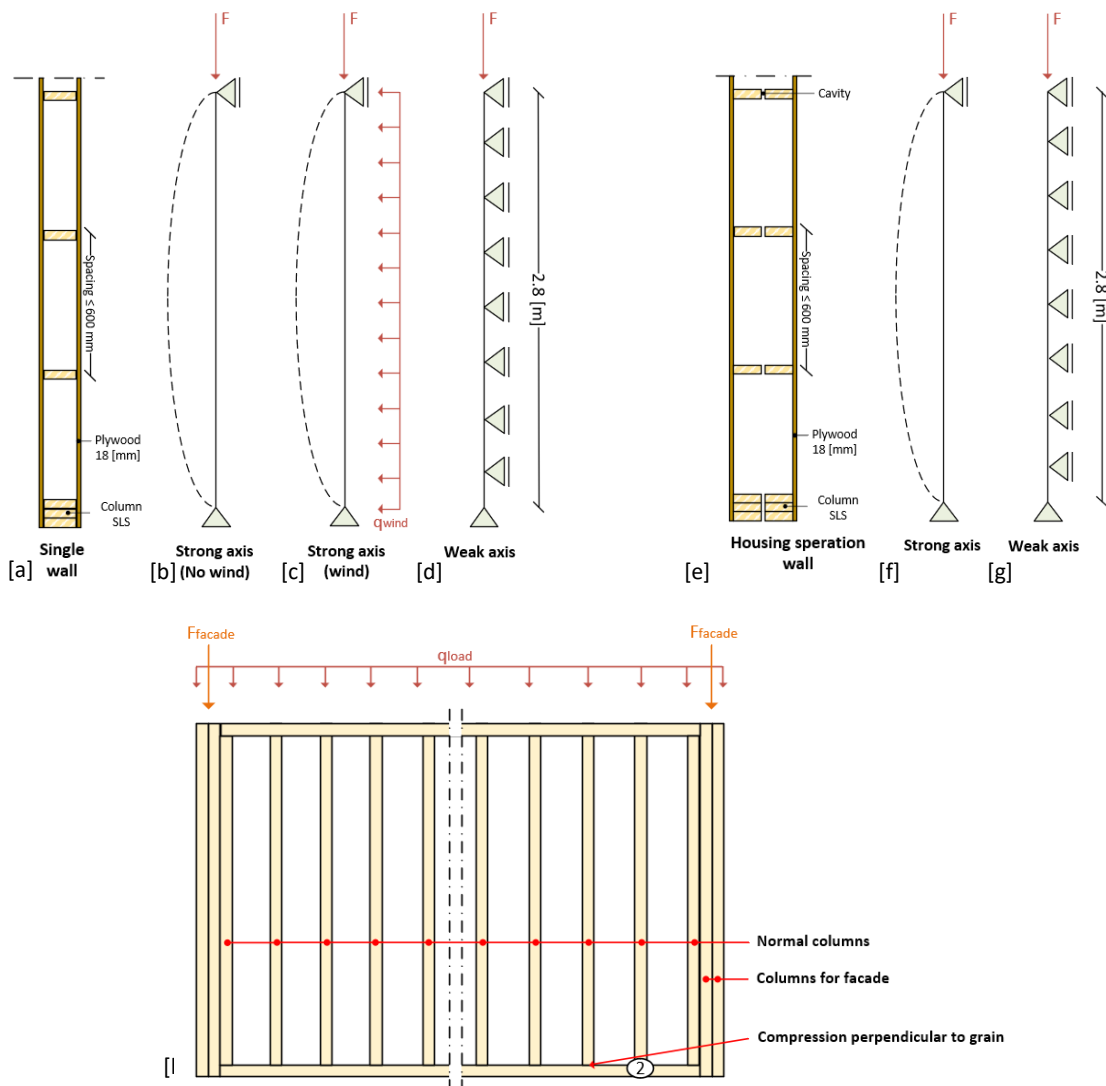


Figure D.23: HSB walls, including structural schematization; [a] Top section single wall; [b] Str. scheme 1 strong axis (Single wall); [c] Str. scheme 2 strong axis (Normal wall); [d] Str. scheme weak axis (Normal wall); [e] Top section housing separation wall; [f] Str. scheme strong axis (Housing sep. wall); [g] Str. scheme weak axis (Housing sep. wall); [h] Side view of wall element (Self-produced).

This appendix describes only the structural verification of a 3-storey top-up (CFL=6.70 m). A summary of the results is provided in appendix D.4.3.4. Table D.31 and D.32 describes the acting loads on the distinct walls. Equation D.1 and D.2 are used to calculate the design load.

Table D.31: Load definition normal wall (CFL=6.70 m).

Single wall	Value
Column dimensions (C24)	38x140 -spaced 400 mm
Number of storeys	3 [-]
Combination 1: No wind	
$F_{ed, normal\ column}$	19.55 kN
$F_{ed, 2\ columns\ for\ facade}$	21.26 kN
Combination 2: Wind	
$F_{ed, normal\ column}$	9.29 kN
$Q_{wind, ed, normal\ column}$	0.45 kN/m
$F_{ed, 2\ columns\ for\ facade}$	9.20 kN
$Q_{wind, ed, 2\ columns\ for\ facade}$	0.11 kN/m

Table D.32: Load definition housing sep. wall (CFL=6.70 m).

Property	Value
Column dimensions (C24)	38x120 -spaced 600 mm
Number of storeys	3 [-]
Combination: No wind	
$F_{ed, normal\ column}$	14.43 kN
$F_{ed, 2\ columns\ for\ facade}$	16.00 kN
Combination: Wind	
$F_{ed, normal\ column}$	9.29 kN
$Q_{wind, ed, normal\ column}$	0.45 kN/m
$F_{ed, 2\ columns\ for\ facade}$	9.20 kN
$Q_{wind, ed, 2\ columns\ for\ facade}$	0.11 kN/m

D.4.3.1 Verification single wall column – Combination 1

Climate class 1

$$\begin{aligned}
 k_{mod} &= 0.7 \\
 \gamma_M &= 1.3 \\
 f_{c,0,k} &= 21 \text{ [N/mm}^2\text{]} \\
 f_{c,0,d} &= \frac{f_{c,0,k} \cdot k_{mod}}{\gamma_M} = 11.31 \text{ [N/mm}^2\text{]}
 \end{aligned}$$

1. Buckling factor

$$b = 38 \text{ [mm]}$$

$$h = 140 \text{ [mm]}$$

$$I_y = \frac{1}{12} b h^3 = 8.69 \cdot 10^6 \text{ [mm}^4\text{]}$$

$$i_y = \sqrt{\frac{I_y}{A}} = 40.41 \text{ [mm]}$$

$$\lambda_y = \frac{l_{buc,y}}{i_y} \quad (L_{buc} = 2800 \text{ [mm]}) = 69.28 \text{ [-]}$$

$$\lambda_{rel} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = 1.17 \text{ [-]}$$

$$k_y = 0.5(1 + 0.2(\lambda_{rel} - 0.3) + \lambda_{rel}^2) = 1.28 \text{ [-]}$$

$$k_c = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel}^2}} = 0.56 \text{ [-]}$$

2. Normal column

$$F_{ed} = 19.55 \text{ [kN]}$$

$$\sigma_N = \frac{F_{ed}}{b \cdot h} = 3.68 \text{ [N/mm}^2\text{]}$$

$$UC_{N; buckling} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \underline{0.58} < 1.0$$

3. 2 Columns for façade

$$F_{ed} = 21.26 \text{ [kN]} \quad (\text{is divided by factor 2, as two columns are placed})$$

$$\sigma_N = \frac{F_{ed}}{b \cdot h} = 4.00 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \underline{0.63} \text{ [-]} < 1.0$$

4. Compression perpendicular to grain (beam below columns)

Note that the beam is constructed in the same size as the columns.

$$k_{c,90} = 1.25 \text{ [-]}$$

$$x_i = \text{MIN}(30; 400/2; 38) = 30 \text{ [mm]}$$

$$l_{\text{eff}} = 38 + (2 \cdot 30) = 98 \text{ [mm]}$$

$$A_{\text{eff}} = l_{\text{eff}} h_{\text{column}} = 13720 \text{ [mm}^2\text{]}$$

$$F_{c,90,k} = 2.5 \text{ [N/mm}^2\text{]}; k_{\text{mod}} = 0.7$$

$$\sigma_N = \frac{F}{A_{\text{eff}}} \leq k_{c,90} f_{c,90,d} = \frac{k_{c,90} k_{\text{mod}} f_{c,90,k}}{\gamma_M} \rightarrow 1.43 < 1.68 \quad (\text{UC}_{\text{beam}\perp} = 0.85)$$

5. Optimization

The chosen column is optimized. Below the results for 1 size larger spacing.

38x140 – spaced 600: UC normal column, perpendicular to grain: 1.27 > 1.0.

D.4.3.2 Verification single wall column – Combination 2

From combination 1

$$k_c = 0.56 \text{ [-]}$$

$$f_{c,0,d} = 11.31 \text{ [N/mm}^2\text{]}$$

1. Design bending strength

$$f_{m,k} = 24 \text{ [N/mm}^2\text{]}$$

$$k_h = \min\left(\left(\frac{150}{h}\right)^{0.2}; 1.3\right) = 1.01 \text{ [-]}$$

$$f_{m,d} = \frac{f_{c,0,k} \cdot k_{\text{mod}} \cdot k_h}{\gamma_M} = 13.10 \text{ [N/mm}^2\text{]}$$

2. Normal column

$$W = \frac{1}{6} b h^2 = 1.24 \cdot 10^5 \text{ [mm}^3\text{]}$$

$$M_{\text{wind,Ed}} = \frac{1}{8} q_{\text{wind,ed}} H^2 = 4.37 \cdot 10^5 \text{ [Nmm]}$$

$$\sigma_M = \frac{M_{\text{wind,Ed}}}{W} = 3.52 \text{ [N/mm}^2\text{]}$$

$$\sigma_N = \frac{F_{\text{ed}}}{b \cdot h} = 1.75 \text{ [N/mm}^2\text{]}$$

$$\text{UC}_{\text{N; buckling}} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} + \frac{\sigma_M}{f_{m,d}} = \underline{0.54} \text{ [-]} < 1.0$$

3. 2 Columns for façade

$$W = \frac{1}{6} b h^2 = 1.24 \cdot 10^5 \text{ [mm}^3\text{]}$$

$$M_{\text{wind,Ed}} = \frac{1}{8} q_{\text{wind,ed}} H^2 = 1.09 \cdot 10^5 \text{ [Nmm]}$$

$$\sigma_M = \frac{M_{\text{wind,Ed}}}{W} = 0.88 \text{ [N/mm}^2\text{]}$$

$$\sigma_N = \frac{F_{\text{ed}}}{b \cdot h} = 1.73 \text{ [N/mm}^2\text{]}$$

$$\text{UC} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} + \frac{\sigma_M}{f_{m,d}} = \underline{0.34} \text{ [-]} < 1.0$$

4. Compression perpendicular to grain (beam below columns)

$A_{\text{eff}} = 13720 \text{ [mm}^2\text{]}; k_{c,90} = 1.25 \text{ [-]}$ (Combination 1)

$$F_{c,90,k} = 2.5 \text{ [N/mm}^2\text{]}; k_{\text{mod}} = 0.7$$

$$\sigma_N = \frac{F}{A_{\text{eff}}} \leq k_{c,90} f_{c,90,d} = \frac{k_{c,90} k_{\text{mod}} f_{c,90,k}}{\gamma_M} \rightarrow 1.43 < 1.68 \quad (\text{UC}_{\text{beam}\perp} = 0.40)$$

D.4.3.3 Verification housing separation wall column

Climate class 1

$$\begin{aligned} k_{\text{mod}} &= 0.7 \\ \gamma_M &= 1.3 \\ f_{c,0,k} &= 21 \text{ [N/mm}^2\text{]} \\ f_{c,0,d} &= \frac{f_{c,0,k} k_{\text{mod}}}{\gamma_M} = 11.31 \text{ [N/mm}^2\text{]} \end{aligned}$$

1. Buckling factor

$$b = 38 \text{ [mm]}$$

$$h = 120 \text{ [mm]}$$

$$I_y = \frac{1}{12} b h^3 = 5.47 \cdot 10^6 \text{ [mm}^4\text{]}$$

$$i_y = \sqrt{\frac{I_y}{A}} = 34.64 \text{ [mm]}$$

$$\lambda_y = \frac{l_{\text{buc},y}}{i_y} \quad (L_{\text{buc}} = 2800 \text{ [mm]}) = 80.83 \text{ [-]}$$

$$\lambda_{\text{rel}} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} = 1.37 \text{ [-]}$$

$$k_y = 0.5(1 + 0.2(\lambda_{\text{rel}} - 0.3) + \lambda_{\text{rel}}^2) = 1.55 \text{ [-]}$$

$$k_c = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel}}^2}} = 0.44 \text{ [-]}$$

2. Normal column

$$F_{\text{ed}} = 14.43 \text{ [kN]}$$

$$\sigma_N = \frac{F_{\text{ed}}}{b \cdot h} = 3.16 \text{ [N/mm}^2\text{]}$$

$$\text{UC}_{\text{N; buckling}} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \mathbf{0.63} < 1.0$$

3. 2 Columns for façade

$$F_{\text{ed}} = 16.00 \text{ [kN]} \quad (\text{is divided by factor 2, as two columns are placed})$$

$$\sigma_N = \frac{F_{\text{ed}}}{b \cdot h} = 3.51 \text{ [N/mm}^2\text{]}$$

$$\text{UC} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \mathbf{0.70} \text{ [-]} < 1.0$$

4. Compression perpendicular to grain (beam below columns)

$$k_{c,90} = 1.25 \text{ [-]}$$

$$x_i = \text{MIN}(30; 600/2; 38) = 30 \text{ [mm]}$$

$$l_{\text{eff}} = 38 + (2 \cdot 30) = 98 \text{ [mm]}$$

$$A_{\text{eff}} = l_{\text{eff}} h_{\text{column}} = 11760 \text{ [mm}^2\text{]}$$

$$F_{c,90,k} = 2.5 \text{ [N/mm}^2\text{]}; k_{\text{mod}} = 0.7$$

$$\sigma_N = \frac{F}{A_{\text{eff}}} \leq k_{c,90} f_{c,90,d} = \frac{k_{c,90} k_{\text{mod}} f_{c,90,k}}{\gamma_M} \rightarrow 1.23 < 1.68 \quad (\text{UC}_{\text{beam} \perp} = \mathbf{0.73})$$

5. Optimization

The chosen column is optimized; size smaller (38x89 – spaced 600): $\text{UC}_{\text{N; buckling}} = 1.44$; $\text{UC}_{\text{facade column}} = 1.60$.

D.4.3.4 Summary of the results

Table D.33 till D.35 provides an overview of the dimensions and unity checks for the distinct walls. Note that all walls are optimized. Sometimes the unity checks are low. For those 3 points.

1. 38x89 is kept as minimum size for a wall, to have enough lay-on length for floor beams.
2. Reduction from 38x120 ($k_c = 0.44$) to 38x89 ($k_c = 0.22$) has significant impact on the buckling resistance.
3. 50x100 beams result in a higher unity check compared to 38x120. Unfortunately, the material use is higher ($A_{50 \times 100} = 5000 \text{ mm}^2$; $A_{38 \times 120} = 4560 \text{ mm}^2$), making the beam unfavourable.

Table D.33: Results for a 1-storey top-up (columns in wall).

1-storey top-up				
Single wall (No wind; Combination 1)				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x89 – spaced 600	0.78	0.53	0.67 (2)
8.49	SLS 38x89 – spaced 600	0.95	0.65	0.84 (2)
10.28	SLS 38x120 – spaced 600	0.50	0.57	0.44 (2)
Single wall (Wind; Combination 2)**				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x120 – spaced 600	0.61	0.15	0.46 (1)
8.49	SLS 38x120 – spaced 600	0.62	0.17	0.51 (1)
10.28	SLS 38x120 – spaced 600	0.64	0.18	0.56 (1)
Housing separation wall				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x89 – spaced 600	0.38	0.26	0.93 (1)
8.49	SLS 38x89 – spaced 600	0.47	0.32	0.58 (2)
10.28	SLS 38x89 – spaced 600	0.56	0.38	0.70 (2)

* Number provides the needed columns at the end of the wall, to transfer the load of the façade.

** Other walls needs to be placed at the two end facades, than at the inner walls. Otherwise, a lot of material will be wasted.

Table D.34: Results for a 2-storey top-up (columns in wall).

2-storey top-up				
Single wall (No wind; Combination 1)				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x140 – spaced 600	0.61	0.89	0.94 (1)
8.49	SLS 38x140 – spaced 400	0.51	0.74	0.55 (2)
10.28	SLS 38x140 – spaced 400	0.60	0.88	0.66 (2)
Single wall (Wind; Combination 2)**				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x140 – spaced 600	0.64	0.36	0.57 (1)
8.49	SLS 38x140 – spaced 400	0.45	0.28	0.28 (2)
10.28	SLS 38x140 – spaced 400	0.48	0.32	0.32 (2)
Housing separation wall				
CFL	Column size [mm]	$UC_{N; \text{buckling}}$	$UC_{\text{beam} \perp}$	$UC_{\text{column for façade (*)}$
6.70	SLS 38x120 – spaced 600	0.45	0.51	0.97 (1)
8.49	SLS 38 x 120 – spaced 600	0.55	0.64	0.61 (2)
10.28	SLS 38 x 120 – spaced 600	0.66	0.76	0.74 (2)

* Number provides the needed columns at the end of the wall, to transfer the load of the façade.

** Size is kept similar to inner-walls, to provide more uniformity in the top-up. Will not increase drastically the material use, as these are only 2 walls.

Table D.35: Results for a 3-storey top-up (columns in wall).

3-storey top-up				
Single wall (No wind; Combination 1)				
CFL	Column size [mm]	UC _N ; buckling	UC _{beam⊥}	UC _{column for façade (*)}
6.70	SLS 38x140 – spaced 400	0.58	0.85	0.63 (2)
8.49	SLS 38x170 – spaced 400	0.47	0.87	0.51 (2)
10.28	SLS 38x184 – spaced 400	0.48	0.95	0.53 (2)
Single wall (Wind; Combination 2)**				
CFL	Column size [mm]	UC _N ; buckling	UC _{beam⊥}	UC _{column for façade (*)}
6.70	SLS 38x140 – spaced 400	0.54	0.40	0.34 (2)
8.49	SLS 38x170 – spaced 400	0.39	0.39	0.27 (2)
10.28	SLS 38x184 – spaced 400	0.36	0.41	0.26 (2)
Housing separation wall				
CFL	Column size [mm]	UC _N ; buckling	UC _{beam⊥}	UC _{column for façade (*)}
6.70	SLS 38x120 – spaced 600	0.63	0.73	0.70 (2)
8.49	SLS 38x120 – spaced 600	0.78	0.90	0.88 (2)
10.28	SLS 38x140 – spaced 600	0.63	0.92	0.72 (2)

* Number provides the needed columns at the end of the wall, to transfer the load of the façade.

** Size is kept similar to inner-walls, to provide more uniformity in the top-up. Will not increase drastically the material use, as these are only 2 walls.

D.4.4 Structural design of the gallery beams

Gallery beams span between the column at the gallery and the HSB wall. The floor and roof beams span between the gallery beams, clad with plywood [Fig. D.24a]. Unfortunately, the gallery beam can be constructed much smaller, compared to the floor and roof beams. The span is much lower (1.2 m) and the calculation does not include vibrations. To make the connection between floor/roof and gallery beams, two options are considered.

- 1. Placement of the floor/roof beams on top of the gallery beams.** A minimum lay-up length is needed (≈ 90 mm), which will result in massive beams. Additionally, it increases the structural height.
- 2. Placement of floor/roof beams between the gallery beams [Fig. D.24a].** To make this solution practically and architecturally feasible, the gallery beams must be chosen in an equivalent size to the floor/roof beams.

The total material use of these beams has hardly any impact on the total material use. Chosen is for the second solution, as it lowers the structural height, and is architecturally more pleasant. This appendix verifies for both floor- and roof level, the most unfavourable situation (Smallest beam; CFL = 10.28 (maximum)). Appendix X.11.4 provides an excel sheet of the calculation, including a hand validation.

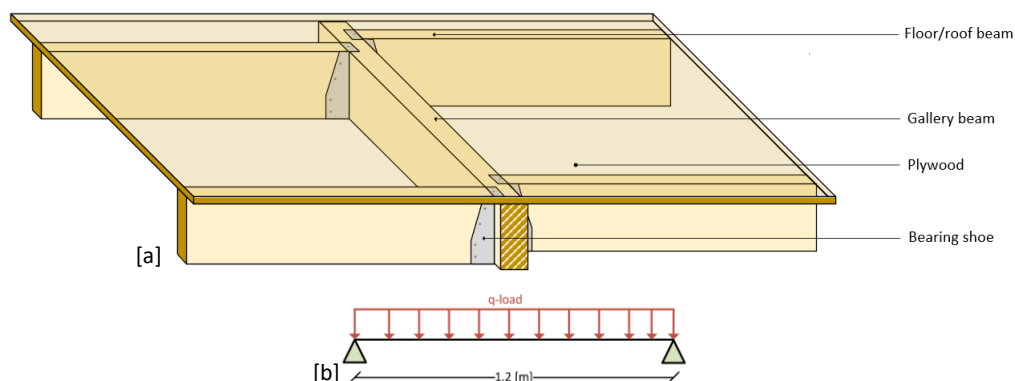


Figure D.24: Gallery beam; [a] 3D drawing of the connection. [b] Simplified structural scheme of gallery beam (Self-produced).

D.4.4.1 Verification of gallery beam (at floor level)

1. Applied loading and properties

Table D.36 provides an overview of the used loads. Equation D.1 and D.2 are used to calculate the design load. Table D.37 indicates the properties of the gallery beam.

Table D.36: Load definition gallery beam (at floor level).

Load	Value [kN/m]
Permanent	
Floor structure (1.00 kN/m ²)	5.14
Self-weight beam	0.08
Variable	
Residential load (1.75 kN/m ²)	9.00
Non-structural walls (0.5 kN/m ²)	2.57
$q_{Ed,eq D.1}$	13.99
$q_{Ed,eq D.2}$	21.30

Table D.37: Wood properties gallery beam.

Property	Value	Unit
Beam (C24)		
Dimensions	70 x 270	mm
E-modulus	11000	N/mm ²
ρ_{mean}	420	kg/m ³
$f_{m,k}$	24	N/mm ²
$f_{v,k}$	4	N/mm ²
Span	1200	mm

$$k_{mod} = 0.7 \text{ (long term; Service class 1)}$$

$$\gamma_M = 1.3 \text{ [-]}$$

$$k_h = 1.0 \text{ (} h > 150 \text{ mm)}$$

2. Bending [ULS]

$$W_z = \frac{1}{6} b h^2 = 8.51 \cdot 10^5 \text{ [mm}^3\text{]}$$

$$M_d = \frac{q_d L^2}{8} = 3.83 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W_z} < f_{M,Rd} = \frac{f_{M,k}}{\gamma_M} \cdot k_{mod} \cdot k_h \rightarrow 4.51 < 12.92 \text{ [N/mm}^2\text{]} \quad (\mathbf{UC_M = 0.35})$$

3. Shear [ULS]

$$V_d = 0.5 q_d L = 12.78 \text{ [kN]}$$

$$\tau_d = \frac{3V_d}{2bh} \leq f_{v,d} = \frac{f_{v,k} \cdot k_{mod}}{\gamma_M} \rightarrow 1.01 < 2.15 \text{ [N/mm}^2\text{]} \quad (\mathbf{UC_v = 0.47})$$

4. Force perpendicular to the grain [ULS] (Beam on HSB wall)

$$V_d = 0.5 q_d L = 12.78 \text{ [kN]}$$

$$\text{Width} = 2 \times 38 = 76 \text{ [mm]} \quad (2 \text{ columns } 38 \times 89 \text{ at the end assumed})$$

$$X_i = \min(30; \text{width}; \text{Span}) = \min(30; 76; 1200) = 30$$

$$L_{eff} = 76 + 2 \cdot 30 = 136 \text{ [mm]} \rightarrow 2 \text{ sides } 30 \text{ mm needed!}$$

$$A_{eff} = 136 \cdot \text{MIN}(\text{Column height}=89; \text{beam thickness}=70) = 9520$$

$$K_{c,90} = 1.25 \text{ [-]}$$

$$\sigma_{c,90} = \frac{V_d}{A_{eff}} = 1.34 \text{ [N/mm}^2\text{]}$$

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} = \frac{k_{c,90} \cdot k_{mod} \cdot f_{c,90,k}}{\gamma_M} \rightarrow 1.34 < 1.69 \quad (\mathbf{UC_{90} = 0.80})$$

Note: In case 1 column will be placed at the end of the wall, it should always be recalculated. In case it is not sufficient, 2 columns should be placed at the end of the wall, than it will be sufficient.

5. Deflection [SLS]

$$k_{def} = 0.8 \text{ (Service class 1)}$$

$$\psi_0 = 0.3; \psi_2 = 0.8 \text{ [-]}$$

$$I_z = \frac{1}{12} b h^3 = 1.15 \cdot 10^8 \text{ [mm}^4\text{]}$$

$$u_{fin,G} = u_{inst,G}(1 + k_{def}) = \frac{5}{384} \frac{q_g L^4}{EI} (1 + k_{def}) = 0.20 \text{ [mm]}$$

$$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Q1} L^4}{EI} (1 + \psi_2 k_{def}) = 0.24 \text{ [mm]}$$

$$u_{fin,Qi} = u_{inst,Qi}(\psi_0 + \psi_2 k_{def}) = \frac{5}{384} \frac{q_{Qi} L^4}{EI} (\psi_0 + \psi_2 k_{def}) = 0.04 \text{ [mm]}$$

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + u_{fin,Qi} = 0.47 \text{ [mm]}$$

$$u_{fin} < 0.004L \rightarrow 0.47 < 4.8 \text{ [mm]} \quad (\text{UC}_w = 0.10)$$

D.4.4.2 Verification of gallery beam (at roof level)

1. Applied loading and properties

Table D.38 provides an overview of the used loads. Equation D.1 and D.2 are used to calculate the design load. Table D.39 indicates the properties of the gallery beam.

Table D.38: Load definition gallery beam (at floorlevel).

Load	Value [kN/m]
Permanent	
Floor structure (0.70 kN/m ²)	3.60
Self-weight beam	0.05
Variable	
Imposed load (1.00 kN/m ²)	5.14
q _{Ed,eq D.1}	4.92
q _{Ed,eq D.2}	12.09

Table D.39: Wood properties gallery beam.

Property	Value	Unit
Beam (C24)		
Dimensions	70 x 170	mm
E-modulus	11000	N/mm ²
ρ _{mean}	420	kg/m ³
f _{m,k}	24	N/mm ²
f _{v,k}	4	N/mm ²
Span	1200	mm

$$k_{mod} = 0.7;$$

$$\gamma_M = 1.3 [-];$$

$$k_h = 1.0 \text{ (} h > 150 \text{ mm)}$$

2. Bending [ULS]

$$W_z = \frac{1}{6} b h^2 = 3.37 \cdot 10^5 \text{ [mm}^3\text{]}$$

$$M_d = \frac{q_d L^2}{8} = 2.18 \text{ [kNm]}$$

$$\sigma_{M;d} = \frac{M_d}{W_z} < f_{M,Rd} = \frac{f_{M,k}}{\gamma_M} \cdot k_{mod} \cdot k_h \rightarrow 6.45 < 12.92 \text{ [N/mm}^2\text{]} \quad (\text{UC}_M = 0.50)$$

3. Shear [ULS]

$$V_d = 0.5 q_d L = 7.25 \text{ [kN]}$$

$$\tau_d = \frac{3V_d}{2bh} \leq f_{v,d} = \frac{f_{v,k} \cdot k_{mod}}{\gamma_M} \rightarrow 0.91 < 2.15 \text{ [N/mm}^2\text{]} \quad (\text{UC}_V = 0.42)$$

4. Force perpendicular to the grain [ULS] (Beam on HSB-Wall)

$$V_d = 0.5 q_d L = 7.25 \text{ [kN]}$$

$$\text{Width} = 2 \times 38 = 76 \text{ [mm]} \quad (2 \text{ columns } 38 \times 89 \text{ at the end assumed})$$

$$X_i = \min(30; \text{width}; \text{Span}) = \min(30; 76; 1200) = 30$$

$$L_{eff} = 76 + 2 \cdot 30 = 136 \text{ [mm]} \rightarrow 2 \text{ sides } 30 \text{ mm needed!}$$

$$A_{eff} = 136 \cdot \text{MIN}(\text{Column height}=89; \text{beam thickness}=70) = 9520$$

$$K_{c,90} = 1.25 [-]$$

$$\sigma_{c,90} = \frac{V_d}{A_{eff}} = 0.76 \text{ [N/mm}^2\text{]}$$

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} = \frac{k_{c,90} \cdot k_{mod} \cdot f_{c,90,k}}{\gamma_M} \rightarrow 0.76 < 1.69 \quad (\text{UC}_{90} = 0.45)$$

5. Deflection [SLS]

$k_{def} = 0.8$ (Service class 1)

$\psi_2 = 0$ [-]

$I_z = \frac{1}{12}bh^3 = 2.87 * 10^7$ [mm⁴]

$u_{fin,G} = u_{inst,G}(1 + k_{def}) = \frac{5}{384} \frac{qgL^4}{EI} (1 + k_{def}) = 0.56$ [mm]

$u_{fin,Q1} = u_{inst,Q1}(1 + \psi_2k_{def}) = \frac{5}{384} \frac{qQ_1L^4}{EI} (1 + \psi_2k_{def}) = 0.44$ [mm]

$u_{fin} = u_{fin,G} + u_{fin,Q1} = 1.00$ [mm]

$u_{fin} < 0.004L \rightarrow 1.00 < 4.8$ [mm]

(UC_w = 0.21)

D.4.5 Structural design of the gallery columns

The gallery columns transfer half of the gallery load by an axial force. The columns are unsupported for 1 floor in both directions ($L_{buc,y} = L_{buc,z} = 2.8$ m) [Fig. D.25c]. Resultingly, chosen is for a square column. The columns are verified for strength [ULS]. Equation D.1 and D.2 are used to calculate the design load. This appendix will only provide the calculation for a 3-storey top-up (CFL=10.28 m) [Table D.40]. The used excel sheets can be found in appendix X.11.5 (including validation).

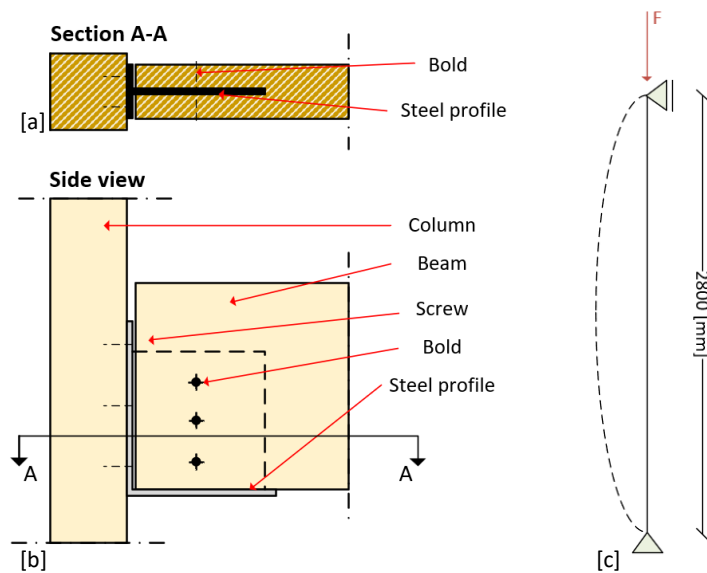


Figure D.25: Gallery column; [a] Top section connection column-beam; [b] Side view connection column-beam; [c] Schematization column (Self-produced).

Table D.40: Starting points for calculation column gallery.

Aspect	Value
Number of storeys top-up	3
CFL	10.28 m
Column size	95x95 mm
F _{ed}	28.42 kN

D.4.5.1 Verification of gallery column (3-storey; CFL=10.28)

Climate class 1

$$\begin{aligned}
 k_{\text{mod}} &= 0.7 \\
 \gamma_M &= 1.3 \\
 f_{c,0,k} &= 21 \text{ [N/mm}^2\text{]} \\
 f_{c,0,d} &= \frac{f_{c,0,k} \cdot k_{\text{mod}}}{\gamma_M} = 11.31 \text{ [N/mm}^2\text{]}
 \end{aligned}$$

1. Buckling factor

$$b = 95 \text{ [mm]}$$

$$h = 95 \text{ [mm]}$$

$$I_y = I_z = \frac{1}{12} b h^3 = 6.79 \cdot 10^6 \text{ [mm}^4\text{]}$$

$$i_y = \sqrt{\frac{I_y}{A}} = 27.42 \text{ [mm]}$$

$$\lambda_y = \frac{l_{\text{buc},y}}{i_y} \quad (l_{\text{buc}} = 2800 \text{ [mm]}) = 102.10 \text{ [-]}$$

$$\lambda_{\text{rel}} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = 1.73 \text{ [-]}$$

$$k_y = 0.5(1 + 0.2(\lambda_{\text{rel}} - 0.3) + \lambda_{\text{rel}}^2) = 2.14 \text{ [-]}$$

$$k_c = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{\text{rel}}^2}} = 0.29 \text{ [-]}$$

2. Check

$$F_{\text{ed}} = 28.42 \text{ [kN]}$$

$$\sigma_N = \frac{F_{\text{ed}}}{b \cdot h} = 3.15 \text{ [N/mm}^2\text{]}$$

$$\text{UC} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \underline{\underline{0.95}} < 1.0$$

D.4.5.2 Summary of results

Table D.41 till D.43 provide an overview of the results for distinct CFL and number of storeys. Also the unity check for 1 column size smaller is added, to show that the column is optimized.

Table D.41: Results for 1-storey top-up (column size gallery).

1-storey top-up			
CFL [m]	Column size [mm]	UC	UC (44x70 mm)
6.70 – 8.49	70 x 70	0.65	2.54
8.49 – 10.28	70 x 70	0.79	3.07

Table D.42: Results for 2-storey top-up (column size gallery).

2-storey top-up			
CFL [m]	Column size [mm]	UC	UC (70x70 mm)
6.70 – 8.49	95 x 95	0.56	1.80
8.49 – 10.28	95 x 95	0.67	2.18

Table D.43: Results for 3-storey top-up (column size gallery).

3-storey top-up			
CFL [m]	Column size [mm]	UC	UC (70x70 mm)
6.70 – 8.49	95 x 95	0.78	2.53
8.49 – 10.28	95 x 95	0.95	3.06

D.4.6 Structural design of the stability wall

The structural system of the existing structure continues to the top-up. Resultingly, the stability wall is located above the existing one. That indicates that the size of the stability wall strongly depends on the existing ratio: depth building/ length stability wall.

Although the stability of a structure forms a major point in the structural design, it has not a major impact on the total material use of the top-up. For this research, only the most unfavourable situation is calculated (**Ratio = 4.5; 3-storey top-up**). Further properties are based on the Helena Mercierstraat. Note that for future practise, the stability always must be recalculated.

For HSB-walls, it is uncommon to calculate the deflection of the structure (the norm provides no rules for the stiffness of an HSB-wall). In practice, 18 mm plywood should be applied to provide diaphragm action (Van Rossum B.V.; Internal communication). Two aspects must be calculated:

1. Tension caused by the wind load;
2. Needed number of nails in wall;

Two options are though of during the design process, to provide a stable structure.

1. Provide a stable structure for a single building block (2 porches).
2. Connect two blocks to distribute the wind load over more stability walls.

For this situation (Ratio = 4.5; 3-storey top-up; No additional room), option 1 is unfavourable. The tension force becomes large, as the wind load is only distributed over two walls. Chosen is for option 2, as it does not increase the material use. Additionally, it is highly assumable that the substructure needs to be linked horizontally, to provide enough resistance against the horizontal load [Fig. 6.15]. Table D.44 provides properties of the stability wall. Figure D.26 provides a drawing of the stability principle. The calculation can be found in appendix X.11.6 (including validation).

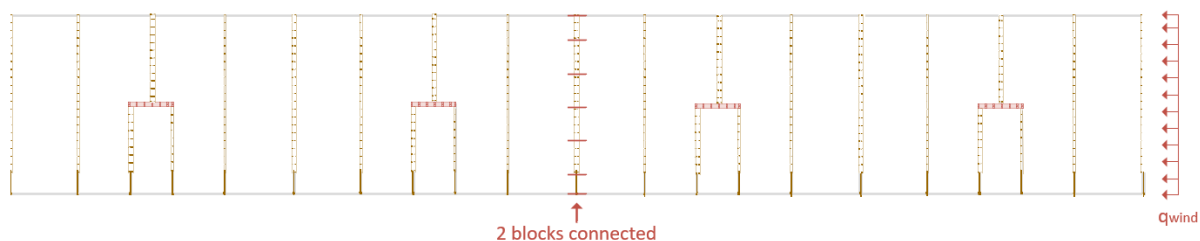


Figure D.26: Stability system top-up HSB (Self-produced).

Table D.44: Stability wall properties.

Aspect	Value
General	
Type of wall	Housing sep. wall
Number of storeys top-up	3
Ratio depth/length stab. wall	4.5 [-]
Length stab. wall	2.41 m
Depth building	10.85 m
Columns wall (C24)	
Dimensions	38x120 mm
Spacing	600 mm
Plywood	
Thickness	18 mm

D.4.6.1 Verification in longitudinal direction

1. Tension caused by wind

Wind area II; Urban

Number of walls = 4

$H_{\text{building}} = 8.4 \text{ [m]}$

$$q_{w;ULS} = \frac{q_p(22.4) \cdot C_s C_d \cdot C_f \cdot \text{Depth building}}{\text{Number of walls}} \cdot \gamma_Q (= 1.5) = 3.03 \text{ [kN/m]}$$

$$M_{ULS} = \frac{1}{2} q_{w;ULS} H^2 = 106.90 \text{ [kNm]}$$

$$T_{\text{ed}} = \frac{M_{ULS}}{L_{\text{stab.wall}}} = 44.29 \text{ [kN]}$$

2. Compression caused by self-weight

Self-weight wall = 4.55 kN/m -> it is assumed that the floors transfer no weight to the wall.

$$F_c = \frac{1}{2} \cdot q_{\text{self}} \cdot L_{\text{stab.wall}} \cdot \gamma_G (= 0.9) = 4.94 \text{ [kN]}$$

3. Tension anchoring

$$F_{t,\text{net}} = T_{\text{ed}} - F_c = 39.36 \text{ [kN]}$$

$$F_{t,\text{net; per wall side}} = 39.36 / 2 = 19.68 \text{ [kN]}$$

Tension capacity anchor (WHT440) = 21.3 [kN] [Fig. D.27] -> Each wall side 1 anchor (4 total)

STATIC VALUES - CEMENT/WOOD TENSILE JOINT - TOTAL NAILING

Code	Washer	Anker Nails Ø 4.0x60 n _{conn} [pc.]	Threaded Bar + Vinylester resin [pcs.]	Ø x L [mm]	R _{1,zul} (admissible)		R _{1,k} (characteristic)	
					wood side [kN]	steel side [kN]	wood side [kN]	steel side [kN]
WHT340	-	20	1	16 x 190	14.2	21.5	38.6	42.0
WHT440	H=10mm ULSS05610	30	1	16 x 190	21.3	32.4	57.9	63.4
WHT540	H=10mm ULSS05610	42	1	16 x 190	29.8	32.4	81.1	63.4
WHT620	H=20mm ULS707720	52	1	20 x 240	36.9	46.8	100.4	85.2

Figure D.27: Tension capacity of tension anchors (Van Rossum B.V.; Internal communication).

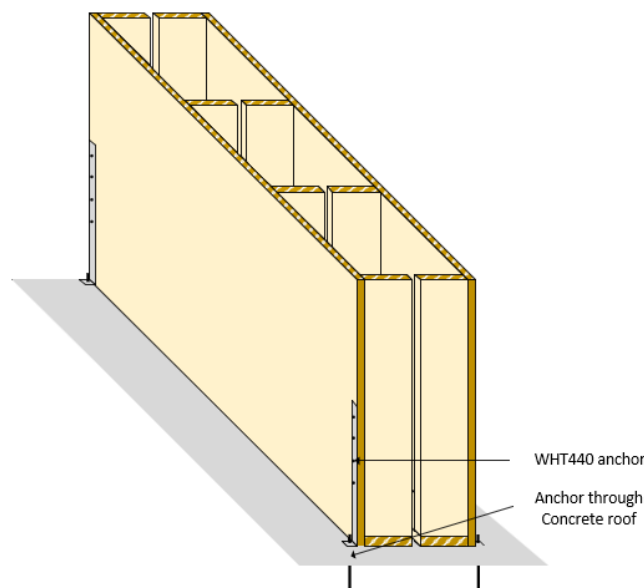


Figure D.28: Anchoring stability wall.

4. Verification maximum compression

$$F_{C,max} = T_{ed} + F_c = 44.29 + 4.94 = 49.23 \text{ [kN]}$$

$$\text{Number of columns (per half wall)} = 4 \text{ [-]}$$

$$F_{C, \text{ per column}} = 49.23 / 4 = 12.31 \text{ [kN]}$$

$$A_{\text{Column: } 38 \times 120} = 4560 \text{ [mm}^2\text{]}$$

$$\sigma_N = F_{C, \text{ per column}} / A = 2.69 \text{ [N/mm}^2\text{]}$$

$$k_c = 0.44 \text{ (From excel walls)}$$

$$k_{mod} = 0.7; \gamma_M = 1.3 \text{ [-]; } f_{c,0,k} = 21 \text{ [N/mm}^2\text{]}$$

$$f_{c,0,d} = \frac{f_{c,0,k} \cdot k_{mod}}{\gamma_M} = 11.31 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \mathbf{0.54} \text{ [-]} < 1.0 \text{ [-]}$$

5. Verification compression perpendicular to the grain (beam below columns) [Fig. D.28]

$$K_{c,90} = 1.25$$

$$X_i = \min(30; 600/2; 38) = 30 \text{ [mm]}$$

$$L_{eff} = 38 + (2 \cdot 30) = 98 \text{ [mm]}$$

$$A_{eff} = B_{beam} L_{eff} = 11760 \text{ [mm}^2\text{]}$$

$$\sigma_N = F_{C, \text{ per column}} / A_{eff} = 1.04 \text{ [N/mm}^2\text{]}$$

$$k_{mod} = 0.7; \gamma_M = 1.3 \text{ [-]; } f_{c,90,k} = 2.5 \text{ [N/mm}^2\text{]}$$

$$f_{c,90,d} = \frac{f_{c,90,k} \cdot k_{mod}}{\gamma_M} = 1.35 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_N}{f_{c,90,d} K_{c,90}} = \mathbf{0.62} \text{ [-]} < 1.0 \text{ [-]}$$

D.4.6.2 Calculation needed nails (longitudinal direction)**1. Properties nail**

$$\text{Thickness (d)} = 3.1 \text{ [mm]}$$

$$\text{Length} = 50 \text{ [mm]}$$

$$f_{u,k} = 400 \text{ [N/mm}^2\text{]}$$

$$M_{y,Rk} = 0.3 f_{u,k} d^{2.6} = 2273.64 \text{ [Nmm]}$$

2. Properties of plywood

$$\text{Thickness (t}_1\text{)} = 18 \text{ [mm]}$$

$$\rho_k = 470 \text{ [kg/m}^3\text{]}$$

$$f_{h,1,k} = 0.11 \rho_k d^{-0.3} = 36.82 \text{ [N/mm}^2\text{]}$$

3. Properties of columns

$$t_2 = \text{length nail} - t_1 = 32 \text{ [mm]}$$

$$\rho_k = 350 \text{ [kg/m}^3\text{]}$$

$$f_{h,2,k} = 0.082(1 - 0.01d) \rho_k = 27.81$$

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} = 0.76 \text{ [-]}$$

4. Estimation $F_{ax,Rk}$

$$\frac{1}{4} F_{ax,Rk} = \frac{1}{4} \cdot \frac{1}{4} \pi d^2 \cdot f_{u,k} = 754.77 \text{ [kN]} \rightarrow \text{Self-produced formula. Most presumable a bit too high}$$

value, but $\frac{1}{4} F_{ax,Rk}$ is limited to 15% of johansen part.

5. Calculation $F_{V,Rk}$

Red numbers reflect to values in excel sheet [Appendix X.11.6.2].

$$F_{V,Rk} = \min \left\{ \begin{array}{l} f_{h,1,k}t_1d \text{ (a)} \\ f_{h,2,k}t_2d \text{ (b)} \\ \frac{f_{h,1,k}t_1d}{1+\beta} \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2 \right] + \beta^3 \left(\frac{t_2}{t_1}\right)^2} - \beta \left(1 + \frac{t_2}{t_1} \right) \right] + \frac{F_{ax,Rk}}{4} \text{ (c)} \\ 1.05 \frac{f_{h,1,k}t_1d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k}dt_1^2} - \beta} \right] + \frac{F_{ax,Rk}}{4} \text{ (d)} \\ 1.05 \frac{f_{h,1,k}t_2d}{1+2\beta} \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k}dt_2^2} - \beta} \right] + \frac{F_{ax,Rk}}{4} \text{ (e)} \\ 1.15 \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2M_{y,Rk}f_{h,1,k}d} + \frac{F_{ax,Rk}}{4} \text{ (f)} \end{array} \right. = \min \left\{ \begin{array}{l} 2054.55 \\ 2758.78 \\ 1189.86 \\ 1397.23 \\ 2074.86 \\ 883.88 \end{array} \right. = 883.88 \text{ [N]}$$

6. Calculation $F_{V,Rd}$

$$k_{mod} = 0.7; \gamma_M = 1.3$$

$$F_{V,Rd} = \frac{F_{V,Rk} \cdot k_{mod}}{\gamma_M} = 475.94 \text{ [N]}$$

7. Calculation shear

$$H_{tot} = 8.4 \text{ [m]}$$

$$H_{storey} = 2.8 \text{ [m]}$$

$$F_2 \text{ per side} = \frac{q_p(22.4) \cdot C_s C_d \cdot C_f}{\text{Number of walls} \cdot 2 \text{ (sides)}} \cdot \gamma_Q (= 1.5) \cdot (H_{tot} - 0.5H_{storey}) = 10.60 \text{ [kN]}$$

$$F_1 \text{ per side} = \frac{F_2 \cdot H_{storey}}{L_{stab.wall}} = 12.30 \text{ [kN]}$$

$$\tau_1 = \tau_2 = \frac{F_1 \text{ per side}}{H_{storey}} = 4.39 \text{ [kN/m]}$$

$$\text{Needed nails per meter} = \frac{\tau_1}{F_{v,Rd}} = 10 \text{ [-]} \rightarrow \text{spacing} = 100 \text{ [mm]}$$

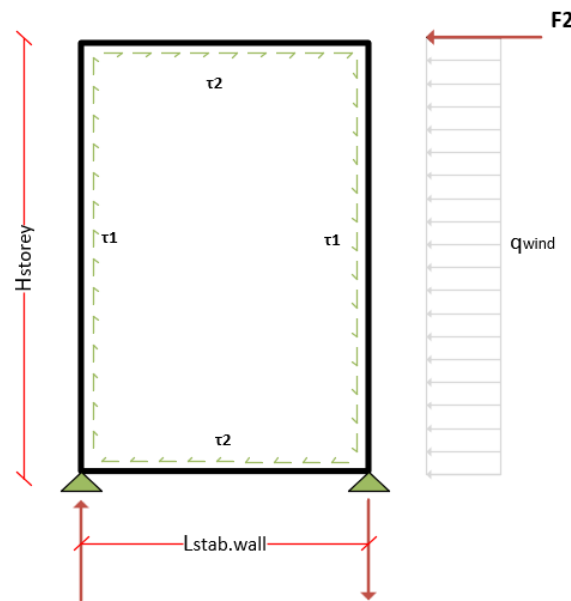


Figure D.29: Shear between columns and plywood.

D.4.6.3 Verification in transversal direction

Just a small verification, although it is known that this direction is not normative.

1. Tension caused by wind

Wind area II; Urban

$$H_{\text{building}} = 8.4 \text{ [m]}$$

$$L_{\text{wall}} = 10.85 \text{ [m]}$$

Width that transfers wind load = 2.94 [m] (estimation)

$$q_{w;ULS} = q_p(22.4) \cdot C_s C_d \cdot C_f \cdot \text{Width} (= 2.94 \text{ [m]}) \cdot \gamma_Q (= 1.5) = 6.15 \text{ [kN/m]}$$

$$M_{ULS} = \frac{1}{2} q_{w;ULS} H^2 = 217.0 \text{ [kNm]}$$

$$T_{\text{ed}} = \frac{M_{ULS}}{L_{\text{wall}}} = 20.00 \text{ [kN]}$$

2. Compression caused by self-weight

$$\text{Roof} = 1.38 \text{ [kN/m]}$$

$$3^{\text{rd}} \text{ floor} = 1.96 \text{ [kN/m]}$$

$$2^{\text{nd}} \text{ floor} = 1.96 \text{ [kN/m]}$$

$$\text{Wall} = 4.55 \text{ [kN/m]}$$

$$\text{Total} = 9.84 \text{ [kN/m]}$$

$$F_{c,Ed} = 0.5qL_{\text{wall}} \gamma_G = 48.06 \text{ [kN]}$$

Tension to substructure: $T_{\text{ed}} < F_{c,Ed}$ -> No tension

3. Maximum compression

$$F_{c,\text{max}} = T_{\text{ed}} + F_{c,Ed} = 68.06 \text{ [kN]}$$

Number of columns considered -> 9

$$F_{c,\text{max,per column}} = 7.53 \text{ [kN]}$$

$$A_{\text{Column: } 38 \times 120} = 4560 \text{ [mm}^2\text{]}$$

$$\sigma_N = F_{c,\text{per column}} / A = 1.65 \text{ [N/mm}^2\text{]}$$

$k_c = 0.44$ (From excel walls)

$$k_{\text{mod}} = 0.7; \gamma_M = 1.3 [-]; f_{c,0,k} = 21 \text{ [N/mm}^2\text{]}$$

$$f_{c,0,d} = \frac{f_{c,0,k} \cdot k_{\text{mod}}}{\gamma_M} = 11.31 \text{ [N/mm}^2\text{]}$$

$$\text{UC} = \frac{\sigma_N}{k_c \cdot f_{c,0,d}} = \mathbf{0.33} [-] < 1.0 [-]$$

4. Verification compression perpendicular to the grain (beam below columns)

$$K_{c,90} = 1.25$$

$$X_i = \min(30; 600/2; 38) = 30 \text{ [mm]}$$

$$L_{\text{eff}} = 38 + (2 \cdot 30) = 98 \text{ [mm]}$$

$$A_{\text{eff}} = B_{\text{beam}} L_{\text{eff}} = 11760 \text{ [mm}^2\text{]}$$

$$\sigma_N = F_{c,\text{per column}} / A_{\text{eff}} = 0.64 \text{ [N/mm}^2\text{]}$$

$$k_{\text{mod}} = 0.7; \gamma_M = 1.3 [-]; f_{c,90,k} = 2.5 \text{ [N/mm}^2\text{]}$$

$$f_{c,90,d} = \frac{f_{c,90,k} \cdot k_{\text{mod}}}{\gamma_M} = 1.35 \text{ [N/mm}^2\text{]}$$

$$\text{UC} = \frac{\sigma_N}{f_{c,90,d} \cdot K_{c,90}} = \mathbf{0.38} [-] < 1.0 [-]$$

D.4.7 Weight of the HSB top-up

The weight of the top-up is calculated for one, two, and three added layers. The new calculated weight is a bit higher than the estimated value (4.92 kN/m^2 / storey). This is caused by and higher values for floor and roof weight (estimated too low: 0.45 kN/m^2). Note that a 5% exceeding of the overcapacity is used (Van Rossum B.V.; Internal communication). The calculation of the new weight can be found in appendix [X.11.7](#).

- 1 added floor: 5.5 kN/m^2
- 2 added floors: 10.1 kN/m^2
- 3 added floors: 13.4 kN/m^2

D.5 Environmental impact HSB extension

D.5.1 Global warming potential

D.5.1.1 Line graphs of total

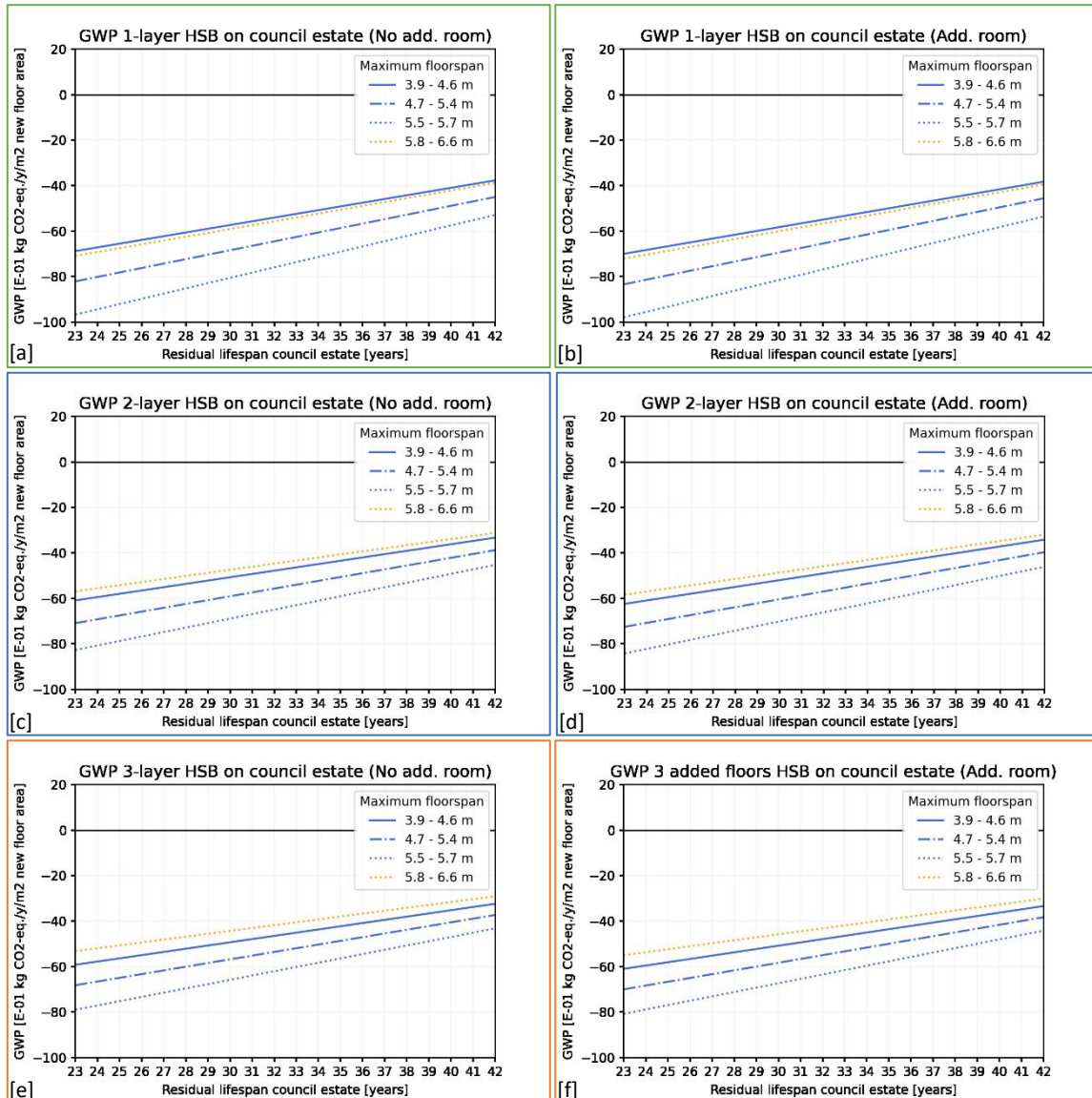


Figure D.30: Results total global warming potential HSB top-up (Self-produced).

D.5.1.2 Bar graphs of GWP separated per element

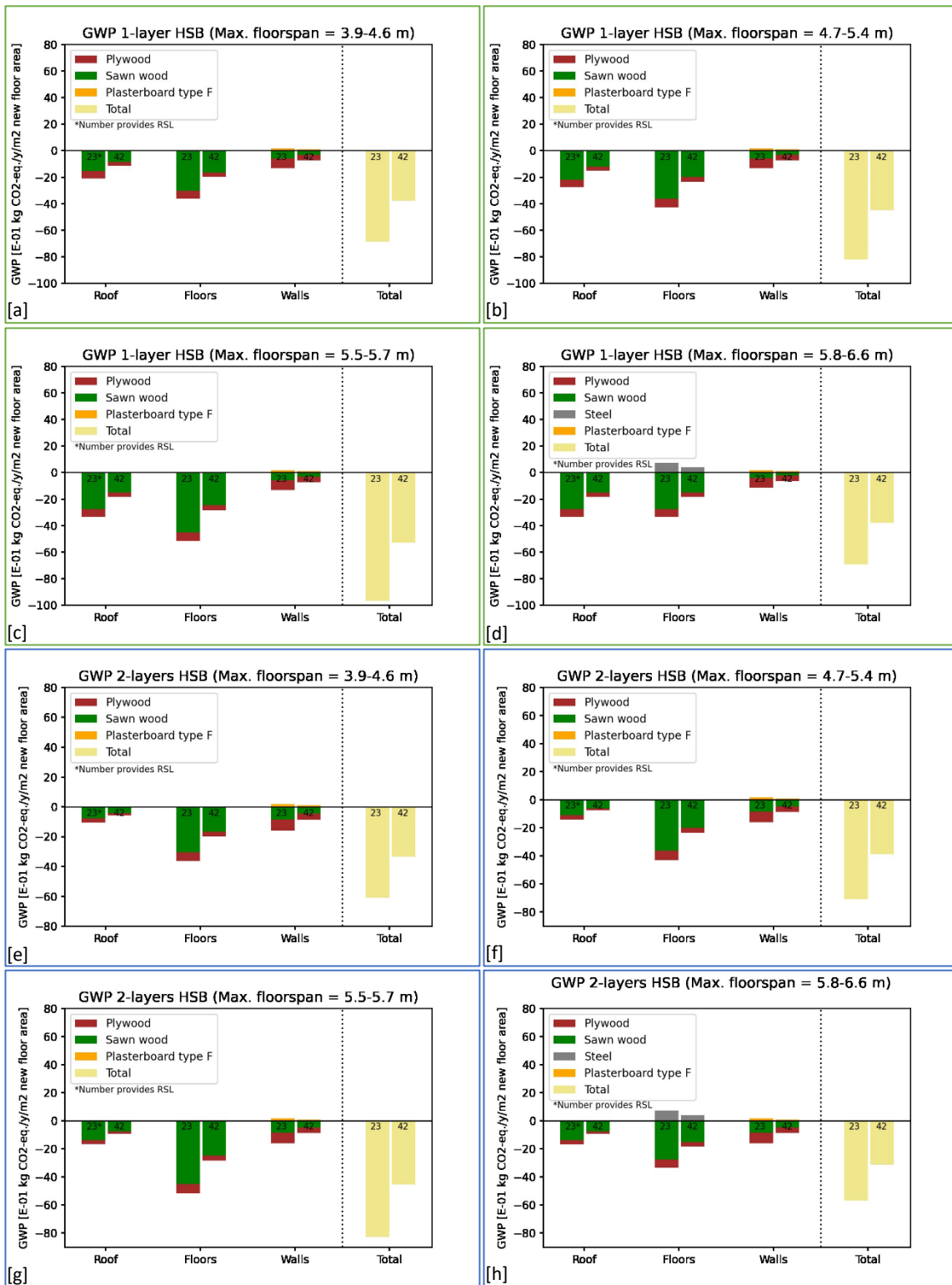


Figure D.31: Bar graphs for global warming potential HSB, separated per element (No additional room considered) (Self-produced).

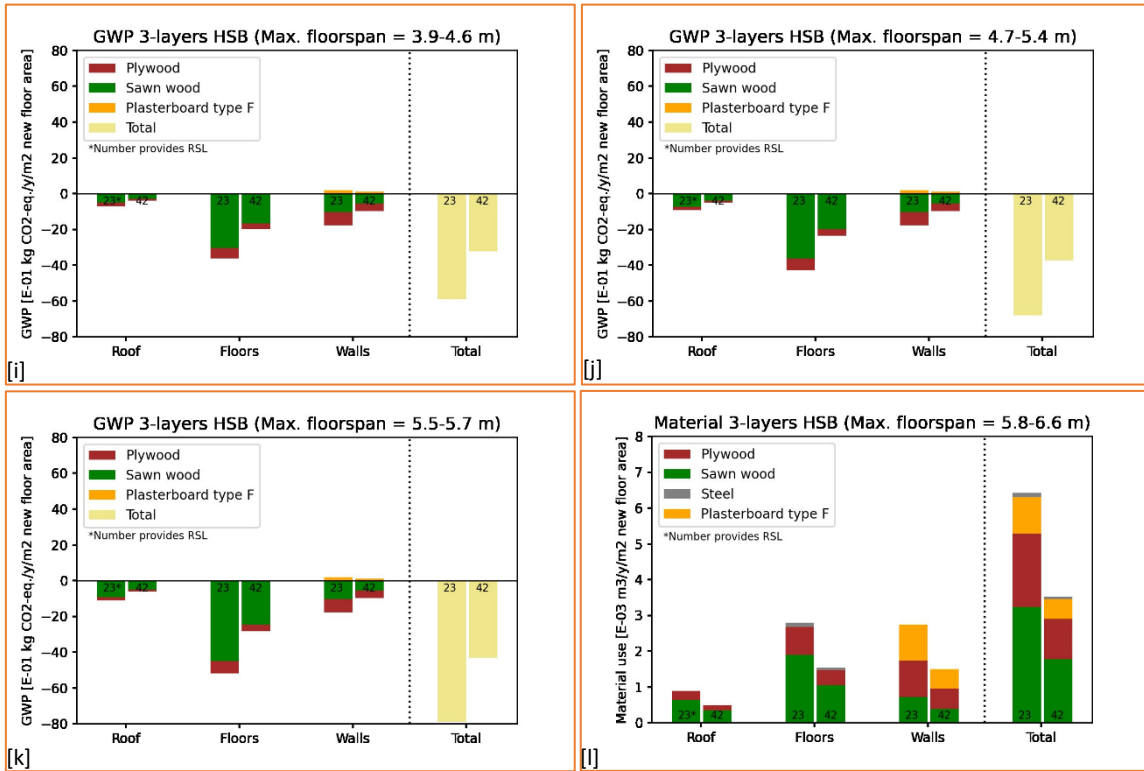


Figure D.31 (Continue): Bar graphs for global warming potential HSB, separated per element (No additional room considered) (Self-produced).

D.5.2 Material use

D.5.2.1 Line graphs of total

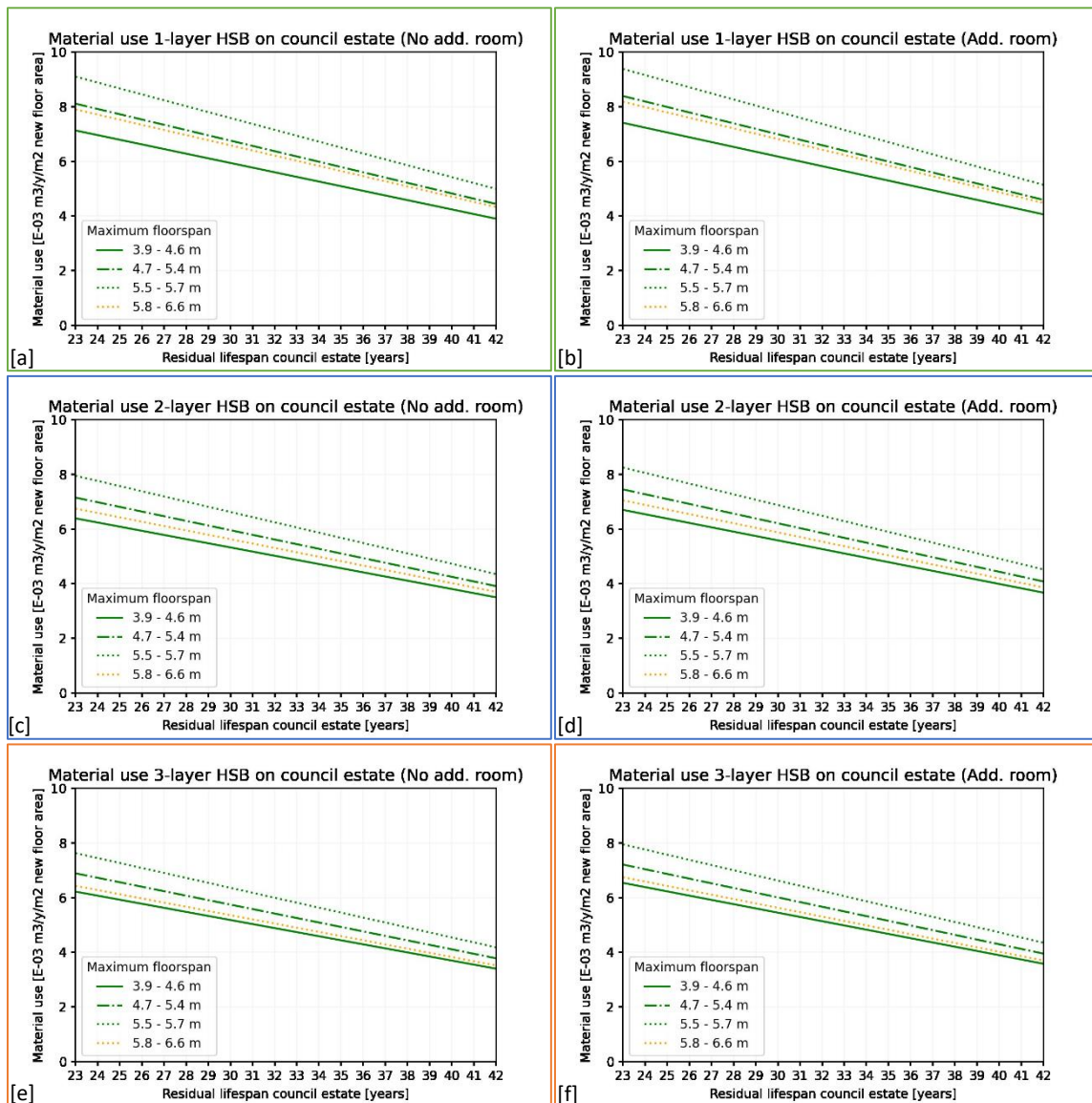


Figure D.32: Results total material use HSB top-up (Self-produced).

D.5.2.2 Bar graphs of material use separated per element

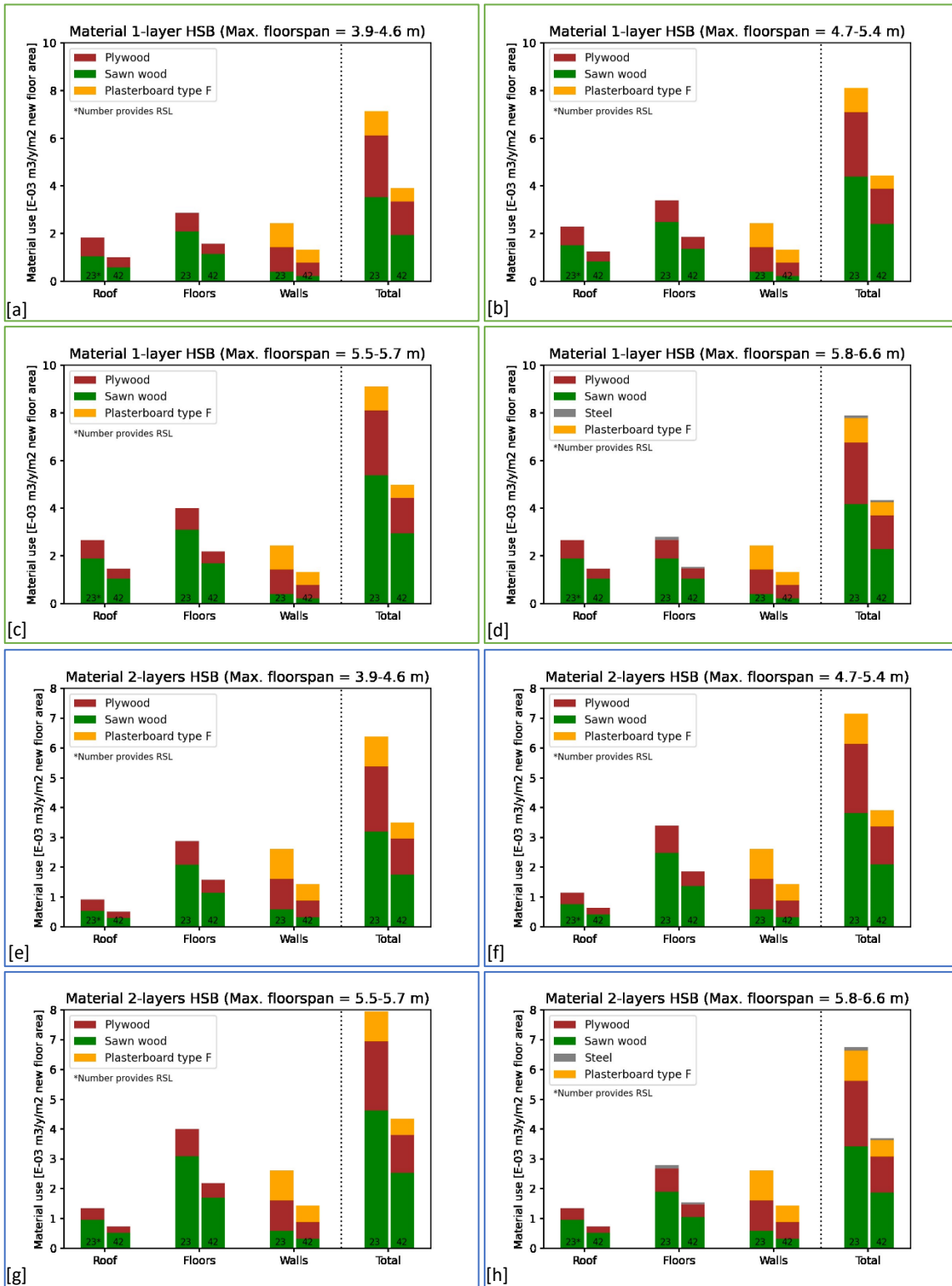


Figure D.33: Bar graphs for material use HSB, separated per element (No additional room considered) (Self-produced).

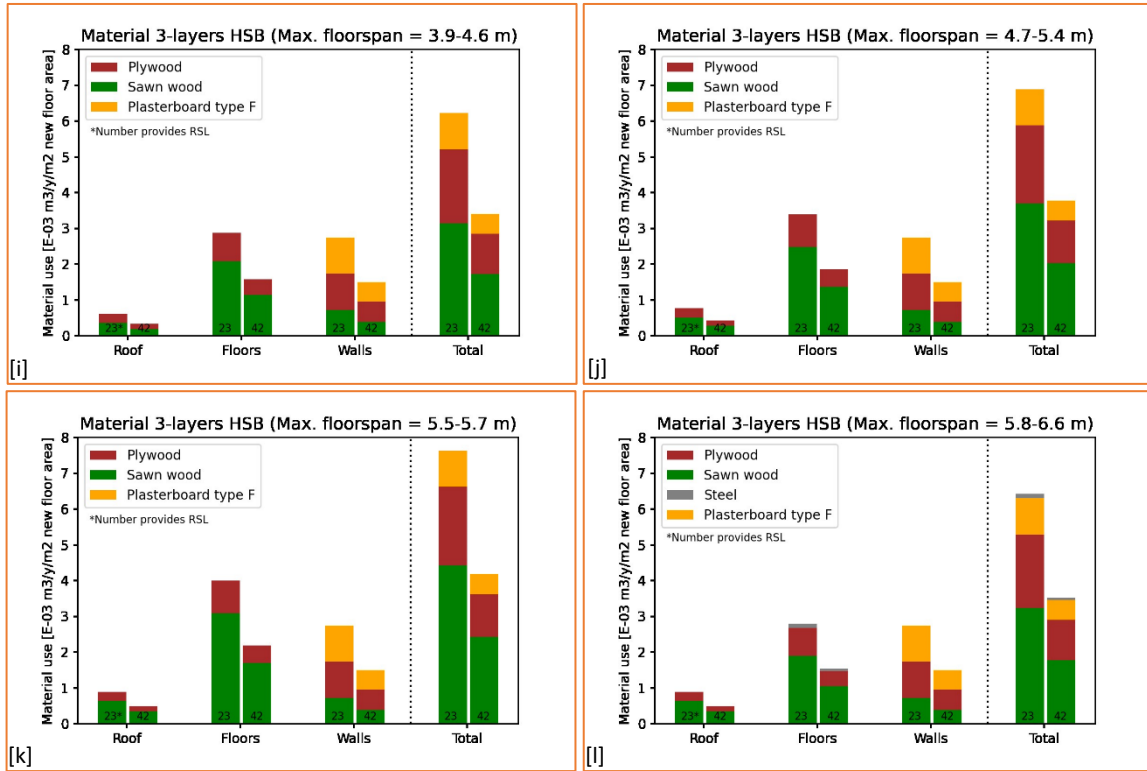


Figure D.33 (Continue): Bar graphs for material use HSB, separated per element (No additional room considered) (Self-produced).

D.5.3 Tables with results

Table D.45: Results for variants excluding an additional room.

No additional room									
RSL - 23 y				RSL - 42 y					
Material use [10 ⁻³ m ³ /y/m ² new floor area]				Material use [10 ⁻³ m ³ /y/m ² new floor area]					
Stories	1	2	3	Stories	1	2	3		
3.9 - 4.6	7,13	6,39	6,22	3.9 - 4.6	3,9	3,5	3,4		
4.7 - 5.4	8,11	7,15	6,89	4.7 - 5.4	4,44	3,91	3,78		
5.4 - 5.7	9,1	7,95	7,63	5.4 - 5.7	4,99	4,35	4,18		
5.7 - 6.6	7,9	6,75	6,43	5.7 - 6.6	4,33	3,7	3,52		
GWP [10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area]				GWP [10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area]					
Stories	1	2	3	Stories	1	2	3		
3.9 - 4.6	-68,8	-60,9	-59,2	3.9 - 4.6	-37,7	-33,3	-32,4		
4.7 - 5.4	-82,1	-70,9	-68,2	4.7 - 5.4	-45	-38,8	-37,3		
5.4 - 5.7	-96,7	-82,7	-79	5.4 - 5.7	-52,9	-45,3	-43,2		
5.7 - 6.6	-70,9	-56,9	-53,2	5.7 - 6.6	-38,8	-31,2	-29,1		

Table D.46: Results for variants including an additional room.

Additional room									
RSL - 23 y				RSL - 42 y					
Material use [10 ⁻³ m ³ /y/m ² new floor area]				Material use [10 ⁻³ m ³ /y/m ² new floor area]					
Stories	1	2	3	Stories	1	2	3		
3.9 - 4.6	7,41	6,7	6,54	3.9 - 4.6	4,06	3,67	3,58		
4.7 - 5.4	8,39	7,45	7,21	4.7 - 5.4	4,59	4,08	3,95		
5.5 - 5.7	9,38	8,25	7,95	5.5 - 5.7	5,14	4,52	4,35		
5.8 - 6.6	8,18	7,05	6,75	5.8 - 6.6	4,48	3,86	3,7		
GWP [10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area]				GWP [10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area]					
Stories	1	2	3	Stories	1	2	3		
3.9 - 4.6	-70	-62,4	-61	3.9 - 4.6	-38,3	-34,2	-33,4		
4.7 - 5.4	-83,4	-72,5	-70	4.7 - 5.4	-45,6	-39,7	-38,3		
5.4 - 5.7	-97,9	-84,2	-80,8	5.4 - 5.7	-53,6	-46,1	-44,2		
5.8 - 6.6	-72,1	-58,4	-55	5.8 - 6.6	-39,5	-32	-30,1		

E

Results

Appendix E will provide background information to chapter 9.

Content appendix E

- E.1** Material optimisation graph's: council estates incl. add. room
- E.2** Example: How to use the method?

E.1 Material optimisation graph's: council estates incl. add. room

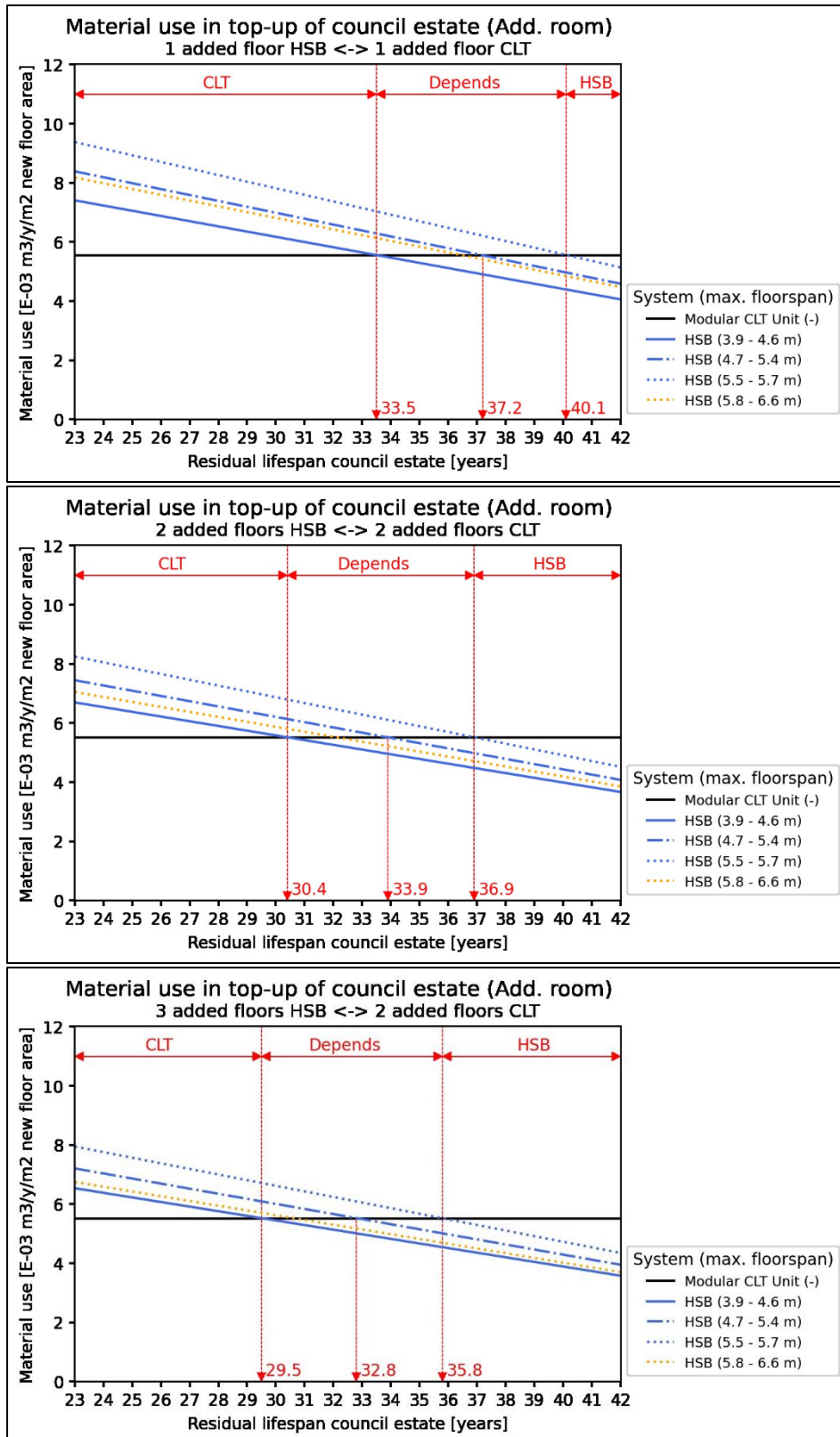
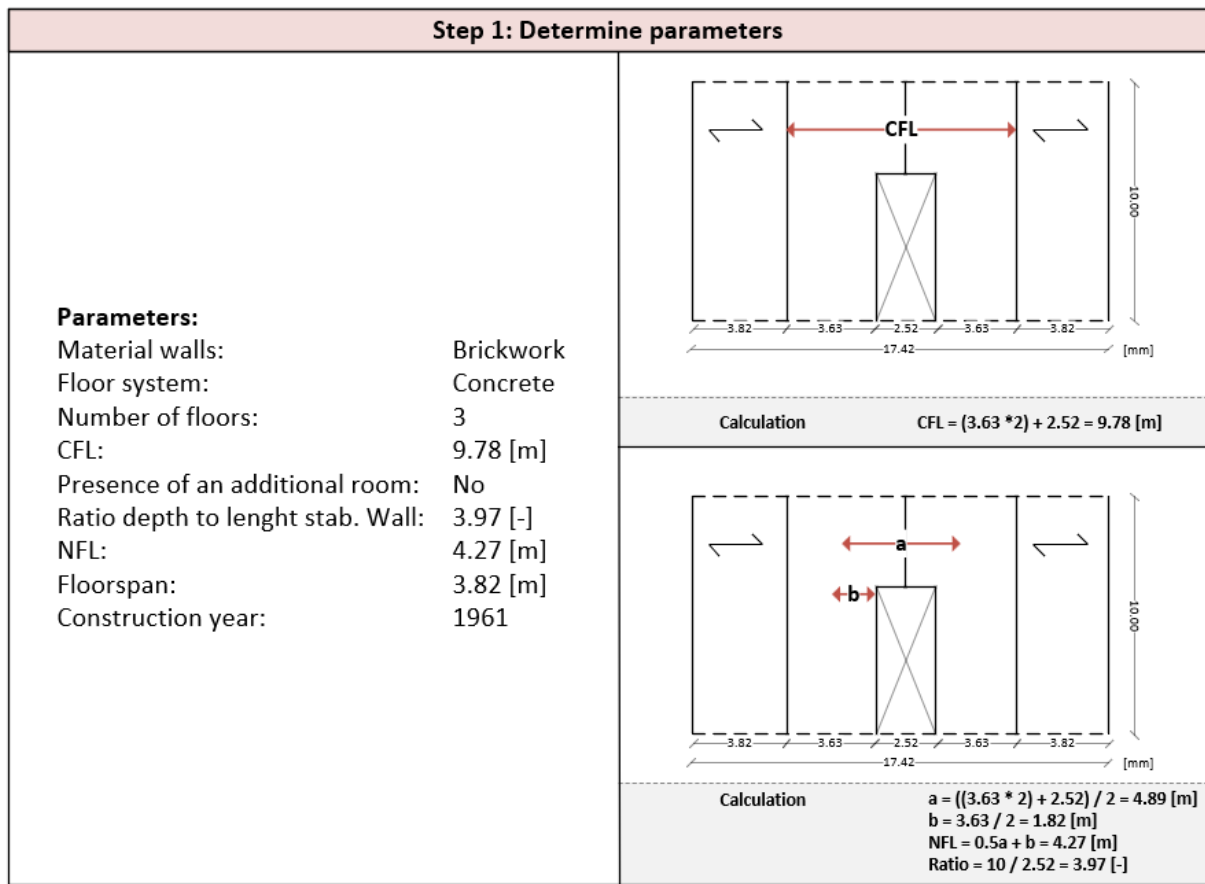


Figure E.1: Minimalization of material use for vertical extension on council estate (Additional room); [a] 1-layer HSB - 1-layer CLT; [b] 2 layers HSB - 2 layers CLT; [c] 3 layers HSB - 2 layers CLT (Self-produced).

E.2 Example: How to use the method?



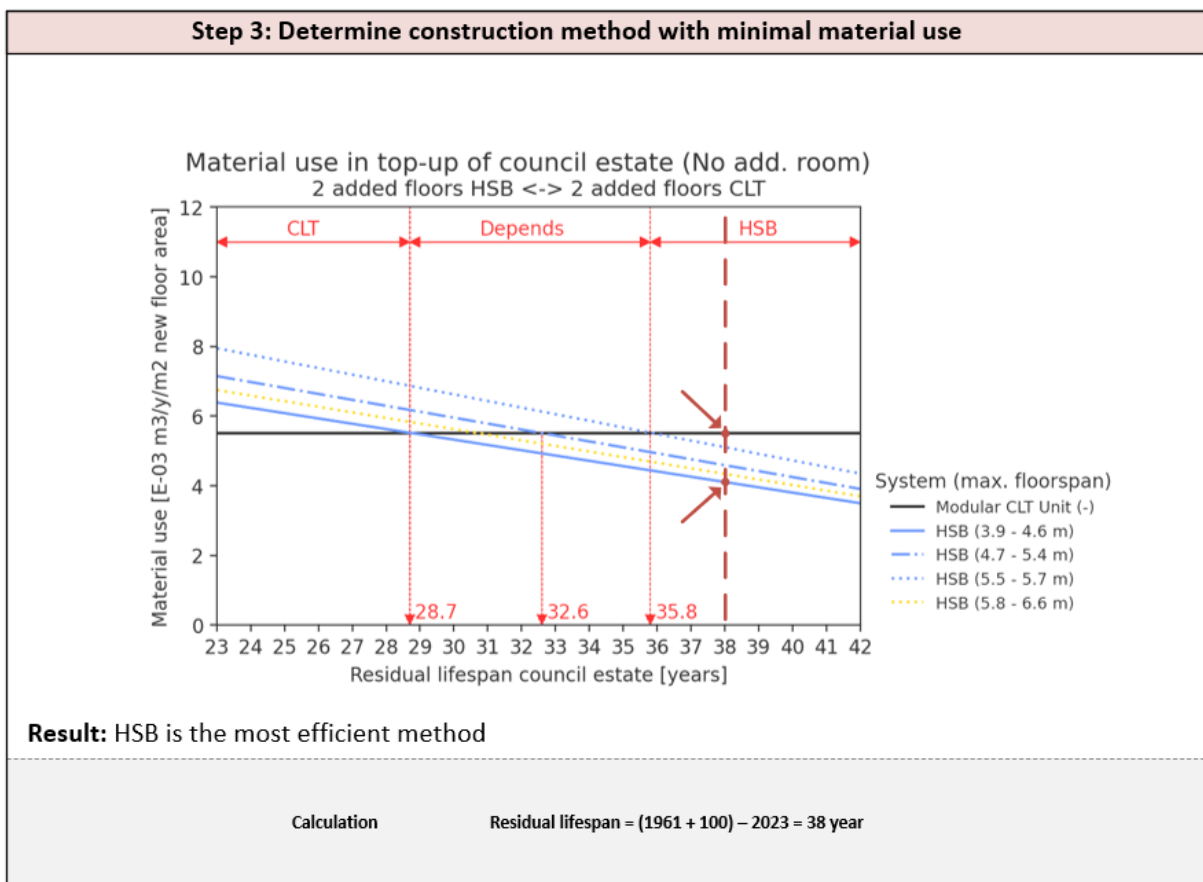
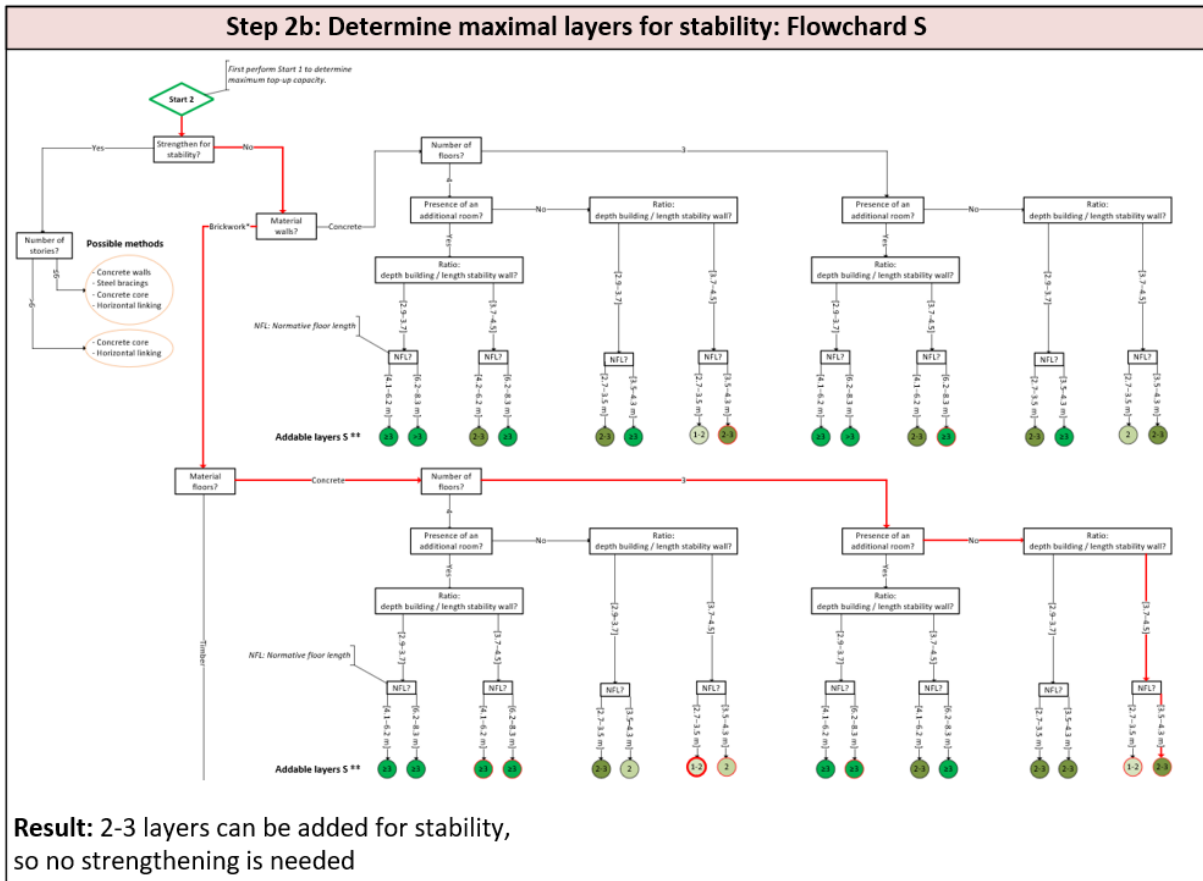


Figure E.3: Method explanation (Self-produced) (2).



Calculation sheets

Appendix X will provide the created excel sheets, including hand validations.

Content appendix X

- X.1** Vertical load study [[Section 5.5.2](#)]
- X.2** Stability Helena Mercierstraat [[Section 5.5.3](#)]
- X.3** Parameter study [[Section 5.6](#)]
- X.4** Overcapacity VL [[Section 6.4](#)]
- X.5** Overcapacity S [[Section 6.4](#)]
- X.6** Strengthening for stability [[Section 6.3.5](#)]
- X.7** Environmental impact HSB and CLT [[Section 7.2.2](#)]
- X.8** Environmental impact strengthening methods [[Section 7.2.4](#)]
- X.9** LCA-calculation concrete building [[Section 7.3](#)]
- X.10** Structural design of CLT Unit [[Appendix D.3](#)]
- X.11** Structural design of HSB top-up [[Appendix D.4](#)]

X.1 Vertical load study [Section 5.5.2]

X.1.1 Traditional brickwork construction

	A	B	C	D	E	F	G	H	I	J	K	
1	Brickwork variant with largest span with four stories, variant 10											
2	Parameters											
3	Dimensional Parameters											
4	Depth building	7,94	m									
5	Height building	14	m									
6	Wall height	2,8	m									
7	Opening factor (100 - opening percentage)	84	%	2x door 830 [mm] width								
8	Connected floor length	7,94	m									
9	Stories	4										
10												
11	Material properties											
12	Weight brickwork	1900	kg/m3	=	19	kn/m3	B12*10/1000					
13	Weight concrete	2400	kg/m3	=	24	kn/m3	B13*10/1000					
14	Gravel	1600	kg/m3	=	16	kn/m3	B14*10/1000					
15												
16	Safety factors (CC2)	6.10b		6.10a								
17	Variable load	1,5		1,5	Ψ ₀							
18	Permanent load	1,2		1,35								
19												
20												
21	Load definition											
22	Roof structure											
23	Permanent											
24	Gravel	Thickness	30	mm	=	0,48	kn/m2	(B25/1000)*E14				
25	Schewil isolation plate	Thickness	70	mm	=	0,2	kn/m2					
26	Reinforced concrete	Thickness	125	mm	=	3	kn/m2	(B29/1000)*E13				
27	Ceiling	Weight system ceiling	20	kg/m2	=	0,2	kn/m2	B31*10/1000				
28												
29												
30												
31												
32												
33												
34	Variable											
35	Snow											
36	s	0,7	kn/m2									
37	μ ₁	0,8										
38	C _s	1										
39	C _i	1										
40	Load				=	0,56	kn/m2	B36*B37*B38*B39	Ψ ₀	Ψ ₁	Ψ ₂	
41												
42	Wind (On roof)											
43	C _s C _d	1										
44	q _p (14)	0,78	kn/m2	Windarea 2, Urban								
45	C _{f,F}	-1,8	Normative									
46	C _{f,S}	-1,2										
47	C _{f,H}	-0,7										
48	C _f (+/-) 0,2											
49	Load				=	-1,404	kn/m2	B43*B44*B45		0	0,2	
50												
51	Imposed load				=	1	kn/m2			0	0	
52												
53												
54												
55	Story 1 till 4											
56	Permanent											
57	Wall brickwork	Thickness	220	mm	=	2,48	kn/m2	(B58/1000)*B6*(B7/100)*E12/(B8/2)				
58	Floor	Reinforced concrete floor	150	mm	=	3,6	kn/m2	(B60/1000)*E13				
59		Topping concrete	40	kg/m2	=	0,4	kn/m2	B61*10/1000				
60	Ceiling	Weight system ceiling	20	kg/m2	=	0,2	kn/m2	B63*10/1000				
61												
62												
63	Non loadbearing walls				=	0,5	kn/m2					
64												
65												
66												
67	Variable											
68	Imposed load	Load (Also considering stairs)	2	kn/m2	=	7,94	kn/m	B68*B8/2				
69												
70												
71	Walls on groundfloor											
72	Permanent											
73	Wall brickwork	Thickness	220	mm	=	9,83	kn/m	(B74/1000) * (B7/100) * B6 * E12				
74												
75												
76	Variable											
77	Not present											
78												
79	Total load on wall											
80	Roof structure	Permanent	15,40	H82	Formule B	Variable	3,97	I52	kn/m	Ψ ₀	Q*Ψ ₀	Formule H
81	4th floor_Volbelast		28,49	H65			7,94	E68	kn/m	0	0	D81*G81
82	3rd floor_Volbelast		28,49	H65			7,94	E68	kn/m	0,4	3,18	D82*G82
83	2nd floor_Volbelast		28,49	H65			7,94	E68	kn/m	0,4	3,18	D83*G83
84	1st floor		28,49	H65			7,94	E68	kn/m	0,4	3,18	D84*G84
85	Walls on groundfloor		9,83	E74			0	B77	kn/m	0,4	3,18	D85*G85
86		Total	139,1964	SOM(B81:B86)			35,73	SOM(D81:D86)	kn/m	0	0	D86*G86
87												
88												
89	U _{L5} 6.10a	206,97	kn/m		Formule B							
90	U _{L5} 6.10b	200,38	kn/m		(C18*B87) + (C17*H87)							
91	Load SLS	174,93	kn/m		(B18*B87) + (B17*(D82+D83)) + (B17 * (H81+H84+H85+H86))							
92					B87+D87							
93												
94												
95	Verification Stresses_ULS											
96	Wall thickness	220	mm		Formule B							
97	Normal stress	1119,97	kn/m2		B74							
98	Material factor (Cat. I)	1,7			B90/((B96/1000)*(B7/100))	=	1,12	N/mm2			Formule F	
99	Brickwork (fbc = 5, M5)	2,55	N/mm2								B97*1000/1000000	
100	Brickwork (fbc = 30, M20)	11,58	N/mm2								B100/B98	
101											B101/B98	
102												
103												
104												
105	Unity checks											
106	Unity check_most worst case	0,75										
107	Unity check_most optimal case	0,16										
108												
109	Experimentation other variants_performed in brickwork											
110	Biggest brickwork span (independent of floors)_Variant 8	Brickwork	9,78				0,86		0,19			
111	Largest span of all investigated variants_Variant 3	Alreiy system	10,28				0,9		0,2			
112	Normative variant_Variant 10	Brickwork	7,94				0,75		0,16			

Figure X.1: Traditional brickwork construction_Excel sheet

X.1.2 Non-traditional in-situ structure

	A	B	C	D	E	F	G	H	I	J	K
1	Concrete design in-situ (variant 13)										
2	Parameters										
3	Dimensional Parameters										
4	Depth building	14.1	m								
5	Height building	10.8	m								
6	Wall height	2.8	m								
7	Opening factor (100 - opening percentage)	84	%								
8	Connected floor length	9.9	m								
9	Stories	4									
11	Material properties										
12	Weight concrete	2400	kg/m ³	=		24	kN/m ³	Formule E			
13	Gravel	1600	kg/m ³	=		16	kN/m ³	B12*10/1000			
14								B13*10/1000			
15	Safety factors (CC2)										
16	Variable load	6.10b									
17	Permanent load	1.5				1.5	Ψ ₀				
18		1.2				1.35					
20	Load definition										
21	Roof structure										
22	Permanent										
23	Gravel	Thickness	30	mm	=		0.48	kN/m ²	(B24/1000)*E13		
25	Schewil isolation plate	Thickness	70	mm	=		0.2	kN/m ²			
27	Reinforced concrete	Thickness	125	mm	=		3	kN/m ²	(B28/1000)*E12		
29	Ceiling	Weight system ceiling	20	kg/m ²	=		0.2	kN/m ²	B30*10/1000		
31							3.88	kN/m ²	E24+E26+E28+E30		Formule I
32											E31*B8/2
33	Variable										
34	Snow	s	0.7	kN/m ²							
35		μ	0.8								
37		C _s	1								
38		C _i	1								
39		Load			=		0.56	kN/m ²	B35*B36*B37*B38	Ψ ₀	Ψ ₁
40										0	0.2
41	Wind (On roof)										
42		C _s C _d	1								
43		q _p (10.8)	0.7	kN/m ²	Windarea 2, Urban						
44		C _f F	-1.8	Normative							
45		C _s G	-1.2								
46		C _s H	-0.7								
47		C _i	0.2								
48		Load			=		-1.26	kN/m ²	B42*B43*B44	0	0.2
49										0	0
50		Imposed load			=		1	kN/m ²		0	0
51							1	kN/m ²	MAX(E39;E48;E50)	4.95	kN/m
52											Formule I
53											E51*B8/2
54	Story 1 till 4										
55	Permanent										
56	Wall concrete	Thickness	200	mm	=		2.28	kN/m ²	(B57/1000)*(B7/100)*B6*E12/(B8/2)		
58	Floor	Reinforced concrete floor	150	mm	=		3.6	kN/m ²	(B59/1000)*E12		
60		Topping concrete	40	kg/m ²	=		0.4	kN/m ²	B60*10/1000		
61	Ceiling	Weight system ceiling	20	kg/m ²	=		0.2	kN/m ²	B62*10/1000		
62	Non loadbearing walls				=		0.8	kN/m ²			
64							6.98	kN/m ²	E57+E59+E60+E62+E63		Formule I
65											E64*0.5*B8
66											164*B9
67	Variable										
68		Imposed load			=		9.9	kN/m	B68*B8/2		
69		Load (Also considering stairs)	2	kN/m ²	=		39.6	kN/m	E68*B9		
70											
71											
72	Walls on groundfloor										
73	Permanent										
74	Wall concrete	Thickness	200	mm	=		11.29	kN/m	(B75/1000)*(B7/100)*B6*E12		
76											
77	Variable										
78	Not present										
79											
80											
81	Total load on wall										
82		Permanent	Formule B	Variable	Formule D		Ψ ₀	Q*Ψ ₀	Formule H		
83	Roof structure	19.21	B31		4.95	B51	kN/m	0	0.00	B82*B83	
84	4th floor_Voibelaast	34.55	B64		9.9	B68	kN/m	0.4	3.96	B83*B83	
85	3rd floor_Voibelaast	34.55	B64		9.9	B68	kN/m	0.4	3.96	B84*B84	
86	2nd floor	34.55	B64		9.9	B68	kN/m	0.4	3.96	B85*B85	
87	1st floor	34.55	B64		9.9	B68	kN/m	0.4	3.96	B86*B86	
88	Walls on groundfloor	11.29	B75		0.878	B87	kN/m	0	0.00	B87*B87	
89		Total	168.71	SOM(B82:B87)	44.55	SOM(D82:D87)	kN/m		15.84	SOM(H82:H87)	kN/m
90											
91	ULS_6.10a	251.52	kN/m		Formule B						
92	ULS_6.10b	244.04	kN/m		(C17*B88)+(C16*B88)						
93	Load SLS	213.26	kN/m		(B17*B88) + (B16*(D83+D84))+B(16)*H82+H85+H86+H87)						
94					B88+D88						
95	Verification Stresses -ULS										
96	Wall thickness	200	mm		Formule B				Formule F		
97	Normal stress	1497.17	kN/m ²	=	B91/(B97/1000)*(B7/100)		1.50	N/mm ²	B98*1000/1000000		
99	Material factor (Cat. I)	1.5									
100							fd				
101	Low strength concrete (C12/15)	12	N/mm ²	=			8	N/mm ²	B101/B99		
102	Normal strength concrete (C30/37)	30	N/mm ²	=			20	N/mm ²	B102/B99		
103											
104											
105											
106	Unity checks										
107	Unity check_low strength concrete	0.19			F98/F101						
108	Unity check_high strength concrete	0.07			F98/F102						
109											
110											
111	Comparison										
112	Applied on variant 13 (Real building)	Actual system	Span	Lowest unity check	Normal unity check						
113	Largest span	Concrete, Non-trad	9.9	0.19	0.07						
		Airey system	10.28	0.19	0.08						

Figure X.2: Non-traditional; in-situ structure_Excel sheet

X.1.3 Non-traditional prefab structure

	A	B	C	D	E	F	G	H	I	J	K
1	Concrete design with prefab floor (Variant 13)										
2	Parameters										
3	Dimensional Parameters										
4	Depth building	14,1	m								
5	Height building	10,8	m								
6	Wall height	2,8	m								
7	Opening factor (100 - opening percentage)	84	%	2-3 x door 830 [mm] width							
8	Connected floor length	9,9	m								
9	Stories	4									
10	Material properties										
12	Weight concrete (assume muvi same weight)	2400	kg/m3	=	24	kn/m3	Formule E B12*10/1000				
13	Gravel	1600	kg/m3	=	16	kn/m3	B13*10/1000				
15	Safety factors (CC2)	6.10b		6.10a							
16	Variable load	1,5		1,5	ψ ₀						
17	Permanent load	1,2		1,35							
20	Load definition										
21	Roof structure (Use values from muvi-prefab system)										
22	Permanent										
23	Prefab system floor (From archive)	220	kg/m2	=	2,2	kn/m2	B23*10/1000				
24	Roof finishing	92	kg/m2	=	0,92	kn/m2	B24*10/1000				
25	Ceiling	20	kg/m2	=	0,2	kn/m2	B25*10/1000				
26					Total	3,32	kn/m2	SOM(E23:E25)	=	16,43	kn/m
27											Formule I E26*B8/2
28	Variable loading										
30	Snow										
31	s	0,7	kn/m2								
32	ψ ₀	0,8									
33	C _e	1									
34	C _i	1									
35	Load			=	0,56	kn/m2	B31*B32*B33*B34	ψ ₀	ψ ₁	ψ ₂	
36								0	0,2	0	
37	Wind (On roof)										
38	C _r C _d	1									
39	q _f (10,8)	0,7	kn/m2	Windarea 2, Urban							
40	C _{r,F}	-1,8	Normative								
41	C _{r,G}	-1,2									
42	C _{r,H}	-0,7									
43	C _{r,I}	(+/-) 0,2									
44	Load			=	-1,26	kn/m2	B38*B39*B40		0	0,2	0
45											
46	Imposed load			=	1	kn/m2			0	0	0
47					Normative load	1	kn/m2	MAX(E35,E44,E46)	=	4,95	kn/m
48											Formule I E47*B8/2
50	Story 1 till 4										
51	Permanent										
52	Walls concrete	Thickness	200	mm	=	2,28	kn/m2	(B53/1000) * B6 * (B7/100) * E12 / (B8/2)			
54	Floor										
55	Prefab system floor (From archive)	220	kg/m2								
56	Finishing	40	kg/m2								
57	Ceiling	20	kg/m2								
58	Total	280	kg/m2	=	2,8	kn/m2	B58*10/1000				
59	Non loadbearing walls										
60					Total	0,5	kn/m2				
61								5,58	kn/m2	E53+E58+E60	=
62											Formule I E61*B8/2
63	Variable										
64	Imposed load										
65	Load (Also considering stairs)	2	kn/m2	=	9,9	kn/m	B66*B8/2				
70	Walls on groundfloor										
71	Permanent										
72	Walls concrete	Thickness	200	mm	=	11,29	kn/m	(B73/1000)*(B7/100)*B6*E12			
73	Variable										
74	Not present										
79	Total load on wall										
80	Roof structure	Permanent	Formule B	Variable	Formule D		ψ ₀	Q*ψ ₀	Formule H		
81	4th floor_Volbelast	16,43	I26	4,95	I47	kn/m	0	0	D80*G80		
82	3rd floor_Volbelast	27,62	I61	9,9	E66	kn/m	0,4	3,96	D81*G81		
83	2nd floor_Volbelast	27,62	I61	9,9	E66	kn/m	0,4	3,96	D82*G82		
84	1st floor	27,62	I61	9,9	E66	kn/m	0,4	3,96	D83*G83		
85	Walls on groundfloor	11,29	E73	0	B76	kn/m	0	0	D85*G85		
86	Total	138,22	SOM(B80:B85)	44,55	SOM(D80:D85)	kn/m			15,84	SOM(H80:H85)	kn/m
89	ULS_6_10a	210,36	kn/m	Formule B (C17*B86) + (C16*H86)							
90	ULS_6_10b	207,45	kn/m	(B86*B17) + (B16*(D81+D82)) + (B16*(H80+H83+H84+H85))							
91	Load SLS	182,77	kn/m	B86+D86							
94	Verification Stresses_ ULS Concrete										
95	Wall thickness	200	mm	B73							Formule F
96	Normal stress	1252,14	kn/m2	B89/(B95/1000)*(B7/100)	=	1,25	N/mm2	B95*1000/1000000			
97	Material factor concrete	1,5									
99	Low strength concrete (C12/15)	12	N/mm2		=	8	N/mm2	B99/B97			
100	Normal strength concrete (C30/37)	30	N/mm2		=	20	N/mm2	B100/B97			
103	Unity checks										
104	Unity check_Low strength	0,16		F96/F99							
105	Unity check_Normal strength	0,06		F96/F100							
108	Comparison										
109	Applied on variant 13 (Real building)	Actual system	Span	Lowest unity check	Normal unity check						
110	Largest span	Concrete, Non-trad	9,9	0,16	0,06						
		Airey system	10,28	0,16	0,06						

Figure X.3: Non-traditional; prefab structure_Excel sheet

X.1.4 Non-traditional Muwi-system

	A	B	C	D	E	F	G	H	I	J	K
1	Muwi system applied on variant 3										
2	Parameters										
3	Dimensional Parameters										
4	Depth building	14,1	m								
5	Height building	10,8	m								
6	Wall height	2,8	m								
7	Opening factor (100 - opening percentage)	84	%								
8	Connected floor length	10,28	m								
9	Stories	4									
10											
11	Material properties										
12	Weight Muwi blocks	2000	kg/m ³	=	20	kN/m ³	Formule E B12*10/1000				
13											
14	Safety factors (CC2)										
15	Variable load	1,5		1,5	Ψ ₀						
16	Permanent load	1,2		1,35							
17											
18											
19	Load definition										
20	Roof structure (Use values from muwi-prefab system)										
21	Permanent										
22	Prefab system floor (From archive)	188	kg/m ²	=	1,88	kN/m ²	B22*10/1000				
23	Roof finishing	92	kg/m ²	=	0,92	kN/m ²	B23*10/1000				
24	Ceiling	22	kg/m ²	=	0,22	kN/m ²	B24*10/1000				
25											
26											
27											
28	Variable loading										
29	Snow										
30	s	0,7	kN/m ²								
31	μ _i	0,8									
32	C _e	1									
33	C _i	1									
34	Load			=	0,56	kN/m ²	B30*B31*B32*B33	Ψ ₀	Ψ ₁	Ψ ₂	
35								0	0,2	0	
36	Wind (On roof)										
37	C _s C _d	1									
38	q _p (10,8)	0,7	kN/m ²								
39	C _f	-1,8	Normative								
40	C _{t,G}	-1,2									
41	C _{t,H}	-0,7									
42	C _{t,I} (+/-) 0,2										
43	Load			=	-1,26	kN/m ²	B37*B38*B39		0	0,2	0
44											
45	Imposed load			=	1	kN/m ²			0	0	0
46											
47											
48											
49	Story 1 till 4										
50	Permanent										
51	Walls concrete										
52	Thickness	210	mm	=	1,92	kN/m ²	(B52/1000) * B6 * (B7/100) * E12 / (B8/2)				
53	Floor										
54	Prefab system floor (From archive)	188	kg/m ²								
55	Floor finishing	40	kg/m ²								
56	Ceiling	22	kg/m ²								
57	Total	250	kg/m ²	=	2,5	kN/m ²	B57*10/1000				
58											
59	Non loadbearing walls										
60											
61											
62											
63	Variable										
64	Imposed load										
65	Load (Also considering stairs)	2	kN/m ²	=	10,28	kN/m	B65*B8/2				
66	4 stories			=	41,12	kN/m	E65*B9				
67											
68											
69	Walls on groundfloor										
70	Permanent										
71	Walls concrete										
72	Thickness	210	mm	=	9,88	kN/m	(B72/1000)*(B7/100)*B6*E12				
73											
74	Variable										
75	Not present										
76											
77											
78	Total load on wall										
79	Roof structure	15,52	I25		5,14	I46		Ψ ₀	Q*Ψ ₀	Formule H	
80	4th floor_Voelbelast	25,30	I60		10,28	E65		0	0,00	D79*G79	
81	3th floor_Voelbelast	25,30	I60		10,28	E65		0,4	4,11	D80*G80	
82	2nd floor	25,30	I60		10,28	E65		0,4	4,11	D81*G81	
83	1st floor	25,30	I60		10,28	E65		0,4	4,11	D82*G82	
84	Walls on groundfloor	9,88	E72		0	B75		0	0,00	D83*G83	
85	Total	126,59	SOM(B79:B84)		46,26	SOM(D79:D84)			16,45	SOM(H79:H84)	kN/m
86											
87											
88	ULS_6.10a	195,57	kN/m								
89	ULS_6.10b	195,09	kN/m								
90	Load SLS	172,85	kN/m								
91											
92											
93	Verification Stresses - ULS Concrete										
94	Wall thickness	210	mm								
95	Normal stress	1108,70	kN/m ²					1,11	N/mm ²	B95*1000/1000000	
96	Material factor concrete	1,5									
97											
98	Hollow blocks	6	N/mm ²					4	N/mm ²	B98/B96	
99											
100											
101											
102	Unity checks										
103	Unity check_Muwi blocks	0,28									

Figure X.4: Non-traditional; Muwi-system_Excel sheet

X.1.5 Traditional structure variant 7

	A	B	C	D	E	F	G	H	I	J	K
1	Brickwork variant with smallest walls_Variant 7										
2	Parameters										
3	Dimensional Parameters										
4	Depth building	10,9	m								
5	Height building	14	m								
6	Wall height	2,9	m								
7	Opening factor (100 - opening percentage)	84	%								
8	Connected floor length	7,01	m								
9	Stories (till wall 110 mm)	3									
10	Roof angle	30									
11											
12	Material properties										
13	Weight brickwork	1900	kg/m ³	=	19	kN/m ³	B13*10/1000				
14	Weight concrete	2400	kg/m ³	=	24	kN/m ³	B14*10/1000				
15	Gravel	1600	kg/m ³	=	16	kN/m ³	B15*10/1000				
16											
17	Safety factors (CC2)										
18	Variable load	6.10b									
19	Permanent load	1,5			1,5		Ψ_0				
20		1,2			1,35						
21											
22	Load definition										
23	Roof structure										
24	Permanent										
25	Roof structure (roof tiles + wood)										
26	Weight (from their calculations)	56	kg/m ²	=	0,56	kN/m ²	B26*10/1000	=	1,96	kN/m	Formule I E26*B8/2
27											
28	Variable										
29	Snow load										
30	s	0,7	kN/m ²								
31	μ (angle =30)	0,8	-								
32	C _s	1									
33	C _t	1									
34	Load			=	0,65	kN/m ²	B30*B31*B32*B33/ COS(B10*2*PI()/360)	Ψ_0	Ψ_1	Ψ_2	
35								1	0,2	0	
36	Wind load										
37	C _s C _d	1									
38	q _f (14)	0,78	kN/m ²								
39	C _{f,F}	0,7	(only pressure given here)								
40	C _{f,G}	0,7	(only pressure given here)								
41	C _{f,H}	0,4	(only pressure given here)								
42	C _{f,I}	0	(only pressure given here)								
43	C _{f,J}	0	(only pressure given here)								
44	Load			=	0,55	kN/m ²	B37*B38*B39	0	0,2	0	
45											
46	Imposed load				1	kN/m ²					
47					1	kN/m ²	MAX(E34;E44;E46)	=	3,51	kN/m	Formule I E47*B8/2
48											
49	Attic structure										
50	Permanent										
51	Concrete floor										
52	Thickness	100	mm	=	2,4	kN/m ²	(B52/1000)*E14				
53	Topping	40	kg/m ²	=	0,4	kN/m ²	B53*10/1000				
54	Ceiling	20	kg/m ²	=	0,2	kN/m ²	B54*10/1000				
55											
56											
57	Variable										
58	Imposed load	1,75	kN/m ²	=	6,13	kN/m	B58*B8/2				
59											
60											
61	Floors										
62	Permanent										
63	Concrete floor										
64	Thickness	100	mm	=	2,4	kN/m ²	(B64/1000)*E14				
65	Topping	40	kg/m ²	=	0,4	kN/m ²	B65*10/1000				
66											
67	Interior walls										
68	Ceiling	20	kg/m ²	=	0,2	kN/m ²	B67*10/1000				
69											
70											
71	Variable										
72	Imposed load	2	kN/m ²	=	7,01	kN/m	B71*B8/2				
73											
74											
75	Brickwork walls_Permanent										
76	Thickness	110	mm	=	5,09	kN/m	(B75/1000)*(B7/100)*B6*E13				
77	Number of walls	4		=	20,36	kN/m	E75*B76				
78											
79	Total loads										
80	Roof structure	1,96	I26		3,51	I47	kN/m ²	Ψ_0	$Q^*\Psi_0$		
81	Attic	10,52	I55		6,13	E58	kN/m ²	0	0,00	D80*G80	
82	4th floor	12,27	I68		7,01	E71	kN/m ²	0,4	2,45	D81*G81	
83	3rd floor	12,27	I68		7,01	E71	kN/m ²	0,4	2,80	D82*G82	
84	Walls	20,36	E76		0,00		kN/m ²	0	0,00	D83*G83	
85								0	0,00	D84*G84	
86											
87											
88	Formule B										
89	ULS_6_10a	89,55	kN/m								
90	ULS_6_10b	93,56	kN/m								
91	Load SLS	81,04	kN/m								
92	Verification ULS										
93	Formule B										
94	Thickness wall	110	mm								
95	Stress	1,01	N/mm ²								
96	Material factor (Cat. I)	1,7	-								
97	Brickwork (fbc = 5; M5)	2,55	N/mm ²								
98	Brickwork (fbc = 30; M20)	11,58	N/mm ²								
99											
100	Unity checks										
101	Unity check_most worst case	0,68	-								
102	Unity check_most optimal case	0,15	-								

Figure X.5: Traditional structure; Variant 7_Excel sheet

X.1.6 Verification excel sheets appendix X.1

Order ✓
 Blad nr 1/2
 Deel Verificatie excel sheet vertical load study.
 Datum 26/04/2023



SHEET X.1.1
 ↳ principle hold for sheet x.1.1 1.11 x.1.5

- Parameters

depth building	=	7.94	[m]
Height building	=	14.00	[m]
Wall height	=	2.80	[m]
Opening factor	=	84	[%]
CFA d *(CFA = CFL)	=	7.94	[m]
Stories	=	4	[L-J]

Weight brickwork (γ_m) = 19 [kN/m³]
 Weight concrete (γ_c) = 25 [kN/m³]
 Gravel (γ_g) = 16 [kN/m³]
- Safety equations (NEN 1990)

eg 6.10a = $1.35 G + 1.5 \psi_{0,0} Q_1 + \sum 1.5 \psi_{0,i} Q_i$
 eg 6.10b = $1.2 G + 1.5 Q_1 + \sum 1.5 \psi_{0,i} Q_i$
- Loads (Normative defined exact in excel)

Roof

G_e	=	3.88 [kN/m ²]			
Q_e	=	1.00 [kN/m ²]			
			x 0.5 x 7.94	=	15.40 [kN/m]
			x 0.5 x 7.94	=	7.94 [kN/m]

Storage 1 till 4

G_R (incl. str. walls)	=	7.18 [kN/m ²]	x 0.5 x 7.94	=	28.50 [kN/m]
Q_R (assuming stairs)	=	2.00 [kN/m ²]	x 0.5 x 7.94	=	7.94 [kN/m]

Wall on ground floor

$G_e = \frac{\text{thickness}}{100} \cdot \frac{\text{opening fac}}{100} \cdot \text{wall height} \cdot \gamma_s = \frac{220}{100} \cdot \frac{84}{100} \cdot 2.8 \cdot 19 = 9.83 [kN/m]$
- Design value normal load SLS

1. Eq 6.10a

Roof:	$g_{d1} = (1.35 \cdot 15.40) + (1.5 \cdot 0.04 \cdot 1) = 21.79 [kN/m]$
4 th floor:	$g_{d1} = (1.35 \cdot 28.50) + (1.5 \cdot 0.4 \cdot 7.94) = 43.24 [kN/m]$
3 rd floor:	$g_{d1} = \dots = 43.24 [kN/m]$
2 nd floor:	$g_{d1} = \dots = 43.24 [kN/m]$
1 st floor:	$g_{d1} = \dots = 43.24 [kN/m]$
Walls on ground floor	= $1.35 \cdot 9.83 = 13.27 [kN/m]$
	209.0 [kN/m]

↳ 208.97 [kN/m] steel value.

Figure X.6: Verification excel sheet appendix X.1 (X.1.1) (1)

Order ✓
 Blad nr 2/2
 Deel Verificatie excel sheet vertical load study
 Datum 26/04/2023

VAN ROSSUM
 RAADGEVENDE
 INGENIEURS

SHEET X.1.1

↳ principle hold for sheet X.1.1 till X.1.5

2. Eq 6.10.b

Roof: $g_d = (1.2 \cdot 15.40) + (1.5 \cdot 1.0) = 18.40 \text{ [kN/m]}$
 4th floor: $g_d = (1.2 \cdot 18.50) + (1.5 \cdot 7.94) = 46.11 \text{ [kN/m]}$
 3rd floor: $g_d = (1.2 \cdot 20.50) + (1.5 \cdot 7.94) = 46.11 \text{ [kN/m]}$
 2nd floor: $g_d = (1.2 \cdot 20.50) + (1.5 \cdot 0.4 \cdot 7.94) = 30.96 \text{ [kN/m]}$
 1st floor: $g_d = (1.2 \cdot 20.50) + (1.5 \cdot 0.4 \cdot 7.94) = 30.96 \text{ [kN/m]}$
 Walls on groundfl.: $g_d = 1.2 \cdot 9.83 = 11.80 \text{ [kN/m]}$

$g_d = 200.42 \text{ [kN/m]}$ S
 $\Leftrightarrow 200.39 \text{ [kN/m]}$ (steel)

$g_{d, \text{ult}} = 200.00 \text{ [kN/m]}$

• Verification normal stress

$\sigma_N = \frac{g_d}{\text{wall thickness} \cdot \text{opening factor}} = \frac{200}{220 \cdot 0.24} = 1.12 \text{ [N/mm}^2\text{]} \text{ S}$
 $\Leftrightarrow 1.12 \text{ [N/mm}^2\text{]}$ (steel value)

Brickwork - HQ $\Rightarrow f_{td} = \frac{11.5}{1.7} = 6.8 \text{ [N/mm}^2\text{]} \text{ S}$
 ($f_{tk} = 30$; M20)
 ← material factor

Brickwork - LQ $\Rightarrow f_{td} = \frac{2.55}{1.7} = 1.5 \text{ [N/mm}^2\text{]} \text{ S}$
 ($f_{tk} = 5$; M5)
 ← material factor.

UC - worst case = $\frac{1.12}{1.5} = 0.75 \text{ [-]} \text{ S} \Leftrightarrow 0.75 \text{ (steel value)}$
 LQ brickwork

UC - optimal case = $\frac{1.12}{6.8} = 0.16 \text{ [-]} \text{ S} \Leftrightarrow 0.16 \text{ (steel value)}$
 HQ brickwork

Figure X.7: Verification excel sheet appendix X.1 (X.1.1) (2)

X.2 Stability Helena Mercierstraat [Section 5.5.3]

	A	B	C	D	E	F	G	H	I
1	Brickwork walls/ No additional room/ Concrete floors								
2	Parameters geometry								
3	Ratio: Depth / length stability wall		4,14	Formules B		Formules E			
4	Depth building		10,85	m					
5	Width unit building		18	m					
6	Storey height		2,8	m					
7	Number of floors_existing building		4						
8	Number of layers top-up		0						
9	Contributing width angle		30	degree		=			
10	Number of buildings between dilatation		2						
11	Height building_extended		14	m		(B8+B9+1)*B7			
12	Height existing building		14	m		(B8+1)*B7			
13	Wall parameters								
14	Wall thickness		220	mm					
15	Length stability wall		2,62	m		B5/B4			
16	Longitudinal direction								
17	NFL		3,13	m					
18	Floor length a		2,47	m					
19	Floor length b		1,855	m		B18-(0,5*B19)			
20	Transversal direction								
21	Half floor span (Wall next to dilatation)		1,96	m					
22	Structural material properties								
23	General								
24	Brickwork		19	kN/m3					
25	Concrete		24	kN/m3					
26	Concrete								
27	Concrete floor		4,7	kN/m2					
28	Concrete roof		3,88	kN/m2					
29	Timber floor								
30	Timber floor		0,5	kN/m2					
31	Timber roof		0,56	kN/m2					
32	Safety factors (NEN 8700)								
33	Permanent load		0,9						
34	Variable load		1,5						
35	Verification in longitudinal direction								
36	Compression caused by self-weight								
37		kN/m2	Pointload left/right [kN]	Formules C	Pointload middle [kN]	[kN/m]	Formules G		
38	1.Roof structure		3,88	23,65	B47*B20*H10	30,99	B47*B19*H10		
39	2.Floor load (story 1 till 4)								
40	First floor		4,7	14,32	B49*B20*G10	18,77	B49*B19*G10		
41	Other floors_per floor		4,7	28,64	B50*B20*H10	37,53	B50*B19*H10		
42	Total for floors			100,25	C49+((B8-1)*C50)	131,37	E49+((B8-1)*E50)		
43	3.Walk								
44	Stability wall							58,52	(B15/1000)*B7*(B8+1)*B28
45	Pointloads from contributing walls								
46	First floor			18,92	(B15/1000)*B7*B28*G10	18,92	(B15/1000)*B28*B7*G10		
47	Contributing width other walls			113,52	(B15/1000)*H10*B7*B28*(B8-1)	113,52	(B15/1000)*B7*(B8-1)*H10*B28		
48	Total overview of loads			256,34	C47+C51+C55+C56	294,80	E47+E51+E55+E56	58,52	G53
49	Compression force SLS		480,43	kN		Formules B			
50	Compression force ULS		432,38	kN		(0,5*E58)+C58+(0,5*G58*B16)			
51						B60*B40			
52	Tension caused by wind								
53	C _s C _d		1					Total stories	Height
54	q _p		0,78	kN/m2				3	11,2
55	C _{f,D}		0,8					4	14
56	C _{f,E}		0,8					5	16,8
57	C _{f,ext}		0,8			B67*B68		6	19,6
58	q _{wind_per wall}		3,39	kN/m		B65*B66*B69*B5/B11		7	22,4
59	M _{wind}		331,75	kNm		(1/2)*B70*B12*2			0,93
60	M _{wind_Ed}		497,62	kNm		B71*B41			
61	Tension on the sides		189,88	kN		B72/B16			
62	Tension?		0,44			B73/B61			
63	Compression maximum								
64	Maximum compression force		622,26	kN		B61+B73			
65	Area		355647,8	mm2		(G10*1000)*B15			
66	Compressive stress_UIS		1,75	N/mm2		B78*1000/B79			
67	Brickwork (Fbc=10; M10)								
68	Material factor		4,77	N/mm2					
69	Compressive capacity		2,81	N/mm2		B82/B83			
70	UC		0,62			B80/B84			
71	Verification in cross direction								
72	Compression caused by self-weight								
73		kN/m2	kN/m	Formules C					
74	Roof		3,88	7,60	B90*B23				
75	Floor load		4,7	36,85	B91*B8*B23				
76	Walls			58,52	(B15/1000)*B7*(B8+1)*B28				
77	Total			102,97	C90+C91+C92				
78	Compressive stress		0,47	N/mm2		Formules B			
79	Compressive stress_ED		0,42	N/mm2		C93/B15			
80						B95*B40			
81	Tensile stress (caused by wind)								
82	C _s C _d		1						
83	q _p (h)		0,78	kN/m2					
84	C _{f,D}		0,8						
85	C _{f,E}		0,7						
86	C _{f,ext}		1,5			B102-B103			
87	q _{wind}		3,44	kN/m		B100*B101*B104*B23*2*0,75			
88	M _{wind_Ed}		5,06E+08	Nmm		(1/2)*B105*(B12*1000)*2*B41			
89	W _{wall}		4,32E+09	mm3		(1/6)*B15*(B5*1000)*2			
90	Sigma M		0,12	N/mm2		B106/B107			
91	UC_cross direction		0,28			B108/B96			
92	Compression maximum								
93	Maximum compressive stress_UIS		0,54	N/mm2		B96+B108			
94	Brickwork (Fbc = 10; M5)								
95	Material factor		4,77	N/mm2					
96	Compressive capacity		2,81	N/mm2		B116/B117			
97	UC		0,19			B114/B118			

Figure X.8: Stability calculation Helena Mercierstraat_Excel sheet

X.3 Parameter study [Section 5.6]

For the vertical load study, the sheet of X.1.1 is used. The results are provided in figure X.9.

Bay width effect + Type of floor					
CFL [m]	Floor + roof	Weight floor	Weight roof	Worst	Optimal
6,7	Wood	1	0,56	0,38	0,08
6,7	Concrete	4,7	3,88	0,67	0,15
10,28	Wood	1	0,56	0,47	0,1
10,28	Concrete	4,7	3,88	0,9	0,2

Number of floors				
Number of stories	CFL [m]	Type floor	Worst	Optimal
3	6,7	Concrete	0,53	0,12
3	10,28	Concrete	0,71	0,16
4	6,7	Concrete	0,67	0,15
4	10,28	Concrete	0,9	0,2

Bay width effect -> including outer walls					
CFL [m]	Floor + roof	Weight floor	Weight roof	Worst	Optimal
7,84	Concrete	4,7	3,88	0,66	0,15
10,84	Concrete	4,7	3,88	0,82	0,18
13,2	Concrete	4,7	3,88	0,95	0,21

Opening percentage				
Percentage	Floor + roof	Worst	Optimal	
15	Concrete	0,74	0,16	
23	Concrete	0,79	0,17	
31	Concrete	0,86	0,19	

Figure X.9: Results vertical load study

For the stability study, the sheets: Calculations overcapacity S are used. The results are provided in figure X.10.

	A	B	C	D	E	F	G	H	
1	1. Effect of ratio	Ratio	NFL (No add. room)	Type floors	Height building [m]	UC_longitudinal direction			
2	Variant 1.1	2,87	3,12	Concrete	14 m	0,28			
3	Variant 1.2	2,87	3,12	Concrete	11,2 m	0,22			
4									
5	Variant 2.1	4,5	3,12	Concrete	14 m	0,48			
6	Variant 2.2	4,5	3,12	Concrete	11,2 m	0,38			
7									
8									
9	2. Material floors	Ratio	NFL (No add. Room)	Floors	Roof	Height	UC_longitudinal direction	UC_cross direction	
10	Variant 1.1	4,14	3,12	Concrete (4.70 kN/m2)	3.88 kN/m2	14	0,44	0,28	
11	Variant 1.2	4,14	3,12	Concrete (4.70 kN/m2)	3.88 kN/m2	11,2	0,34	0,2	
12									
13	Variant 2.1	4,14	3,12	Timber (0.5 kN/m2)*	0.56 kN/m2	14	0,67	0,42	
14	Variant 2.2	4,14	3,12	Timber (0.5 kN/m2)*	0.56 kN/m2	11,2	0,52	0,31	
15									
16	*Excluding non-loadbearing walls, so total weight equals 1.0 [kN/m2]								
17									
18	3. Effect of additional room	Additional room	Ratio	Structure	Height	NFL	UC_longitudinal		
19	Variant 1.1	No		4,14 Concrete	14	2,67	0,47		
20	Variant 1.2	No		4,14 Concrete	14	4,26	0,38		
21									
22	Variant 2.1	Yes		4,14 Concrete	14	4,07	0,35		
23	Variant 2.2	Yes		4,14 Concrete	14	8,25	0,24		

Figure X.10: Results stability study

X.4 Overcapacity VL [Section 6.4]

X.4.1 Calculation 4 storey, brickwork walls, concrete floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	
1	4 stories, Brickwork walls, Concrete floors															
2	Safety factors															
3		G	Q	psi-factor												
4	Eq. 6.1	1,2	1,3	0,4												
5	Eq. 6.2	1,15	1,3	0,4												
6	Load factor_Historical	1,4														
7																
8	Structural parameters															
9	CFL	10,28	m													
10	Number of floors	4	-													
11	Wall thickness	220	mm													
12	Floor height	2,8	m													
13																
14	Load definition															
15	General															
16	Weight concrete	24	kN/m3													
17	Weight brickwork	19	kN/m3													
18	Timber															
19	Timber floor	1	kN/m2													
20	Timber roof	0,56	kN/m2													
21	Concrete															
22	Concrete floor	4,7	kN/m2													
23	Concrete roof	3,88	kN/m2													
24	Foundation floor	5,1	kN/m2													
25																
26																
27	Historical situation N1055							Recalculating (NEN8700)								
28																
29	Roof structure	[%]	[kN/m2]	kN/m	Formule O							eq 6.1 [kN/m]	Formule L		eq 6.2 [kN/m]	Formule N
30	Permanent			3,88	19,94 C30*S859/2							23,93 B4*J31			22,93 B5*J31	
31	Variable	100	Not considered		0 B31*O											
32																
33	4th floor															
34	Permanent															
35	Floor load			4,7	24,16 C35*B9/2											
36	Wall				11,70 (B11/1000)*B17*B12											
37					35,86 SOM(D35:D36)											
38	Variable	100	2	10,28	C39*(B9/2)*B39/100											
39																
40	3th floor															
41	Permanent															
42	Floor load			4,7	24,16 C43*B9/2											
43	Wall				11,70 (B11/1000)*B12*B17											
44					35,86 SOM(D43:D44)											
45	Variable	90	2	9,25	C47*(B47/100)*(B9/2)											
46																
47	2nd floor															
48	Permanent															
49	Floor load			4,7	24,16 C51*B9/2											
50	Wall				11,70 (B11/1000)*B12*B17											
51					35,86 SOM(D51:D52)											
52	Variable	80	2	8,22	(B55/100)*C55*B9/2											
53																
54	1st floor															
55	Permanent															
56	Floor load			4,7	24,16 C59*B9/2											
57	Wall				11,70 (B11/1000)*B12*B17											
58					35,86 SOM(D59:D60)											
59	Variable	70	2	7,20	(B63/100)*C63*(B9/2)											
60																
61	Ground floor															
62	Permanent															
63	Floor load			5,1	26,21 C67*B9/2											
64	Wall				11,70 (B11/1000)*B12*B17											
65					37,92 SOM(D67:D68)											
66	Variable	60	3	9,25	(B71/100)*C71*B9/2											
67																
68	Load historical															
69	Total load_kar	245,51	kN/m		D30+D31+D37+D39+D45+D47+D53+D55+D61+D63+D69+D71											
70	Total load_ed	343,72	kN/m		B75*B6											
71																
72																
73																
74	Summary table															
75	CFA	6,70 - 8,49		15,75 - 14,83	kN/m2							kN/m2				
76		8,49 - 10,28		14,83 - 14,23	kN/m2							kN/m2				

Figure X.11: Overcapacity VL; 4-storey; Brickwork walls; Concrete floors_Excel sheet

X.4.2 Calculation 4 storey, brickwork walls, timber floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	
1	4 stories Brickwork walls Timber floors															
2	Safety factors															
3		G	Q	psi-factor												
4	Eq. 6.1	1,2	1,3	0,4			1,2G + 1,3*0,4*Q									
5	Eq. 6.2	1,15	1,3	0,4			1,15G + 1,3Q + 1,3*0,4*Q									
6	Load factor_Historical	1,4														
7	Structural parameters															
8	CFL	10,28	m													
9	Number of floors	4	-													
10	Wall thickness	220	mm	Brickwork 220 [mm]; Concrete 200 [mm]												
11	Floor height	2,8	m													
12	Load definition															
13	General															
14	Weight concrete	24	kN/m ³													
15	Weight brickwork	19	kN/m ³													
16	Timber															
17	Timber floor	1	kN/m ²													
18	Timber roof	0,56	kN/m ²													
19	Concrete															
20	Concrete floor	4,7	kN/m ²													
21	Concrete roof	3,88	kN/m ²													
22	Foundation floor	5,1	kN/m ²													
23	Historical situation N1055															
24	Recalculating (NEN8700)															
25	Roof structure	[%]	[kN/m ²]	kN/m	Formule D							eq 6.1 [kN/m]	Formule L		eq 6.2 [kN/m]	Formule N
26	Permanent		0,56	2,88	C30*S8S9/2							3,45	B4*J31		3,31	B5*J31
27	Variable	100	Not considered	0	B31*O											
28	4th floor															
29	Permanent															
30	Floor load		1	5,14	C35*B9/2							24,89	(B4*J35) + (C4*D4*J36)		31,06	(B5*J35) + (C5*J36)
31	Wall			11,70	(B11/1000)*B12*B12											
32				16,84	SOM(D35-D36)											
33	Variable	100	2	10,28	C39*(B9/2)*B39/100											
34	3th floor															
35	Permanent															
36	Floor load		1	5,14	C43*B9/2							24,89	(B4*J39) + (C4*D4*J40)		24,05	(B5*J39) + (C5*D5*J40)
37	Wall			11,70	(B11/1000)*B12*B17											
38				16,84	SOM(D43-D44)											
39	Variable	100	2	9,25	C47*(B47/100)*(B9/2)											
40	2nd floor															
41	Permanent															
42	Floor load		1	5,14	C49*B9/2							24,89	(B4*J43) + (C4*D4*J44)		24,05	(B5*J43) + (C5*D5*J44)
43	Wall			11,70	(B11/1000)*B12*B17											
44				16,84	SOM(D43-D44)											
45	Variable	90	2	9,25	C47*(B47/100)*(B9/2)											
46	1st floor															
47	Permanent															
48	Floor load		1	5,14	C51*B9/2							24,89	(B4*J47) + (C4*D4*J48)		24,05	(B5*J47) + (C5*D5*J48)
49	Wall			11,70	(B11/1000)*B12*B17											
50				16,84	SOM(D51-D52)											
51	Variable	80	2	8,22	(B55/100)*C55*B9/2											
52	Ground floor															
53	Permanent															
54	Floor load		5,1	26,21	C67*B9/2							50,85	(B4*J51) + (C4*D4*J52)		56,97	(B5*J51) + (C5*J52)
55	Wall			11,70	(B11/1000)*B12*B17											
56				37,92	SOM(D67-D68)											
57	Variable	60	3	9,25	(B71/100)*C71*B9/2											
58	Overcapacity structure															
59	Overcapacity wall			49,84	kN/m											
60	Overcapacity/m ²			9,70	kN/m ²											
61	Summary table															
62	CFA			6,70 - 8,49					11,22 - 10,30							
63				8,49 - 10,28					10,30 - 9,70							
64																
65	Load historical															
66	Total load_kar	152,38	kN/m		Formule											
67					D30+D31+D37+D39+D45+D47+D53+D55+D61+D63+D69+D71											
68	Total load_ed	213,33	kN/m		B75*B86											

Figure X.12: Overcapacity VL; 4-storey; Brickwork walls; Timber floors_Excel sheet

X.4.3 Calculation 4 storey, concrete walls, concrete floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	4 stories_Concrete walls_Concrete floors														
2	Safety factors														
3	G	Q	psi-factor												
4	Eq. 6.1	1,2	1,3	0,4			1.2G + 1.3*0.4*Q								
5	Eq. 6.2	1,15	1,3	0,4			1.15G + 1.3Q + 1.3*0.4*Q								
6	Load factor_Historic	1,4													
7	Structural parameters														
8	CFL	10,28	m												
9	Number of floors	4													
10	Wall thickness	200	mm				Brickwork 220 [mm]; Concrete 200 [mm]								
11	Floor height	2,8	m												
12	Load definition														
13	General														
14	Weight concrete	24	kN/m ³												
15	Weight brickwork	19	kN/m ³												
16	Timber														
17	Timber floor	1	kN/m ²												
18	Timber roof	0,56	kN/m ²												
19	Concrete														
20	Concrete floor	4,7	kN/m ²												
21	Concrete roof	3,88	kN/m ²												
22	Foundation floor	5,1	kN/m ²												
23	Historical situation N1055														
24	Roof structure	[%]	[kN/m ²]	kN/m	Formule D										
25	Permanent		3,88	19,94	C30*B59/2										
26	Variable	100	Not considered	0	B31*0										
27	Recalculating (NEN8700)														
28	Roof structure				Formule J	eq. 6.1 [kN/m]	Formule L		eq. 6.2 [kN/m]	Formule N					
29	Permanent					23,93	B4*J31		22,93	B5*J31					
30	Variable					0	I32								
31	4th floor														
32	4th floor_Full loaded														
33	Permanent					49,80	(B4*J35) + (C4*D4*J36)		54,93	(B5*J35) + (C5*J36)					
34	Floor load		4,7	24,16	C35*B9/2										
35	Wall			13,44	(B11/1000)*B12*B16				37,60	SOM(D35-D36)					
36	Variable	100	2	10,28	C39*(B9/2)*B39/100										
37	3th floor														
38	Permanent					49,80	(B4*J39) + (C4*D4*J40)		47,92	(B5*J39) + (C5*D5*J40)					
39	Floor load		4,7	24,16	C43*B9/2										
40	Wall			13,44	(B11/1000)*B12*B16				37,60	SOM(D43+D44)					
41	Variable	90	2	9,25	C47*(B47/100)*(B9/2)										
42	2nd floor														
43	Permanent					49,80	(B4*J43) + (C4*D4*J44)		47,92	(B5*J43) + (C5*D5*J44)					
44	Floor load		4,7	24,16	C43*B9/2										
45	Wall			13,44	(B11/1000)*B12*B16				37,60	SOM(D43+D44)					
46	Variable	80	2	8,22	(B55/100)*C55*B9/2										
47	1st floor														
48	Permanent					49,80	(B4*J47) + (C4*D4*J48)		47,92	(B5*J47) + (C5*D5*J48)					
49	Floor load		4,7	24,16	C59*B9/2										
50	Wall			13,44	(B11/1000)*B12*B16				37,60	SOM(D51+D52)					
51	Variable	70	2	7,20	(B63/100)*C63*(B9/2)										
52	Ground floor														
53	Permanent					276,04	SOM(L30-L52)		280,58	SOM(N30-N50)					
54	Floor load		5,1	26,21	C67*B9/2										
55	Wall			13,44	(B11/1000)*B12*B16				39,65	SOM(D67-D68)					
56	Variable	60	3	9,25	(B71/100)*C71*B9/2										
57	Overcapacity structure														
58	Overcapacity wall					75,29	kN/m	B76-I56							
59	Overcapacity/m2					14,65	kN/m ²	I62/(B9/2)							
60	Summary table														
61	CFA					6.70 - 8.49		16.39 - 15.34		kN/m ²					
62						8.49 - 10.28		15.34 - 14.65		kN/m ²					
63	Load historical														
64	Total load_kar		254,19	kN/m	Formule										
65						D30+D31+D37+D39+D45+D47+D53+D55+D61+D63+D69+D71									
66	Total load_ed		355,87	kN/m	B75*B6										

Figure X.13: Overcapacity VL; 4-storey; Concrete walls; Concrete floors_Excel sheet

X.4.4 Calculation 3 storey, brickwork walls, concrete floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	3 stories_Brickwork walls_Concrete floors														
2	Safety factors														
3		G	Q	psi-factor											
4	Eq. 6.1	1,2	1,3	0,4	1,2G + 1,3*0,4*Q										
5	Eq. 6.2	1,15	1,3	0,4	1,15G + 1,3Q + 1,3*0,4*Q										
6	Load factor_Historic	1,4													
7	Structural parameters														
9	CFL	10,28 m													
10	Number of floors	3													
11	Wall thickness	220 mm Brickwork 220 [mm]; Concrete 200 [mm]													
12	Floor height	2,8 m													
13	Load definition														
14	General														
16	Weight concrete	24 kN/m3													
17	Weight brickwork	19 kN/m3													
18	Timber														
19	Timber floor	1 kN/m2													
20	Timber roof	0,56 kN/m2													
21	Concrete														
22	Concrete floor	4,7 kN/m2													
23	Concrete roof	3,88 kN/m2													
24	Foundation floor	5,1 kN/m2													
25	Historical situation N1055														
26	Recalculating (NEN8700)														
27	Roof structure														
29	Roof structure	[%]	[kN/m2]	kN/m	Formule D										
30	Permanent		3,88	19,94	C30*5B59/2										
31	Variable	100	Not considered	0	B31*0										
32	3th floor														
34	Permanent														
35	Floor load		4,7	24,16	C35*B9/2										
36	Wall			11,70	(B11/1000)*B17*B12										
37				35,86	SOM(D35;D36)										
38	Variable	100	2	10,28	C39*(B9/2)*B39/100										
39	2nd floor														
41	Permanent														
42	Floor load		4,7	24,16	C43*B9/2										
43	Wall			11,70	(B11/1000)*B12*B17										
44				35,86	SOM(D43+D44)										
45	Variable	90	2	9,25	C47*(B47/100)*(B9/2)										
46	1st floor														
48	Permanent														
49	Floor load		4,7	24,16	C51*B9/2										
50	Wall			11,70	(B11/1000)*B12*B17										
51				35,86	SOM(D51+D52)										
52	Variable	80	2	8,22	(B55/100)*C55*B9/2										
53	Ground floor														
55	Permanent														
56	Floor load		5,1	26,21	C59*B9/2										
57	Wall			11,70	(B11/1000)*B12*B17										
58				37,92	SOM(D59;D60)										
59	Variable	70	3	10,79	(B63/100)*C63*B9/2										
60	Overcapacity structure														
61	Overcapacity wall	60,92 kN/m B68-I52													
62	Overcapacity/m2	11,85 kN/m2 I56/(B9/2)													
63	Summary table														
64	CFA	6,70 - 8,49 13,07 - 12,33 kN/m2													
65		8,49 - 10,28 12,33 - 11,85 kN/m2													
66	Load historical														
67	Total load_kar	204,00 kN/m D30+D31+D37+D39+D45+D47+D53+D55+D61+D63													
68	Total load_ed	285,60 kN/m B67*B6													

Figure X.14: Overcapacity VL; 3-storey; Brickwork walls; Concrete floors_Excel sheet

X.4.5 Calculation 3 storey, brickwork walls, timber floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	3 stories_Brickwork walls_Timber floors														
2	Safety factors														
3		G	Q	psi-factor											
4	Eq. 6.1	1,2	1,3	0,4	1,2G + 1,3*0,4*Q										
5	Eq. 6.2	1,15	1,3	0,4	1,15G + 1,3Q + 1,3*0,4*Q										
6	Load factor_Historic	1,4													
7	Structural parameters														
9	CFL	10,28	m												
10	Number of floors	3													
11	Wall thickness	220	mm Brickwork 220 [mm]; Concrete 200 [mm]												
12	Floor height	2,8	m												
13	Load definition														
14	General														
16	Weight concrete	24	kN/m3												
17	Weight brickwork	19	kN/m3												
18	Timber														
19	Timber floor	1	kN/m2												
20	Timber roof	0,56	kN/m2												
21	Concrete														
22	Concrete floor	4,7	kN/m2												
23	Concrete roof	3,88	kN/m2												
24	Foundation floor	5,1	kN/m2												
25															
27	Historical situation N1055							Recalculating (NEN8700)							
29	Roof structure	[%]	[kN/m2]	kN/m	Formule D			kN/m2	kN/m	Formule J	eq 6.1 [kN/m]	Formule L	eq 6.2 [kN/m]	Formule N	
30	Permanent		0,56	2,88	C30*S859/2			0,56	2,88	I31*B9/2	3,45	B4*J31	3,31	B5*J31	
31	Variable	100	Not considered	0	B31*0			0	0	I32					
32															
33	3th floor														
34	Permanent														
35	Floor load		1	5,14	C35*B9/2				16,84	D37	24,89	(B4*J35) + (C4*D4*J36)	31,06	(B5*J35) + (C5*J36)	
36	Wall			11,70	(B11/1000)*B17*B12				9,00	I36*B9/2					
37				16,84	SOM(D35;D36)										
38	Variable	100	2	10,28	C39*(B9/2)*B39/100										
39															
40	2nd floor														
41	Permanent														
42	Floor load		1	5,14	C43*B9/2				16,84	D53	24,89	(B4*J39) + (C4*D4*J40)	24,05	(B5*J39) + (C5*D5*J40)	
43	Wall			11,70	(B11/1000)*B12*B17				9,00	I44*B9/2					
44				16,84	SOM(D43+D44)										
45	Variable	90	2	9,25	C47*(B47/100)*(B9/2)										
46															
47	1st floor														
48	Permanent														
49	Floor load		1	5,14	C51*B9/2				16,84	D61	50,85	(B4*J43) + (C4*D4*J44)	56,97	(B5*J43) + (C5*D5*J44)	
50	Wall			11,70	(B11/1000)*B12*B17				9,00	I48*B9/2					
51				16,84	SOM(D51+D52)										
52	Variable	80	2	8,22	(B55/100)*C55*B9/2										
53															
54	Ground floor														
55	Permanent														
56	Floor load		5,1	26,21	C59*B9/2				37,92	D61	128,97	SOM(L30;L48)	139,44	SOM(N30;N46)	
57	Wall			11,70	(B11/1000)*B12*B17				8,25	I56*(B9/2)					
58				37,92	SOM(D59;D60)										
59	Variable	70	3	10,79	(B63/100)*C63*B9/2										
60															
61	Summary table														
62	CFA														
63	6.70 - 8.49			9.46 - 8.73			kN/m2								
64	8.49 - 10.28			8.73 - 8.25			kN/m2								
65															
66	Load historical														
67	Total load_kar	129,88			kN/m D30+D31+D37+D39+D45+D47+D53+D55+D61+D63										
68	Total load_ed	181,83			kN/m B67*B6										

Figure X.15: Overcapacity VL; 3-storey; Brickwork walls; Timber floors_Excel sheet

X.4.6 Calculation 3 storey, concrete walls, concrete floors

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	3 stories_Concrete walls_Concrete floors														
2	Safety factors														
3		G	Q	psi-factor											
4	Eq. 6.1	1,2	1,3	0,4	1,2G + 1,3*0,4*Q										
5	Eq. 6.2	1,15	1,3	0,4	1,15G + 1,3Q + 1,3*0,4*Q										
6	Load factor_Historic	1,4													
7	Structural parameters														
9	CFL	10,28	m												
10	Number of floors	3													
11	Wall thickness	200	mm Brickwork 220 [mm], Concrete 200 [mm]												
12	Floor height	2,8	m												
13	Load definition														
15	General														
16	Weight concrete	24	kN/m3												
17	Weight brickwork	19	kN/m3												
18	Timber														
19	Timber floor	1	kN/m2												
20	Timber roof	0,56	kN/m2												
21	Concrete														
22	Concrete floor	4,7	kN/m2												
23	Concrete roof	3,88	kN/m2												
24	Foundation floor	5,1	kN/m2												
25															
26	Historical situation N1055														
27	Recalculating (NEN8700)														
28															
29	Roof structure	[%]	[kN/m2]	kN/m	Formule D										
30	Permanent		3,88	19,94	C30*S859/2	kN/m2 kN/m Formule J eq 6.1 [kN/m] Formule L eq 6.2 [kN/m] Formule N									
31	Variable	100	Not considered	0	B31*0	3,88 19,94 I31*I89/2 23,93 B4*I31 22,93 B5*I31									
32															
33	3th floor														
34	Permanent														
35	Floor load		4,7	24,16	C35*B9/2	49,80 (B4*I35) + (C4*D4*I36) 54,93 (B5*I35) + (C5*I36)									
36	Wall			13,44	(B11/1000)*B16*B12	37,60 D37 9,00 I36*B9/2									
37				37,60	SOM(D35-D36)										
38	Variable	100	2	10,28	C39*(B9/2)*B39/100	49,80 (B4*I39) + (C4*D4*I40) 47,92 (B5*I39) + (C5*D5*I40)									
39	2nd floor														
40	Permanent														
41	Floor load		4,7	24,16	C43*B9/2	49,80 (B4*I43) + (C4*D4*I44) 47,92 (B5*I43) + (C5*D5*I44)									
42	Wall			13,44	(B11/1000)*B12*B16	37,60 D53 9,00 I44*B9/2									
43				37,60	SOM(D43+D44)										
44	Variable	90	2	9,25	C47*(B47/100)*(B9/2)	52,93 (B4*I47) + (C4*D4*I48) 58,97 (B5*I47) + (C5*I48)									
45	1st floor														
46	Permanent														
47	Floor load		4,7	24,16	C51*B9/2	52,93 (B4*I47) + (C4*D4*I48) 58,97 (B5*I47) + (C5*I48)									
48	Wall			13,44	(B11/1000)*B12*B16	37,60 D52 9,00 I48*B9/2									
49				37,60	SOM(D51+D52)										
50	Variable	80	2	8,22	(B55/100)*C55*B9/2	226,25 SOM(L30:L48) 232,66 SOM(N30:N46)									
51	Ground floor														
52	Permanent														
53	Floor load		5,1	26,21	C59*B9/2	232,66 kN/m MAX(L49:N49)									
54	Wall			13,44	(B11/1000)*B12*B16	62,66 kN/m B68-I52									
55				39,65	SOM(D59-D60)	12,19 kN/m2 I56/(B9/2)									
56	Variable	70	3	10,79	(B63/100)*C63*B9/2										
57	Summary table														
58	CFA														
59	6.70 - 8.49	13.59 - 12.74 kN/m2													
60	8.49 - 10.28	12.74 - 12.19 kN/m2													
61															
62															
63															
64															
65															
66	Load historical														
66	Total load_kar	210,94	kN/m Formule												
67	Total load_ed	295,32	kN/m D30+D31+D37+D39+D45+D47+D53+D55+D61+D63												
68			B67*B6												

Figure X.16: Overcapacity VL; 3-storey; Concrete walls; Concrete floors_Excel sheet

X.4.7 Preliminary used weight for HSB and CLT construction


	A	B	C	D	E	F	G	H	I	J	
1	Weight calculation per added floor										
2	Safety factors										
3		G	Q	Psi							
4	Eq. 6.1	1,35	1,5	0,4	1,35G + 1,5*0,4*Q						
5	Eq. 6.2	1,2	1,5	0,4	1,2G + 1,5Q + 1,5*0,4*Q						
6											
7	HSB					CLT					
8	Permanent load					Permanent load					
9	Walls	0,7	kN/m2			Walls	0,99	kN/m2			
10	Floors_excl. Finishing	0,45	kN/m2			Floors and roof_excl. Finishing	0,96	kN/m2			
11	Fermacell finishing	0,25	kN/m2			Fermacell finishing	0,25	kN/m2			
12	Ceiling	0,2	kN/m2			Ceiling	0,2	kN/m2			
13		1,603	kN/m2				2,403	kN/m2			
14	SOM(B9:B12)					SOM(H9:H12)					
15	Variable loading					Variable loading					
16	Imposed load	1,75	kN/m2			Imposed load	1,75	kN/m2			
17	Non-loadbearing wall	0,5	kN/m2			Non-loadbearing walls	0,5	kN/m2			
18		2,25	kN/m2				2,25	kN/m2			
19	SOM(B16:B17)					SOM(H16:H17)					
20	eq 6.1	3,51	kN/m2			eq 6.1	4,59	kN/m2			
21	eq 6.2	4,85	kN/m2			eq 6.2	5,81	kN/m2			
22											
23											
24	Addable floors per overcapacity										
25	1 < 9.70										
26	9.70 < 1-2 < 11.62										
27	11.62 < 2 < 14.55;										
28	14.55 < 2-3 < 17.43:										

Figure X.17: Weight estimation; HSB and CLT construction Excel sheet

X.4.8 Verification excel sheet: Overcapacity VL (X.4)

X.4.8.1: 4-storey flats

Order ✓
Blad nr 1/4
Deel Verification parameter study VL (1)
Datum 26/04/2023



Verification excel sheet vertical overcapacity


- 4 storey
- Brickwork walls
- concrete floors

SHEET X.4.1

→ principle valid for sheet X.4.1 till X.4.5

- Safety factors (NEN 8700)
- Eq 6.1: $\gamma_G = 1.2$, $\gamma_Q = 1.3$; $\psi_0 = 0.4$ (residential building)
- Eq 6.2: $\gamma_G = 1.15$; $\gamma_Q = 1.3$; $\psi_0 = 0.4$
- $\gamma_{load} = 1.4$ (chapter 6.3.6) → historical factor

- Structural parameters
- * (CFA = CFL)
- CFA = 10.28 [m]
- number of floors = 4 [-]
- wall thickness = 220 [mm]
- Floor height = 2.80 [m]



CFA = sum of spans on both sides

- Load definition

Weight concrete = 24 [kN/m ³]	= γ_c
Weight brickwork = 19 [kN/m ³]	= γ_b
timber floor = 9.1 [kN/m ²]	= γ_{FF}
timber roof = 0.56 [kN/m ²]	= γ_{rF}
Concrete floor = 4.7 [kN/m ²]	= γ_{cF}
Concrete roof = 3.28 [kN/m ²]	= γ_{cR}
Foundation floor = 5.1 [kN/m ²]	= γ_{FF}

- Calculation according to historical procedure (Nioss)
- Characteristic loads

1. Roof structure
 - 1a permanent = $\gamma_{cR} \cdot CFA/2 = 3.28 \cdot 10.28/2 = 19.54$ [kN/m]
 - 1b Variable = Not considered as floor above will be constructed
2. 4th floor
 - 2a permanent
 - Floor load = $\gamma_{cF} \cdot CFA/2 = 4.7 \cdot 10.28/2 = 24.16$ [kN/m]
 - Wall = $\frac{\gamma_{wall} \cdot \text{floor height}}{1000} \cdot \gamma_b = \frac{220}{1000} \cdot 2.8 \cdot 19 = 11.70$ [kN/m] + 25.36 [kN/m]

Figure X.18: Verification excel sheet (4 stories)_Overcapacity VL (1)

Order ✓
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→ 2b Variable

$$\rightarrow Q = 2 \text{ [rN/m}^2\text{]} \cdot 100\%]$$

$$\rightarrow \text{load} = \frac{Q}{2} \cdot \frac{CFA}{2} \cdot \frac{100\%}{100} = 2 \cdot (10.28/2) \cdot 1 = 10.28 \text{ [rN/m]} \quad \text{Ⓜ}$$

3. 3th floor
 → 3a permanent

$$\rightarrow \text{Floor load} = \gamma_{cf} \cdot CFA/2 = 4.7 \cdot 10.28/2 = 24.16 \text{ [rN/m]}$$

$$\rightarrow \text{wall} = \frac{t_{\text{wall}}}{1000} \cdot \text{Floor height} \cdot \gamma_b = \frac{220}{1000} \cdot 2.8 \cdot 19 = 11.70 \text{ [rN/m]} +$$

$$35.86 \text{ [rN/m]} \quad \text{Ⓜ}$$

 → 3b Variable

$$\rightarrow Q = 2 \text{ [rN/m}^2\text{]} \cdot 90\%]$$

$$\rightarrow \text{load} = \frac{Q}{2} \cdot \frac{CFA}{2} \cdot \frac{90\%}{100} = 2 \cdot \frac{10.28}{2} \cdot \frac{90}{100} = 9.25 \text{ [rN/m]} \quad \text{Ⓜ}$$

4. 2nd floor
 → 4a permanent

$$\rightarrow \text{Floor load} = \gamma_{cf} \cdot CFA/2 = 4.7 \cdot 10.28/2 = 24.16 \text{ [rN/m]}$$

$$\rightarrow \text{Wall} = \frac{t_{\text{wall}}}{1000} \cdot \text{Floor height} \cdot \gamma_b = \frac{220}{1000} \cdot 2.8 \cdot 19 = 11.70 \text{ [rN/m]} +$$

$$35.86 \text{ [rN/m]} \quad \text{Ⓜ}$$

 → 4b Variable

$$\rightarrow Q = 2 \text{ [rN/m}^2\text{]} \cdot 80\%]$$

$$\rightarrow \text{load} = \frac{Q}{2} \cdot \frac{CFA}{2} \cdot \frac{80\%}{100} = 2 \cdot \frac{10.28}{2} \cdot \frac{80}{100} = 8.22 \text{ [rN/m]} \quad \text{Ⓜ}$$

5. 1st floor
 → 5a permanent

$$\rightarrow \text{Floor load} = \gamma_{cf} \cdot CFA/2 = 4.7 \cdot 10.28/2 = 24.16 \text{ [rN/m]}$$

$$\rightarrow \text{Wall} = \frac{t_{\text{wall}}}{1000} \cdot \text{Floor height} \cdot \gamma_b = \frac{220}{1000} \cdot 2.8 \cdot 19 = 11.70 \text{ [rN/m]} +$$

$$35.86 \text{ [rN/m]} \quad \text{Ⓜ}$$

 → 5b Variable

$$\rightarrow Q = 2 \text{ [rN/m}^2\text{]} \cdot 70\%]$$

$$\rightarrow \text{load} = \frac{Q}{2} \cdot \frac{CFA}{2} \cdot \frac{70\%}{100} = 2 \cdot \frac{10.28}{2} \cdot \frac{70}{100} = 7.20 \text{ [rN/m]} \quad \text{Ⓜ}$$

6. Ground floor
 → 6a permanent

$$\rightarrow \text{Floor load} = \gamma_{cf} \cdot CFA/2 = 5.1 \cdot 10.28/2 = 26.21 \text{ [rN/m]}$$

$$\rightarrow \text{Wall} = \frac{t_{\text{wall}}}{1000} \cdot \text{Floor height} \cdot \gamma_b = \frac{220}{1000} \cdot 2.8 \cdot 19 = 11.70 \text{ [rN/m]} +$$

$$37.91 \text{ [rN/m]} \quad \text{Ⓜ}$$

Figure X.19: Verification excel sheet (4 stories)_Overcapacity VL (2)

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→ 6b Variable:
 $Q = 23 \text{ [kN/m}^2\text{]}$; 60%
 $\rightarrow \text{load} = Q \cdot \frac{AFA}{2} \cdot \frac{60}{100} = 3 \cdot \frac{10.28}{2} \cdot \frac{60}{100} = 9.25 \text{ [kN/m]} \text{ } \checkmark$

7. Total load historical

$$g_{tot_cur} = 19.94 + 35.86 + 10.28 + 35.86 + 9.25 + 35.86 + 8.22 + 35.86 + 7.20 + 37.91 + 9.25 = 245.45 \text{ [kN/m]} \text{ } \checkmark \leftrightarrow 245.51 \text{ [kN/m]} \text{ (sheet)}$$

$$g_{tot_ed} = 245.45 \cdot 1.4 = 343.63 \text{ [kN/m]} \text{ } \checkmark \leftrightarrow 343.72 \text{ [kN/m]} \text{ (sheet)}$$

\uparrow γ_{load}

• Recalculation (NEN 3700)

1. Roof structure

→ 1a permanent_{cur} = $\gamma_r \cdot (FA/2) = 3.88 \cdot 10.28/2 = 19.94 \text{ [kN/m]}$

→ 1b Variable = Not considered.

design values:

$$g_{y 6.1} = 1.2 \cdot 6 = 1.2 \cdot 19.94 = 23.93 \text{ [kN/m]} \text{ } \checkmark$$

$$g_{y 6.2} = 1.15 \cdot 6 = 1.15 \cdot 19.94 = 22.93 \text{ [kN/m]} \text{ } \checkmark$$

2. 4th floor

→ 2a permanent = 35.86 [kN/m] (historical calc.)

→ 2b Variable: $Q = 1.75 \text{ [kN/m}^2\text{]}$

→ load_{cur-Q} = $Q \cdot (FA/2) = 1.75 \cdot 10.28/2 = 9.00 \text{ [kN/m]} \text{ } \checkmark$

design values

Full loaded → $g_{y 6.1} = 1.2G + 1.3 \cdot \psi \cdot Q = (1.2 \cdot 35.86) + (1.3 \cdot 9.00) = 47.71 \text{ [kN/m]} \text{ } \checkmark$
 $g_{y 6.2} = 1.15G + 1.3Q = (1.15 \cdot 35.86) + (1.3 \cdot 9.00) = 52.94 \text{ [kN/m]} \text{ } \checkmark$

3. 1st, 1st, 3rd floor

→ 3a permanent = 35.86 [kN/m] (historical calc.)

→ 3b Variable: $Q = 1.75 \text{ [kN/m}^2\text{]}$

→ load_{cur-Q} = $Q \cdot (FA/2) = 1.75 \cdot 10.28/2 = 9.00 \text{ [kN/m]} \text{ } \checkmark$

Figure X.20: Verification excel sheet (4 stories)_Overcapacity VL (3)

Order /

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design values

$$\text{Eg } 6.1 = 1.2 G + 1.3 \Psi Q = (1.2 \cdot 35.86) + (1.3 \cdot 0.4 g) = 47.71 \text{ [kN/m]} \quad \text{S}$$

$$\text{Eg } 6.2 = 1.15 G + 1.3 \Psi Q = (1.15 \cdot 35.86) + (1.3 \cdot 0.4 g) = 45.92 \text{ [kN/m]} \quad \text{S}$$

4. Ground floor

$$\rightarrow 4a \text{ permanent} = 37.91 \text{ [kN/m]} \text{ (historical calc)}$$

$$\rightarrow 4b \text{ Variable: } Q = 2 \text{ [kN/m}^2\text{]}$$

$$\rightarrow \text{load, car-} Q = Q \cdot CFA/2 = 2 \cdot 10.28/2 = 10.28 \text{ [kN/m]}$$

design values

$$\text{Eg } 6.1 = 1.2 G + 1.3 \Psi Q = (1.2 \cdot 37.91) + (1.3 \cdot 0.4 g) = 50.84 \text{ [kN/m]} \quad \text{S}$$

$$\text{Full load} \rightarrow \text{Eg } 6.2 = 1.15 G + 1.3 \Psi Q = (1.15 \cdot 37.91) + (1.3 \cdot 10.28) = 56.96 \text{ [kN/m]} \quad \text{S}$$

5. Total load Ed

$$\text{Eg } 6.1 = 23.93 + 47.71 + (3 \cdot 47.71) + 50.84 = 265.61 \text{ [kN/m]} \quad \text{S}$$

$\Leftrightarrow 265.63 \text{ [kN/m]}$
(steel value)

$$\text{Eg } 6.2 = 22.93 + 52.94 + (3 \cdot 45.92) + 56.96 = 270.59 \text{ [kN/m]} \quad \text{S}$$

$\Leftrightarrow 270.60 \text{ [kN/m]}$
(steel value)

$$\text{Eurocode design value} = 270.59 \text{ [kN/m]}.$$

• Overcapacity structure

$$\text{Overcapacity-} m_2 = \frac{(\text{Historical} - \text{Eurocode})}{0.5 \text{ CFA}} = \frac{(343.65 - 270.59)}{0.5 \cdot 10.28}$$


$$14.22 \text{ [kN/m}^2\text{]} \quad \text{S}$$

$\Leftrightarrow 14.23 \text{ [kN/m}^2\text{]}$
(steel value)

Figure X.21: Verification excel sheet (4 stories)_Overcapacity VL (4)

X.4.8.2: 3-storey flats

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Verification excel sheet vertical overcapacity (2)

→ 3 storey
 → brickwork walls
 → timber floors

SHEET X.4.5


→ principle valid for sheet X.4.4 till X.4.6

• Safety factors (NEN 990)

- Eq 6.1: $\gamma_G = 1.2$; $\gamma_Q = 1.3$; $\psi_0 = 0.4$ (residential building)
 - Eq 6.2: $\gamma_G = 1.15$; $\gamma_Q = 1.3$; $\psi_0 = 0.4$
 - $\gamma_{load} = 1.4$ [-] (chapter 6.3.6) \Rightarrow historical factor

• Structural parameters

* (CFA = CFL)
 CFA = 10.20 [m]
 number of floors = 3 [-]
 wall thickness = 220 [mm]
 floor height = 2.80 [m]



(CFA = sum of spans on both sides.)

• Load definition

Weight concrete (γ_c) = 24 [kN/m³]
 Weight brickwork (γ_b) = 19 [kN/m³]
 Timber floor (γ_{tr}) = 1.0 [kN/m²]
 Timber roof (γ_{tr}) = 0.56 [kN/m²]
 concrete floor (γ_{cF}) = 4.7 [kN/m²]
 concrete roof (γ_{cR}) = 3.88 [kN/m²]
 Foundation floor (γ_{FF}) = 5.1 [kN/m²]

• Calculation according to historical procedure (NEN 990) (Nuss)

Characteristic loads

1. Roof structure
 → 1a permanent = $\gamma_{tr} \cdot (CFA/2) = 0.56 \cdot 10.20/2 + 2.88$ [kN/m] ≈ 8
 → 1b Variable = Not considered for roof

2. 4/3 Floor
 → 2a permanent
 → Floor load = $\gamma_{cF} \cdot (CFA/2) = 1.0 \cdot 10.20/2 = 5.14$ [kN/m]
 → Wall = $\gamma_b \cdot \frac{wall}{1000} \cdot floor\ height = 19 \cdot \frac{220}{1000} \cdot 2.8 = 11.70$ [kN/m] \rightarrow
16.84 [kN/m] ≈ 8

Figure X.22: Verification excel sheet (3 stories)_Overcapacity VL (1)

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→ 2b Variable
 → $Q = 2 \text{ [kN/m}^2\text{]} ; 100\%$
 → $\text{load} = Q \cdot \frac{CFA}{2} \cdot \frac{100\%}{100} = 2 \cdot \frac{10,20^2}{2} \cdot \frac{100}{100} = 10,20 \text{ [kN/m]} \text{ Ⓢ}$

3. 2nd Floor
 → 3a permanent
 → Floorload = $\gamma_{17} \cdot CFA/2 = 1 \cdot 10,20^2/2 = 5,10 \text{ [kN/m]}$
 → Wall = $\gamma_b \cdot \frac{t_{\text{wall}}}{1000} \cdot \text{Floor height} \cdot 19 \cdot \frac{220}{100} \cdot 2,8 = 11,70 \text{ [kN/m]}$
 +
 16,80 $\text{[kN/m]} \text{ Ⓢ}$

→ 3b Variable
 → $Q = 2 \text{ [kN/m}^2\text{]} ; 90\%$
 → $\text{load} = Q \cdot \frac{CFA}{2} \cdot \frac{90\%}{100} = 2 \cdot \frac{10,20^2}{2} \cdot 0,9 = 9,25 \text{ [kN/m]} \text{ Ⓢ}$

4. 1st Floor
 → 4a permanent
 → Floorload = $\gamma_{17} \cdot CFA/2 = 1 \cdot 10,20^2/2 = 5,10 \text{ [kN/m]}$
 → Wall = $\gamma_b \cdot \frac{t_{\text{wall}}}{1000} \cdot \text{storey height} = 19 \cdot \frac{220}{1000} \cdot 2,8 = 11,70 \text{ [kN/m]}$
 +
 16,80 $\text{[kN/m]} \text{ Ⓢ}$

→ 4b Variable
 → $Q = 2 \text{ [kN/m}^2\text{]} ; 80\%$
 → $\text{load} = Q \cdot \frac{CFA}{2} \cdot \frac{80\%}{100} = 2 \cdot \frac{10,20^2}{2} \cdot \frac{80}{100} = 8,22 \text{ [kN/m]} \text{ Ⓢ}$

5. ground floor
 → 5a permanent
 → Floorload = $\gamma_{15} \cdot CFA/2 = 5,1 \cdot 10,20^2/2 = 26,21 \text{ [kN/m]}$
 → Wall = $\gamma_b \cdot \frac{t_{\text{wall}}}{1000} \cdot \text{storey height} = 19 \cdot \frac{220}{1000} \cdot 2,8 = 11,70 \text{ [kN/m]}$
 +
 37,91 $\text{[kN/m]} \text{ Ⓢ}$

→ 5b Variable
 → $Q = 3 \text{ [kN/m}^2\text{]} ; 70\%$
 → $\text{load} = Q \cdot \frac{CFA}{2} \cdot \frac{70}{100} = 3 \cdot \frac{10,20^2}{2} \cdot \frac{70}{100} = 10,75 \text{ [kN/m]} \text{ Ⓢ}$

6. Total load historical
 $q_{\text{tot. kar}} = 2,82 + 16,80 + 10,20 + 16,80 + 9,25 + 16,80 + 8,22 + 37,91 + 10,75 = 129,85 \text{ Ⓢ}$
 $q_{\text{tot. des. ja}} = 129,85 \cdot 1,4 = 181,79 \text{ [kN/m]} \text{ Ⓢ} \leftrightarrow 181,82$
 (steel value)

Figure X.23: Verification excel sheet (3 stories)_Overcapacity VL (2)

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- o Recalculation (NEN 8700)
- Roof structure
 - 1a permanent = 2.28 kN/m^2 (historical rule)
 - 1b variable = not considered

design values

eg 6.1: $1.2 G = 1.2 \cdot 2.28 = 3.46 \text{ kN/m}^2$ ⓧ
 eg 6.2: $1.15 G = 1.15 \cdot 2.28 = 3.31 \text{ kN/m}^2$ ⓧ
 - 3rd floor
 - 2a permanent = 16.84 kN/m^2 (historical rule)
 - 2b variable = $Q = 1.75 \text{ kN/m}^2$
 - load var. Q = $Q \cdot CFA/2 = 1.75 \cdot 10.28/2 = 9.00 \text{ kN/m}^2$ ⓧ

design values

Full loaded → eg 6.1 = $1.2 G + 1.3 \psi Q = (1.2 \cdot 16.84) + (1.3 \cdot 0.4 \cdot 9) = 24.8 \text{ kN/m}^2$ ⓧ
 eg 6.2 = $1.15 G + 1.3 Q = (1.15 \cdot 16.84) + (1.3 \cdot 9) = 31.07 \text{ kN/m}^2$ ⓧ
 - 1st & 2nd floor
 - 3a permanent = 16.84 kN/m^2 (historical ^{calc} value)
 - 3b Variable: $Q = 1.75 \text{ kN/m}^2$
 - load var. Q = $Q \cdot CFA/2 = 1.75 \cdot 10.28/2 = 9.00 \text{ kN/m}^2$ ⓧ

design values

eg 6.1 = $1.2 G + 1.3 \psi Q = (1.2 \cdot 16.84) + (1.3 \cdot 0.4 \cdot 9) = 24.8 \text{ kN/m}^2$ ⓧ
 eg 6.2 = $1.15 G + 1.3 Q = (1.15 \cdot 16.84) + (1.3 \cdot 9) = 24.05 \text{ kN/m}^2$ ⓧ
 - Ground floor
 - 4a permanent = 37.91 kN/m^2 (historical rule)
 - 4b Variable: $Q = 2 \text{ kN/m}^2$
 - load var. Q = $Q \cdot CFA/2 = 2 \cdot 10.28/2 = 10.28 \text{ kN/m}^2$ ⓧ

design values

Full loaded → eg 6.1 = $1.2 G + 1.3 \psi Q = (1.2 \cdot 37.91) + (1.3 \cdot 0.4 \cdot 10.28) = 50.84 \text{ kN/m}^2$ ⓧ
 eg 6.2 = $1.15 G + 1.3 Q = (1.15 \cdot 37.91) + (1.3 \cdot 10.28) = 56.96 \text{ kN/m}^2$ ⓧ
 - Total load Ed

eg 6.1 = $3.46 + 24.8 + (2 \cdot 24.8) + 50.84 = 128.97 \text{ kN/m}^2$ ⓧ $\leftrightarrow 128.97 \text{ kN/m}^2$ (steel)
 eg 6.2 = $3.31 + 31.07 + (2 \cdot 24.05) + 56.96 = 139.44 \text{ kN/m}^2$ ⓧ $\leftrightarrow 139.44 \text{ kN/m}^2$ (steel)
 Eurocode design value = 139.44 kN/m^2

Figure X.24: Verification excel sheet (3 stories)_Overcapacity VL (3)

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• Calculatie overcapaciteit

$$\text{Over capacity}_{M_2} = \frac{(\text{historical} - \text{Eurocode})}{0.5 \text{ CFA}} = \frac{181.75 - 135.44}{0.5 \cdot 10.28}$$

$8.24 \text{ [kN/m}^2\text{]} \rightarrow 8.25 \text{ [kN/m}^2\text{]}$
 (steel value)

Figure X.25: Verification excel sheet (3 stories)_Overcapacity VL (4)

X.5 Overcapacity S [Section 6.4]

X.5.1 Brickwork walls; No additional room; concrete floors

	A	B	C	D	E	F	G	H	I
Brickwork walls/ No additional room/ Concrete floors									
Parameters geometry			Formules B		Formules E				
4	Ratio: Depth / length stability wall	4,5 -							
5	Depth building	10,85 m							
6	Width unit building	18 m							
7	Storey height	2,8 m							
8	Number of floors_existing building	4							
9	Number of layers top-up	2 -					Contr. Width 1st	Contr. width other	3,23 m
10	Contributing width angle	90 degree							
11	Number of buildings between dilatation	2 -							
12	Height building_extended	19,6 m		(B8+B9+1)*B7					
13	Height existing building	14 m		(B8+1)*B7					
Wall parameters									
15	Wall thickness	220 mm							
16	Length stability wall	2,41 m		B5/B4					
Longitudinal direction									
18	NFL	4,26 -							
19	Floor length a	2,47 m							
20	Floor length b	3,025 m		B18-(0,5*B19)					
Transversal direction									
23	Half floor span (Wall next to dilatation)	1,96 m							
Structural material properties									
General									
28	Brickwork	19 kN/m3							
29	Concrete	24 kN/m3							
Concrete									
32	Concrete floor	4,7 kN/m2							
33	Concrete roof	3,88 kN/m2							
Timber floor									
36	Timber floor	0,5 kN/m2							
37	Timber roof	0,56 kN/m2							
Safety factors (NEN 8700)									
40	Permanent load	0,9 -							
41	Variable load	1,4 -							
Verification in longitudinal direction									
Compression caused by self-weight									
46	1.Roof structure	kN/m2	Pointload left/right [kN]	Formules C	Pointload middle [kN]	[kN/m]	Formules G		
47		3,88		37,95 B47*B20*H10	30,99 B47*B19*H10				
48	2.Floor load (story 1 till 4)								
49	First floor	4,7	22,98 B49*B20*C10		18,77 B49*B19*C10				
50	Other floors_per floor	4,7	45,87 B50*B20*H10		37,53 B50*B19*H10				
51	Total for floors		160,89 C49+(B8-1)*C50		131,37 E49+(B8-1)*E50				
3.Walls									
53	Stability wall						58,52	(B15/1000)*B7*(B8+1)*B28	
54	Pointloads from contributing walls								
55	First floor		18,92	(B15/1000)*B7*B28*G10	18,92	(B15/1000)*B28*B7*G10			
56	Contributing width other walls		113,52	(B15/1000)*H10*B7*B28*(B8-1)	113,52	(B15/1000)*B7*(B8-1)*H10*B28			
57	Total overview of loads		331,28	C47+C51+C55+C56	294,80	E47+E51+E55+E56	58,52	G53	
Formules B									
60	Compression force SLS	549,22 kN		(0,5*E58)+C58+(0,5*G58*B16)					
61	Compression force ULS	494,30 kN		B60*B40					
Tension caused by wind									
64	CsCd	1 -					Total stories	Height	op [kN/m2]
65	qs	0,9 kN/m2					3	11,2	0,71
66	Ct,D	0,8 -					4	14	0,78
67	Ct,E	0 -					5	16,8	0,84
68	Ct,ro	0,8 -		B67+B68			6	19,6	0,9
69	q_wind_per wall	3,91 kN/m		B65*B66*B69*B5/B11			7	22,4	0,93
70	M_wind	750,26 kNm		(1/2)*B70*B12*2					
71	M_wind_Ed	1050,37 kNm		B71*B41					
72	Tension on the sides	435,64 kN		B72/B16					
73	Tension?	0,88 -		B73/B61					
Compression maximum									
77	Maximum compression force	929,94 kN		B61+B73					
78	Area	355647,8 mm2		(G10*1000)*B15					
79	Compressive stress_ULS	2,61 N/mm2		B78*1000/B79					
Brickwork (Fbc=10; M10)									
82	Material factor	1,7 -							
83	Compressive capacity	2,81 N/mm2		B82/B83					
84	UC	0,93 -		B80/B84					
Verification in cross direction									
Compression caused by self-weight									
88	Roof	kN/m2	kN/m	Formules C					
89		3,88		7,60 B90*B23					
90	Floor load	4,7		36,85 B91*B8*B23					
91	Walls			58,52 (B15/1000)*B7*(B8+1)*B28					
92	Total			102,97 C90+C91+C92					
Formules B									
95	Compressive stress	0,47 N/mm2		C93/B15					
96	Compressive stress_ED	0,42 N/mm2		B95*B40					
Tensile stress (caused by wind)									
100	CsCd	1 -							
101	qs(h)	0,9 kN/m2							
102	Ct,D	0,8 -							
103	Ct,E	-0,7 -							
104	Ct,ro	1,5 -		B102-B103					
105	q_wind	3,97 kN/m		B100*B101*B104*B23*2*o,75					
106	M_Wind_Ed	1,07E+09 Nmm		(1/2)*B105*(B12*1000)^2*B41					
107	W_wall	4,32E+09 mm3		(1/6)*B15*(B5*1000)^2					
108	Sigma M	0,25 N/mm2		B106/B107					
109	UC_cross direction	0,59 -		B108/B96					
Compression maximum									
114	Maximum compressive stress_ULS	0,67 N/mm2		B96+B108					
Brickwork (Fbc=10; M10)									
116	Material factor	1,7 -							
117	Compressive capacity	2,81 N/mm2		B116/B117					
118	UC	0,24 -		B114/B118					

Figure X.26: Overcapacity S; No additional room; Brickwork walls; Concrete floors_Excel

X.5.2 Concrete walls; No additional room; concrete floors

	A	B	C	D	E	F	G	H	I
Concrete Walls / No additional room/ Concrete floors									
Parameters geometry			Formules B		Formules E				
Ratio: Depth / length stability wall		4,5 -							
Depth building		10,85 m							
Width unit building		18 m							
Storey height		2,8 m							
Number of floors_existing building		4 -							
Number of layers_top-up		2 -					Contr. Width 1st	Contr. width other	
Contributing width angle		30 degree					1,62	3,23 m	
Number of buildings between dilatation		2 -					TAN(B10*2*PI()/360) * B; TAN(B10*2*PI()/360) * B7 * 2		
Height building_extended		19,6 m		(B8+B9+1)*B7					
Height existing building		14 m		(B8+1)*B7					
Wall parameters									
Wall thickness		200 mm							
Length stability wall		2,41 m		B5/B4					
Longitudinal direction									
NFL		4,26 -							
Floor length a		2,47 m							
Floor length b		3,025 m		B18*(0,5*B19)					
Transversal direction									
Half floor span (Wall next to dilatation)		1,96 m							
Structural material properties									
General									
Brickwork		19 kN/m3							
Concrete		24 kN/m3							
Concrete									
Concrete floor		4,7 kN/m2							
Concrete roof		3,88 kN/m2							
Timber floor									
Timber floor		0,5 kN/m2							
Timber roof		0,56 kN/m2							
Safety factors (NEN 8700)									
Permanent load		0,9 -							
Variable load		1,4 -							
Verification in longitudinal direction									
Compression caused by self-weight									
1.Roof structure	kN/m2	3,88	Pointload left/right [kN]	Formules C	Pointload middle [kN]		[kN/m]	Formules G	
2.Floor load (story 1 till 4)									
First floor		4,7	22,98	B49*B30*G10	18,77	B49*B19*G10			
Other floors_per floor		4,7	45,97	B50*B20*H10	37,53	B50*B19*H10			
Total for floors			160,89	C49+((B8-1)*C50)	131,37	E49+((B8-1)*E50)			
3.Walk									
Stability wall								67,2	(B15/1000)*B7*(B8+1)*B29
Pointloads from contributing walls									
First floor			21,73	(B15/1000)*B7*B29*G10	21,73	(B15/1000)*B29*B7*G10			
Contributing width other walls			130,36	(B15/1000)*H10*B7*B29*(B8-1)	130,36	(B15/1000)*B7*(B8-1)*H10*B29			
Total overview of loads			350,92	C47+C51+C55+C56	314,44	E47+E51+E55+E56		67,2	G53
Formules B									
Compression force SLS		589,16 kN		(0,5*E58)+C58+(0,5*G58*B16)					
Compression force ULS		530,24 kN		B60*B40					
Tension caused by wind									
CsCd		1 -							
qp		0,9 kN/m2			Total stories	Height	qp [kN/m2]		
Cf,D		0,8 -			3		11,2	0,71	
Cf,E		0 -			4		14	0,78	
Cf,tot		0,8 -		B67*B68	5		16,8	0,84	
q_wind_per wall		3,91 kN/m		B65*B66*B69*B5/B11	6		19,6	0,9	
M_wind		750,26 kNm		(1/2)*B70*B12*2	7		22,4	0,93	
M_wind_Ed		1050,37 kNm		B71*B41					
Tension on the sides		435,64 kN		B72/B16					
Tension?		0,82 -		B73/B61					
Compression maximum									
Maximum compression force		965,88 kN		B61*B73					
Area		323316,2 mm2		(G10*1000)*B15					
Compressive stress_ULS		2,99 N/mm2		B78*1000/B79					
Concrete C12/15		12 N/mm2							
Material factor		1,5 -							
Compressive capacity		8 N/mm2		B82/B83					
UC		0,37 -		B80/B84					
Verification in cross direction									
Compression caused by self-weight									
Roof	kN/m2	3,88	kN/m	Formules C					
Floor load		4,7		7,60	B90*B23				
Walls				36,85	B91*B8*B23				
Total				67,2	(B15/1000)*B7*(B8+1)*B29				
				111,65	C90+C91+C92				
Formules B									
Compressive stress		0,56 N/mm2		C93/B15					
Compressive stress_ED		0,50 N/mm2		B95*B40					
Tensile stress (caused by wind)									
CsCd		1 -							
qp(h)		0,9 kN/m2							
Cf,D		0,8 -							
Cf,F		-0,7 -							
Cf,tot		1,5 -		B102-B103					
q_wind		3,97 kN/m		B100*B101*B104*B23*2*0,75					
M_Wind_Ed		1,07E+09 Nmm		(1/2)*B105*(B12*1000)*2*B41					
W_wall		3,92E+09 mm3		(1/6)*B15*(B5*1000)*2					
Sigma M		0,27 N/mm2		B106/B107					
UC_cross direction		0,54 -		B108/B96					
Compression maximum									
Maximum compressive stress_ULS		0,77 N/mm2		B96*B108					
Concrete C12/15		12 N/mm2							
Material factor		1,5 -							
Compressive capacity		8 N/mm2		B116/B117					
UC		0,10 -		B114/B118					

Figure X.27: Overcapacity S; No additional room; Concrete walls; Concrete floors_Excel

X.5.3 Brickwork walls; No additional room; timber floors

	A	B	C	D	E	F	G	H	I
Brickwork walls/ No additional room/ Timber floors									
Parameters geometry			Formules B		Formules E				
4	Ratio: Depth / length stability wall	4,5 -							
5	Depth building	10,85 m							
6	Width unit building	1,8 m							
7	Storey height	2,8 m							
8	Number of floors	4 -							
9	Number of layers_Top-up	2 -							
10	Contributing width angle	30 degree							
11	Number of buildings between dilatation	2 -							
12	Height building_extended	19,6 -		(B8+B9+1)*B7					
13	Height existing building	14 -		(B8+1)*B7					
Wall parameters									
15	Wall thickness	220 mm							
16	Length stability wall	2,41 m		B5/B4					
Longitudinal direction									
18	NFL	4,26 -							
19	Floor length a	2,47 m							
20	Floor length b	3,025 m		B18-(0,5*B19)					
Transversal direction									
23	Half floor span (Wall next to dilatation)	1,96 m							
Structural material properties									
General									
28	Brickwork	19 kN/m3							
29	Concrete	24 kN/m3							
Concrete									
32	Concrete floor	4,7 kN/m2							
33	Concrete roof	3,88 kN/m2							
Timber floor									
36	Timber floor	1 kN/m2							
37	Timber roof	0,56 kN/m2							
Safety factors (NEN 8700)									
40	Permanent load	0,9 -							
41	Variable load	1,4 -							
Verification in longitudinal direction									
Compression caused by self-weight									
46		kN/m2	Pointload left/right [kN]	Formules C	Pointload middle [kN]		[kN/m]	Formules G	
47	1.Roof structure		0,56	5,48 B47*B20*H10	4,47 B47*B19*H10				
48	2.Floor load (story 1 till 4)								
49	First floor	1	4,89 B49*B20*G10		3,99 B49*B19*G10				
50	Other floors_per floor	1	9,78 B50*B20*H10		7,99 B50*B19*H10				
51	Total for floors		34,23 C49+(B8-1)*C50		27,95 E49+(B8-1)*E50				
Walls									
53	Stability wall								
54	Pointloads from contributing walls								58,52 (B15/1000)*B7*(B8+1)*B28
55	First floor		18,92 (B15/1000)*B7*B28*G10		18,92 (B15/1000)*B28*B7*G10				
56	Contributing width other walls		113,52 (B15/1000)*H10*B7*B28*(B8-1)		113,52 (B15/1000)*B7*(B8-1)*H10*B28				
57	Total overview of loads		172,15 C47+C51+C55+C56		164,87 E47+E51+E55+E56			58,52 G53	
58				Formules B					
59	Compression force SLS	325,13 kN		(0,5*E58)+C58+(0,5*G58*B16)					
60	Compression force ULS	292,62 kN		B60*B40					
Tension caused by wind									
65	CsCd	1 -							
66	qp	0,9 kN/m2							
67	Cr,D	0,8 -							
68	Cr,E	0 -							
69	Cr,F	-0,7 -							
70	q_wind_per wall	3,91 kN/m		B67*B68					
71	M_wind	750,26 kNm		B65*B66*B69*B5/B11					
72	M_wind_Ed	1050,37 kNm		(1/2)*B70*B12*2					
73	Tension on the sides	435,64 kN		B71*B41					
74				B72/B16					
75	Tension?	1,49 -		B73/B61					
Compression maximum									
77	Maximum compression force	728,26 kN		B61+B73					
78	Area	355647,77 mm2		(G10*1000)*B15					
79	Compressive stress_ULS	2,05 N/mm2		B78*1000/B79					
81	Brickwork (Fbc=10; M10)	4,77 N/mm2							
82	Material factor	1,7 -							
84	Compressive capacity	2,81 N/mm2		B82/B83					
85	UC	0,73 -		B80/B84					
Verification in cross direction									
Compression caused by self-weight									
89		kN/m2	kN/m	Formules C					
90	Roof		0,56	1,10 B90*B23					
91	Floor load		1	7,84 B91*B8*B23					
92	Walls			58,52 (B15/1000)*B7*(B8+1)*B28					
93	Total			67,46 C90+C91+C92					
94				Formules B					
95	Compressive stress	0,31 N/mm2		C95/B15					
96	Compressive stress_ED	0,28 N/mm2		B95*B40					
Tensile stress (caused by wind)									
100	CsCd	1 -							
101	qp(h)	0,9 kN/m2							
102	Cr,D	0,8 -							
103	Cr,F	-0,7 -							
104	Cr,Fot	1,5 -		B102-B103					
105	q_wind	3,97 kN/m		B100*B101*B104*B23*2*0,75					
106	M_Wind_Ed	1067311728,0 Nmm		(1/2)*B105*(B12*1000)^2*B41					
107	W_wall	4316491666,7 mm3		(1/6)*B15*(B5*1000)^2					
108	Sigma M	0,25 N/mm2		B106/B107					
109	UC_cross direction	0,90 -		B108/B96					
Compression maximum									
113	Maximum compressive stress_ULS	0,52 N/mm2		B96+B108					
115	Brickwork (Fbc=10; M10)	4,77 N/mm2							
117	Material factor	1,7 -							
118	Compressive capacity	2,81 N/mm2		B116/B117					
119	UC	0,19 -		B114/B118					

Figure X.28: Overcapacity S; No additional room; Brickwork walls; Timber floors_Excel

X.5.4 Brickwork walls; Additional room; concrete floors

	A	B	C	D	E	F	G	H	I
Brickwork walls / Additional room / Concrete floors									
Parameters geometry									
Ratio: Depth / length stability wall		4,5 -		Formules B		Formules E			
Depth building		10,85 m							
Width unit building		1,8 m							
Storey height		2,8 m							
Number of floors_Existing building		4 -							
Number of layers_top-up		2 -					Contr. Width 1st	1,62	Contr. width other
Contributing width angle		30 degree							3,23 m
Number of buildings between dilatation		2 -					$TAN(B10*2*PI()/360) * B7$		
Height building_extended		19,6 m		$(B8+B9+1)*B7$			$TAN(B10*2*PI()/360) * B7 * 2$		
Height existing building		14 m		$(B8+1)*B7$					
Wall parameters									
Wall thickness		220 mm							
Length stability wall		2,41 m		B5/B4					
Longitudinal direction									
NFL		4,26 -							
Floor length d		2,41 m							
Floor length c		1,53 m		$(B18-(0,5*B19))/2$					
Transversal direction									
Half Floor span (Wall next to dilatation)		1,96 m							
Structural material properties									
General									
Brickwork		19 kN/m3							
Concrete		24 kN/m3							
Concrete									
Concrete floor		4,7 kN/m2							
Concrete roof		3,88 kN/m2							
Timber floor									
Timber floor		0,5 kN/m2							
Timber roof		0,56 kN/m2							
Safety factors (NEN 8700)									
Permanent load		0,9 -							
Variable load		1,4 -							
Verification in longitudinal direction									
Compression caused by self-weight									
1.Roof structure		3,88 kN/m2	Pointload left/right [kN]	Formules C	53,44	Pointload middle [kN]	0,00	[kN/m]	Formules G
2.Floor load (story 1 till 4)									
First floor		4,7			32,37		0,00		
Other floors_per floor		4,7			64,73		0,00		
Total for floors					226,57		0,00	$E49+((B8-1)*E50)$	
3.Walls									
Stability wall								58,52	$(B15/1000)*B7*(B8+1)*B28$
Pointloads from contributing walls									
First floor					37,84		0,00		
Contributing width other walls					227,05		0,00		
Total overview of loads					544,90		0,00	$E47+E51+E55+E56$	58,52 G53
Compression force SLS		615,45 kN		Formules B					
Compression force ULS		553,90 kN		$C58 + (0,5 * G58 * B16)$					
Tension caused by wind									
CsCd		1 -							
qp		0,9 kN/m2				Total stories	Height	qp [kN/m2]	
Cr_D		0,8 -				3	11,2	0,71	
Cr_E		0 -				4	14	0,78	
Ct_tot		0,8 -		B67*B68		5	16,8	0,84	
q_wind_per wall		3,91 kN/m		B65*B66*B69*B5/B11		6	19,6	0,9	
M_wind		750,26 kNm		$(1/2)*B70*B12*2$		7	22,4	0,93	
M_wind_Ed		1050,37 kNm		B71*B41					
Tension on the sides		435,64 kN		B72/B16					
Tension?		0,79 -		B73/B61					
Compression maximum									
Maximum compression force		989,54 kN		B61*B73					
Area		711295,5 mm2		$(G10*1000*2)*B15$					
Compressive stress_ULS		1,39 N/mm2		$B78*1000/B79$					
Brickwork (Fbc=10; M10)		4,77 N/mm2							
Material factor		1,7 -							
Compressive capacity		2,81 N/mm2		B82/B83					
UC		0,50 -		B80/B84					
Verification in cross direction									
Compression caused by self-weight									
Roof		3,88 kN/m2	kN/m	Formules C	7,60				
Floor load		4,7		B90*B23	36,85				
Walls				B91*B8*B23	58,52				$(B15/1000)*B7*(B8+1)*B28$
Total				C90+C91+C92	102,97				
Compressive stress		0,47 N/mm2		Formules B					
Compressive stress_ED		0,42 N/mm2		C93/B15					
Tensile stress (caused by wind)									
CsCd		1 -							
qp(h)		0,9 kN/m2							
Cr_D		0,8 -							
Cr_E		-0,7 -							
Ct_tot		1,5 -		B102-B103					
q_wind		3,97 kN/m		$B100*B101*B104*B23*2*0,75$					
M_wind_Ed		1,07E+09 Nmm		$(1/2)*B105*(B12*1000)*2*B41$					
W_wall		4,32E+09 mm3		$(1/6)*B15*(B5*1000)*2$					
Sigma M		0,25 N/mm2		B106/B107					
UC_cross direction		0,59 -		B108/B96					
Compression maximum									
Maximum compressive stress_ULS		0,67 N/mm2		B96*B108					
Brickwork (Fbc=10; M10)		4,77 N/mm2							
Material factor		1,7 -							
Compressive capacity		2,81 N/mm2		B116/B117					
UC		0,24 -		B114/B118					

Figure X.29: Overcapacity S; Additional room; Brickwork walls; Concrete floors_Excel

X.5.5 Concrete walls; Additional room; concrete floors

	A	B	C	D	E	F	G	H	I	
Concrete walls / Additional room/ Concrete floor										
Parameters geometry			Formules B		Formules E					
4	Ratio: Depth / length stability wall	4,5 -								
5	Depth building	10,85 m								
6	Width unit building	18 m								
7	Storey height	2,8 m								
8	Number of floors_Existing building	4 -								
9	Number of layers top-up	2 -					Contr. Width 1st	Contr. width other		
10	Contributing width angle	30 degree					1,62	3,23 m		
11	Number of buildings between dilatation	2 -					TAN(B10°*2*PI()/360) * B7 TAN(B10°*2*PI()/360) * B7 * 2			
12	Height building_extended	19,6 m		(B8+B9+1)*B7						
13	Height existing building	14 m		(B8+1)*B7						
Wall parameters										
15	Wall thickness	200 mm								
16	Length stability wall	2,41 m		B5/B4						
Longitudinal direction										
18	NFL	4,26 -								
19	Floor length d	2,41 m								
20	Floor length c	1,53 m		(B18-(0,5*B19))/2						
Transversal direction										
23	Half floor span (Wall next to dilatation)	1,96 m								
Structural material properties										
General										
28	Brickwork	19 kN/m3								
29	Concrete	24 kN/m3								
Concrete										
32	Concrete floor	4,7 kN/m2								
33	Concrete roof	3,88 kN/m2								
Timber floor										
36	Timber floor	0,5 kN/m2								
37	Timber roof	0,56 kN/m2								
Safety factors (NEN 8700)										
40	Permanent load	0,9 -								
41	Variable load	1,4 -								
Verification in longitudinal direction										
Compression caused by self-weight			Formules C		Formules G					
46		kN/m2	Pointload left/right [kN]		Pointload middle [kN]		kN/m			
47	1.Roof structure	3,88	53,44	B47*B18*H10	0,00					
48	2.Floor load (story 1 till 4)									
49	First floor	4,7	32,37	B49*B18*G10	0,00					
50	Other floors_per floor	4,7	64,73	B50*B18*H10	0,00					
51	Total for floors		226,57	C49+((B8-1)*C50)	0,00	E49H((B8-1)*E50)				
3.Walls										
53	Stability wall						53,2	(B15/1000)*B7*(B8+1)*B28		
54	Pointloads from contributing walls									
55	First floor		43,45	(B15/1000)*B7*B29*(G10°*2)	0,00					
56	Contributing width other walls		260,72	(B15/1000)*H(H10°*2)*B7*B29*(B8-1)	0,00					
57	Total overview of loads									
58			584,19	C47+C51+C55+C56	0,00	E47+E51+E55+E56	53,2	G53		
Formules B										
60	Compression force SLS	648,32 kN		C58 + (0,5 * G58 * B16)						
61	Compression force ULS	583,49 kN		B60*B40						
Tension caused by wind										
65	CsCd	1 -								
66	qp	0,9 kN/m2				Total stories	Height	qp [kN/m2]		
67	Cr_D	0,8 -				3	11,2	0,71		
68	Cr_E	0 -				4	14	0,78		
69	Cr_ox	0,8 -		B67*B68		5	16,8	0,84		
70	q_wind_per wall	3,91 kN/m		B65*B66*B69*B95/B11		6	19,6	0,9		
71	M_wind	750,26 kNm		(1/2)*B70*B12*2		7	22,4	0,93		
72	M_wind_Ed	1050,37 kNm		B71*B41						
73	Tension on the sides	435,64 kN		B72/B16						
74	Tension?	0,75		B73/B61						
Compression maximum										
78	Maximum compression force	1019,13 kN		B61*B73						
79	Area	646632,3 mm2		(G10°*1000°*2)*B15						
80	Compressive stress_ULS	1,58 N/mm2		B78°*1000/B79						
83	C12/15	12 N/mm2								
84	Material factor	1,5 -								
85	Compressive capacity	8 N/mm2								
86	UC	0,20 -		B80/B84						
Verification in cross direction										
Compression caused by self-weight			Formules C		Formules B					
89		kN/m2	kN/m							
90	Roof	3,88	7,60	B90*B23						
91	Floor load	4,7	36,85	B91*B8*B23						
92	Walls		67,2	(B15/1000)*B7*(B8+1)*B29						
93	Total		111,65	C90+C91+C92						
95	Compressive stress	0,56 N/mm2		C93/B15						
96	Compressive stress_ED	0,50 N/mm2		B95*B40						
Tensile stress (caused by wind)										
100	CsCd	1 -								
101	qp(h)	0,9 kN/m2								
102	Cr_D	0,8 -								
103	Cr_E	-0,7 -								
104	Cr_ox	1,5 -		B102-B103						
105	q_wind	3,97 kN/m		B100*B101*B104*B23°*2*0,75						
106	M_Wind_Ed	1,07E+09 Nmm		(1/2)*B105*(B12°*1000°)*2*B41						
107	W_wall	3,92E+09 mm3		(1/6)*B15*(B5°*1000°)*2						
108	Sigma M	0,27 N/mm2		B106/B107						
110	UC_cross direction	0,54 -		B108/B96						
Compression maximum										
114	Maximum compressive stress_ULS	0,77 N/mm2		B96+B108						
116	Concrete C12/15	12 N/mm2								
117	Material factor	1,5 -								
118	Compressive capacity	8 N/mm2		B116/B117						
119	UC	0,10 -		B114/B118						

Figure X.30: Overcapacity S; Additional room; Concrete walls; Concrete floors_Excel

X.5.6 Brickwork walls; Additional room; timber floors

	A	B	C	D	E	F	G	H	I
Brickwork walls/ Additional room / Timber floors									
Parameters geometry			Formules B		Formules E				
Ratio: Depth / length stability wall		4,5 -							
Depth building		10,85 m							
Width unit building		1,8 m							
Storey height		2,8 m							
Number of floors		4 -							
Number of layers_Top-up		2 -							
Contributing width angle		30 degree							
Number of buildings between dilatation		2 -							
Height building_extended		19,6 -		(B8+B9+1)*B7					
Height existing building		14 -		(B8+1)*B7					
Wall parameters									
Wall thickness		220 mm							
Length stability wall		2,41 m		B5/B4					
Longitudinal direction									
NFL		4,26 -							
Floor length d		2,41 m							
Floor length c		1,53 m		(B18-(0,5*B19))/2					
Transversal direction									
Half floor span (Wall next to dilatation)		1,96 m							
Structural material properties									
General									
Brickwork		19 kN/m3							
Concrete		24 kN/m3							
Concrete									
Concrete floor		4,7 kN/m2							
Concrete roof		3,88 kN/m2							
Timber floor									
Timber floor		1 kN/m2							
Timber roof		0,56 kN/m2							
Safety factors (NEN 8700)									
Permanent load		0,9							
Variable load		1,4							
Verification in longitudinal direction									
Compression caused by self-weight									
1.Roof structure		0,56 kN/m2		Pointload left/right [kN] Formules C		Pointload middle [kN]		[kN/m]	
2.Floor load (story 1 till 4)				7,71 B47*B18*H10		0,00			
First floor		1		6,89 B49*B18*G10		0,00			
Other floors_per floor		1		13,77 B50*B18*H10		0,00			
Total for floors				48,21 C49+((B8-1)*C50)		0,00 E49+((B8-1)*E50)			
3.Walls									
Stability wall									
Pointloads from contributing walls									
First floor				37,84 (B15/1000)*B7*B28*(G10*2)		0,00			
Contributing width other walls				227,05 (B15/1000)*(H10*2)*B7*B28*(B8-1)		0,00			
Total overview of loads				320,81 C47+C51+C55+C56		0,00 E47+E51+E55+E56		58,52 G53	
Formules B									
Compression force SLS		391,36 kN		C58 + (0,5 * G58 * B16)					
Compression force ULS		352,22 kN		B60*B40					
Tension caused by wind									
CsCd		1 -							
qp		0,9 kN/m2							
CtD		0,8 -							
CtE		0 -							
Ct,rot		0,8 -		B67*B68					
q_wind_per wall		3,91 kN/m		B65*B66*B69*B5/B11					
M_wind		750,26 kNm		(1/2)*B70*B12*2					
M_wind_Ed		1050,37 kNm		B71*B41					
Tension on the sides		435,64 kN		B72/B16					
Tension?		1,24 -		B73/B61					
Compression maximum									
Maximum compression force		787,86 kN		B61*B73					
Area		711295,5 mm2		(G10*1000*2)*B15					
Compressive stress_ULS		1,11 N/mm2		B78*1000/B79					
Brickwork (Fbc=10; M10)		4,77 N/mm2							
Material factor		1,7 -							
Compressive capacity		2,81 N/mm2		B82/B83					
UC		0,39 -		B80/B84					
Verification in cross direction									
Compression caused by self-weight									
Roof		0,56 kN/m2		1,10 B90*B23					
Floor load		1		7,84 B91*B8*B23					
Walls				58,52 (B15/1000)*B7*(B8+1)*B28					
Total				67,46 C90+C91+C92					
Formules B									
Compressive stress		0,31 N/mm2		C93/B15					
Compressive stress_ED		0,28 N/mm2		B95*B40					
Tensile stress (caused by wind)									
CsCd		1 -							
qp(h)		0,9 kN/m2							
CtD		0,8 -							
CtE		-0,7 -							
Ct,rot		1,5 -		B102-B103					
q_wind		3,97 kN/m		B100*B101*B104*B23*2*0,75					
M_Wind_Ed		1,07E+09 Nmm		(1/2)*B105*(B12*1000)*2*B41					
W_wall		4,32E+09 mm3		(1/6)*B15*(B5*1000)*2					
Sigma M		0,25 N/mm2		B106/B107					
UC_cross direction		0,90 -		B108/B96					
Compression maximum									
Maximum compressive stress_ULS		0,52 N/mm2		B96+B108					
Brickwork (Fbc=10; M10)		4,77 N/mm2							
Material factor		1,7 -							
Compressive capacity		2,81 N/mm2		B116/B117					
UC		0,19 -		B114/B118					

Figure X.31: Overcapacity S; Additional room; Brickwork walls; Timber floors_Excel sheet

X.5.7 Results for flowchart stability

Brickwork walls/ No additional room/ Concrete floors						
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors
2.67	2.87	3		2.67	2.87	3
2.67	3.69	2		2.67	3.69	2
3.47	2.87	>3		3.47	2.87	3
3.47	3.69	2		3.47	3.69	2
3.47	2.87	>3		3.47	2.87	3
3.47	3.69	2		3.47	3.69	2
4.26	2.87	2		4.26	2.87	>3
4.26	3.69	2		4.26	3.69	3
2.67	3.69	2		2.67	3.69	2
2.67	4.50	1		2.67	4.50	1
3.47	3.69	2		3.47	3.69	2
3.47	4.50	2		3.47	4.50	2
3.47	3.69	2		3.47	3.69	2
3.47	4.50	2		3.47	4.50	2
4.26	3.69	3		4.26	3.69	3
4.26	4.50	2		4.26	4.50	2

Concrete Walls / No additional room/ Concrete floors						
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors
2.67	2.87	>3		2.67	2.87	3
2.67	3.69	2		2.67	3.69	2
3.47	2.87	>3		3.47	2.87	>3
3.47	3.69	3		3.47	3.69	3
3.47	2.87	>3		3.47	2.87	>3
3.47	3.69	3		3.47	3.69	3
4.26	2.87	>3		4.26	2.87	>3
4.26	3.69	3		4.26	3.69	3
2.67	3.69	2		2.67	3.69	2
2.67	4.50	1		2.67	4.50	2
3.47	3.69	3		3.47	3.69	3
3.47	4.50	2		3.47	4.50	2
3.47	3.69	3		3.47	3.69	3
3.47	4.50	2		3.47	4.50	2
4.26	3.69	3		4.26	3.69	3
4.26	4.50	2		4.26	4.50	2

Brickwork walls/ No additional room/ Timber floors						
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors
2.67	2.87	2		2.67	2.87	2
2.67	3.69	1		2.67	3.69	1
3.47	2.87	2		3.47	2.87	2
3.47	3.69	1		3.47	3.69	1
3.47	2.87	2		3.47	2.87	2
3.47	3.69	1		3.47	3.69	1
4.26	2.87	2		4.26	2.87	2
4.26	3.69	1		4.26	3.69	1
2.67	3.69	1		2.67	3.69	1
2.67	4.50	0		2.67	4.50	0
3.47	3.69	1		3.47	3.69	1
3.47	4.50	0		3.47	4.50	1
3.47	3.69	1		3.47	3.69	1
3.47	4.50	1		3.47	4.50	1
4.26	3.69	1		4.26	3.69	1
4.26	4.50	0		4.26	4.50	1

Brickwork walls / Additional room/ Concrete floors							
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors	
	4,07	2,87	>3		4,07	2,87	>3
	4,07	3,69	3		4,07	3,69	3
	6,16	2,87	>3		6,16	2,87	>3
	6,16	3,69	>3		6,16	3,69	>3
	6,16	2,87	>3		6,16	2,87	>3
	6,16	3,69	>3		6,16	3,69	3
	8,25	2,87	>3		8,25	2,87	>3
	8,25	3,69	>3		8,25	3,69	>3
	4,07	3,69	3		4,07	3,69	3
	4,07	4,5	>3		4,07	4,5	2
	6,16	3,69	>3		6,16	3,69	>3
	6,16	4,5	>3		6,16	4,5	3
	6,16	3,69	>3		6,16	3,69	>3
	6,16	4,5	>3		6,16	4,5	3
	8,25	3,69	>3		8,25	3,69	>3
	8,25	4,5	>3		8,25	4,5	>3

Figure X.32: Results for flowchart S (1)

Concrete walls / Additional room/ Concrete floor							
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors	
	4,07	2,87	>3		4,07	2,87	>3
	4,07	3,69	3		4,07	3,69	3
	6,16	2,87	>3		6,16	2,87	>3
	6,16	3,69	>3		6,16	3,69	>3
	6,16	2,87	>3		6,16	2,87	>3
	6,16	3,69	>3		6,16	3,69	>3
	8,25	2,87	>3		8,25	2,87	>3
	8,25	3,69	>3		8,25	3,69	>3
	4,07	3,69	3		4,07	3,69	3
	4,07	4,5	2		4,07	4,5	2
	6,16	3,69	>3		6,16	3,69	>3
	6,16	4,5	3		6,16	4,5	3
	6,16	3,69	>3		6,16	3,69	>3
	6,16	4,5	3		6,16	4,5	3
	8,25	3,69	3		8,25	3,69	>3
	8,25	4,5	>3		8,25	4,5	>3
Brickwork walls/ Additional room / Timber floors							
4 stories NFL	Ratio	Addable floors		3 stories NFL	Ratio	Addable floors	
	4,07	2,87	2		4,07	2,87	2
	4,07	3,69	2		4,07	3,69	2
	6,16	2,87	2		6,16	2,87	2
	6,16	3,69	2		6,16	3,69	2
	6,16	2,87	2		6,16	2,87	2
	6,16	3,69	2		6,16	3,69	2
	8,25	2,87	2		8,25	2,87	2
	8,25	3,69	2		8,25	3,69	2
	4,07	3,69	2		4,07	3,69	2
	4,07	4,5	1		4,07	4,5	1
	6,16	3,69	2		6,16	3,69	2
	6,16	4,5	1		6,16	4,5	1
	6,16	3,69	2		6,16	3,69	2
	6,16	4,5	1		6,16	4,5	1
	8,25	3,69	2		8,25	3,69	2
	8,25	4,5	1		8,25	4,5	1

Figure X.33: Results for flowchart S (2)

X.5.8 Verification excel sheet stability (X.2; X.5)

X.5.8.1: No additional room

Order /
 Blad nr 1/6
 Deel Verification parameter study stability (1)
 Datum 25/04/2023



Verification excel stability study : No additional room
 Brickwork walls ;
 concrete floors

• Parameters geometry

Ratio: Depth / length stab. wall = 4.5 [-]
 depth building = 10.85 [m]
 width unit building = 1.8 [m]
 storey height = 2.8 [m]
 Floors existing building = 4 [-]
 number of layers top up = 2 [-]
 contributing width unit = 30 [°]
 number of buildings between dilatation = 2 [-]

Height existing building = $5 \cdot 2.8 = 14.00$ [m] Ⓢ
 Height extended building = $7 \cdot 2.8 = 19.60$ [m] Ⓢ

SHEET X.S.1
 ↳ principle valid for X.S.1 till X.S.3

contributing width
 (a) 1st Floor
 $\tan(30) = \frac{\text{contr. width } 1^{\text{st}}}{\text{storey height}}$
 $\Rightarrow \text{contr. width } 1^{\text{st}} = \tan(30) \cdot 2.8 = 1.62$ [m] Ⓢ
 (b) 2nd Floor and other
 $\Rightarrow \text{contr. width } 2^{\text{nd}} = \tan(30) \cdot 2.8 \cdot 3 = 2.3$ [m] Ⓢ

• Wall parameters

Wall thickness = 220 [mm]
 length stability wall = $\frac{\text{depth building}}{\text{Ratio}} = \frac{10.85}{4.5} = 2.41$ [-] Ⓢ

longitudinal direction

NFA = 4.26 [-] *(NFA = NFL)
 a = 2.47 [m]
 b = NFA - 0.5a = 4.26 - (0.5 \cdot 2.47) = 3.025 [m] Ⓢ

transversal direction
 half floor span = 1.96 [m]

Figure X.34: Verification excel overcapacity S (No additional room) (1)

Order ✓
 Blad nr 2/6
 Deel Verification parameter study stab. (1)
 Datum 25/04/2023

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• Structural material properties

Brickwork = 19 $\text{[kN/m}^3\text{]}$
 Concrete = 24 $\text{[kN/m}^3\text{]}$
 Concrete floor = 4.7 $\text{[kN/m}^2\text{]}$
 Concrete roof = 3.00 $\text{[kN/m}^2\text{]}$
 Timber floor = 0.5 $\text{[kN/m}^2\text{]}$
 Timber roof = 0.56 $\text{[kN/m}^2\text{]}$

• Safety factors (NEN 8700)

permanent: $\gamma_G = 0.9$ [-]
 variable: $\gamma_Q = 1.4$ [-]

• Verification longitudinal direction

A Self-weight \Rightarrow compression

1. Roof-structure

\rightarrow point load left/right = $b \cdot \text{cont. width other} \cdot \text{weight}$
 $= 3.025 \cdot 3.23 \cdot 3.00 = 31.91 \text{ [kN]} \quad \text{R}$
 \rightarrow point load middle = $\text{weight} \cdot \text{cont. width other} \cdot a$
 $= 3.00 \cdot 3.23 \cdot 2.47 = 30.96 \text{ [kN]} \quad \text{R}$

2. Floors

1st floor

\rightarrow point load left/right = $b \cdot \text{cont. width 1st} \cdot \text{weight}$
 $= 3.025 \cdot 1.66 \cdot 4.7 = 23.63 \text{ [kN]} \quad \text{R}$
 \rightarrow point load middle = $\text{weight} \cdot \text{cont. width 1st} \cdot a$
 $= 4.7 \cdot \frac{3.23}{1.66} \cdot 2.47 = \frac{37.80}{18.80} \text{ [kN]} \quad \text{R}$

Other floors

\rightarrow point load left/right = $b \cdot \text{cont. width other} \cdot \text{weight}$
 $= 3.025 \cdot 3.23 \cdot 4.7 = 45.92 \text{ [kN]} \quad \text{R}$
 \rightarrow point load middle = $\text{weight} \cdot \text{cont. width other} \cdot a$
 $= 4.7 \cdot 3.23 \cdot 2.47 = 37.50 \text{ [kN]} \quad \text{R}$

Total floors

\rightarrow point load left/right = $23.63 + 3 \cdot 45.92 = 160.79 \text{ [kN]} \quad \text{R}$
 \rightarrow point load middle = $18.80 + 3 \cdot 37.50 = 131.30 \text{ [kN]} \quad \text{R}$

Figure X.35: Verification excel overcapacity S (No additional room) (2)

Order ✓
 Blad nr 3/6
 Deel Verification parameter study stability (1)
 Datum 25/04/2023

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3 Walls

Stability wall

$$\rightarrow g = \frac{\text{th. cross}}{1000} \cdot \text{storey height} \cdot \text{number of walls} \cdot \text{width}$$

$$= \frac{220}{1000} \cdot 2.8 \cdot 5 \cdot 19 = 50.52 \text{ [kN/m]} \quad \text{S}$$

Contributing walls
 (a) First floor

\rightarrow point load left/right = $\frac{220}{1000} \cdot 2.8 \cdot 1.62 \cdot 19 = 18.96 \text{ [kN]} \quad \text{S}$

\rightarrow point load middle = $\frac{\text{th. cross}}{1000} \cdot \text{storey height} \cdot \text{cont.} \cdot \text{P. number}$

$$= \frac{220}{1000} \cdot 2.8 \cdot 1.62 \cdot 19 = 18.96 \text{ [kN]} \quad \text{S}$$

(b) other floors

\rightarrow point load left/right = $\frac{\text{th. cross}}{1000} \cdot \text{storey height} \cdot \text{cont. other} \cdot \text{P. number}$

$$= \frac{220}{1000} \cdot 2.8 \cdot 3.23 \cdot 19 = 113.41 \text{ [kN]} \quad \text{S}$$

\rightarrow point load middle = $\frac{\text{th. cross}}{1000} \cdot \text{storey height} \cdot \text{cont. other} \cdot \text{P. number}$

$$= \frac{220}{1000} \cdot 2.8 \cdot 3.23 \cdot 19 = 113.41 \text{ [kN]} \quad \text{S}$$

(c) Total contributing walls

point load left/right = $18.96 + 113.41 = 132.37 \text{ [kN]}$

point load middle = $18.96 + 113.41 = 132.37 \text{ [kN]}$

4. Total overview of loads

Point load left/right = $37.92 + 160.79 + 132.37 = 331.07 \text{ [kN]} \quad \text{S}$

Point load middle = $30.96 + 131.30 + 132.37 = 294.63 \text{ [kN]} \quad \text{S}$

g-load = $50.52 \text{ [kN]} \quad \text{S}$

Figure X.36: Verification excel overcapacity S (No additional room) (3)

Order /
 Blad nr 4/6
 Deel Verification parameter study stability (1)
 Datum 25/04/2023



Compression Force SLS = point load left/right + 0.5 middle + 0.5 g · L_{wall}

$$= 331.07 + (0.5 \cdot 294.05) + (0.5 \cdot 3052.241)$$

$$= 548.60 \text{ [kN]} \quad \rightarrow \quad 548.22 \text{ [kN]} \text{ (sheet)}$$

Compression Force ULS = 548.60 · γ_G = 548.60 · 0.9 = 493.74 [kN] Ⓢ

Point load L/R Point load middle Point load L/R.

↳ 493.74 [kN] (sheet)

B tension caused by wind.

$c_{s,c,d} = 1$ [-]
 $g_p = 0.5$ [kN/m²]
 $c_{pe} = 0.8$ [-]
 $c_{fe} = 0$ [-] dilatation
 $c_{pe,tot} = 0.8 + 0 = 0.8$ [-]

number of buildings per dilatation

Wind load = $c_{s,c,d} \cdot g_p \cdot c_{pe,tot} \cdot \text{depth building} / \text{number of walls}$

$$= 1 \cdot 0.5 \cdot 0.8 \cdot 10.85 / 2 = 3.51 \text{ [kN/m]}$$

$M_{wind} = \frac{1}{2} g l^2 = \frac{1}{2} \cdot 3.51 \cdot 19.6^2 = 751.03 \text{ [kNm]} \quad \rightarrow$

$M_{wind-ud} = M_{wind} \cdot \gamma_G = 751.03 \cdot 1.4 = 1051.45 \text{ [kNm]} \quad \rightarrow$

Tension per side = $\frac{M_{wind-ud}}{L_{wall}} = \frac{1051.45}{2.41} = 436.28 \text{ [kN]} \quad \rightarrow \quad 435.64 \text{ kN (sheet)}$

~~ULC = tension per side / compression force ULS = 436.28 / 493.74 = 0.88 Ⓢ~~

ULC = $\frac{\text{tension per side}}{\text{compression force ULS}} = \frac{436.28}{493.74} = 0.88 \quad \rightarrow \quad 0.88 \text{ (sheet)}$

Figure X.37: Verification excel overcapacity S (No additional room) (4)

Order ✓
 Blad nr 5/16
 Deel Verification parameter stability (1)
 Datum 25/04/2023

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C maximum compression

Max compressive force - uts = $493.74 + 436.28 = 930.02 \text{ [kN]}$ ✓

Area = $\text{Cont. } i^2 \cdot 160 \cdot \frac{\text{thickness}}{1000} = 1.62 \cdot \frac{220}{1000} \cdot 10^6 = 356400 \text{ [mm}^2\text{]}$ ✓

$\sigma_{N, uts} = \frac{F_u}{A} = \frac{930.02 \cdot 1000}{356400} = 2.61 \text{ [N/mm}^2\text{]}$ ✓ $\leftrightarrow 2.61 \text{ [N/mm}^2\text{]}$ (sheet)

brickwork ($f_m = 10$, M30) $\rightarrow 4.77 \text{ N/mm}^2$
 Material factor = 1.7 [7]
 $f_{cd} = \frac{4.77}{1.7} = 2.81 \text{ [N/mm}^2\text{]}$ ✓

$UC = \frac{2.61}{2.81} = 0.93 \text{ [-]}$ ✓ $\leftrightarrow 0.93 \text{ [-]}$ (sheet)

• Verification in cross-direction

Compression caused by self-weight

1. Roof: $g = \text{weight} \cdot \text{span} = 3.88 \cdot 1.96 = 7.60 \text{ [kN/m]}$ ✓

2. Floors: $g = \text{weight} \cdot \text{span} \cdot \text{amount} = 4.7 \cdot 1.96 \cdot 4 = 36.85 \text{ [kN/m]}$ ✓

3. Walls: $g = \frac{\text{thickness}}{1000} \cdot \text{storey height} \cdot \text{number} \cdot \text{brickwork}$
 $= \frac{220}{1000} \cdot 2.8 \cdot 5 \cdot 19 = 58.52 \text{ [kN/m]}$ ✓

Total = $g_{tot} = 7.60 + 36.85 + 58.52 = 102.97 \text{ [kN/m]}$ ✓

$\sigma_{N, n} = \frac{g_{tot}}{\text{wall thickness}} = \frac{102.97}{220} = 0.47 \text{ [N/mm}^2\text{]}$ ✓

$\sigma_{N, ed} = \sigma_{N, n} \cdot \gamma_G = 0.47 \cdot 0.9 = 0.42 \text{ [N/mm}^2\text{]}$ ✓ $\leftrightarrow 0.42$ (sheet)

Figure X.38: Verification excel overcapacity S (No additional room) (5)

Order /
 Blad nr 6/6
 Deel Verificatie parameter study (1)
 Datum 25/04/2023

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Tension caused by wind

$C_s(d) = 1$ [-]
 $g_p(6) = 0.9$ [N/m²] \Rightarrow 6 stories
 $C_{f,p} = 0.8$ [-]
 $C_{f,e} = -0.7$ [-]
 $C_{f,vel} = 0.8 - (-0.7) = 1.5$ [-]

75% of span transfers
 to wall (see report)

$q_{wind} = C_s(d) \cdot g_p \cdot C_{f,p} \cdot \text{span} \cdot 2 \cdot 0.75 = 1 \cdot 0.9 \cdot 1.5 \cdot 1.96 \cdot 2 \cdot 0.75 = 3.97$ kN/m
 $M_{wind} = \frac{1}{2} q l^2 = \frac{1}{2} \cdot 3.97 \cdot 19.6^2 = 762.56$ [kNm]
 $M_{wind,act} = 762.56 \cdot 10^6 \cdot 1.4 \cdot 1.07 \cdot 10^9$ [Nmm] S
 $W_{wall} = \frac{1}{6} t_{wall} h_{wall}^3 = \frac{1}{6} \cdot 220 \cdot (1035 \cdot 1000)^3 = 4.32 \cdot 10^9$ [mm³] S
 $\sigma_{M,act} = \frac{M}{W} = \frac{1.07 \cdot 10^9}{4.32 \cdot 10^9} = 0.25$ [N/mm²] S \Leftrightarrow 0.25 [N/mm²] (sheet)

$UC_{cross} = \frac{0.25}{0.42} = 0.59$ [-] S \Leftrightarrow 0.59 (sheet)

Compression maximum

$\sigma_{c,max} = 0.25 + 0.42 = 0.67$ [N/mm²]
 $f_{c,d} = 2.8$ [N/mm²]
 $UC = \frac{0.67}{2.8} = 0.24$ [-] S \Leftrightarrow 0.24 (sheet)

Figure X.39: Verification excel overcapacity S (No additional room) (6)

X.5.8.2: Including additional room

Order ✓
 Blad nr 1 / 5
 Deel Verification parameter study stability (2)
 Datum 25/04/2023



Verification excel stability study

- Brickwork walls ;
- Additional room
- Concrete Beam Floors

• Parameters geometry

Ratio: Depth / length stab. wall = 4.5 [-]

Depth building = 10.85 [m]

Width unit building = 18.00 [m]

Storey height = 2.8 [m]

Number of floors - existing building = 4 [-]

Number of layers - top floor = 2 [-]

Contributing angle = 30°

Number of buildings between dilutions = 2 [-]

Height existing building = 5 · 2.8 = 14.00 [m] ✓

Height extended building = 7 · 2.8 = 19.60 [m] ✓

SHEET X.5.4

↳ principle valid for sheet X.5.4 & X.5.6

Contributing width

1st floor: $\tan(30) \cdot 2.8 = 1.62$ [m] ✓

other floors: $\tan(20) \cdot (2 \cdot 2.8) = 3.23$ [m] ✓

• Wall parameters

wall thickness = 220 [mm]

length stab. wall = $\frac{\text{depth building}}{\text{Ratio}} = \frac{10.85}{4.5} = 2.41$ [-] ✓

- longitudinal direction

NFA = 4.26 [-]

d = 2.41 [m]

$c = \frac{NFA - 0.5}{2} = \text{*(NFA = NFL)}$

$= \frac{4.26 - (2 \cdot 41 \cdot 0.5)}{2} = 1.53$ [m]

- cross-direction

half floor span = 1.96 [m]

Figure X.40: Verification excel overcapacity S (Additional room) (1)

Order ✓
 Blad nr 2/5
 Deel Verificatie parameter study stability (2)
 Datum 25/04/2023

• Structural material properties

Brickwork = $19 \text{ [kN/m}^3\text{]}$
 Concrete = $24 \text{ [kN/m}^3\text{]}$
 Concrete floor = $4.7 \text{ [kN/m}^2\text{]}$
 Concrete roof = $3.88 \text{ [kN/m}^2\text{]}$
 Timber floor = $0.5 \text{ [kN/m}^2\text{]}$
 Timber roof = $0.56 \text{ [kN/m}^2\text{]}$

• Safety factors (NEN 200)

Permanent : $\gamma_G = 1.35$
 Variable : $\gamma_Q = 1.4$

1. Verification in longitudinal direction
 Compression caused by self-weight.

1. Roof structure
 point load left/right = NFA · contr. width other · load
 $= 4.26 \cdot 3.23 \cdot 3.88 = 53.39 \text{ [kN]} \quad S$

2. Floors

1st floor
 point load left/right = NFA · contr. width 1st · load
 $= 4.26 \cdot 1.62 \cdot 4.7 = 32.44 \text{ [kN]} \quad S$

Other floors
 point load left/right = NFA · contr. width other · load
 $= 4.26 \cdot 3.23 \cdot 4.7 = 64.67 \text{ [kN]} \quad S$

Total
 point load left/right = $32.44 + 3 \cdot 64.67 = 226.45 \text{ [kN]} \quad S$

3. Walls

Stability wall
 $\gamma = \frac{\text{thickness}}{100} \cdot \text{storey height number} \cdot \text{pb. coeff.}$
 $= \frac{220}{1000} \cdot 2.8 \cdot 5 \cdot \gamma = 58.52 \text{ [kN/m]} \quad S$

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Figure X.41: Verification excel overcapacity S (Additional room) (2)

Order ✓
 Blad nr 3/5
 Deel Verificatie parameter study stability (2)
 Datum 25/04/2023



Pointloads from contributing walls

1st Floor: pointload L/R = $\frac{\text{thickness}}{1000} \cdot \text{storey height (cont. 2)} \cdot \text{price}$
 $= \frac{220}{1000} \cdot 2,8 \cdot (1,62 \cdot 2) \cdot 15 = 37,92 \text{ [kN]} \quad \text{S}$

Other floors: pointload L/R = $\frac{\text{thickness}}{1000} \cdot \text{storey length} \cdot \text{amount (cont. other 2)} \cdot \text{price}$
 $= \frac{220}{1000} \cdot 2,8 \cdot 3 \cdot (3,23 \cdot 2) \cdot 15 = 226,82 \text{ [kN]} \quad \text{S}$

4. Total overview of loads

Pointload L/R = $53,39 + 226,45 + 37,92 + 226,82 = 544,58 \text{ [kN]} \quad \text{S}$
 g-load = $50,52 \text{ [kN/m]} \quad \text{S}$

→ Compressive Force SLS = pointload L/R + $\frac{1}{2} g \cdot L_{\text{wall}}$
 $= 544,58 + 0,5 \cdot 50,52 \cdot 2,41 = 615,10 \text{ [kN]} \quad \text{S}$

→ Compressive Force ULS = $615,10 \cdot 0,9 = 553,59 \text{ [kN]} \quad \text{S}$
 $\hookrightarrow 615,45 \text{ [kN]} \quad \text{(sheet)}$

B Tension caused by wind

$C_s \cdot C_d = 1 \quad [-]$
 $g_p = 0,5 \text{ kN/m}^2$
 $C_{f,p} = 0,8$
 $C_{f,e} = 0$
 $C_{f,\text{tot}} = 0,8 - 0 = 0,8 \quad [-]$

Wind load = $C_s \cdot C_d \cdot g_p \cdot C_{f,\text{tot}} \cdot \text{depth building} / \text{number of buildings per elevation}$
 $= 1 \cdot 0,5 \cdot 0,8 \cdot 10,85 / 2 = 3,9 \text{ [kN/m]} \quad \text{S}$

$M_{\text{wind}} = \frac{1}{2} g L^2 = \frac{1}{2} \cdot 3,9 \cdot 19,60^2 = 751,03 \text{ [kNm]} \quad \text{S}$

$M_{\text{wind-ed}} = 751,03 \cdot 0,4 = 1051,44 \text{ [kNm]}$

Tension per side = $\frac{M_{\text{wind-ed}}}{L_{\text{wall}}} = \frac{1051,44}{2,41} = 436,28 \text{ [kN]} \quad \text{S}$

$u_c = \frac{436,28}{553,59} = 0,79 \quad \text{S} \quad \hookrightarrow 0,79 \text{ (sheet)} \quad \hookrightarrow 436,64 \text{ [kN]} \quad \text{(sheet)}$

Figure X.42: Verification excel overcapacity S (Additional room) (3)

Order ✓
 Blad nr 4/5
 Deel Verification parameter study stability (2)
 Datum 25/04/2023



Maximum compression

$$F_{c, max} = 436.28 + 553.90 = 990.18 \text{ [kN]} \quad \text{S}$$

$$\text{Area} = (\text{Centr. width} \cdot 2 \cdot 1000) \cdot \text{thickness}$$

$$= (1.62 \cdot 2 \cdot 1000) \cdot 220 = 712800 \text{ [mm}^2\text{]} \quad \text{S}$$

$$\sigma_N = \frac{F_{c, max}}{\text{Area}} = \frac{990.18 \cdot 10^3}{712800} = 1.39 \text{ [N/mm}^2\text{]} \quad \text{S} \Leftrightarrow 1.39 \text{ [N/mm}^2\text{]} \text{ (sheet)}$$

Brickwork ($F_{bc} = 10$; $M20$) = $4.77 \text{ [N/mm}^2\text{]}$
 Material factor = 1.7
 $f_{cd} = \frac{4.77}{1.7} = 2.81 \text{ [N/mm}^2\text{]} \quad \text{S}$

$$\text{UC} = \frac{1.39}{2.81} = 0.49 \quad \text{S} \Leftrightarrow 0.5 \text{ (sheet)}$$

• Verification in cross-direction

Compression caused by self-weight

Roof: $g = \text{load} \cdot \text{span} = 2.88 \cdot 1; g_6 = 7.60 \text{ [kN/m]} \quad \text{S}$
 Floors: $g = \text{load} \cdot \text{span} \cdot \text{amount} = 4.7 \cdot 1.6 \cdot 4 = 36.85 \text{ [kN/m]} \quad \text{S}$
 Walls: $g = \frac{\text{thickness} \cdot \text{storey height} \cdot \text{amount} \cdot \rho_w \cdot g_c}{1000}$
 $\frac{220}{1000} \cdot 2.8 \cdot 5 \cdot 15 = 58.52 \text{ [kN/m]} \quad \text{S}$

Total: $g = 7.60 + 36.85 + 58.52 = 102.97 \text{ [kN/m]} \quad \text{S} \Leftrightarrow 102.97 \text{ [kN/m]} \text{ (sheet)}$

$$\sigma_{N, s} = \frac{g}{\text{wall thickness}} = \frac{102.97}{220} = 0.47 \text{ [N/mm}^2\text{]} \quad \text{S}$$

$$\sigma_{N, ed} = 0.47 \cdot 0.9 = 0.42 \text{ [N/mm}^2\text{]} \quad \text{S} \Leftrightarrow 0.42 \text{ [N/mm}^2\text{]} \text{ (sheet)}$$

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Figure X.43: Verification excel overcapacity S (Additional room) (4)

Order /
 Blad nr 5/5
 Deel Verificatie parameter study stability (2)
 Datum 25/04/2023

VAN RAADGEVENDE
ROSSUM INGENIEURS

Tension caused by wind

$$C_s C_d = 1.0$$

$$q_p = 0.5 \text{ kN/m}^2$$

$$C_{f, tot} = 0.8 + 0.7 = 1.5 \text{ [-]} \quad \rightarrow 75\% \text{ of span to wall (see report)}$$

$$q_{wind} = C_s C_d q_p C_{f, tot} s_{pan} \cdot 2 \cdot 0.75$$

$$= 1 \cdot 0.5 \cdot 1.5 \cdot 19.6 \cdot 2 \cdot 0.75 = 357 \text{ [kNm]} \quad \&$$

$$M_{wind, ed} = \frac{1}{2} q l^2 \cdot \gamma_Q = \frac{1}{2} \cdot 3.57 \cdot (19.6 \text{ m})^2 \cdot 1.4 = 1.07 \cdot 10^9 \text{ [Nmm]} \quad \&$$

$$W_{wall} = \frac{1}{6} \cdot t \cdot h^2 = \frac{1}{6} \cdot 220 \cdot (10.85 \cdot 1000)^2 = 4.31 \cdot 10^9 \text{ [mm}^3] \quad \&$$

$$\sigma_m = \frac{M}{W} = \frac{1.07 \cdot 10^9}{4.31 \cdot 10^9} = 0.25 \text{ [N/mm}^2] \quad \& \Leftrightarrow 0.25 \text{ [N/mm}^2] \text{ (steel)}$$

$$UC = \frac{0.25}{0.42} = 0.59 \text{ [-]} \quad \&$$

Compression maximum

$$\sigma_{comp} = 0.25 + 0.42 = 0.67 \text{ [N/mm}^2]$$

$$f_{cd} = 2.0 \text{ [N/mm}^2]$$

$$uc = \frac{0.67}{2.0} = 0.34 \text{ [-]} \quad \& \Leftrightarrow 0.34 \text{ (steel)}$$

Figure X.44: Verification excel overcapacity S (Additional room) (5)

X.6 Strengthening for stability [Section 6.3.5]

X.6.1 Concrete core

Excel used for preliminary determination of core dimensions		Note: All values change if 1 parameter is changed. So the values are obtained per height.	
General			
3	Safety factor permanent loading	0.9	
4	Safety factor variable loading	1.4	
5	Weight concrete	24 kN/m ³	
Properties elevator			
7	Length_long	2180 mm	
8	Length_cross	2500 mm	
9	Wall thickness	200 mm	
10	Area core	2105000 mm ²	$(B7*B8) - ((B7-2*B9)) * ((B8-2*B9))$
11	I_longitudinal	1.86007E+12 mm ⁴	$((1/12)*B8*B7^3) - ((1/12)*(B8-B9^2))*B7*(2*B9)^3$
12	I_cross	1.79289E+12 mm ⁴	$((1/12)*B7*B8^3) - ((1/12)*(B7-2*B9))*B8*(2*B9)^3$
13	E_concrete	31000 N/mm ²	
14	Reduction factor stiffness openings	0.7	
15	Factor foundation	2	
17	EI_concrete long_reduced	1,04725E+16 Nmm ²	B11*B13*B14
18	EI_concrete cross_reduced	1,38052E+16 Nmm ²	B12*B13*B14
Building properties			
22	Depth building	10.85 m	
23	Width unit building	3.8 m	
Stiffness estimation in cross-direction			
27	Brickwork, E-modules (1000*1k)	4770 N/mm ²	
28	Outer wall left		
29	Length	10.85 m	B22
30	thickness	200 mm	
31	EI	1,11699E+17 Nmm ²	$(1/12)*B30*(B29^3*1000)^3*B27$
32	Outer wall right		
33	Length	10.85 m	B22
34	thickness	200 mm	
35	EI	1,11699E+17 Nmm ²	$(1/12)*B34*(B33^3*1000)^3*B27$
37	Wall (between 2 appartments)		
38	Length	5.43 m	B22/2
39	Thickness	200 mm	
40	EI	1,39824E+16 Nmm ²	$(1/12)*B39*(B38^3*1000)^3*B27$
43	EI_total brickwork	2,3736E+17 Nmm ²	B31+B35+B40
44	EI_total concrete	1,38052E+16 Nmm ²	B18
45	EI_total	2,51105E+17 Nmm ²	B43+B44
46	Part elevator	0.05	B44/B45
Wind properties longitudinal			
51	Area II, Urban	1	
52	C _{1,G}	0.8	
53	C _{1,E}	0	
54	C _{1,sw}	0.8	
55	qp(16.8m)	0.84 kN/m ²	
56	qp(19.6m)	0.9 kN/m ²	
57	qp(22.4m)	0.93 kN/m ²	
58	Buildings linked	2	
62	Height [m]	19.6	
63	q _w [kN/m]	3.91	$851*B54*B55*B22/B58$
64	M _{w,ed} [Nmm]	1050370272	$(1/2)*B63*B4*A63^2*10^6$
65	w [mm]	13.76	$(1/8)*B63*(A63^4*1000^4*B15)/(S8517)$
66	UC	39.2	$(1/8)*B64*(A64^4*1000^4*B15)/(S8517)$
67	Formula I	0.35	F63/F63
68	Formula I	0.54	F64/F64
69	Weight structure_ED	576523.8	
71	Continue	om [N/mm ²]	
72	om [N/mm ²]	0.83	$(0.61*S857/2)/(S8511)$
73	om [N/mm ²]	1.13	$(0.64*S857/2)/(S8511)$
74	om [N/mm ²]	0.42	$(0.48*S855*1000)/(10^6)*B3$
75	om [N/mm ²]	0.48	$(0.48*S855*1000)/(10^6)*B3$
76	om [N/mm ²]	0.27	$(0.27*S858/10)$
77	om [N/mm ²]	0.27	$(0.27*S858/10)$
78	om [N/mm ²]	0.27	$(0.27*S858/10)$
79	om [N/mm ²]	0.27	$(0.27*S858/10)$
80	om [N/mm ²]	0.27	$(0.27*S858/10)$
81	om [N/mm ²]	0.27	$(0.27*S858/10)$
82	om [N/mm ²]	0.27	$(0.27*S858/10)$
83	om [N/mm ²]	0.27	$(0.27*S858/10)$
84	om [N/mm ²]	0.27	$(0.27*S858/10)$
85	om [N/mm ²]	0.27	$(0.27*S858/10)$
86	om [N/mm ²]	0.27	$(0.27*S858/10)$
87	om [N/mm ²]	0.27	$(0.27*S858/10)$
88	om [N/mm ²]	0.27	$(0.27*S858/10)$
89	Height [m]	19.6	
90	q _w [kN/m]	1.24	$861*B84*B85*B22/B47$
91	M _{w,ed} [Nmm]	358170392.4	$(1/2)*B83*B4*A83^2*10^6$
92	w [mm]	5.57	$(1/8)*B83*(A83^4*1000^4*B15)/(S8518)$
93	UC	39.2	$(1/8)*B84*(A84^4*1000^4*B15)/(S8518)$
94	Formula I	0.29	F90/F90
95	Formula I	0.14	F90/F90
96	Formula I	0.44	C95-E95-G95
97	Formula I	-0.41	C96-E96-G96
98	Formula I	-0.41	C96-E96-G96
99	Reinforcement calculation_Cross		
100	19.6m	0	$(0.41*S95+0.95*B97*B99,0)$
101	19.2m	0	$(0.41*S95+0.95*B97*B99,0)$
102	18.8m	0	$(0.41*S95+0.95*B97*B99,0)$
103	18.4m	0	$(0.41*S95+0.95*B97*B99,0)$
104	18.0m	0	$(0.41*S95+0.95*B97*B99,0)$
105	17.6m	0	$(0.41*S95+0.95*B97*B99,0)$
106	17.2m	0	$(0.41*S95+0.95*B97*B99,0)$
107	16.8m	0	$(0.41*S95+0.95*B97*B99,0)$
108	16.4m	0	$(0.41*S95+0.95*B97*B99,0)$
109	16.0m	0	$(0.41*S95+0.95*B97*B99,0)$
110	15.6m	0	$(0.41*S95+0.95*B97*B99,0)$
111	15.2m	0	$(0.41*S95+0.95*B97*B99,0)$
112	14.8m	0	$(0.41*S95+0.95*B97*B99,0)$
113	14.4m	0	$(0.41*S95+0.95*B97*B99,0)$
114	14.0m	0	$(0.41*S95+0.95*B97*B99,0)$
115	13.6m	0	$(0.41*S95+0.95*B97*B99,0)$
116	13.2m	0	$(0.41*S95+0.95*B97*B99,0)$
117	12.8m	0	$(0.41*S95+0.95*B97*B99,0)$
118	12.4m	0	$(0.41*S95+0.95*B97*B99,0)$
119	12.0m	0	$(0.41*S95+0.95*B97*B99,0)$
120	11.6m	0	$(0.41*S95+0.95*B97*B99,0)$
121	11.2m	0	$(0.41*S95+0.95*B97*B99,0)$
122	10.8m	0	$(0.41*S95+0.95*B97*B99,0)$
123	10.4m	0	$(0.41*S95+0.95*B97*B99,0)$
124	10.0m	0	$(0.41*S95+0.95*B97*B99,0)$
125	9.6m	0	$(0.41*S95+0.95*B97*B99,0)$
126	9.2m	0	$(0.41*S95+0.95*B97*B99,0)$
127	8.8m	0	$(0.41*S95+0.95*B97*B99,0)$
128	8.4m	0	$(0.41*S95+0.95*B97*B99,0)$
129	8.0m	0	$(0.41*S95+0.95*B97*B99,0)$
130	7.6m	0	$(0.41*S95+0.95*B97*B99,0)$
131	7.2m	0	$(0.41*S95+0.95*B97*B99,0)$
132	6.8m	0	$(0.41*S95+0.95*B97*B99,0)$
133	6.4m	0	$(0.41*S95+0.95*B97*B99,0)$
134	6.0m	0	$(0.41*S95+0.95*B97*B99,0)$
135	5.6m	0	$(0.41*S95+0.95*B97*B99,0)$
136	5.2m	0	$(0.41*S95+0.95*B97*B99,0)$
137	4.8m	0	$(0.41*S95+0.95*B97*B99,0)$
138	4.4m	0	$(0.41*S95+0.95*B97*B99,0)$
139	4.0m	0	$(0.41*S95+0.95*B97*B99,0)$
140	3.6m	0	$(0.41*S95+0.95*B97*B99,0)$
141	3.2m	0	$(0.41*S95+0.95*B97*B99,0)$
142	2.8m	0	$(0.41*S95+0.95*B97*B99,0)$
143	2.4m	0	$(0.41*S95+0.95*B97*B99,0)$
144	2.0m	0	$(0.41*S95+0.95*B97*B99,0)$
145	1.6m	0	$(0.41*S95+0.95*B97*B99,0)$
146	1.2m	0	$(0.41*S95+0.95*B97*B99,0)$
147	0.8m	0	$(0.41*S95+0.95*B97*B99,0)$
148	0.4m	0	$(0.41*S95+0.95*B97*B99,0)$

Figure X.45: Strengthening with concrete core_Excel sheet

X.6.2 Concrete thickened stability wall

	A	B	C	D	E	F	G	H	
1	Sheet for preliminary design stability wall concrete								
2	General								
3	Safety factor variable loading	1.4						Note: All values (except piles) change if 1 parameter is changed. So the values are obtained per height.	
4	Safety factor permanent loading	0.9							
5	Weight concrete	24 kN/m3							
6	Buildings linked	2							
7	Variant	Timber_No exchange room_NFA3.47; Ratio=4.2							
8									
9	Wind loads (sheet core)_kar		Longitudinal						
10	19.6m	3.9 kN/m							
11	22.4m	4.8 kN/m							
12									
13	Wall dimensioning		Formule B						
14	Length wall	2000 mm							
15	Thickness wall	150 mm							
16	l_wall	2,197E+11 mm4			(1/12)*B15*B14^3				
17	E_wall	31000 N/mm2							
18	EI_wall	2,4167E+15 Nmm2			816*B17				
19	Factor foundation	2							
20									
21	Calculation stiffness differences								
22	Brickwork wall								
23	E (*1000E6)	4770 N/mm2							
24	Wall thickness	200 mm							
25	Length wall	2000 mm							
26	Contribution due to wall for stiffness (estimation)								
27	l_outer	2,60571E+11 mm4			((1/12)*B26*B24^3)+((B26*B24)*(B25/2)^2)*2				
28	l_wall	3,22227E+11 mm4			((1/12)*B14*B25^3)				
29	l_total	5,82797E+11 mm4			B27+B28				
30	EI_brickwork	2,77994E+15 Nmm2			B29*B23				
31	EI_total	5,19664E+15 Nmm2			B18*B30				
32	Part wind_load brickwork	0.5			B30/B31				
33	Part wind_load concrete	0.5			B18/B31				
34	* w is reduced based on the stiffnesses								
35	Calculation longitudinal							Tension force [kN] (from sheet appendix X)	
36	Height building [m]	w [mm]*	Formule B	H/500 [mm]	Formule O	UC_SLS	Formule F		
37	19.6	27.69	(1/8)*B10*B33*(A37*1000)^4*SBS19/SBS18	39.2	A37*1000/500	0.71	B37/D37	118.77	
38	22.4	48.45	(1/8)*B11*B33*(A38*1000)^4*SBS19/SBS18	44.8	A38*1000/500	1.08	B38/D38	260.94	
39									
40	Calculation longitudinal (continue)		Self-weight of wall/side [kN]	Formule B	Tension [kN]	Formule O	Fe8500 [N/mm2]	Area reinf. [mm2/m/row]	Formule G
41	>>>	59.0	((B15/1000)*(B14/2/1000)*B5*14)*B4	59.80	H37-B41	435	52.9	D41*1000/(F41*2)*(SBS14/2000)	
42		59.0	((B15/1000)*(B14/2/1000)*B5*14)*B4	201.97	H38-B42	435	178.6	D42*1000/(F42*2)*(SBS14/2000)	
43									
44									
45	Calculation needed tension piles_Not parametric								
46	Capacity tension pile	100 kN							
47	Longitudinal direction		Tension force [kN]	piles_needed	Applied piles/wall				
48	19.6m	19.6	0.60	4					
49	22.4m	183.32	1.82	4					
50									
51									
52									
53	Estimation diameter piles								
54	Compressive capacity	10 N/mm2							
55	Height								
56	Number of piles/ side	Compressive force/pile [kN]	Area needed [mm2]	Formule O	Applied diameter [mm]	Area applied [mm2]			
57	19.6	1	396.00	39600	C57*1000/B54	273	58535		
58	22.4	2	467.44	46744	C58*1000/B54	273	58535		
59									
60	Results								
61	Height building	Wall thickness [mm]	Wall length [mm]	Tension [kN]	Area reinf [mm2/m/row]				
62	19.6	150	2600	59.8	52.9				
63	22.4	200	2600	183.32	161.3				

Figure X.46: Strengthening with concrete wall_Excel sheet

X.6.3 Steel bracings

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Dimensioning of bracings_ longitudinal direction													
2	Safety factor G													
3	Safety factor G													
4	Height	q_sls	q_ULS	Formule C	Width [mm]	Ø Ties [mm]	Profile	Nties [kN]*	Atie [mm2]	Formule I	σN [N/mm2]	Formule K	UC_ULS_ties	Formule M
5	19.6	3.905	5.47	B5*B2	2600	40	HEA200_235	146.27	1256.64	(1/4)*PI()*F5^2	116.40	H5*1000/I5	0.50	K5/235
6	22.4	4.036	5.65	B6*B2	2600	40	HEA240_235	166.2	1256.64	(1/4)*PI()*F6^2	132.26	H6*1000/I6	0.56	K6/235
7														
8														
9	Continue		Nprof. [kN]*	Aprof [mm2]	σN [N/mm2]	Formule D	UC_ULS_prof	Formule F	w [mm]*	H/500 [mm]	Formule I	UC_SLS	Formule K	
10	>>>	404.11	5380	75.11	B10*1000/C10	0.32	D10/235	27.4	39.2	A5*1000/500	0.70	H10/I10		
11		545.18	7680	70.99	B11*1000/C11	0.30	D11/235	30.4	44.8	A6*1000/500	0.68	H11/I11		
12														
13														
14	Calculation tension piles													
15	Capacity	100 kN												
16	Height	Tension force [kN]G_Ed [kN]*	G_bracings_ed [kN]**	Force to foundation [kN]	piles needed	Applied/truss								
17	19.6	404.11	787.82	37.36	79.03	0.79								
18	22.4	545.18	287.82	42.93	214.43	2.14								
19														
20	* downward load for a timber floor_no exchange room. NFA=3.47; Ratio=4.2													
21	** Excel for impact calculation													
22														
23	Estimation pile diameter													
24	Compressive capacity	10 N/mm2												
25	Height building													
26	piles	Compressive force/pile [kN]	Formule C	Area needed [mm2]	Formule E	Applied diameter [mm]	Applied area [mm2]	Formule H						
27	19.6	2	364.60	(B17+C17+D17)/B27	36460	C27*1000/B24	273	58535	(1/4)*PI()*G27^2					
28	22.4	2	437.97	(B18+C18+D18)/B28	43797	C28*1000/B24	273	58535	(1/4)*PI()*G28^2					

Figure X.47: Strengthening with steel bracings_Excel sheet

X.6.4 Verification appendix X.6

X.6.4.1 Concrete core

Order /

Blad nr 1/84

Deel Verification strengthening with concrete core

Datum 28/04/2023

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SHEET X 6.1

0 General safety factors

permanent: $\gamma_G = 0.9$ [-]

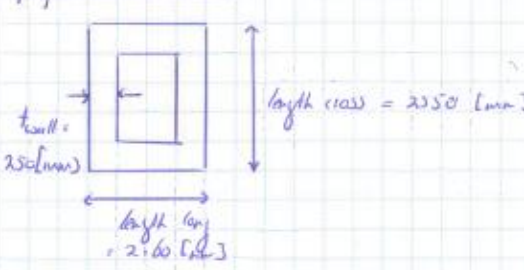
variable: $\gamma_Q = 1.4$ [-]

Building properties

Depth building = 10.85 [m]

Width building = 10 [m]

0 Properties elevator



length (wall) = 2550 [mm]


length (core) = 2160 [mm]

$\gamma_c = 24$ [kN/m³]

Area core = $(2550 \cdot 2160) - ((2550 - 2 \cdot 195) \cdot (2160 - 2 \cdot 195)) = 550000 - (2050 \cdot 1660) = 2105000$ [mm²] $\leftrightarrow 2105000$ [mm²]

$I_{long} = (\frac{1}{12} \cdot 2550 \cdot 2160^3) - (\frac{1}{12} \cdot (2550 - 500) \cdot (2160 - 500)^3) = 2.14 \cdot 10^{12} - 147.8 \cdot 10^{12} = 1.36 \cdot 10^{12}$ [mm⁴] $\leftrightarrow 1.36 \cdot 10^{12}$ [mm⁴]

$I_{cross} = (\frac{1}{12} \cdot 2160 \cdot 2550^3) - (\frac{1}{12} \cdot (2160 - 500) \cdot (2550 - 500)^3) = 2.98 \cdot 10^{12} - 1.15 \cdot 10^{12} = 1.73 \cdot 10^{12}$ [mm⁴] $\leftrightarrow 1.73 \cdot 10^{12}$ [mm⁴]



w \rightarrow width

h \rightarrow height

Reduction factors

Openings $\rightarrow 0.7$ [-]

$E_{concrete} = 11000$ [N/mm²]

Factor foundation = 2 [-]

\hookrightarrow \hookrightarrow deflection calculation

$EI_{long, reduced} = E_c I_{long, openings} = 11000 \cdot 1.36 \cdot 10^{12} \cdot 0.7 = 1.047 \cdot 10^{16}$ [Nmm²]

$EI_{cross, reduced} = E_c I_{cross, openings} = 11000 \cdot 1.73 \cdot 10^{12} \cdot 0.7 = 1.301 \cdot 10^{16}$ [Nmm²]

0 Stability in longitudinal direction

\hookrightarrow Assumed: core provides all stiffness as $E_{core} \gg E_{wall}$

Wind load

$C_{s,d} = 1.5$ [-]

$C_{p,d} = 0.8$ [-]

$C_{pe} = 0$ [-]

$g_p (s, b) = 0.5$ [kN/m²]

$C_{F, tot} = 0.8 - 0 = 0.8$ [-]

Figure X.48: Strengthening concrete core_Verification (1)


Order ✓
 Blad nr 2/14
 Deel Verificatie strengthening with core
 Datum 28/04/2023



Buildings linked between dilatation = 2 -

$$q_{wind} = \frac{C_{ed} C_{pe} C_{pe} \cdot g_{wind} \cdot \text{Depth building}}{\text{Number of buildings dilatation } D} = \frac{1 \cdot 0,9 \cdot 0,9 \cdot 10 \cdot 25}{2} = 3,91 \text{ [kN/m]} \quad \text{✓}$$

⇒ deflections SLS



$$w = \frac{1}{8} \frac{q_{wind} H^4}{EI_{reduced}} \cdot \text{factor } \alpha_{deflection}$$

$$= \frac{1}{8} \cdot \frac{3,91 \cdot (19,6 \cdot 1000)^4}{1,36 \cdot 10^{10}} \cdot 0,2 = 10,61 \text{ [mm]} \quad \text{✓}$$

↔ 13,76 [mm]
(rounded numbers)

$$w_{allowed} = \frac{H}{500} = \frac{19,6 \cdot 1000}{500} = 39,2 \text{ [mm]} \quad \text{✓}$$

$$w_c = \frac{10,61}{39,2} = 0,27 \text{ [-]} \quad \text{✓} \quad \leftrightarrow 0,35$$

⇒ Tension ULS

$$M_{wind} = \frac{1}{2} q_{wind} H^2 \cdot \gamma_Q = \frac{1}{2} \cdot 3,91 \cdot (19,6 \cdot 1000)^2 \cdot 1,4 = 1051445720 \text{ [Nm]} \quad \text{✓}$$

3,91 initial ✓ ↔ 1050310272 [Nm]

$$\sigma_M = \frac{M_{ed} \cdot \left(\frac{L_{long}}{3}\right)}{I_{long}} = \frac{1051445720 \cdot 2160}{1,36 \cdot 10^{12}} = 0,83 \text{ [N/mm}^2\text{]} \quad \text{✓}$$

$$\sigma_{N, core} = \frac{H \cdot \gamma_G \cdot 1000}{10} \cdot \gamma_R = \frac{19,6 \cdot 24 \cdot 1000}{10,6} \cdot 0,9 = 0,42 \text{ [N/mm}^2\text{]} \quad \text{✓}$$

Normal force structure = 576523,8 [N] ⇒ $\sigma_{N, add} = \frac{N}{A} = \frac{576523,8}{2109000} = 0,27 \text{ [N/mm}^2\text{]} \quad \text{✓}$
 (excl steel stability)

Tension = $\sigma_N - \sigma_{N, add} - \sigma_M = 0,42 + 0,27 - 0,83 = -0,14 \text{ [N/mm}^2\text{]} \quad \text{✓}$
 tension ↔ exl [N/mm²] (steel)

Tension force = $\sigma_T \cdot A_{steel} \cdot f_{t,add} = 0,14 \cdot 2550 \cdot 250 = 92250 \text{ [N]} \quad \text{✓}$
 FeB500 ⇒ 435 [N/mm²]

Area, S. = $\frac{F_{tension}}{435 \cdot 2 \cdot \left(\frac{A_{steel}}{1000}\right)} = \frac{92250}{435 \cdot 2 \cdot \left(\frac{2550}{1000}\right)} = 40,22 \text{ [mm}^2\text{/m /row]} \quad \text{✓}$
 ↑ 2 rows ↔ 35,32 [mm²/m /row]

Figure X.49: Strengthening concrete core_Verification (2)

Order /
 Blad nr 3 / 04
 Deel Verfication stahtlenig with concrete core
 Datum 28/04/2023



o Stability in cross-direction

Wind load to core

$E_{b, \text{concrete}} = 1000$	$f_c = 4770$	$[N/mm^2]$
length	x	th. cross
Wall 1: 10.85 [m]		220 [mm]
Wall 2: "		"
Wall 3: 5.43 [m]		220 [mm]
Core		

$$E_{I, \text{wall}} = \frac{1}{12} \cdot \text{th. cross} \cdot (L \cdot 1000)^3 \cdot E_b$$

$$E_{I, \text{wall 1}} = \frac{1}{12} \cdot 220 \cdot (10.85 \cdot 1000)^3 \cdot 4770 = 1.11 \cdot 10^{17} [Nmm^2]$$

$$E_{I, \text{wall 2}} = 1.11 \cdot 10^{17} [Nmm^2]$$

$$E_{I, \text{wall 3}} = \frac{1}{12} \cdot 220 \cdot (5.43 \cdot 1000)^3 \cdot 4770 = 1.40 \cdot 10^{16} [Nmm^2]$$

$$E_{I, \text{core}} = 1.38 \cdot 10^{16} [Nmm^2]$$

$E_{I, \text{total}} = 2.45 \cdot 10^{17} [Nmm^2]$

$I_{\text{part core}} = \frac{1.38 \cdot 10^{16}}{2.45 \cdot 10^{17}} = 0.0558 \rightarrow 0.05$

Wind load cross-direction

$C_{s, \text{cl}} = 1.0$
 $C_{f, \text{p}} = 0.8$
 $C_{f, \text{e}} = 0.7$
 $g_p (s, 0) = 0.9 [kN/m^2]$
 $C_{f, \text{tot}} = 0.8 + 0.7 = 1.5 [-]$

Wind = $C_{s, \text{cl}} \cdot C_{f, \text{tot}} \cdot g_p (s, 0)$ width building : part core
 $= 1.0 \cdot 1.5 \cdot 0.9 \cdot 10^3 \cdot 0.055 = 1.34 [kN/m]$

Deflection SLS

$w = \frac{1}{8} \frac{q_{\text{wind}} H^4}{EI_{\text{reduced}}} \cdot 2 = \frac{1}{8} \frac{1.34 \cdot (5.6 \cdot 1000)^4}{1.38 \cdot 10^{16}} = 3.58 [mm]$

$w_{\text{allowed}} = H/500 = 4600/500 = 39.2 [mm]$
 $w/c = 3.58/39.2 = 0.0909 [-]$

Tension ULS

$M_{\text{wind, uls}} = \frac{1}{2} q_{\text{wind}} H^2 \cdot 4 = \frac{1}{2} \cdot 1.34 \cdot (5.6 \cdot 1000)^2 \cdot 1.4 = 360342080 [Nmm]$

$\sigma_m = \frac{M_{\text{ul}} \cdot \left(\frac{t_{\text{cross}}}{2}\right)}{I_{\text{cross}}} = \frac{360342080 \cdot \left(\frac{220}{2}\right)}{1.77 \cdot 10^{12}} = 0.26 [N/mm^2]$

$\sigma_{u, \text{core}} = 0.42 [N/mm^2]$
 $\sigma_{u, \text{add}} = 0.27 [N/mm^2]$
 } see calc longitudinal direction

$\sigma_T = \sigma_m - \sigma_{u, \text{core}} - \sigma_{u, \text{add}} = 0.26 - 0.42 - 0.27 = -0.43 [N/mm^2]$
 } No tension

Figure X.50: Strengthening concrete core_Verfication (3)

Order ✓
 Blad nr 4/4
 Deel Verification strengthening with concrete core
 Datum 28/04/2023

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o Piles needed (estimation)

⇒ tensile force

Tension force = 89250 [N]
 pile capacity tension = 100000 [N]

↳ needed piles = $89250 / 100000 = 0.89$ ⇒ 2 needed at least per side.
 (4 total)

⇒ Compression force

$$F_c = \frac{(0.83 + 0.42 + 0.27) \cdot 250 \cdot 2550}{2} = 484500 \text{ [N]} \quad \text{8}$$

$$\phi \text{ piles} = 273 \text{ [mm]} \Rightarrow A_{\text{area}} = \frac{\pi \cdot 273^2}{4} = 58535 \text{ [mm}^2\text{]}$$

$$\text{↳ } \sigma_c = \frac{484500}{58535} = 8.3 \text{ [N/mm}^2\text{]} < 10 \text{ [N/mm}^2\text{]} \quad \text{8}$$

↳ compressive capacity

Figure X.51: Strengthening concrete core_Verification (4)

X.6.4.2 Concrete thickened stability wall

Order /
 Blad nr 1 / 2
 Deel Verification strengthening with concrete wall
 Datum 28/04/2023

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General properties

$$\gamma_R = 1.4 \quad \gamma_G = 0.9 \quad \left. \begin{array}{l} \\ \end{array} \right\} \text{NEN 8700}$$

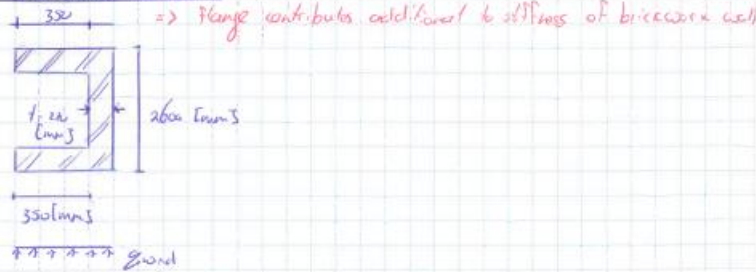
$E_{\text{concrete}} = 24 \text{ [kN/mm}^2\text{]}$
 buildings/occupation = 2 [C-3]
 Variant: Timber; No exchange room; NEA = 3.47; Ratio depth/wall = 4.2 [-]

Wind load = 3.9 [kN/m] \Rightarrow See verification steel concrete core (in longitudinal direction)

Wall properties concrete

$E_{\text{wall}} = 26000 \text{ [kN/m}^2\text{]}$
 $C_{\text{wall}} = 150 \text{ [kN/m]}$
 $I_{\text{wall}} = \frac{1}{12} t_{\text{wall}} h_{\text{wall}}^3 = \frac{1}{12} \cdot 150 \cdot 2600^3 = 2.157 \cdot 10^9 \text{ [mm}^4\text{]}$
 $E_{\text{wall,c}} = 11000 \text{ [N/mm}^2\text{]}$
 $E_{\text{wall,t}} = I_{\text{wall}} E_c = 2.157 \cdot 10^9 \cdot 11000 = 2.4167 \cdot 10^{15} \text{ [Nmm}^2\text{]}$

Wind distribution to wall



$$I_{\text{brick}} = \left(\frac{1}{12} \cdot 220 \cdot 2600^3 \right) + 2 \cdot \left(\left(\frac{1}{12} \cdot 350 \cdot 2600^3 \right) + \left(220 \cdot 350 \cdot \left(\frac{2600}{2} \right)^2 \right) \right)$$

$$= 5.83 \cdot 10^9 \text{ [mm}^4\text{]} \quad \text{or} \quad 5.83 \cdot 10^9 \text{ [mm}^4\text{]}$$

$E_{\text{brick}} = 1000 f_{c,c} = 4770 \text{ [N/mm}^2\text{]}$
 $E_{\text{brick}} = 4770 \cdot 5.83 \cdot 10^9 = 2.78 \cdot 10^{13} \text{ [Nmm}^2\text{]}$

$E_{\text{total}} = E_{\text{brick}} + E_{\text{c}} = 2.4167 \cdot 10^{15} + 2.78 \cdot 10^{13} = 5.158 \cdot 10^{15} \text{ [Nmm}^2\text{]}$

part concrete wall = $\frac{E_{\text{c}}}{E_{\text{total}}} = \frac{2.4167 \cdot 10^{15}}{5.158 \cdot 10^{15}} = 0.46 \quad \text{or} \quad 0.5 \text{ (steel)}$

Figure X.52: Strengthening by concrete thickened stability wall_Verification (1)

Order /
 Blad nr 2/2
 Deel Verification strengthening with concrete wall
 Datum 28/04/2023

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o Calculation deflection (SLS)

$$q_w = 0.46 \cdot 3g = 1.754 \text{ [kN/m]}$$

$$w = \frac{1}{8} \frac{q_w L^4}{EI_c} \cdot 2 = \frac{1}{8} \cdot \frac{1.754 \cdot 10600^4}{2.4167 \cdot 10^9} \cdot 2 = 27.3g \text{ [mm]} \quad \text{S}$$

↳ Factor Foundation ↔ 27.6g [mm] (sheet)

$$w_{\text{allowed}} = 11/500 = 15600/500 = 39.2 \text{ [mm]}$$

$$UC = 27.3g / 39.2 = 0.7 \text{ [-]}$$

o Reinforcement needed (ULS)

$$\text{Tension Force} = 118.77 \text{ [kN]} \text{ (sheet stability)}$$

↳ Design value; combines already existing normal and tension force

$$\text{Self-weight wall} = \frac{t_{\text{wall}}}{1000} \cdot \frac{L_{\text{wall}}}{1000} \cdot \gamma_c \cdot t_{\text{wall}} \cdot \gamma_c$$

$$= \frac{200}{1000} \cdot \frac{2600}{1000} \cdot 24 \cdot 14.00 \cdot 0.9 = 59.0 \text{ [kN]} \quad \text{S}$$

$$\text{Resulting tensile force} = 118.77 - 59.0 = 59.77 \text{ [kN]} \quad \text{S} \quad \leftrightarrow 59.80 \text{ [kN]} \text{ (sheet value)}$$

$$f_{ct,BS00} \Rightarrow f_{yd} = \frac{f_{yc}}{\gamma_s} = \frac{5000}{1.15} = 435 \text{ [N/mm}^2\text{]}$$

$$A_{\text{reinf.}} = \frac{F_{\text{tension}} \cdot 1000}{f_{yd} \cdot \left(\frac{t_{\text{wall}}}{2000}\right) \cdot 2} = \frac{59.77 \cdot 1000}{435 \cdot \left(\frac{2000}{2000}\right) \cdot 2} = 52.85 \text{ [mm}^2\text{/m /row]}$$

S ↔ 52.9 [mm²/m /row]

o Piles

$$\text{Tensile capacity: } 100 \text{ [kN]} \Rightarrow 59.8/100 = 0.6 \text{ piles} \Rightarrow 4 \text{ applied.} \quad \text{S}$$

$$\text{Compressive capacity } 10 \text{ [N/mm}^2\text{]} \Rightarrow 2 \cdot 273 \text{ per side}$$

$$= 2 \cdot 2 \cdot 273 \cdot 10 / 1000 = 1170 \text{ [kN]}$$

$$\Rightarrow \text{max compression} = 792 \text{ [kN]} = \text{sheet} + 59.8 \text{ [kN]}$$

S sufficient

Figure X.53: Strengthening by concrete thickened stability wall_Verification (2)

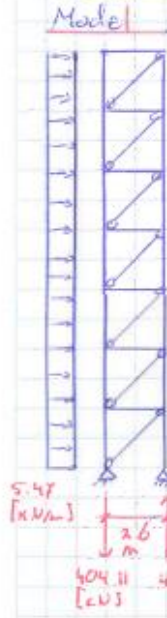
X.6.4.3 Steel bracings

Order ✓
 Blad nr 1 / 2
 Deel Verification strengthening with stability bracings
 Datum 28/04/2023



SHEET x63

o Verification matrix frame model



Check model

$$q_{w-sls} = 3.906 \text{ [kN/m]}$$

$$q_{w-uls} = q_{w-sls} \cdot \gamma_0 = 3.906 \cdot 1.4 = 5.47 \text{ [kN/m]}$$

$$M = \frac{1}{2} q_{w-uls} H^2 = \frac{1}{2} \cdot 5.47 \cdot 6^2 = 1050.6 \text{ [kNm]}$$

$$N = \frac{M}{z} = \frac{1050.6}{2.60} = 404.11 \text{ [kN]}$$

$$\sum F_U = 0 \quad q_w \cdot H = V_{bottom}$$

$$V_{bottom} = 5.47 \cdot 6 = 107.21 \text{ [kN]}$$

o Verification corresponding excel sheet

1. Ties (ULS)

$$N_{ties} = 146.27 \text{ [kN]} \quad (\text{Matrix Frame})$$

$$A_{ties} = 40 \text{ [mm}^2] \Rightarrow A_{ties} = \frac{1}{2} \pi \cdot 40^2 = 1256.64 \text{ [mm}^2]}$$

$$\sigma_{N, ties} = \frac{N}{A} = \frac{146.27 \cdot 1000}{1256.64} = 116.40 \text{ [N/mm}^2]}$$

$$S_{235} \Rightarrow f_{y,t} = 235 \text{ [N/mm}^2]}$$

$$UC_{ties} = \frac{116.40}{235} = 0.50 \text{ [-]} \quad \Leftrightarrow \quad 0.50 \text{ [-]} \quad (\text{steel value})$$

o 2. Columns (HEA200) (ULS)

$$N_{column} = 404.11 \text{ [kN]}$$

$$A_{column} = 5380 \text{ [mm}^2]}$$

$$S_{235} \Rightarrow f_{y,t} = 235 \text{ [N/mm}^2]}$$

$$\sigma_N = \frac{N}{A} = \frac{404.11 \cdot 1000}{5380} = 75.11 \text{ [N/mm}^2]}$$

$$UC = \frac{75.11}{235} = 0.32 \text{ [-]} \quad \Leftrightarrow \quad 0.32 \text{ (steel value)}$$

o 3. Webstiffeners (SL5)

$$w = 27.4 \text{ [mm]} \quad (\text{matrix frame})$$

$$w_{allow} = H/500 = 19600/500 = 39.2 \text{ [mm]}$$

$$UC = \frac{27.4}{39.2} = 0.70 \text{ [-]} \quad \Leftrightarrow \quad 0.70 \text{ (steel value)}$$

Figure X.54: Strengthening with steel bracings_Verification (1)

Order /
 Blad nr 2/2
 Deel Verification strengthening with stability bracings
 Datum 28/04/2023

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o Calculation piles needed

$$\begin{aligned} \text{Tension Force} &= 404.11 \text{ [kN]} \\ G_{\text{structure}} &= -207.82 \text{ [kN]} \\ G_{\text{bracings}} &= -37.26 \text{ [kN]} \end{aligned}$$

$$\text{Resulting tension} = 75.03 \text{ [kN]} \quad S$$

$$\text{tensile capacity per pile} = 100 \text{ [kN]}$$

$$\hookrightarrow \text{Needed piles} = \frac{75.03}{100} = 0.75 \Rightarrow 4 \text{ piles minimum needed. } S$$

o Pile diameter

$$\text{Compressive force per pile} = \frac{404.11 + 2 \cdot 207.82 + 37.26}{2} = 364.60 \text{ [kN]} \quad S$$

$2 \leftarrow 2 \text{ per side}$

$$\text{pile } \phi 273 \Rightarrow \text{Area} = \pi \cdot 273^2 = 50534.94 \text{ [mm}^2\text{]}$$

$$\sigma_p = \frac{N}{A} = \frac{364.60 \cdot 1000}{50534.94} = 6.22 \text{ [N/mm}^2\text{]} < 10 \text{ [N/mm}^2\text{]} \quad S$$

\uparrow Compressive capacity

Figure X.55: Strengthening with steel bracings_Verification (2)

X.7 Environmental impact HSB and CLT [Section 7.2.2]

X.7.1 HSB


	A	B	C	D	E	F	G
1	Environmental impact Parelhof Heerhugowaard						
2	HSB-construction						
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19	Inventarisatie structural materials						
20	HSB-wall						
21	Columns						
22	Dimensions	38x89x3000	mm3	Formules B			
23	Spacing		600 mm per pair				
24	Length wall		11700 mm				
25	Number of columns per wall		39	B25*(B24)			
26	Volume of columns (1 single wall)		0,40 m3	B26*B38*89*3000*10^-9			
27	Beams						
28	Number of beams (top and bottom)		4				
29	Dimensions	38x89	mm2				
30	Length wall		11700 mm	B25			
31	Volume of beams (1 single wall)		0,16 m3	B32*B30*38*89*10^-9			
32	Woodbased panel						
33	Thickness		15 mm				
34	Number of sides		2				
35	Height		3000 mm				
36	Length		11700 mm				
37	Volume of woodbased panel (1 single wall)		1,05 m3	B36*B37*B38*B39*10^-9			
38	Stability wall						
39	Length	1/3 of normal wall					
40	Volume columns		0,07 m3	((B25/3)/B24)*38 * 89* 3000*10^-9			
41	Volume beams		0,03 m3	(B32/3)*[B30/2]*38*89*10^-9			
42	Volume panels		0,35 m3	B36*B37*B38*(B39/3)*10^-9			
43	Total overview walls						
44	Number of walls		10				
45	Number of stability walls		4				
46	Volume columns		3,83 m3	(B27*(B49-1))+[B44*B50]			
47	Volume beams		1,53 m3	(B33*(B49-1))+[B45*B50]			
48	Volume panels		11,93 m3	(B40*B49)+[B46*B50]			
49	HSB floor & roof						
50	Beams						
51	Dimensions	71 x 221	mm2				
52	Total considered length		35100 mm				
53	Spacing		600 mm				
54	Amount of beams		20				
55	Total volume beams		11,02 m3	71*221*B60*B62*10^-9			
56	Floor panels						
57	Thickness		18 mm				
58	Area		410,67 m2	B60*B25*10^-6			
59	Volume floorpanels		7,39 m3	B67*(B66/1000)			
60	Total (Considering floor and roof similar)						
61	Volume beams		22,03 m3	B63*2			
62	Volume floorpanels		14,78 m3	B68*2			
63	Inventarisatie insulation materials						
64	HSB-facade wall (outer walls)						
65	Area (single side)		35,1 m2	B38*B39*10^-6			
66	Insulation						
67	Gypsum plasterboard	90 mm thick layer	35,1 m2	B77			
68	Gypsum plasterboard	15 mm one layer	35,1 m2	B77			
69	HSB-facade wall (inner walls)						
70	Area (single side)		35,1 m2	B77			
71	Insulation						
72	Gypsum plasterboard	90 mm thick layer 2 layers	70,2 m2	B85*2			
73	Gypsum plasterboard	15 mm two layers	70,2 m2	B85*2			
74	Total overview walls						
75	Number of facade walls		2				
76	Number of inner walls (stab. wall = 1/3 of normal wall)		9,33				
77	Insulation 90 mm		725,4 m2	(B79*B93)+(B87*B94)			
78	Gypsum plasterboard 15 mm		725,4 m2	(B81*B93) + (B89*B94)			
79	Floor						
80	Area single		410,67 m2	B67			
81	Insulation 100mm		410,67 m2	B101			
82	Fermacell layer		410,67 m2	B101			
83	Total						
84	Number of floor/roof		2				
85	Insulation 100mm		821,34 m2	B102*B106			
86	Fermacell layer 22 mm		821,34 m2	B103*B106			

Figure X.56: Environmental impact estimation HSB construction_Excel sheet (1)

Life cycle inventory list					
111	Structural elements				
112	Columns and beams	Considered wood	kg CO2-eq/m3 (A1 - A3)	Formule C	
113	Columns and beams	Sawn wood	-1460		
114	Floor panels	Plywood	-784		
115					
116					
117	Element	Volume [m3]	GWP [kg CO2-eq] (A1-A3)		
118	Columns	3,83	-5584,56	B118*C113	
119	Beams	23,56	-34397,86	B119*C113	
120	Floor panels	26,72	-19611,10	B120*C114	
121		54,10	-95959,53	SOM(C118:C120)	
122					
123					
124	Non-structural elements				
125	Walls	Insulation 90 mm	1,09		
126	Walls	Gypsum plasterboard 15 mm	2,59		
127	Floor/roof	Insulation 100 mm	1,21		
128	Floor/roof	Fermacell 22 mm	2,01		
129					
130					
131	Element	Area [m2]	GWP [kg CO2-eq] (A1-A3)		
132	Insulation 90 mm	725,40	790,69	B132*C125	
133	Insulation 100 mm	821,34	998,82	B133*C127	
134	Gypsum plasterboard 15 mm	725,40	1878,79	B134*C126	
135	Fermacell	821,34	1650,89	B135*C128	
136			5314,19	SOM(C132:C135)	
137					
138					
139					
140	Environmental impact				
140	Intended lifetime	28 y		Formule B	
141	Floor area	410,67 m2		B67	
142					
143	Environmental impact_Structural		-63,09 *10^-1 kg CO2-eq/y/m2 floor area	(C121)/(B140*B141)*10	
144	Environmental impact_Non-structural		5,63 *10^-1 kg CO2-eq/y/m2 floor area	(C136)/(B140*B141)*10	
145	Environmental impact_total		-57,47 *10^-1 kg CO2-eq/y/m2 floor area	B144*B145	
146					
147					
148	Results				
149	23y	Structural	-63,09	5,63	percent non-structural from structural
150	42y		-34,55	3,08	-57,47 *10^-1 kg CO2-eq/y/m2 floor area
151	50y		-29,02	2,59	-31,47 *10^-1 kg CO2-eq/y/m2 floor area
152					-26,43 *10^-1 kg CO2-eq/y/m2 floor area
153	Material use (parameterisch)				
154	23 y		5,73 *10^-3 m3 wood/ m2 floor area/storey	B121*10^9/(23*B142)	
155	42 y		3,14 *10^-3 m3 wood/ m2 floor area/storey	B121*10^9/(42*B142)	

Figure X.57: Environmental impact estimation HSB construction_Excel sheet (2)

X.7.2 CLT

	A	B	C	D	E
1	Environmental impact Parelhof Heerhugowaard_CLT				
2					
3	CLT-construction				
4					
5					
6					
7					
8					
9					
10					
11					
12					
13					
14					
15					
16					
17					
18					
19					
20	Inventarisation structural materials				
21	CLT-wall				
22	Normal wall				
23	Lenght		11700 mm		Formules B
24	Height		3000 mm		
25	Thickness		120 mm		
26	Volume (single wall)		4,21 m3		B23*B24*B25*10^-9
27					
28	Stability wall				
29	Lenght		3900 mm		B23/3
30	Height		3000 mm		B24
31	Thickness		120 mm		B25
32	Volume (single wall)		1,40 m3		B29*B30*B31*10^-9
33					
34	Floor and roof				
35	Considered width		35100,00 mm		
36	Area		410,67 m2		B35*B23*10^-6
37	Thickness		120 mm		B25
38	Volume (single floor/roof)		49,28 m3		B36*(B37/1000)
39					
40	Total overview of materials				
41	Number of normal walls		10 -		
42	Number of stability walls		4,00 -		
43	Number of slabs		2 -		
44	Volume walls		47,74 m3		(B26*B41) + (B32*B42)
45	Volume slabs		98,56 m3		B38*B43
46					
47					
48	Inventarisation insulation materials				
49	Facade wall (outer walls)				
50	Area		35,1 m2		B23*B24*10^-6
51	Insulation				

Figure X.58: Environmental impact estimation CLT construction_Excel sheet (1)

	A	B	C	D	E
52	Insulation 170 mm		35,1 m2		B50
53	Gypsum plasterboard				
54	Gypsum board 15 mm one layer		35,1 m2		B50
55					
56	Inner wall				
57	Area		35,1 m2		B50
58	Gypsum plasterboard				
59	Gypsum board 15 mm two layers		70,2 m2		B57*2
60					
61	Total walls				
62	Number of facade walls	2	-		
63	Number of inner walls	9,33	-		
64					
65	Insulation 170 mm		70,2 m2		B52*B62
66	Gypsum plasterboard 15 mm		725,4 m2		(B54*B62) + (B59*B63)
67					
68	Floor				
69	Area single		410,67 m2		B36
70	Insulation 100 mm		410,67 m2		B69
71	Fermacell 22 mm		410,67 m2		B69
72					
73	Total				
74	Number of floor/roof	2	-		
75	Insulation 100 mm		821,34 m2		B70*B74
76	Fermacell 22 mm		821,34 m2		B71*B74
77					
78					
79	Life cycle inventory list				
80	Structural materials	Considered wood	kg CO2-eq/m3 (A1 - A3)		Formules C
81	All materials	CLT		-714	
82					
83	Element	Volume [m3]		GWP [kg CO2-eq] (A1-A3)	
84	Walls	47,74		-34083,50	B84*C81
85	Slabs	98,56		-70372,41	B85*C81
86		146,30		-104455,92	SOM(C84:C85)
87					
88					
89	Non-structural elements	Considered aspect		kg CO2-eq/m2 (A1-A3)	
90	Walls	Insulation 170 mm		2,06	
91	Walls	Gypsum plasterboard 15 mm		2,59	
92	Floor/roof	Insulation 100 mm		1,21	
93	Floor/roof	Fermacell 22 mm		2,01	
94					
95					
96	Element	Area [m2]		GWP [kg CO2-eq] (A1-A3)	
97	Insulation 100 mm	821,34		993,82	B97*C92
98	Insulation 170 mm	70,20		144,61	B98*C90
99	Gypsum plasterboard 15 mm	725,40		1878,79	B99*C91
100	Fermacell	821,34		1650,89	B100*C93
101				4668,11	SOM(C97:C100)
102					
103	Environmental impact				
104	Intended lifetime	23	y		Formule B
105	Floor area	410,67	m2		B36
106					
107	Environmental impact_Structural	-110,59	*10 ⁻¹ kg CO2-eq/y/m2 floor area		(C86/(B104*B105))*10
108	Environmental impact_Non-structural	4,94	*10 ⁻¹ kg CO2-eq/y/m2 floor area		(C101/(B104*B105))*10
109	Environmental impact_total	-105,65	*10 ⁻¹ kg CO2-eq/y/m2 floor area		B108+B109
110					
111		Structural	Non-structural	Total	Percent non-structural from structural
112	23y	-110,59	4,94	-105,65	-4,5
113	42y	-60,56	2,71	-57,85	-4,5
114	50y	-50,87	2,27	-48,6	-4,5
115					
116					
117	Material use_23	15,49	*10 ⁻³ m3 wood/ m2 floor area/y		B86*10 ^{^3} /(B106*23)
118	Material use_42	8,48	*10 ⁻³ m3 wood/ m2 floor area/y		B86*10 ^{^3} /(B106*42)

Figure X.59: Environmental impact estimation CLT construction_Excel sheet (2)

X.8 Environmental impact strengthening methods [Section 7.2.4]

X.8.1 Concrete core

	A	B	C	D	E	F	G	H
1	Impact of concrete core							
2	Core							
3	Height	Length_long [m]	Length_cross [m]	Thickness [m]	Volume [m3]	Formule E		
4	19,6	2,16	2,55	0,25	41,26	((B4*C4) - ((B4-(2*D4)) * (C4-(2*D4))))*A4		
5	22,4	2,16	2,55	0,25	47,15	(((B5*C5) - ((B5-(2*D5)) * (C5-(2*D5))))*A5		
6	Assumed building dimensions							
7	Width	18 m	Formule B					
8	Depth	10,85 m						
9	Floor area_added floor	195,3 m2						
10	Height [m]	added floors	New floor area [m2]	Formule C				
11	19,6	2	390,6	B13*B10				
12	22,4	3	585,9	B14*B10				
13	Environmental impact							
14	Shadow costs CO2	0,05 C/ kg CO2-eq						
15	Reinforced concrete	210 kg CO2-eq/m3						
16	RFL_short	23 year						
17	RFL_long	42 year						
18	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
19	19,6 m	96,44	((E4*\$B518/(\$B519*C13))*100	52,81	(E4*\$B518/(\$B520*C13))*100			
20	22,4 m	73,48	((E5*\$B518/(\$B519*C14))*100	40,24	(E5*\$B518/(\$B520*C14))*100			
21	Piles							
22	Pile dimensions							
23	Weight steel	7850 kg/m3						
24	Height [m]	Length pile [mm]	Number of piles	Diameter [mm]	Thickness steel ring [mm]	Volume steel [m3]	Weight steel [kg]	Volume Concrete [m3]
25	19,6	20000	4	273	10	0,17	1297,20	1,01
26	22,4	20000	4	324	10	0,20	1548,74	1,45
27	Environmental impact							
28	Reinforced concrete	210 kg CO2-eq/m3						
29	Steel	719 kg CO2-eq/ 1000 kg						
30	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
31	19,6 m	12,73	((G31*\$B536/1000) + (\$B35*H31))/(\$B520*C13))*100	6,97	(((G31*\$B536/1000) + (\$B35*H31))/(\$B520*C13))*100			
32	22,4 m	10,53	((G32*\$B536/1000) + (\$B35*H32))/(\$B520*C14))*100	5,76	(((G32*\$B536/1000) + (\$B35*H32))/(\$B520*C14))*100			
33	Total impact							
34	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
35	19,6 m	109,17	B23+H39	59,79	D23+H39			
36	22,4 m	84,01	B24+H40	46,00	D24+H40			

Figure X.60: Environmental impact concrete core_Excel sheet

X.8.2 Concrete thickened stability wall

	A	B	C	D	E	F	G	H
1	Impact of concrete wall in Longitudinal direction							
2	Wall							
3	Building height	Wall height [m]	Length [m]	Thickness [m]	Volume [m3]	Formule E		
4	19,6	14	2,6	0,15	5,46	B4*C4*D4		
5	22,4	14	2,6	0,2	7,28	B5*C5*D5		
6	Assumed building dimensions							
7	Width	18 m	Formule B					
8	Depth	10,85 m						
9	Floor area_added floor	195,3 m2						
10	Height [m]	added floors	New floor area [m2]	Formule C				
11	19,6	2	390,6	B13*B10				
12	22,4	3	585,9	B14*B10				
13	Environmental impact							
14	Reinforced concrete	210 kg CO2-eq/m3						
15	RFL_short	23 year						
16	RFL_long	42 year						
17	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
18	19,6 m	12,76	((G31*\$B536/1000)+H31*\$B535))/(\$B520*C13))*100	6,99	(((G31*\$B536/1000)+H31*\$B535))/(\$B520*C13))*100			
19	22,4 m	11,34	((G32*\$B536/1000)+H32*\$B535))/(\$B520*C14))*100	6,21	(((G32*\$B536/1000)+H32*\$B535))/(\$B520*C14))*100			
20	Piles							
21	Pile dimensions							
22	Weight steel	7850 kg/m3						
23	Height [m]	Length pile [mm]	Number of piles	Diameter [mm]	Thickness steel ring [mm]	Volume steel [m3]	Weight steel [kg]	Volume Concrete [m3]
24	19,6	20000	4	273	10	0,17	1297,20	1,01
25	22,4	20000	4	273	10	0,17	1297,20	1,01
26	Environmental impact							
27	Reinforced concrete	210 kg CO2-eq/m3						
28	Steel	719 kg CO2-eq/ 1000 kg						
29	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
30	19,6 m	12,73	((G31*\$B536/1000)+H31*\$B535))/(\$B520*C13))*100	6,97	(((G31*\$B536/1000)+H31*\$B535))/(\$B520*C13))*100			
31	22,4 m	8,49	((G32*\$B536/1000)+H32*\$B535))/(\$B520*C14))*100	4,85	(((G32*\$B536/1000)+H32*\$B535))/(\$B520*C14))*100			
32	Total impact							
33	Height [m]	Carbon footprint_23 y [*10^-2 €/y/m2 new floor area]	Formule B	Carbon footprint_42 y [*10^-2 €/y/m2 new floor area]	Formule D			
34	19,6 m	25,50	B23+H39	13,96	D23+H39			
35	22,4 m	19,83	B24+H40	10,86	D24+H40			

Figure X.61: Environmental impact concrete wall_Excel sheet

X.8.3 Steel bracings

	A	B	C	D	E	F	G	H	I	J	K	
1	Impact of steel bracings in Longitudinal direction											
2	Amount of weight	Note the bracings only continue to the 14 meters										
3	Weight steel	7850 kg/m3										
4	Ties											
5	Building height [m]	Tie diameter [mm]	Cross-section [mm2]	Formule C	Length 1 tie [m]	Number of ties	Volume ties [m3]	Formule G	Weight [kg]	Formule I		
6	19.6	40	1256,64	$1/4 \cdot \pi \cdot d^2$	3,82	10	0,05	$(C7^{10} \cdot d^4) \cdot E \cdot \pi \cdot F$	376,83	$G7^{10} \cdot B$		
7	22.4	40	1256,64	$1/4 \cdot \pi \cdot d^2$	3,82	10	0,05	$(C8^{10} \cdot d^4) \cdot E \cdot \pi \cdot F$	376,83	$B3^{10} \cdot B$		
8	Columns/ beams											
9	Building height [m]	Profile	Area [mm2]	Length column [m]	Number of column	Length beam [m]	Number of beams	Total length profiles [m]	Formule H	Vol. profiles_total [m3]	Weight [kg]	
10	19.6	HEA200_235	8580	2,8	10	2,6	5	41	$(D12^{10} \cdot H) \cdot (F12^{10} \cdot G12)$	0,22	1731,55	
11	22.4	HEA240_235	7680	2,8	10	2,6	5	41	$(D13^{10} \cdot H) \cdot (F13^{10} \cdot G13)$	0,31	2471,81	
12	Assumed building dimensions											
13	Width	18 m										
14	Depth	10,85 m										
15	Floor area_added floor	195,3 m2										
16	Height											
17	added floors	New floor area [m2]										
18	19.6	2										
19	22.4	8										
20	Environmental impact											
21	Ties	298 kg CO2-eq/1000 kg										
22	Profiles	718 kg CO2-eq/1000 kg										
23	RFL_short	23 year										
24	RFL_long	40 year										
25	CO2_impact_23 y											
26	Environmental impact	*10^-2 kg CO2-eq/m2 new floor area	Formule B	CO2_impact_42 y							Formule D	
27	19.6 m	15,11	$((I7^{10} \cdot S8526 / 1000) + (K12^{10} \cdot S8527 / 1000) + (S8528 \cdot C21)) \cdot 100$	$(I7^{10} \cdot S8526 / 1000) + (K12^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							8,27	
28	22.4 m	14,02	$((I8^{10} \cdot S8526 / 1000) + (K13^{10} \cdot S8527 / 1000) + (S8528 \cdot C21)) \cdot 100$	$(I8^{10} \cdot S8526 / 1000) + (K13^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							7,68	
29	Piles											
30	Pile dimensions											
31	Weight steel	7850 kg/m3										
32	Height [m]	Length pile [mm]	Number of piles	Diameter [mm]	Thickness steel ring [mm]	Volume steel [m3]	Weight steel [kg]	Volume Concrete [m3]				
33	19.6	20000	4	273	10	0,17	1297,20	1,01				
34	22.4	20000	4	273	10	0,17	1297,20	1,01				
35	Environmental impact											
36	Reinforced concrete	210 kg CO2-eq/m3										
37	Steel	718 kg CO2-eq/ 1000 kg										
38	CO2_impact_23 y											
39	Environmental impact	*10^-2 kg CO2-eq/m2 new floor area	Formule B	CO2_impact_42 y							Formule D	
40	19.6 m	12,79	$((G41^{10} \cdot S8546 / 1000) + (H41^{10} \cdot S8545) / (C21^{10} \cdot S8539)) \cdot 100$	$(I7^{10} \cdot S8526 / 1000) + (K12^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							6,97	
41	22.4 m	8,49	$((G42^{10} \cdot S8546 / 1000) + (H42^{10} \cdot S8545) / (C22^{10} \cdot S8539)) \cdot 100$	$(I8^{10} \cdot S8526 / 1000) + (K13^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							6,65	
42	Total impact											
43	CO2_impact_23 y											
44	Environmental impact	*10^-2 kg CO2-eq/m2 new floor area	Formule B	CO2_impact_42 y							Formule D	
45	19.6 m	27,84	$S51 \cdot B34$	$(I7^{10} \cdot S8526 / 1000) + (K12^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							15,25	$D51 \cdot B34$
46	22.4 m	22,51	$S51 \cdot B34$	$(I8^{10} \cdot S8526 / 1000) + (K13^{10} \cdot S8527 / 1000) + (S8529 \cdot C21) \cdot 100$							13,33	$D51 \cdot B34$

Figure X.62: Environmental impact steel bracings_Excel sheet

X.9 LCA-calculation concrete building [Section 7.3]

X.9.1 Actual building

	A	B	C	D	E	F	G	H	I	J	
1	Material use Swietenhof										
2											
3	Foundation										
4	Piles	Diameter [mm]	Area [m2]	Length [m]	Amount [-]	Volume_tot [m3]					
5		410	0,13	20	32	84,50					
6		410	0,13	21	8	22,18					
7		410	0,13	22	4	11,62					
8		410	0,13	21,5	24	68,13					
9		460	0,17	22	30	109,69					
10		460	0,17	23,5	7	27,34					
11		460	0,17	21,5	28	100,05					
12		460	0,17	21	5	17,45					
13						440,94	m3 concrete (C30/37; XC4)				
14											
15	Foundation beams	Width [mm]	Height [mm]	Length [mm]	Amount [-]	Volume_tot [m3]					
17		600	500	11356	2	6,81					
18		500	500	11356	5	14,20					
19		500	500	4500	1	1,13					
20		500	500	1970	9	4,43					
21		500	500	9836	8	19,67					
22		500	500	9856	1	2,46					
23		650	500	9836	1	3,20					
24		450	500	94550	1	21,27					
25		350	500	1500	4	1,05					
26		350	500	4650	1	0,81					
27		400	500	4650	1	0,93					
28		500	500	4650	2	2,33					
29		350	500	19850	1	3,47					
30						81,77	m3 concrete (C30/37; XC4)				
31											
32											
33	Ground floor										
34											
35	Ground floorsystem	Length [mm]	width [mm]	thickness [mm]	amount [-]	area_tot [m2]					
37	Hollow core slab	9836	4650	350	10	457,37					
38		11356	4650	350	5	264,03					
39	Deleting area	4000	4650	350	1	18,60					
40		4650	1500	350	2	6,98					
41						695,83	m2 hollow core slab				
42											
43	Concrete_in situ					Volume_tot [m3]					
44		4000	4650	250	1	4,65	m3 concrete (C30/37; XC1)				
45											
46											
47	Walls	Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]					
48	In-situ	11475	3270	200	2	15,01					
49		10110	3270	200	2	13,22					
50		10110	3270	250	9	74,38					
51		9856	3270	250	1	8,06					
52		1365	3270	180	4	3,21					
53		11356	3270	250	4	37,13					
54		4650	3270	250	1	3,80					
55						154,82	m3 concrete (C30/37; XC1)				
56											
57	Prefab	1520	3270	390	7	13,57					
58		2300	3270	180	1	1,35					
59		4650	3270	200	1	3,04					
60		2845	3270	150	2	2,79					
61		1600	3270	150	2	1,57					
62		200	3270	390	9	2,30					
63		1520	3270	90	4	1,79					
64						26,41	m3 concrete (C45/55; XC1)				
65											
66											
67											
68	1st floor										
69											
70	1st floor	Length [mm]	width [mm]	thickness [mm]	amount [-]	Volume_tot [m3]					
71	In_situ	74590	8491	230	1	145,67					
72		55800	1619	230	1	20,78					
73	Deleted area	4000	595	230	1	0,55					
74		2800	1200	230	9	6,96					
75		1000	500	230	15	1,73					
76		4650	1600	230	1	1,71					
77		4650	1619	150	3	3,39					
78		4650	1520	150	4	4,24					
79		4650	1520	230	3	4,88					
80						163,14	m3 Concrete (C30/37; XC1)				
81											
82	Prefab	4650	1619	150	1	1,13	m3 concrete (C45/55; XC1)				
83											
84											
85											
86	Walls	Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]					
87	In_situ	10110	2770	200	4	22,40					
88		10110	2770	250	10	70,01					
89		8491	2770	250	4	23,52					
90		1365	2770	180	6	4,08					
91		4650	2770	250	1	3,22					
92						123,24	m3 Concrete (C30/37; XC1)				
93											
94											

Figure X.63: Environmental impact new concrete building_Excel sheet (1)

95	Prefab								
96		1520	2770	390	11			18,06	
97		200	2770	390	6			1,30	
98	beam	4650	600	250	4			2,79	
99	beam	4650	390	200	4			1,45	
100		1600	2770	150	2			1,33	
101		2845	2770	150	2			2,36	
102		2300	2770	180	1			1,15	
103		1520	2770	90	6			2,27	
104		3000	2770	200	1			1,66	
105								32,38 m3 concrete (C45/55, XC1)	
106									
107									
108	2nd floor								
109	2nd floor	Length [mm]	width [mm]	thickness [mm]	amount [-]			Volume_tot [m3]	
110									
111	In-situ	74590	8491	230	1			145,67	
112	Deleted area	1500	1000	230	1			0,35	
113		4650	1600	230	1			1,71	
114		1000	500	230	15			1,73	
115		4650	1619	150	15			16,94	
116		4650	1619	230	1			1,73	
117		4650	1520	230	14			22,76	
118		4650	1520	150	2			2,12	
119								185,44 m3 Concrete (C30/37, XC1)	
120									
121	Prefab								
122		4650	1619	150	1			1,13	
123		4650	1520	150	2			2,12	
124								3,25 m3 concrete (C45/55, XC1)	
125									
126									
127									
128	Walls	Length [mm]	Height [mm]	Thickness [mm]	Amount [-]			Volume_tot [m3]	
129									
130	In-situ	10110	2770	200	2			11,20	
131		8491	2770	200	2			9,41	
132		8491	2770	250	12			70,56	
133		9856	2770	250	1			6,83	
134		10110	2770	250	1			7,00	
135		1365	2770	180	2			1,36	
136		4650	2770	250	1			3,22	
137								109,58 m3 Concrete (C30/37, XC1)	
138									
139	Prefab								
140		1520	2770	390	15			24,63	
141		2300	2770	180	1			1,15	
142		3000	2770	200	1			1,66	
143		1600	2770	150	2			1,33	
144		2845	2770	150	2			2,36	
145		390	2770	200	2			0,43	
146		1520	2770	90	2			0,76	
147	beam	4650	185	397	16			5,46	
148	beam	4650	200	390	1			0,36	
149								38,15 m3 concrete (C45/55, XC1)	
150									
151									
152	3rd floor								
153	3rd floor	Length [mm]	width [mm]	thickness [mm]	amount [-]			Volume_tot [m3]	
154									
155	In-situ	74590	10110	230	1			173,44	
156	Deleted area	4650	1619	230	1			1,73	
157		1500	1000	230	14			4,83	
158		1000	500	230	15			1,73	
159		4650	1600	230	1			1,71	
160		4650	1520	230	2			3,25	
161								166,70 m3 Concrete (C30/37, XC1)	
162									
163	Prefab	4650	1619	150	1			1,13 m3 concrete (C45/55, XC1)	
164									
165									
166									
167	Walls	Length [mm]	Height [mm]	Thickness [mm]	Amount [-]			Volume_tot [m3]	
168									
169	In-situ	10110	2770	200	4			22,40	
170		10110	2770	250	14			98,02	
171		4650	2770	250	1			3,22	
172		1365	2770	180	2			1,36	
173								125,00 m3 Concrete (C30/37, XC1)	
174									
175	Prefab	1520	2770	390	15			24,63	
176		1520	2770	90	2			0,76	
177		2300	2770	180	1			1,15	
178		4650	2770	200	1			2,58	
179		390	2770	200	2			0,43	
180		1600	2770	150	2			1,33	
181		2845	2770	150	2			2,36	
182								33,24 m3 concrete (C45/55, XC1)	
183									
184									
185	4rd floor								
186	4rd floor	Length [mm]	width [mm]	thickness [mm]	amount [-]			Volume_tot [m3]	
187									
188	In-situ	74590	9856	230	1			169,09	
189		4650	1619	150	14			15,81	
190		4650	1619	230	1			1,73	
191	Deleted area	4650	1600	230	1			1,71	

Figure X.64: Environmental impact new concrete building_Excel sheet (2)

192		1000	500	230	15	1,73	
193						183,19	m3 Concrete (C30/37, XC1)
194							
195	Prefab	4650	1619	150	1	1,13	m3 concrete (C45/55, XC1)
196							
197	Walls						
198		Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]	
199	In-situ	10110	2770	200	2	11,20	
200		8491	2770	200	2	9,41	
201		8491	2770	250	14	82,32	
202		1365	2770	180	2	1,36	
203		4650	2770	250	1	3,22	
204						107,51	m3 Concrete (C30/37, XC1)
205							
206	Prefab	1520	2770	390	15	24,63	
207		1520	2770	90	2	0,76	
208		2300	2770	180	1	1,15	
209		3000	2770	200	1	1,66	
210		1600	2770	150	2	1,33	
211		390	2770	200	2	0,43	
212		2845	2770	150	2	2,36	
213	beam	4650	397	185	15	5,12	
214	beam	4650	422	250	1	0,49	
215						37,94	m3 concrete (C45/55, XC1)
216							
217							
218							
219	5th floor						
220	5th floor	Length [mm]	width [mm]	thickness [mm]	amount [-]	Volume_tot [m3]	
221	In-situ	74590	10110	230	1	173,44	
222		4650	1520	230	2	3,25	
223	Deleted area	4650	1619	230	1	1,73	
224		4650	1600	230	1	1,71	
225		1500	1000	230	14	4,83	
226						168,42	m3 Concrete (C30/37, XC1)
227							
228	Prefab	4650	1619	150	1	1,13	m3 concrete (C45/55, XC1)
229							
230	Walls						
231		Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]	
232	In-situ	10110	2670	200	4	21,59	
233		10110	2670	250	14	94,48	
234		4650	2670	250	1	3,10	
235		1365	2670	180	2	1,31	
236						120,49	m3 Concrete (C30/37, XC1)
237							
238	Prefab	1520	2670	390	15	23,74	
239		1520	2670	90	2	0,73	
240		1600	2670	150	2	1,28	
241		2845	2670	150	2	2,28	
242		390	2670	200	2	0,42	
243						28,45	m3 concrete (C45/55, XC1)
244							
245							
246							
247	6th floor						
248	6th floor	Length [mm]	width [mm]	thickness [mm]	amount [-]	Volume_tot [m3]	
249	In-situ	74590	8936	230	1	153,30	m3 Concrete (C30/37, XC1)
250	Deleted area	1000	800	230	1	0,18	
251						153,12	m3 Concrete (C30/37, XC1)
252							
253	Walls						
254		Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]	
255	In-situ	11356	3600	200	4	15,78	
256		11356	3600	250	14	69,02	
257						84,79	m3 Concrete (C30/37, XC1)
258							
259	Prefab	1520	950	390	16	4,51	
260	beam	4650	399	170	15	4,73	
261	beam	4650	482	424	1	0,95	
262	beam	4650	485	450	16	16,24	
263						26,42	m3 concrete (C45/55, XC1)
264							
265							
266	Roof						
267	Plates						
268		Length [mm]	width [mm]	thickness [mm]	amount [-]	Volume_tot [m3]	
269		74590	6742	5	4	10,06	m3 woodbased panel
270							
271	Timber beams						
272		Length [mm]	Height [mm]	Thickness [mm]	Amount [-]	Volume_tot [m3]	
273		74590	171	20	4	1,02	m3 Sawn timber
274							
275							
276	Summation of volumes/amounts						
277		Ribcassettefloor					
278	Foundation	Area [m2]	C30/37; XC1 [m3]	C30/37; XC4 [m3]	C45/55; XC1	Sawn timber [m3]	Woodbased panel [m3]
279		Piles			440,94		
280		Beams			81,77		+
281		Total			522,71		
282	Groundfloor						
283		Floor	695,83	4,65			
284		Walls		154,82	26,41		+
285		Total	695,83	159,47	26,41		
286	1st floor						
287		Floor		163,14	1,13		
288		Walls		123,24	32,38		+
289		Total		286,38	33,51		

Figure X.65: Environmental impact new concrete building_Excel sheet (3)

290	2nd floor								
291	Floor	185,44		3,25					
292	Walls	109,58		38,15				+	
293	Total	295,02		41,40					
294	3rd floor								
295	Floor	166,70		1,13					
296	Walls	125,00		33,24				+	
297	Total	291,70		34,37					
298	4th floor								
299	Floor	183,19		1,13					
300	Walls	107,51		37,94				+	
301	Total	290,70		39,07					
302	5th floor								
303	Floor	168,42		1,13					
304	Walls	120,49		28,45				+	
305	Total	288,91		29,58					
306	6th floor								
307	Floor	153,12							
308	Walls	84,79		26,42				+	
309	Total	237,91		26,42					
310	Roof-structure								
311	Plates						10,06		
312	Beams					1,02		+	
313	Total					1,02	10,06		
314									
315	Total							+	
316		695,83	1850,09	522,71	230,75	1,02	10,06		
317									
318									
319									
320	New floor area								
321		Depth [mm]	Length [mm]	Additional area [m2]	Area [m2]				
322	4th floor (includes balconies)	11356	74590	-	847,04				
323	5th floor	9991	74590	13,95	759,18				
324	Average area/floor				803,11				
325									
326	Added area for 3-storey building	2409,33	m2						
327	Added area for 4-storey building	1606,22	m2						
328									
329									
330	Values for environmental impact								
331		A1-A3	Unit	Amount	Unit	Total impact [kg CO2-eq]			
332	Ribcasette floor	21,84	kg CO2-eq/m2	695,83	m2	15196,84			
333	C30/37; XC1	210	kg CO2-eq/m3	1850,09	m3	388518,79			
334	C30/37; XC4, XF1	217	kg CO2-eq/m3	522,71	m3	113427,06			
335	C45/55; XC1	250	kg CO2-eq/m3	230,75	m3	57687,62			
336	Sawn timber	-1460	kg CO2-eq/m3	1,02	m3	-1489,77	+		
337	Woodbased panel	-734	kg CO2-eq/m3	10,06	m3	-7382,36			
338						565958,17			
339									
340	RSL	75	year						
341	new construction compared to 3-storey porch flat	31,32	*10 ⁻¹ kg CO2-eq./y/m2 new floor area						
342	new construction compared to 4-storey porch flat	46,98	*10 ⁻¹ kg CO2-eq./y/m2 new floor area						

Figure X.66: Environmental impact new concrete building_Excel sheet (4)

X.9.2 Changed to 4-storey building

Summation of volumes/amounts_Changed to 4-storey						
Ribcassettefloor						
Foundation	Area [m2]	C30/37; XC1 [m3]	C30/37; XC4 [m3]	C45/55; XC1	Sawn timber [m3]	Woodbased panel [m3]
	Piles			440,94		
	Beams			81,77		
	Total		522,71			
Groundfloor						
	Floor	695,83	4,65			
	Walls		154,82	26,41		
	Total	695,83	159,47	26,41		
1st floor						
	Floor		163,14	1,13		
	Walls		123,24	32,38		
	Total		286,38	33,51		
2nd floor						
	Floor		185,44	3,25		
	Walls		109,58	38,15		
	Total		295,02	41,40		
3rd floor						
	Floor		166,70	1,13		
	Walls		125,00	33,24		
	Total		291,70	34,37		
4th floor						
	Floor		153,12			
	Walls		84,79	26,42		
	Total		237,91	26,42		
Roof-structure						
	Plates					10,06
	Beams				1,02	
	Total				1,02	10,06
Total						
		695,83	1270,48	522,71	162,11	1,02
						10,06
New floor area						
	Depth [mm]	Length [mm]	Additional area [m2]	Area [m2]		
	11356	74590	-	847,04		
	9991	74590	13,95	759,18		
	Average area/floor			803,11		
	Added area for 3-storey building			803,11 m2		
Values for environmental impact						
	A1-A3	Unit	Amount	Unit	Total impact [kg CO2-eq]	
	Ribcassette floor	21,84 kg CO2-eq/m2	695,83	m2	15196,84	
	C30/37; XC1	210 kg CO2-eq/m3	1270,48	m3	266799,77	
	C30/37; XC4, XF1	217 kg CO2-eq/m3	522,71	m3	113427,06	
	C45/55; XC1	250 kg CO2-eq/m3	162,11	m3	40526,52	
	Sawn timber	-1460 kg CO2-eq/m3	1,02	m3	-1489,77 +	
	Woodbased panel	-734 kg CO2-eq/m3	10,06	m3	-7382,36	
					427078,05	
	RSL	75 year				
	new construction compared to 3-storey porch flat		70,90	*10^-1 kg CO2-eq./y/m2 new floor area		

Figure X.67: Environmental impact new concrete (changed to 4-storey building) building_Excel sheet

X.9.3 Changed to 5-storey building

Summation of volumes/amounts_Changed to 5 storey building						
Ribcassettefloor						
278	Foundation	Area [m2]	C30/37; XC1 [m3]	C30/37; XC4 [m3]	C45/55; XC1	Sawn timber [m3] Woodbased panel [m3]
279		Piles			440,94	
280		Beams			81,77	
281		Total		522,71		
282	Groundfloor					
283		Floor	695,83	4,65		
284		Walls		154,82	26,41	
285		Total	695,83	159,47	26,41	
286	1st floor					
287		Floor		163,14	1,13	
288		Walls		123,24	32,38	
289		Total		286,38	33,51	
290	2nd floor					
291		Floor		185,44	3,25	
292		Walls		109,58	38,15	
293		Total		295,02	41,40	
294	3rd floor					
295		Floor		166,70	1,13	
296		Walls		125,00	33,24	
297		Total		291,70	34,37	
298	4th floor					
299		Floor		183,19	1,13	
300		Walls		107,51	37,94	
301		Total		290,70	39,07	
302	5th floor					
303		Floor		153,12		
304		Walls		84,79	26,42	
305		Total		237,91	26,42	
306	Roof-structure					
307		Plates				10,06
308		Beams			1,02	
309		Total			1,02	10,06
310	Total					
311			695,83	1561,18	522,71	201,17
312					1,02	10,06
313						
314						
315						
316	New floor area					
317		Depth [mm]	Length [mm]	Additional area [m2]	Area [m2]	
318	floor (includes balconies)	11356	74590	-	847,04	
319	floor (excl. Balconies)	9991	74590	13,95	759,18	
320	Average area/floor				803,11	
321						
322	Added area for 3-storey building	1606,22 m2				
323	Added area for 4-storey building	803,11 m2				
324						
325						
326	Values for environmental impact					
327		A1-A3	Unit	Amount	Unit	Total impact [kg CO2-eq]
328	Ribcassette floor	21,84	kg CO2-eq/m2	695,83	m2	15196,84
329	C30/37; XC1	210	kg CO2-eq/m3	1561,18	m3	327847,38
330	C30/37; XC4, XF1	217	kg CO2-eq/m3	522,71	m3	113427,06
331	C45/55; XC1	250	kg CO2-eq/m3	201,17	m3	50293,03
332	Sawn timber	-1460	kg CO2-eq/m3	1,02	m3	-1489,77
333	Woodbased panel	-734	kg CO2-eq/m3	10,06	m3	-7382,36
334						497892,16
335						
336	RSL	75	year			
337	new construction compared to 3-storey porch flat	41,33	*10 ⁻¹ kg CO2-eq./y/m2 new floor area			
338	new construction compared to 4-storey porch flat	82,66	*10 ⁻¹ kg CO2-eq./y/m2 new floor area			

Figure X.68: Environmental impact new concrete (changed to 5-storey building) building_Excel sheet

X.9.4 Changed to 7-storey building

Summation of volumes/amounts_Changed to 7-storey building						
Ribcassettefloor						
Foundation	Area [m2]	C30/37; XC1 [m3]	C30/37; XC4 [m3]	C45/55; XC1	Sawn timber [m3]	Woodbased panel [m3]
	Piles			440,94		
	Beams			81,77		
	Total			522,71		
Groundfloor						
	Floor	695,83	4,65			
	Walls		154,82		26,41	
	Total	695,83	159,47		26,41	
1st floor						
	Floor		163,14		1,13	
	Walls		123,24		32,38	
	Total		286,38		33,51	
2nd floor						
	Floor		185,44		3,25	
	Walls		109,58		38,15	
	Total		295,02		41,40	
3rd floor						
	Floor		166,70		1,13	
	Walls		125,00		33,24	
	Total		291,70		34,37	
4th floor						
	Floor		183,19		1,13	
	Walls		107,51		37,94	
	Total		290,70		39,07	
5th floor						
	Floor		168,42		1,13	
	Walls		120,49		28,45	
	Total		288,91		29,58	
6th floor (fictive)						
	Floor		168,42		1,13	
	Walls		120,49		28,45	
	Total		288,91		29,58	
7th floor						
	Floor		153,12			
	Walls		84,79		26,42	
	Total		237,91		26,42	
Roof-structure						
	Plates					10,06
	Beams				1,02	
	Total				1,02	10,06
Total						
		695,83	2139,00	522,71	260,33	1,02
						10,06
Values for environmental impact						
	A1-A3	Unit	Amount	Unit	Total impact [kg CO2-eq]	
	Ribcassette floor	21,84	kg CO2-eq/m2	695,83 m2	15196,84	
	C30/37; XC1	210	kg CO2-eq/m3	2139,00 m3	449190,21	
	C30/37; XC4, XF1	217	kg CO2-eq/m3	522,71 m3	113427,06	
	C45/55; XC1	250	kg CO2-eq/m3	260,33 m3	65082,21	
	Sawn timber	-1460	kg CO2-eq/m3	1,02 m3	-1489,77 +	
	Woodbased panel	-734	kg CO2-eq/m3	10,06 m3	-7382,36	
					634024,18	
	RSL	75	year			
	new construction compared to 4-storey porch flat	35,09	*10 ⁻¹ kg CO2-eq./y/m2 new floor area			

Figure X.69: Environmental impact new concrete (changed to 7-storey building) building_Excel sheet

X.10 Structural design of CLT unit [Appendix D.3]

X.10.1 CLT properties

X.10.1.1 3-layered panel

3-layered panel				Standard panel sizes			Weight [kg/m ³]	Formule L
Panel number	Total thickness	1	2	3	Unit	E _{mean}		
1	60	20	20	20	mm	25,2	(G3/1000)*S854	
2	70	20	30	20	mm	29,4	(G4/1000)*S854	
3	80	20	40	20	mm	33,6	(G5/1000)*S854	
4	80	30	20	30	mm	33,6	(G6/1000)*S854	
5	90	30	30	30	mm	37,8	(G7/1000)*S854	
6	100	30	40	30	mm	42	(G8/1000)*S854	
7	100	40	20	40	mm	42	(G9/1000)*S854	
8	110	40	30	40	mm	46,2	(G10/1000)*S854	
9	120	40	40	40	mm	50,4	(G11/1000)*S854	

Layer i	t [mm]	a _i [mm]	Formule B
1	40	40	
2	40	0	
3	40	40	

Property	Value	Formule
h _{CLT}	120 mm	SOM(B13:B15)
h ₁	80 mm	B13+B15
h ₂	40 mm	B14
z ₁	60 mm	B16/2
b ₁	1000 mm	
b ₂	1000 mm	

Sectional properties		
Bending along y-axis		
A _{y,brutto}	120000 mm ²	B20*B16
A _{y,netto}	80000 mm ²	B17*B20
I _{y,net}	1,387E+08 mm ⁴	(B20*(B13^3/12) + (B20*B13*C13^2) + (B20*(B15^3/12) + (B20*B15*C15^2)
I _{y,brutto}	666666667 mm ⁴	(B13+B15)*(B20^3)/12
W _{y,net}	2,311E+06 mm ³	(2*B27)/B16
S _{rolling,y,net}	1,60E+06 mm ³	B20*B13*C13
S _{y,net}	1,60E+06 mm ³	B20*B13*C13
Bending along x-axis		
A _{x,brutto}	120000 mm ²	B21*B16
A _{x,netto}	40000 mm ²	B21*B18
I _{x,net}	5,33E+06 mm ⁴	(B21*(B14^3/12) + (B21*B14*C14^2)
I _{x,brutto}	333333333 mm ⁴	B14*B21^3/12
W _{x,net}	8,89E+04 mm ³	2*B36/B16
Rolling deformation Out-of-plane loading only!		
l _{ref} (rspan)	3500 mm	
V ₁	1,00 -	
V ₃	0,78 -	1/(1+((PI)^2*B7*B15/B41^2)*(B14/B8))
I _{ref}	1,245E+08 mm ⁴	B20*((2*B13^3/12) + ((1+B43)*B13*C13^2))
I _{ref}	39,45 mm	WORTE(B44/B26)

Validation Excel																						
Verification excel with table 3.11 (P 45 CLT Handbook) & 3.14 (p 50 CLT Handbook) (Borgström & Fröbel, 2019)																						
Panel build-up	E _{mean} [kg/m ³]		I _{y,ref} (l _{ref} =2.5m) [mm ⁴]		I _{x,ref} (l _{ref} =2.5m) [mm ⁴]		W _{y,net} [mm ³]		S _{rolling,x,net} [mm ³]		I _{y,net} [mm ⁴]		W _{x,net} [mm ³]		A _{y,net} [mm ²]		A _{x,net} [mm ²]		A _{y,brutto} = A _{x,brutto} [mm ²]			
	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table		
20-20-20	25,2	25	1,64E+07	1,64E+07	20,22	20,2	1,78E+07	1,73E+07	5,78E+05	5,78E+05	4,00E+05	6,67E+05	6,70E+05	2,22E+04	2,20E+04	4,00E+04	4,00E+04	2,00E+04	2,00E+04	6,00E+04	6,00E+04	
20-40-20	33,6	34	3,34E+07	3,34E+07	28,9	28,9	3,73E+07	3,73E+07	9,33E+05	9,33E+05	6,00E+05	5,33E+05	5,33E+05	1,33E+05	1,33E+05	4,00E+04	4,00E+04	4,00E+04	4,00E+04	8,00E+04	8,00E+04	
30-40-30	42	42	6,72E+07	6,72E+07	33,46	33,5	7,80E+07	7,80E+07	1,56E+06	1,56E+06	1,05E+06	1,05E+06	5,33E+05	5,33E+05	1,07E+05	1,07E+05	6,00E+04	6,00E+04	4,00E+04	4,00E+04	1,00E+05	1,00E+05

All values correspond, so excel is validated

Figure X.70: CLT section properties 3-layered panel_ Excel incl. validation

X.10.1.2 5-layered panel

5-layered panel										Standard panel sizes					Weight [kg/m ²]	Formule X							
Wood properties										Panel num	Total thid	1	2	3	4	5 Unit	E _{mean}						
1	Strength class									1	100	20	20	20	20	20 mm	42	(Q3/1000)*S854					
2	P _{mean} 420 kg/m ³									2	120	20	30	20	30	20 mm	50.4	(Q4/1000)*S854					
3	P _{max} 350 kg/m ³									3	140	20	40	20	40	20 mm	58.8	(Q5/1000)*S854					
4	E _{0,05} 7400 Mpa									4	110	20	20	30	20	20 mm	46.2	(Q6/1000)*S854					
5	E _{0,1} 11000 Mpa									5	130	20	30	30	30	20 mm	54.6	(Q7/1000)*S854					
6	E _{0,2} 50 Mpa									6	150	20	40	30	40	20 mm	63	(Q8/1000)*S854					
7	E _{0,3} 690 Mpa									7	120	20	20	40	20	20 mm	50.4	(Q9/1000)*S854					
8	E _{0,4} 7400 Mpa									8	140	20	30	40	30	20 mm	58.8	(Q10/1000)*S854					
9	E _{0,5} 11000 Mpa									9	160	20	40	40	40	20 mm	67.2	(Q11/1000)*S854					
10	E _{0,6} 690 Mpa									10	120	30	20	20	20	30 mm	50.4	(Q12/1000)*S854					
11	Layer properties									11	140	30	30	20	30	30 mm	58.8	(Q13/1000)*S854					
12	Layer l	t [mm]	a [mm]							12	160	30	40	20	40	30 mm	67.2	(Q14/1000)*S854					
13	1	20	50							13	130	30	20	30	20	30 mm	54.6	(Q15/1000)*S854					
14	2	30	25							14	150	30	30	30	30	30 mm	63	(Q16/1000)*S854					
15	3	20	0							15	170	30	40	30	40	30 mm	71.4	(Q17/1000)*S854					
16	4	30	25							16	140	30	20	40	20	30 mm	58.8	(Q18/1000)*S854					
17	5	20	50							17	160	30	40	30	40	30 mm	67.2	(Q19/1000)*S854					
18	t _{CLT}	120 mm	Formule 8						18	180	30	40	40	40	30 mm	75.6	(Q20/1000)*S854						
19	h ₁	60 mm	SOM(B13/B17)						19	140	40	20	20	20	40 mm	58.8	(Q21/1000)*S854						
20	h ₂	60 mm	B13+B15+B17						20	160	30	30	40	30	30 mm	67.2	(Q22/1000)*S854						
21	h ₃	60 mm	B14+B16						21	180	30	40	40	40	30 mm	75.6	(Q23/1000)*S854						
22	h ₄	1000 mm	B18/2						22	150	40	20	30	20	40 mm	63	(Q24/1000)*S854						
23	h ₅	1000 mm							23	170	40	30	30	40	30 mm	71.4	(Q25/1000)*S854						
24	Sectional properties									24	190	40	40	30	40	40 mm	79.8	(Q26/1000)*S854					
25	Bending around x-axis									25	160	40	20	40	20	40 mm	67.2	(Q27/1000)*S854					
26	A _{xx,CLT}	1,20E+05 mm ²	B22*B18						26	180	40	30	40	30	40 mm	75.6	(Q28/1000)*S854						
27	A _{yy,CLT}	6,00E+04 mm ²	B19*B22						27	200	40	40	40	40	40 mm	84	(Q29/1000)*S854						
28	I _{xx,CLT}	1,020E+08 mm ⁴	B22*(B13^3/12) + (B22*B13^2*C13^2) + (B22*(B15^3/12) + (B22*B15^2*C15^2) + (B22*(B17^3/12) + (B22*B17^2*C17^2)																				
29	I _{yy,CLT}	5E+09 mm ⁴	B13*(B14^3/12) + (B13*B14^2*C14^2) + (B23*(B16^3/12) + (B23*B16^2*C16^2)																				
30	I _{xy,CLT}	1,700E+06 mm ⁴	B13*(B15+B17)*B22^2/12																				
31	W _{xx,CLT}	1,700E+06 mm ³	I2*B29/B18																				
32	W _{yy,CLT}	1,00E+06 mm ³	B22*B13^2/C13																				
33	W _{xy,CLT}	1050000 mm ³	B22*B13^2/C13 + (B22*(B15^2/8)																				
34	Bending around y-axis																						
35	A _{xx,CLT}	1,20E+05 mm ²	B18*B23																				
36	A _{yy,CLT}	6,00E+04 mm ²	B20*B23																				
37	I _{xx,CLT}	4,300E+07 mm ⁴	B23*(B14^3/12) + (B23*B14^2*C14^2) + (B23*(B16^3/12) + (B23*B16^2*C16^2)																				
38	I _{yy,CLT}	5E+09 mm ⁴	B14*(B16)^2*B23^2/12																				
39	I _{xy,CLT}	7,000E+05 mm ⁴	I2*B38/B18																				
40	W _{xx,CLT}	7,000E+05 mm ³																					
41	Rolling deformation																						
42	l ₀₁ (Espan)	3500 mm	out-of-plane loading only																				
43	l ₀₂ (Espan)	3500 mm																					
44	v ₁	0,90	1/(1 + ((PI)^2*B7*B13/B43^2)*(B14/B8)))																				
45	v ₂	0,90																					
46	v ₃	0,90	1/(1+((PI)^2 *B7*B17/B43^2)*(B16/B8)))																				
47	l ₀₃	9,24E+07 mm ⁴	B22*B13^3/12 + (B44*B22*B13^2/C13^2) + (B22*B15^3/12) + (B22*B17^3/12) + (B46*B22*B17^2/C17^2)																				
48	l ₀₄	39,24 mm	WORTEL(B47/B28)																				
49	Validation Excel																						
50	Verification excel with table 3.12 (P46 CLT Handbook) & 3.15 (p.52 CLT Handbook) (Borgström & Frébel, 2019)																						
51	Excel Table									Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table				
52	Panel build-up									Excel	Table	Excel	Table	Excel	Table	Excel	Table	Excel	Table				
53	20-20-20-20-20	42	42	5,82E+07	5,82E+07	31,14	31,14	6,60E+07	6,60E+07	1,32E+06	1,32E+06	8,00E+05	8,00E+05	1,73E+07	1,73E+07	3,47E+05	3,47E+05	6,00E+04	6,00E+04	4,00E+04	4,00E+04	1,00E+05	1,00E+05
54	30-20-20-20-30	50,4	50,4	1,06E+08	1,06E+08	36,39	36,4	1,27E+08	1,27E+08	2,11E+06	2,11E+06	1,35E+06	1,35E+06	1,73E+07	1,73E+07	2,89E+05	2,89E+05	8,00E+04	8,00E+04	4,00E+04	4,00E+04	1,20E+05	1,20E+05
55	40-20-30-20-40	63	63	2,02E+08	2,02E+08	42,86	42,86	2,55E+08	2,55E+08	3,40E+06	3,40E+06	2,20E+06	2,20E+06	2,63E+07	2,63E+07	3,51E+05	3,51E+05	1,10E+05	1,10E+05	4,00E+04	4,00E+04	1,50E+05	1,50E+05
56	All values correspond, so excel is validated.																						

Figure X.71: CLT section properties 5-layered panel_ Excel incl. validation

X.10.2 CLT roof panels

	A	B	C	D	E	F	G	H	I	J	
78											
79											
80	Verification strenght (ULS)			Formule B							
81	1. Bending										
82	M _{ed}	3741150,00	Nmm	(1/8)*B77*B30*2*10^6							
83	σ _{M,ed}	4,01	N/mm ²	B82/B9							
84	γ _M	1,25	-								
86	Load duration	long-term	-								
87	Service class	1	-								
88	k _{mod}	0,7	-								
89	K _{ti}	1,10	-	MIN((150/B5)^0,2; 1,1)							
90	f _{Md}	14,78	N/mm ²	B27*B88*B89/B85							
91	UC _{Bending}	0,27	-	B83/B90							
92											
93	2. Shear										
94	V _d	4275,60	N	0,5*B77*B30*1000							
95	τ _d	0,07	N/mm ²	B94*B10 / (B7*B6)							
96	τ _{Rv,d}	0,07	N/mm ²	B94*B11/(B7*B6)							
97	γ _M	1,25	-								
99	Load duration	long-term	-								
100	Service class	1	-								
101	k _{mod}	0,7	-								
102	f _{V,d}	2,24	N/mm ²	B24*B101/B98							
103	τ _{Rv,d}	0,39	N/mm ²	B25*B101/B98							
104	UC _{shear}	0,03	-	B95/B102							
105	UC _{Rollingshear}	0,18	-	B96/B103							
106											
107											
108	Verification deflections (SLS)										
109	u _{inst,G}	3,98	mm	(5/384)*(B45*B6/1000)*(B30*1000)^4/(B21*B8)							
110	u _{inst,Q1}	5,06	mm	(5/384)*(B65*B6/1000)*(B30*1000)^4 / (B21*B8)							
111	u _{inst,Qi,1}	2,83	mm	(5/384)*(B53*B6/1000)*(B30*1000)^4/(B21*B8)							
112	u _{inst,Qi,2}	-0,94	mm	(5/384)*(B62*B6/1000)*(B30*1000)^4/(B21*B8)							
113											
114	Load duration	long-term	-								
115	Service class	1	-								
116	k _{def}	0,6	-								
117											
118	u _{rin,G}	6,36	mm	B109*(1+B116)							
119	u _{rin,Q1}	5,06	mm	B110*(1 + (E66*B116))							
120	u _{rin,Qi,1}	0,00	mm	B111*(E53 + (E54*B116))							
121	u _{rin,Qi,2}	0,00	mm	B112*(E62 + (E63*B116))							
122	u _{rin}	11,43	mm	SOM(B118;B121)							
123											
124	w _{allowed} (0.004L)	14	mm	0,004*B30*1000							
125	UC _{deflection}	0,82	-	B122/B124							
126											
127											
128	Results										
129	Panel build-up	UC _{Bending}	UC _{shear}	UC _{Rollingshear}	UC _{deflection}						
130	20-20-20	0,42	0,04	0,24	1,6						
131	20-40-20	0,27	0,03	0,18	0,82	<- Chosen panel					
132	20-30-20	0,33	0,04	0,2	1,11						
133											
134											
135											
136											
137	Verification load perpendicular to the grain (point loads of walls)										
138	Load combinations (NEN1990; CC2)										
139		γ _G	γ _Q								
140	6.10a	1,35	1,5	q _{ed} = G _k γ _G + ΣQ _k γ _Q ψ ₀							
141	6.10b	1,2	1,5	q _{ed} = G _k γ _G + Q _{k,1} γ _Q + ΣQ _{k,i} γ _Q ψ ₀							
142											
143			ψ ₀								
144	Roof	0									
145	Floor	0,4									
146											
147											
148	Roof topfloor	kN/m ²	kN/m	Formule C							
149	Permanent load	0,786	1,38	B149*B30/2							
150	Variable	1,00	1,75	B150*B30/2							
151		Eq_6.10a	1,86	kN/m	(B140*C149) + (C140*B144*C150)						
152		Eq_6.10b	4,28	kN/m	(B141*C149) + (C141*C150)						
153											
154	Floor topfloor	kN/m ²	kN/m								
155	Permanent	0,964	1,69	B155*B30/2							
156	Variable										

Figure X.72: Structural verification CLT roof panel (2)_Excel

	A	B	C	D	E	F	G	H	I	J
156	Variable									
157	Non-structural walls	0,5	0,88			B157*B30/2				
158	Residential loads	1,75	3,06			B158*B30/2				
159		Eq_6.10a	4,64	kN/m		(B140*C155) + (C140*B145*(C157+C158))				
160		Eq_6.10b	7,14	kN/m		(B141*C155) + (C141*C158) + (C141*B145*C157)				
161										
162	Wall (100 mm)									
163	m	42	kg/m2							
164	Height	2,95	m			Formule B				
165	q_k	1,24	kN/m			(B163*10/1000) * B164				
166	Eq_6.10a	1,67	kN/m			B140*B165				
167	Eq_6.10b	1,49	kN/m			B141*B165				
168										
169	Roofpanel 1st floor top-up	kN/m ²	kN/m			Formule C				
170	Permanent	0,786	1,38			B170*B30/2				
171	Variable	Not present								
172		Eq_6.10a	1,86	kN/m		C170*B140				
173		Eq_6.10b	1,65	kN/m		C170*B141				
174										
175	Total load perpendicular to roofpanel					Formule B				
176	Eq_6.10a	10,03	kN/m			C151+C159+B166+C172				
177	Eq_6.10b	14,56	kN/m			C152+C160+B167+C173				
178	Q _{roofpanel} ULS	14,56	kN/m			MAX(B176:B177)				
179										
180	Verification					Formule B				
181	t _{wall}	100	mm							
182	b _{eff}	130	mm			B181+30				
183	σ _{c,r,d}	0,112	N/mm ²			B178/B182				
184										
185	k _{c,90} (at edge)	1,5	-							
186	γ _M	1,25	-							
187	Load duration	long-term	-							
188	Service class	1	-							
189	k _{mod}	0,7	-							
190	f _{c,90,slay,d}	2,1	N/mm ²			B185*B189*B26/B186				
191	UC	0,05	-			B183/B190				

Figure X.73: Structural verification CLT roof panel (3)_Excel

Hand validation roof panel sheet on next page.

Order /
 Blad nr 1/3
 Deel Verificatie CLT roofpanel steel
 Datum 02/05/2023

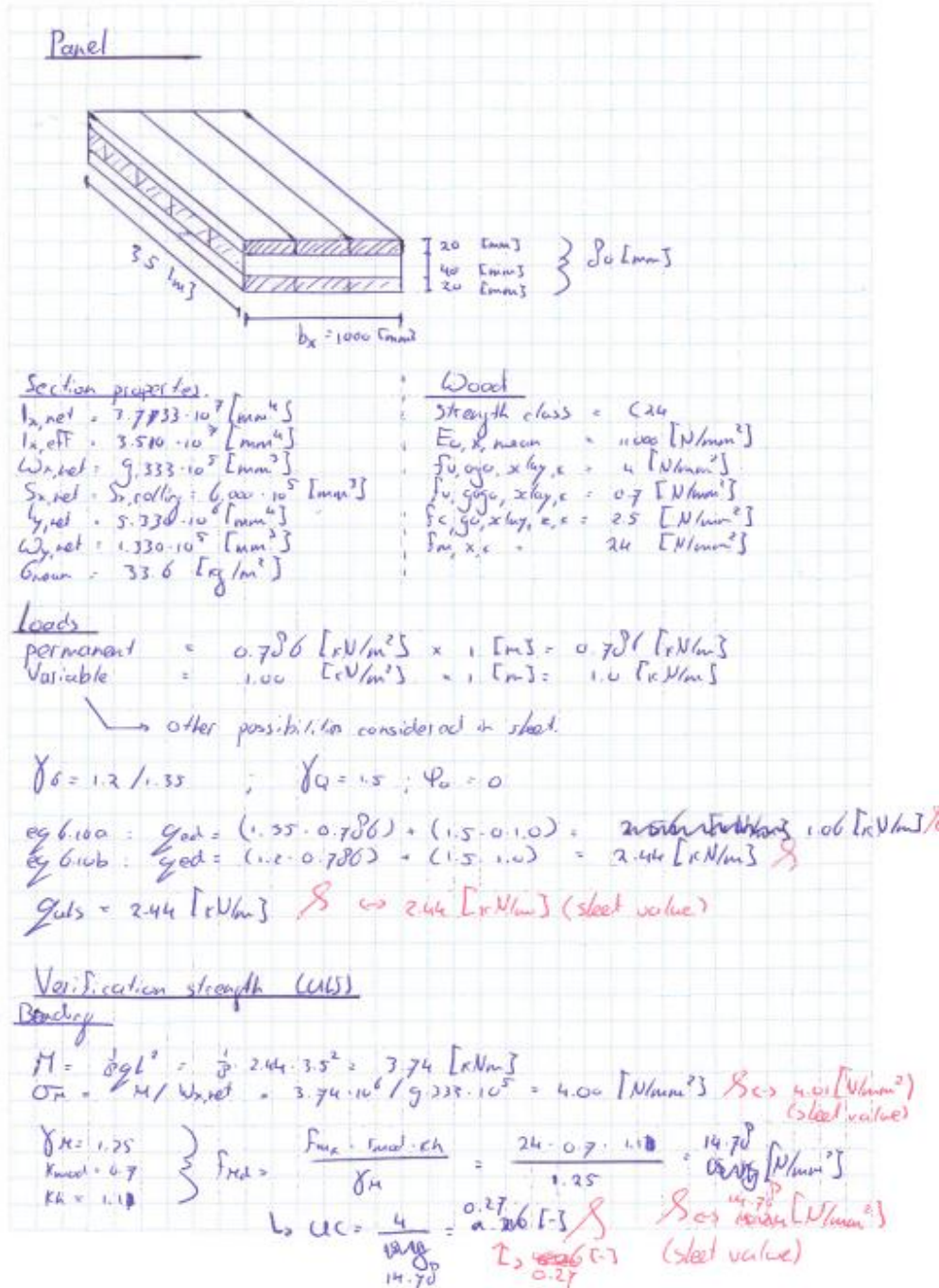


Figure X.74: Validation excel roof panel (1)

Order ✓
 Blad nr 2/3
 Deel Verificatie CCT roofpanel steeet
 Datum 02/05/2023



2. Shear

$$V_{ed} = 0.5 g L = 0.5 \cdot 2.44 \cdot 3.5 = 4.27 \text{ [kN]}$$

$$\tau_{ed} = \frac{V_{ed} \cdot S_{x,rel}}{I_x \cdot t_{x,rel}} = \frac{4.27 \cdot 10^3 \cdot 6.0 \cdot 10^5}{1000 \cdot 3.733 \cdot 10^7} = 0.069 \text{ [N/mm}^2\text{]} \quad \text{OK} \Rightarrow 0.069 \text{ [N/mm}^2\text{]}$$

$$\tau_{Rd,ed} = \frac{V_{ed} \cdot S_{x,rolling}}{I_x \cdot t_{x,rel}} = \tau_{ed} = 0.069 \text{ [N/mm}^2\text{]} \quad \text{OK}$$

$$\begin{aligned} \delta_M = 1.25 & \quad \tau_{Rd,ed} = f_{v,avg} \cdot x_{top} \cdot t \cdot k_{red} / \delta_M = 4 \cdot 0.7 / 1.25 = 2.24 \text{ [N/mm}^2\text{]} \\ k_{wood} = 0.7 & \quad \tau_{Rd,ed} = f_{v,avg} \cdot x_{top} \cdot t \cdot k_{red} / \delta_M = 0.7 \cdot 0.7 / 1.25 = 0.39 \text{ [N/mm}^2\text{]} \end{aligned}$$

$$UC_v = 0.069 / 2.24 = 0.03 \text{ [-]} \quad \text{OK} \Rightarrow 0.03 \text{ (steel value)}$$

$$UC_{rolling} = 0.069 / 0.39 = 0.18 \text{ [-]} \quad \text{OK} \Rightarrow 0.18$$

Verification deflections

$$U_{inst, x} = \frac{5 \cdot g L^4}{384 E_{steel} I_x} \quad ; \quad k_{def} = 0.6 \text{ [-]} \quad ; \quad \psi_2 = 0$$

$$U_{inst, G} = (5/384) \cdot 0.7 \cdot 3500^4 / (11000 \cdot 3.510 \cdot 10^9) = 3.58 \text{ [mm]} \quad \text{OK}$$

$$U_{inst, Q} = (5/384) \cdot 1 \cdot 3500^4 / (11000 \cdot 3.510 \cdot 10^9) = 5.06 \text{ [mm]} \quad \text{OK}$$

↳ Snow and wind load will fall out of first equation as $\psi_0 = \psi_2 = 0$

$$U_{Fin, G} = U_{inst, G} \cdot (1 + k_{def}) = 3.58 \cdot (1 + 0.6) = 6.36 \text{ [mm]} \quad \text{OK}$$

$$U_{Fin, Q} = U_{inst, Q} \cdot (1 + \psi_2 k_{def}) = 5.06 \cdot (1 + 0.6) = 5.06 \text{ [mm]} \quad \text{OK}$$

$$UC = \frac{U_{Fin}}{0.004 \cdot 3500} = \frac{11.42}{0.004 \cdot 3500} = 0.82 \text{ [-]} \quad U_{Fin} = 11.42 \text{ [mm]} \Rightarrow 11.43 \text{ [mm]} \quad \text{OK (steel value)}$$

Verification load/level perpendicular to grain

top roof: $\psi_0 = 0$

	[kN/m ²]	[kN]
permanent load:	$0.7 \cdot 36 \times 3.5 / 2 = 1.38$	[kN/m ²]
variable load:	$1 \times 3.5 / 2 = 1.75$	[kN/m ²]

$$e_{y6.10 a} = (1.35 \cdot 1.38) + (1.5 \cdot 1.75) = 1.86 \text{ [kN/m}^2\text{]} \quad \text{OK}$$

$$e_{y6.10 b} = (1.2 \cdot 1.38) + (1.5 \cdot 1.75) = 1.88 \text{ [kN/m}^2\text{]} \quad \text{OK}$$

Figure X.75: Validation excel roof panel (2)

Order ✓
 Blad nr 3/3
 Deel Verificatie CLT roof panel steel.
 Datum 02/05/2023



Floor top floor

$h_b = 0.4$

permanent:	0.964 kN/m^2	$\times 3.5/2$	$= 1.65 \text{ kN/m}$
variable:			
walks:	0.5	$\times 3.5/2$	$= 0.875 \text{ kN/m}$
imposed load:	1.75	$\times 3.5/2$	$= 3.06 \text{ kN/m}$

eg 6.10a: $(1.35 \cdot 1.65) + (1.5 \cdot 0.4 \cdot (0.875 + 3.06)) = 4.65 \text{ kN/m}$ Ⓝ

eg 6.10b: $(1.2 \cdot 1.65) + (1.5 \cdot 3.06) + (1.5 \cdot 0.4 \cdot 0.875) = 7.15 \text{ kN/m}$ Ⓝ

Wall

→ As wall is not calculated yet, this initial excel assumed a thickness of 120 [mm]. Note that probably in the final excel this thickness is changed. That has no impact on the verification.

mass = 50.4 kg/m^2
 height = 2.95 m

$g_k = \frac{50.4 \cdot 2.95}{1000} = 1.45 \text{ kN/m}$

eg 6.10a: $1.35 \cdot 1.45 = 2.01 \text{ kN/m}$ Ⓝ

eg 6.10b: $1.2 \cdot 1.45 = 1.75 \text{ kN/m}$ Ⓝ

Roof panel itself:

permanent: $0.786 \text{ kN/m}^2 \times 3.5 \text{ m} / 2 = 1.38 \text{ kN/m}$

eg 6.10a: $1.35 \cdot 1.38 = 1.86 \text{ kN/m}$ Ⓝ

eg 6.10b: $1.2 \cdot 1.38 = 1.65 \text{ kN/m}$ Ⓝ

Total:

eg 6.10a: $1.86 + 4.65 + 2.01 + 1.38 = 10.30 \text{ kN/m}$ Ⓝ $\leftrightarrow 10.30 \text{ kN/m}$

eg 6.10b: $1.65 + 7.15 + 1.75 + 1.65 = 14.87 \text{ kN/m}$ Ⓝ $\leftrightarrow 14.87 \text{ kN/m}$

↳ normative value

Verification:

h_{eff} = $120 + 20 = 150 \text{ [mm]}$

$\alpha_{1,2,d} = \frac{g_k}{h_{eff}} = \frac{14.87}{150} = 0.099 \text{ [N/mm}^2\text{]}$ Ⓝ

$k_{1,2,d} = 1.5 \text{ [-]}$
 $\gamma_{1,2,d} = 1.25 \text{ [-]}$
 $f_{1,2,d} = 0.7 \text{ [-]}$

$f_{1,2,d} = \frac{f_{1,2,d} \cdot \gamma_{1,2,d} \cdot k_{1,2,d}}{\gamma_M} = \frac{2.5 \cdot 1.5 \cdot 0.7}{1.25} = 2.1 \text{ [N/mm}^2\text{]}$ Ⓝ

$UL = \frac{0.099}{2.1} = 0.05 \text{ [-]}$ Ⓝ $\leftrightarrow 0.05$ (steel value)

Figure X.76: Validation excel roof panel (2)

X.10.3 CLT floor panels

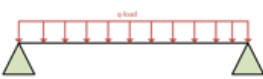
	A	B	C	D	E	F	G	H	I	J
1	Chosen floorpanel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	40-40-40	mm							
5	h_{panel}	120	mm							
6	b_x	1000	mm							
7	$I_{x,net}$	1,387E+08	mm ⁴							
8	$I_{x,eff}$ ($l_{ref} = 3.5$ m)	1,245E+08	mm ⁴							
9	$W_{x,net}$	2,311E+06	mm ³							
10	$S_{x,net}$	1,600E+06	mm ³							
11	$S_{x,rolling}$	1,600E+06	mm ³							
12	$I_{y,net}$	5,330E+06	mm ⁴							
13	$W_{y,net}$	8,890E+04	mm ³							
14	\bar{g}_{mean}	50,4	kg/m ²							
15										
16	Wood properties									
17	Strength class	C24								
18	$\rho_{stem,mean}$	420	kg/m ³							
19	$\rho_{stem,k}$	350	kg/m ³							
20	$E_{0,x,0.05}$	7400	MPa							
21	$E_{0,x,mean}$	11000	MPa							
22	$G_{090,xlay,mean}$	50	MPa							
23	$G_{090,xlay,mean}$	690	MPa							
24	$f_{V,090,xlay,k}$	4	N/mm ²							
25	$f_{V,9090,xlay,k}$	0,7	N/mm ²							
26	$f_{c,90,xlay,z,k}$	2,5	N/mm ²							
27	$f_{m,X,k}$	24	N/mm ²							
28										
29	Structural system									
30	Span	3,5	m							
31	Scheme:									
32										
33										
34										
35										
36										
37										
38										
39										
40										
41	Loads									
42	Permanent loading			Formule B						
43	Self-weight panel	0,504	kN/m ²	B14*10/1000						
44	Fermacell finishing (22 mm)	0,26	kN/m ²							
45	Installations	0,2	kN/m ²							
46	Total permanent	0,964	kN/m ²	SOM(B43:B45)						
47										
48	Variable loading									
49	Non-structural walls	0,5	kN/m ²							
50	Residential loads	1,75	kN/m ²							
51	Total variable	2,25	kN/m ²							
52										
53	Load combinations (NEN1990; CC2)									
54	ψ_0	0,4	-							
55										
56		V_G	V_Q							
57	6.10a	1,35	1,5	$q_{ed} = G_k V_G + \sum Q_{k,i} V_{Q,i} \psi_0$						
58	6.10b	1,2	1,5	$q_{ed} = G_k V_G + Q_{k,1} V_Q + \sum Q_{k,i} V_{Q,i} \psi_0$						
59										
60	$q_{ed,eq6.10a}$	2,65	kN/m ²	(B57*B46)+(C57*B54*B51)						
61	$q_{ed,eq6.10b}$	4,08	kN/m ²	(B58*B46) + (C58*B50) + (C58*B49*B54)						
62										

Figure X.77: Structural verification CLT floor panel (1)_Excel

	A	B	C	D	E	F	G	H	I	J
63	Results									
64	q _{ed,ULS}	4,08 kN/m		MAX(B60:B61) * B6/1000						
65	q _{ed,SLS}	3,21 kN/m		(B46+B51)*B6/1000						
66										
67										
68	Verification strenght (ULS)									
69	1. Bending			Formule B						
70	M _{ed}	6250256,25 Nmm		(1/8)*B64*B30^2*10^6						
71	σ _{M,ed}	2,70 N/mm ²		B70/B9						
72										
73	γ _M	1,25 -								
74	Load duration	long-term	-							
75	Service class	1	-							
76	k _{mod}	0,7	-							
77	k _{th}	1,10	-	MIN((600/B5)^0,1; 1,1)						
78	f _{Md}	14,78 N/mm ²		B27*B76*B77/B73						
79	UC _{Bending}	0,18	-							
80										
81	2. Shear									
82	V _d	7143,15 N		0,5*B64*B30*1000						
83	τ _d	0,08 N/mm ²		B82*B10 / (B7*B6)						
84	τ _{Rv,d}	0,08 N/mm ²		B82*B11/(B7*B6)						
85										
86	γ _M	1,25 -								
87	Load duration	long-term	-							
88	Service class	1	-							
89	k _{mod}	0,7	-							
90	f _{V,d}	2,24 N/mm ²		B24*B89/B86						
91	f _{Rv,d}	0,39 N/mm ²		B25*B89/B86						
92	UC _{shear}	0,04	-							
93	UC _{Rollingshear}	0,21	-							
94										
95										
96	Verification deflections (SLS)									
97	u _{inst,G}	1,38 mm		(5/384)*(B46*B6/1000)*(B30*1000)^4/(B21*B8)						
98	u _{inst,Q1}	2,50 mm		(5/384)*(B50*B6/1000)*(B30*1000)^4 / (B21*B8)						
99	u _{inst,Qi}	0,71 mm		(5/384)*(B49*B6/1000)*(B30*1000)^4/(B21*B8)						
100										
101	Load duration	long-term	-							
102	Service class	1	-							
103	k _{def}	0,6	-							
104	ψ ₂	0,3	-							
105										
106	u _{fin,G}	2,20 mm		B97*(1+B103)						
107	u _{fin,Q1}	2,95 mm		B98*(1 + (B104*B103))						
108	u _{fin,Qi}	0,41 mm		B99*(B54 + (B104*B103))						
109	u _{fin}	5,56 mm		SOM(B106:B108)						
110										
111	w _{allowed (0.004L)}	14 mm		0,004*B30*1000						
112	UC _{deflection}	0,40	-							
113										
114										
115	Verification vibrations (SLS)									
116	1. Fundamental frequency									
117	m	96,4 kg/m ²		B46*1000/10						
118	E _{x,0,mean}	11000000000 N/m ²		B21*10^6						
119	I _{x,ef}	1,245E-04 m ⁴		B8*10^-12						
120	f _{1 (>8 Hz)}	15,28 Hz		(PI()/((2*B30^2)))*WORTEL((B118*B119)/B117)						
121	UC _{vibr1}	0,52	-							
122										
123	2. Point deflection									
124	F	3000 N								
125	w	1,96 mm/3 kN		B124*(B30*1000)^3 / (48 * B21*B8)						
126	w _{per kN}	0,65 mm/1 kN		B125/(B124/1000)						
127	a	1 mm/1 kN								
128	UC _{vibr2}	0,65	-							

Figure X.78: Structural verification CLT floor panel (2)_Excel

	A	B	C	D	E	F	G	H	I	J
129										
130	3. Impulse velocity response									
131	B	2,5	m							
132	$I_{x,net}$	1,387E-04	m ⁴	B7*10 ⁻¹²						
133	$I_{y,net}$	5,330E-06	m ⁴	B12*10 ⁻¹²						
134	E_{mean}	1,10E+10	N/m ²	B21*10 ⁶						
135	n_{40}	2,51	-	(((40/B120) ² - 1) * (B131/B30) ⁴ * ((B134*B132)/(B134*B133))) ^{0,25}						
136	m	96,4	kg/m ²	B117						
137	v	0,0073	m/s	4*((0,4 + (0,6*B135)) / ((B136*B131*B30) + 200))						
138										
139	b	120	m/Ns ²							
140	ξ (damping)	0,025	-	0,025						
141	$b^{(f1\xi-1)}$	0,05	m/s	B139^((B120*B140) - 1)						
142	UC_{vibr3}	0,14	-							
143										
144										
145	Results									
146	Panel build-up	$UC_{Bending}$	UC_{shear}	$UC_{Rollingshear}$	$UC_{deflection}$	UC_{vibr1}	UC_{vibr2}	UC_{vibr3}		
147	20-20-20-20-20	0,31	0,04	0,22	0,77	0,71	1,31	0,19		
148	40-20-40	0,25	0,05	0,26	0,61	0,63	1,04	0,31		
149	20-30-20-30-20	0,25	0,03	0,18	0,54	0,61	0,88	0,12		
150	40-40-40	0,18	0,04	0,21	0,4	0,52	0,65	0,14		

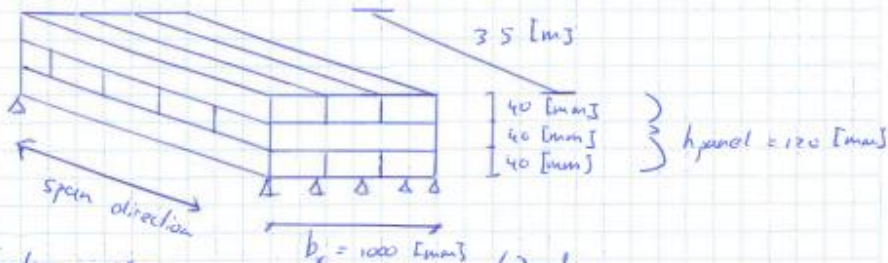
Figure X.79: Structural verification CLT floor panel (3)_Excel

Hand validation floor panel sheet on next page.

Order /
 Blad nr 1 / 3
 Deel Ver. F. Validation excel Floorpanel CLT
 Datum 01/05/2023



o Panel



Section props

$$\begin{aligned}
 I_{x,net} &= 1,307 \cdot 10^8 \text{ [mm}^4\text{]} \\
 I_{x,eff} &= 1,245 \cdot 10^8 \text{ [mm}^4\text{]} \\
 W_{x,net} &= 2,311 \cdot 10^6 \text{ [mm}^3\text{]} \\
 S_{x,net} = S_{x,rolling} &= 1,600 \cdot 10^4 \text{ [mm}^3\text{]} \\
 I_{y,net} &= 5,330 \cdot 10^8 \text{ [mm}^4\text{]} \\
 W_{y,net} &= 8,080 \cdot 10^6 \text{ [mm}^3\text{]} \\
 g_{mean} &= 504 \text{ [kg/m}^3\text{]}
 \end{aligned}$$

$$b_x = 1000 \text{ [mm]}$$

Wood

$$\begin{aligned}
 E_{u,x,mean} &= 11000 \text{ [N/mm}^2\text{]} \\
 F_{u,0.95,x,lay,k} &= 4 \text{ [N/mm}^2\text{]} \\
 F_{v,0.95,x,lay,k} &= 0.7 \text{ [N/mm}^2\text{]} \\
 F_{c,0.2,lay,2,k} &= 2.5 \text{ [N/mm}^2\text{]} \\
 F_{m,x,k} &= 24 \text{ [N/mm}^2\text{]}
 \end{aligned}$$

o Loads

Permanent:

↓ [mm] span width

- Self weight = 0.504 [kN/m ²]	= 0.504 [kN/m]
- Fermocell = 0.26 [kN/m ²]	= 0.26 [kN/m]
- Installations = 0.20 [kN/m ²]	= 0.20 [kN/m]
	<u>0.964 [kN/m]</u>

Variable

- Imposed load = 1.75 [kN/m ²]	= 1.75 [kN/m]
- Non-structural walls = 0.5 [kN/m ²]	= 0.50 [kN/m]
	<u>2.25 [kN/m]</u>
eg 6.10a ⇒ (1.35 · 0.964) + (1.5 · 0.4 · 2.25)	= 2.65 [kN/m]
eg 6.10b ⇒ (1.2 · 0.964) + (1.5 · 1.75) + (1.5 · 0.4 · 0.5)	= 4.03 [kN/m]
g.ed. ult = 4.03 [kN/m]	↔ 4.03 [kN/m] (steel)
g.ed. stl = 0.964 + 2.25 = 3.21 [kN/m]	↔ 3.21 [kN/m] (steel)

Figure X.80: Validation excel floor panel (1)

Order ✓
 Blad nr 2/3
 Deel ~~13/15~~ Ver.ificatie excel floor panel CLT
 Datum 01/05/2023



0 Verifications ULS

1. Bending

$$M = \frac{1}{8} q L^2 = \frac{1}{8} \cdot 4.08 \cdot 3.5^2 = 6.2475 \text{ [kNm]}$$

$$\sigma_M = \frac{M}{W} = \frac{6.2475 \cdot 10^6}{2.311 \cdot 10^6} = 2.70 \text{ [N/mm}^2\text{]} \quad \text{OK} \Rightarrow 2.70 \text{ [N/mm}^2\text{]} \text{ (steel)}$$

$$\left. \begin{array}{l} \delta_M = 1.25 \\ k_{mod} = 0.7 \\ k_{\sigma} = 1.05 \cdot 1.1 \end{array} \right\} \sigma_{M,d} = \frac{F_{M,d} \cdot k_{mod} \cdot k_{\sigma}}{\delta_M} = \frac{24 \cdot 0.7 \cdot 1.05 \cdot 1.1}{1.25} = 14.75 \text{ [N/mm}^2\text{]}$$

$$UL_{bending} = \frac{2.70}{14.75} = \frac{0.183}{0.18} \text{ [L]} \quad \text{OK} \Rightarrow 0.183 \text{ (steel value)}$$

2. Shear

$$V_d = \frac{1}{2} q L = \frac{1}{2} \cdot 4.08 \cdot 3.5 = 7.14 \text{ [kN]}$$

$$\tau_d = \frac{V_d \cdot S_{steel}}{b_s \cdot I_{y,net}} = \frac{7.14 \cdot 10^3 \cdot 1.600 \cdot 10^6}{1000 \cdot 1.387 \cdot 10^8} = 0.80 \text{ [N/mm}^2\text{]} \quad \text{OK} \Rightarrow 0.80 \text{ [N/mm}^2\text{]} \text{ (steel)}$$

$$\tau_d = \tau_{v,d} \quad \text{OK}$$

$$F_{v,d} = \frac{F_{v,avg,slab,c} \cdot k_{mod}}{\delta_M} = \frac{4 \cdot 0.7}{1.25} = 2.24 \text{ [N/mm}^2\text{]} \Rightarrow UL = \frac{0.80}{2.24} = 0.35 \text{ OK} \Rightarrow 0.35 \text{ (steel)}$$

$$F_{v,Red} = \frac{F_{v,avg,slab,c} \cdot k_{mod}}{\delta_M} = \frac{0.7 \cdot 0.7}{1.25} = 0.39 \text{ [N/mm}^2\text{]} \Rightarrow UL = \frac{0.80}{0.39} = 2.05 \text{ OK} \Rightarrow 2.05 \text{ (steel)}$$

0 Verifications deflections (SLS)

$$U_{inst,0} = \frac{5}{384} \frac{q_c L^4}{E_{mean} I_{x,eff}} = \frac{5}{384} \cdot \frac{0.564 \cdot 3500^4}{11000 \cdot 1.245 \cdot 10^9} = 1.30 \text{ [mm]} \quad \text{OK}$$

$$U_{inst,q_1} = \frac{5}{384} \frac{q_q L^4}{E_{mean} I_{x,eff}} = \frac{5}{384} \cdot \frac{1.75 \cdot 3500^4}{11000 \cdot 1.245 \cdot 10^9} = 2.50 \text{ [mm]} \quad \text{OK}$$

$$U_{inst,q_2} = \frac{5}{384} \frac{q_{tot} L^4}{E_{mean} I_{x,eff}} = \frac{5}{384} \cdot \frac{0.5 \cdot 3500^4}{11000 \cdot 1.245 \cdot 10^9} = 0.71 \text{ [mm]} \quad \text{OK}$$

$$k_{def} = 0.6 \quad \left. \begin{array}{l} U_{Fin,0} = U_{inst,0} \cdot (1 + k_{def}) = 1.30 \cdot (1 + 0.6) = 2.20 \text{ [mm]} \\ U_{Fin,q_1} = U_{inst,q_1} \cdot (1 + k_{def} \cdot \psi_2) = 2.50 \cdot (1 + 0.6 \cdot 0.3) = 2.55 \text{ [mm]} \\ U_{Fin,q_2} = U_{inst,q_2} \cdot (\psi_2 + k_{def} \cdot \psi_2) = 0.71 \cdot (0.4 + 0.6 \cdot 0.3) = 0.41 \text{ [mm]} \end{array} \right\}$$

$$UL = \frac{5.56}{0.004 \cdot 3500} = 0.40 \text{ [L]} \quad \text{OK} \Rightarrow 0.40 \text{ [L]} \text{ (steel)}$$

Figure X.81: Validation excel floor panel (2)

Order ✓
 Blad nr 3/3
 Deel Verificatie excel Floorpanel CLT
 Datum 01/05/2023

VAN ROSSUM
 RAADGEVENDE
 INGENIEURS

0 Verificatie vibrations

1.
$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{E_{\text{min}} I_{\text{eff}}}{m}} = \frac{\pi}{2 \cdot 3,5^2} \sqrt{\frac{11000 \cdot 10^6 \cdot 1,745 \cdot 10^8 \cdot 10^{-12}}{96,4}} = 15,28 \text{ Hz}$$

15,28 Hz > 8 Hz, so $\rightarrow UC = 0,52$ $\rightarrow 0,52$ [-] (steel value)

2. Point deflection:

$$\omega = \frac{1}{48} \frac{F L^3}{E_{\text{min}} I_{\text{eff}}} = \frac{1}{48} \frac{3000 \cdot 3500^3}{11000 \cdot 1,745 \cdot 10^8} = 1,56 \text{ [mm / s}^2 \text{V]}$$

$$\omega / \kappa V = \frac{1,56}{3} = 0,65 \text{ [mm/s]}$$

$$\rightarrow UC = \frac{0,65}{1} = 0,65 \text{ [-]} \rightarrow 0,65 \text{ [-]} \text{ (steel)}$$

3. Impulse velocity response

$$n_{40} = \left[\left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{B}{L} \right)^4 \left(\frac{E_{\text{steel}}}{E_{\text{CLT}}} \right) \right]^{0,25} = \left[\left(\left(\frac{40}{15,28} \right)^2 - 1 \right) \left(\frac{3,5}{3,5} \right)^4 \left(\frac{11000 \cdot 10^6 \cdot 1,745 \cdot 10^{-12}}{11000 \cdot 10^6 \cdot 5,33 \cdot 10^{-6}} \right) \right]^{0,25}$$

$$= 2,51 \text{ [-]} \rightarrow 2,51 \text{ (steel)}$$

$$v = \frac{4 \cdot (0,4 + 0,6 n_{40})}{m B L + 200} = \frac{4 \cdot (0,4 + 0,6 \cdot 2,51)}{(96,4 \cdot 3,5 \cdot 3,5) + 200} = 0,0077 \text{ [m/s]}$$

$$v \leq b \frac{F_1 \xi^{-1}}{120} = \frac{((15,28 \cdot 0,025) - 1)}{120} = 0,052 \text{ [m/s]} \rightarrow 0,052 \text{ [m/s]} \text{ (steel)}$$

$$UC = \frac{0,0077}{0,052} = 0,14 \text{ [-]} \rightarrow 0,14 \text{ [-]} \text{ (steel)}$$

Figure X.82: Validation excel floor panel (3)

X.10.4 CLT Wall panel 'beam'

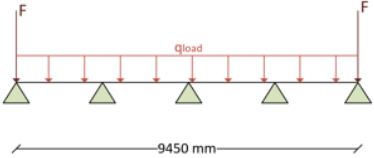
	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Chosen wall panel (Beam verification)													
2	Panel													
3	Number of layers	3	-											
4	Panel build-up	30-40-30	mm											
5	Thickness panel	100	mm											
6	Wall height (l_w)	2950	mm											
7	b_y	2950	mm											
8	b_x	1000	mm											
9	$I_{x,net}$	7,800E+07	mm ⁴											
10	$A_{x,net}$	6,00E+04	mm ²											
11	$W_{x,net}$	1,56E+06	mm ³											
12	$I_{z,y,net}$	8,56E+10	mm ⁴											
13	$I_{x,eff}$ ($l_{ref} = 2950$ mm)	6,95E+07	mm ⁴											
14	$i_{x,eff}$	34,04	mm											
15	m	42	kg/m ²											
16														
17	Wood													
18	Strength class	C24												
19	$\rho_{stem,mean}$	420	kg/m ³											
20	$\rho_{stem,k}$	350	kg/m ³											
21	$E_{0,x,0,05}$	7400	MPa											
22	$E_{0,x,mean}$	11000	MPa											
23	$G_{090,x,lay,mean}$	50	MPa											
24	$G_{090,x,lay,mean}$	690	MPa											
25	$f_{c,0,x,k}$	21	N/mm ²											
26	$f_{m,x,k}$	24	N/mm ²											
27	$f_{v,090,k}$	4	N/mm ²											
28														
29	Structural scheme													
30	Floorspan	3,5	m											
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														
43														
44														
45														
46														
47	Load definition													
48	Load combinations (NEN1990; CC2)	Variable residential load is leading action												
49		γ_G	γ_Q											
50	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$										
51	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_Q + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$										
52														
53			ψ_0											
54	Roof	0												
55	Floor	0,4												
56														
57														
58	Roofpanel topfloor	kN/m ²	kN/m											
59	Permanent	0,786	1,38											Formule C
60	Variable	1	1,75											B59*B30/2
61		Eq 6.10a	1,86											(B60*B30/2)
62		Eq 6.10b	1,65											(C59*B50) + (C60*B54*C50)
63														(B51*C59) + (C51*B54*C60)
64	Floorpanel topfloor	kN/m ²	kN/m											
65	Permanent	0,964	1,69											B65*B30/2
66	Variable													
67	Residential loading	1,75	3,06											B67*B30/2
68	Non-structural walls	0,5	0,88											B68*B30/2
69		Eq 6.10a	4,64											(B50*C65) + (C50*(C67+C68)*B55)
70	Full loaded	Eq 6.10b	7,14											(B51*C65) + (C51*C67) + (C51*C68*B55)
71														
72	Roofpanel 1st floor	kN/m ²	kN/m											
73	Permanent	0,786	1,38											B73* (B30/2)
74	Variable	Not present	-											
75		Eq 6.10a	1,86											B50*C73
76		Eq 6.10b	1,65											B51*C73
77														
78	Floorpanel 1st floor	kN/m ²	kN/m											

Figure X.83: Structural verification CLT wall panel as beam (1)_Excel

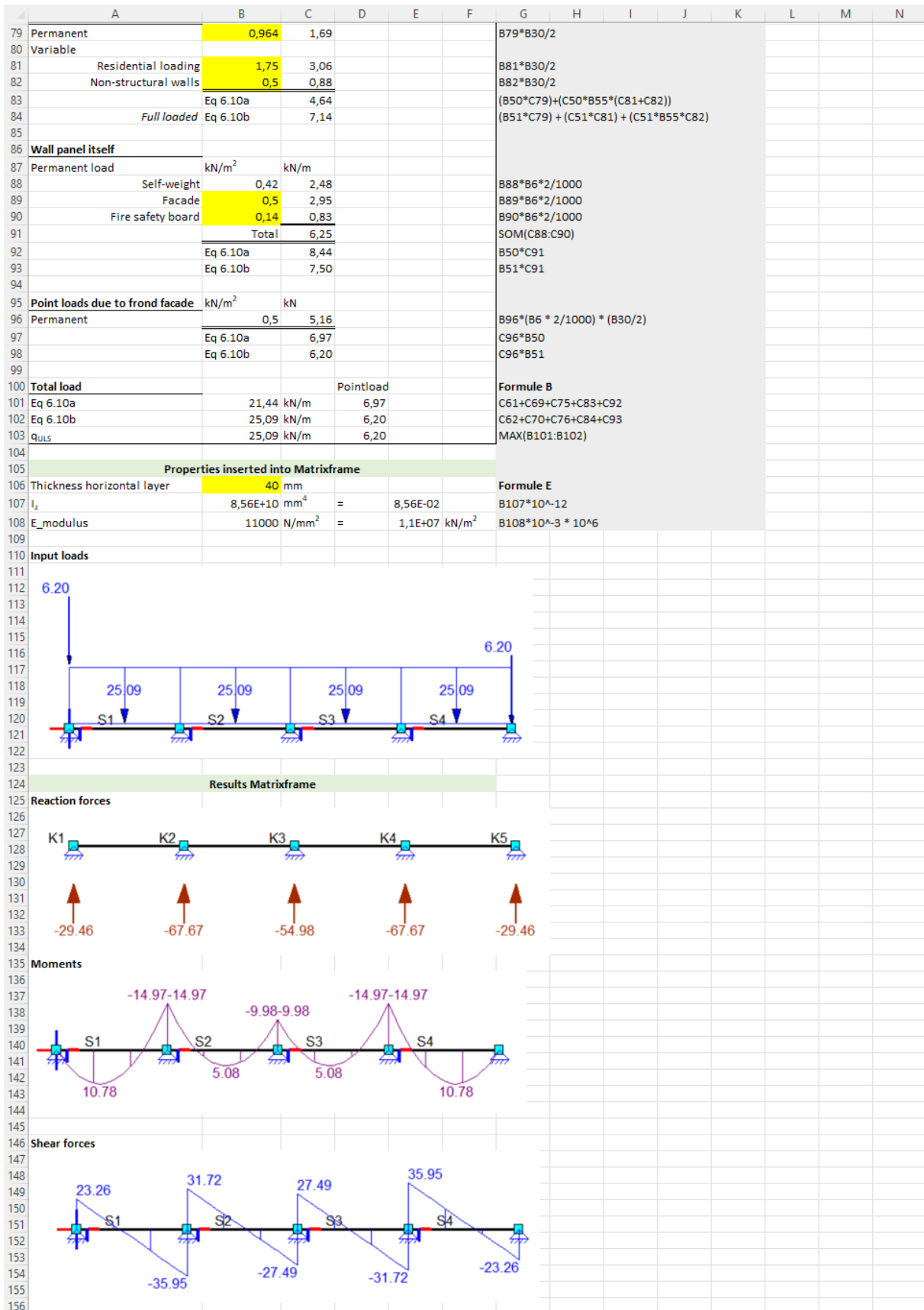
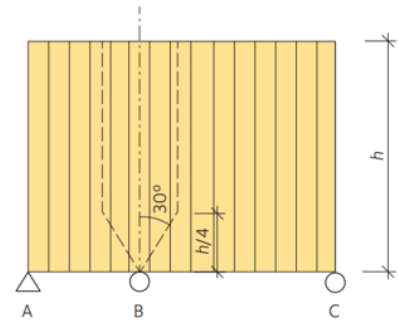


Figure X.84: Structural verification CLT wall panel as beam (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
157														
158														
159	Verification buckling (at support K2)													
160	Reduction factor for buckling			Formule B										
161	λ_s	86,66	-	B6/B14										
162	$\lambda_{rel,x}$	1,47	-	$(B161/PI()) * WORTEL(B25/B21)$										
163	k_y	1,64	-	$0,5 * (1 + (0,1 * (B162-0,3)) + B162^2)$										
164	k_{cy}	0,42	-	$1 / (B163 + WORTEL(B163^2 - B162^2))$										
165														
166	Load and verification													
167	Spread angle	30	degree											
168	h/4	737,5	mm	B7/4										
169	Spread area load	851,59	mm	$2 * TAN(30^2 * PI()/360) * B168$										
170	Normal force _{matrix frame}	67,67	kN											
171	q_{normal}	79,46	kN/m	B170/(B169/1000)										
172	q_{normal} spread for 1 m wall	93,31	kN/m	B171 * 1/(B169/1000)										
173	$\sigma_{c0,d}$	1,56	N/mm ²	B172/60										
174														
175	γ_M	1,25												
176	Load duration	long-term												
177	Service class	1												
178	k_{mod}	0,7												
179	$f_{c0,d}$	11,76	N/mm ²	B25*B178/B175										
180	UC	0,31	-	B173 / (B179*B164)										
181														
182														
183	Verification normal stress k2													
184	Normal force	67,67	kN	B170										
185	Thickness vertical lamella	30	mm											
186	width steel beam	250	mm											
187	Area	15000	mm ²	B186*B185*2										
188	$\sigma_{c0,d}$	4,51	N/mm ²	B184*1000/B187										
189														
190	γ_M	1,25												
191	Load duration	long-term												
192	Service class	1												
193	k_{mod}	0,7												
194	$f_{c0,d}$	11,76	N/mm ²	B193*B25/B190										
195	UC	0,38	-	B188/B194										
196														
197														
198	Verification bending k2/k4													
199	$M_{ULS_Matrixframe}$	14,97	kNm											
200	$\sigma_{rel,s}$	0,26	N/mm ²	B199*10^6 * 0,5 * B7 / (B12)										
201	γ_M	1,25												
202	Load duration	long-term												
203	Service class	1												
204	k_{mod}	0,7												
205	$f_{M,d}$	13,44	N/mm ²	B204*B26/B201										
206	UC	0,02	-	B200/B205										
207														
208	Verification shear stress k2													
209	V_{ULS}	35,95	kN											
210	Thickness horizontal lamella	40	mm											
211	$A_{v,net}$	118000	mm ²	B210*B6										
212	τ_s	0,46	N/mm ²	1,5*B209*1000/B211										
213	γ_M	1,25												
214	Load duration	long-term												
215	Service class	1												
216	k_{mod}	0,7												
217	$f_{v,d}$	2,24	N/mm ²	B27*B216/B213										
218	UC	0,20	-	B212/B217										



Load spread at support (Borgström & Fröbel, 2019: p.131)

Figure X.85: Structural verification CLT wall panel as beam (3)_Excel

Hand validation wall panel as 'beam' sheet on next page.

Order /
 Blad nr 1/4
 Deel Verification CLT wall as 'beam'
 Datum 04/05/2023



Panel properties	Load properties
Layers: 3 build-up: 30-40-30 total thickness: 100mm Wall height (h) = b _y = 2550 mm b _x = 1000 mm I _{x,rel} = 7,800 E+06 [mm ⁴] A _{x,rel} = 6,00 · 10 ⁴ [mm ²] W _{x,rel} = 1,50 · 10 ³ [mm ³] I _{y,rel} = 8,56 · 10 ¹⁰ [mm ⁴] I _{w,rel} = 6,55 · 10 ⁷ [mm ⁴] i _{w,rel} = 34,04 [mm] mass = 42 [kg/m ³]	S ₂₄ E _{x,0.05} = 7400 [N/mm ²] E _{y,mean} = 1100 [N/mm ²] f _{c,0,k} = 21 [N/mm ²] f _{t,0,k} = 24 [N/mm ²] F _{u,0,k} = 4 [N/mm ²]
	Structural scheme
	floor span (between walls) = 3.5 [m]
<u>Load definition</u> eq 6.10a : 1.35 G + 1.5 Q ₁ eq 6.10b : 1.2 G + 1.5 Q ₁ + 1.5 Q ₂	
ψ _{roof} = 0 ψ _{floor} = 0.4	
<u>Roof panel top floor</u>	
permanent:	0.786 × 3.5/2 = 1.38 [kN/m]
variable:	1 × 3.5/2 = 1.75 [kN/m]
eq 6.10a:	1.35 · 1.38 + 1.5 · 0.175 = 1.86 [kN/m]
eq 6.10b:	1.2 · 1.38 + 1.5 · 0.175 = 1.66 [kN/m]
<u>Floor panel top floor</u>	
permanent:	0.564 × 3.5/2 = 0.99 [kN/m]
variable:	1.75 × 3.5/2 = 3.06 [kN/m]
non-str. walls:	0 × 3.5/2 = 0.85 [kN/m]
eq 6.10a:	1.35 · 0.99 + 1.5 · 0.4 · (3.06 + 0.85) = 4.65 [kN/m]
eq 6.10b:	(1.2 · 0.99) + 1.5 · 0.4 · (3.06 + 0.85) = 3.15 [kN/m]
<u>Roof panel 2nd floor</u> (similar to top floor)	
eq 6.10a:	1.38 [kN/m]
eq 6.10b:	1.66 [kN/m]

Figure X.86: Validation excel wall as 'beam' (1)

Order /
 Blad nr 2/4
 Deel Verificatie CLT wall as 'beam'
 Datum 04/05/2022



Floor panel \rightarrow Floor (similar to top floor):
 eg b.w.g: 4.65 [kN/m]
 eg b.w.b: 7.15 [kN/m]

Wall panel itself
 permanent: [kN/m²] [m]
 self-weight: 0.42 \times 2.55 \times 2 = 2.48 [kN/m]
 facade: 0.5 \times 2.55 \times 2 = 2.55 [kN/m]
 Fire safety board: 0.14 \times 2.55 \times 2 = 0.72 [kN/m]
 6.26 [kN/m]

eg b.w.g = 1.55 \times 6.26 = 9.45 [kN/m] \rightarrow S
 eg b.w.b = 1.2 \times 6.26 = 7.51 [kN/m] \rightarrow S

Point loads due to front facade
 permanent: [kN/m²] [m] [m]
 0.5 \times 3.5/2 \times 2.55 \times 2 = 5.16 [kN]

eg b.w.g: 1.55 \times 5.16 = 6.97 [kN] \rightarrow S \rightarrow on 6.97 [kN] (sheet)
 eg b.w.b: 1.2 \times 5.16 = 6.19 [kN] \rightarrow S \rightarrow on 6.20 [kN] (sheet)

Total load
 eg b.w.g: (1.86 \times 2) + (4.65 \times 2) + 9.45 = 21.47 [kN/m] \rightarrow S \rightarrow 21.47 [kN/m]
 eg b.w.b: (1.66 \times 2) + (7.15 \times 2) + 7.51 = 25.03 [kN/m] \rightarrow S \rightarrow 25.03 [kN/m] (sheet values)

Defined loads correct \rightarrow S

Results Matrix Frame
Support reactions
 reaction used for links

Δ	Δ	Δ	Δ	Δ
\uparrow	\uparrow	\uparrow	\uparrow	\uparrow
-25.46	-67.67	-54.95	-67.67	-25.46

$M_{max-rot} = 14.97$ [kNm]
 $V_{max-rot} = 35.95$ [kNm]
 } Forces

Figure X.87: Validation excel wall as 'beam' (2)

Order /
 Blad nr 3 / 4
 Deel Verification CLT wall as 'beam'
 Datum 04/05/2023



1. Verification buckling at support (For 1 m wall)

$$\lambda_y = l_e / i_{y,eff} = 250 / 24.04 = 10.40 [-]$$

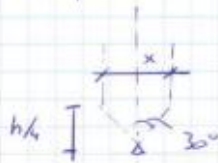
$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{k_{0,y}}{E_{0.05}}} = \frac{10.40}{\pi} \sqrt{\frac{0.31}{71000}} = 1.47 [-]$$

$$k_{y} = 0.5 (1 + 0.5 (1.47 - 0.5) + 1.47^2) = 1.64 [-]$$

$$k_{l,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} = \frac{1}{1.64 + \sqrt{1.64^2 - 1.47^2}} = 0.42 [-]$$

↔ 0.42 (steel)

Load spread:



$$h/4 = 250/4 = 62.5 \text{ [mm]}$$

$$\tan(30) = \frac{x}{h/4} \Rightarrow x = 2 \tan(30) \cdot \frac{h}{4} = 2 \tan(30) \cdot 62.5 = 85.59 \text{ [mm]}$$

$$g_{normal-x} = \frac{N}{x} = \frac{67.67}{85.59} = 79.46 \text{ [kN/m]}$$

$$\text{For 1 [m]} \Rightarrow q = \frac{79.46 \times 1 \text{ [m]}}{0.25 \text{ [m]}} = 317.84 \text{ [kN/m]}$$

$$\sigma_{c,d} = \frac{q}{\sum \text{vertical lamellas}} = \frac{317.84}{2.30} = 138.2 \text{ [N/mm}^2]$$

↔ 1.56 [N/mm²] (steel value)

$$\left. \begin{matrix} \mu = 0.25 \\ \mu_{min} = 0.7 \end{matrix} \right\} f_{c,d} = \frac{2 \cdot 0.25}{0.25} = 1.76 \text{ [N/mm}^2]$$

$$UC = \frac{1.56}{1.76} = 0.88 [-]$$

↔ 0.71

2. Verification normal stress at support

$$N_{ed} = 67.67 \text{ [kN]}$$

$$A = 250 \text{ [mm]} \times (30 \times 2) = 15000 \text{ [mm}^2]$$

(beam 250 [mm] width assumed)

$$\sigma_{c,d} = \frac{N}{A} = \frac{67.67 \cdot 10^3}{15000} = 4.51 \text{ [N/mm}^2]$$

↔ 4.51 [N/mm²] (steel value)

$$UC = \frac{\sigma_{c,d}}{f_{c,d}} = \frac{4.51}{1.76} = 2.56 [-]$$

↔ 0.30 (steel value)

Figure X.88: Validation excel wall as 'beam' (3)

Order /
 Blad nr 4/4
 Deel Verificatie CLT wall as 'beam'
 Datum 04/05/2023

VAN ROSSUM RAADGEVENDE
 INGENIEURS

3. Bending

$$\sigma_{m,ult} = \frac{M \cdot i_{h_{wall}}}{I_{z,wall}} = \frac{14,57 \cdot 10^6}{8,56 \cdot 10^{10}} \cdot \frac{1}{2} \cdot 2530 = 0,26 \text{ [N/mm}^2\text{]}$$

$$\gamma_M = 1,25 \text{ [-]}$$

$$k_{red} = 0,7 \text{ [-]}$$

$$k_{cr,stab}$$

$$\sigma_{M,d} = \frac{24 \cdot 0,7 \cdot 0,26}{1,25} = 13,44 \text{ [N/mm}^2\text{]}$$

$$\sigma \Leftrightarrow 0,26 \text{ [N/mm}^2\text{]} \text{ (steel)}$$

$$UC = \frac{0,26}{13,44} = 0,02 \text{ [-]} \quad \sigma \Leftrightarrow 0,02 \text{ [-]} \text{ (steel)}$$

4. Verification shear stress

$$\tau_d = \frac{3}{2} \frac{V_{ed}}{t_{hor} \cdot h_{panel}} = \frac{3}{2} \frac{35,95 \cdot 10^3}{40 \cdot 2550} = 0,46 \text{ [N/mm}^2\text{]}$$

$$f_{v,ed} = 2,24 \text{ N/mm}^2 \quad (f_{v,red} = 0,7; \gamma_M = 1,25)$$

$$\hookrightarrow UC = \frac{0,46}{2,24} = 0,20 \text{ [-]} \quad \sigma \Leftrightarrow 0,20 \text{ [-]} \text{ (steel value)}$$

(steel correct)

Figure X.89: Validation excel wall as 'beam' (4)

X.10.5 CLT Wall buckling

X.10.5.1 Maximal vertical weight

	A	B	C	D	E	F	G	H	I	J	K
1	Chosen wall panel										
2	Panel										
3	Number of layers	3	-								
4	Panel build-up	30-40-30	mm								
5	Thickness panel	100	mm								
6	Wall height (l_e)	2950	mm								
7	b_x	300	mm								
8	$I_{x,net}$	2,340E+07	mm ⁴								
9	$A_{x,net}$	1,80E+04	mm ²								
10	$W_{x,net}$	4,68E+05	mm ³								
11	$I_{x,eff}$ ($l_{ref} = 2950$ mm)	2,09E+07	mm ⁴								
12	$i_{x,eff}$	34,04	mm								
13	m	42	kg/m ²								
14											
15	Wood										
16	Strength class	C24									
17	$\rho_{slam,mean}$	420	kg/m ³								
18	$\rho_{slam,k}$	350	kg/m ³								
19	$E_{0,x,0.05}$	7400	MPa								
20	$E_{0,x,mean}$	11000	MPa								
21	$G_{090,x,lay,mean}$	50	MPa								
22	$G_{090,x,lay,mean}$	690	MPa								
23	$f_{c,0,x,k}$	21	N/mm ²								
24	$f_{m,X,k}$	24	N/mm ²								
25											
26	Structural scheme										
27	Floorspan	3,5	m								
28	Width column	0,3	m								
29	Width incl. half gallery	0,9	m								
30											
31											
32											
33											
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44	Load definition above column (at gallery)										
45	Load combinations (NEN1990; CC2)	<i>Variable residential load is leading action</i>									
46		γ_G	γ_Q								
47	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$							
48	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_{Q,1} + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$							
49											
50			ψ_0								
51	Roof	0									
52	Floor	0,4									
53											
54											
55	Roofpanel topfloor	kN/m ²	kN/m								
56	Permanent	0,786	4,13								
57	Variable	1	5,25								
58		Eq 6.10a	5,57								
59		Eq 6.10b	4,95								
60											

Formules C
 $B56 * B27 / 2 * (B29 / B28)$
 $(B57 * B27 / 2) * (B29 / B28)$
 $(C56 * B47) + (C57 * B51 * C47)$
 $(B48 * C56) + (C48 * B51 * C57)$

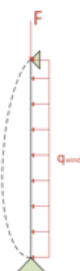
Figure X.90: Structural verification CLT wall panel buckling max vertical load (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
61	Floorpanel topfloor	kN/m ²	kN/m								
62	Permanent	0,964	5,06				B62*B27/2 * (B29/B28)				
63	Variable										
64	Residential loading	1,75	9,19				B64*B27/2 * (B29/B28)				
65	Non-structural walls	0,5	2,63				B65*B27/2 * (B29/B28)				
66		Eq 6.10a	13,92				(B47*C62) + (C47*(C64+C65)*B52)				
67	Full loaded	Eq 6.10b	21,43				(B48*C62) + (C48*C64) + (C48*C65*B52)				
68											
69	Roofpanel 1st floor	kN/m ²	kN/m								
70	Permanent	0,786	4,13				B70* (B27/2) * (B29/B28)				
71	Variable	Not present	-								
72		Eq 6.10a	5,57				B47*C70				
73		Eq 6.10b	4,95				B48*C70				
74											
75	Floorpanel 1st floor	kN/m ²	kN/m								
76	Permanent	0,964	5,06				B76*B27/2 * (B29/B28)				
77	Variable										
78	Residential loading	1,75	9,19				B78*B27/2 * (B29/B28)				
79	Non-structural walls	0,5	2,63				B79*B27/2 * (B29/B28)				
80		Eq 6.10a	13,92				(B47*C76)+(C47*B52*(C78+C79))				
81	Full loaded	Eq 6.10b	21,43				(B48*C76) + (C48*C78) + (C48*B52*C79)				
82											
83	Wall panel itself										
84	h _{door}	2115	mm								
85	Number of doors	2	-				Formule B				
86	h _{reduced} ; height without openings	1670	mm				(2 * B6) - (B85*B84)				
87	A _{total}	2,77	m ²				((B28*(B6/1000)*2) + ((B29-B28) * (B86/1000)))				
88	h _{average} ; 2 floors	9,24	m			(Includes at gallery)	B87/B28				
89											
90	Permanent load	kN/m ²	kN/m				Formule C				
91	Self-weight	0,42	3,88				B91*B88				
92	Facade	0,5	4,62				B92*B88				
93	Fire safety board	0,14	1,29				B93*B88				
94		Total	9,79				SOM(C91:C93)				
95		Eq 6.10a	13,22				B47*C94				
96		Eq 6.10b	11,75				B48*C94				
97											
98	Load due to frond facade	kN/m ²	kN/m								
99	Permanent	0,5	17,21			Devided over width column	B99*(B6*2/1000) * B27/2 / B28				
100		Eq 6.10a	23,23				C99*B47				
101		Eq 6.10b	20,65				C99*B48				
102											
103	Total load						Formule B				
104	Eq 6.10a	75,43	kN/m				C58+C66+C72+C80+C95+C100				
105	Eq 6.10b	85,17	kN/m				C59+C67+C73+C81+C96+C101				
106	q _{ULS}	85,17	kN/m				MAX(B104:B105)				
107											
108											
109											
110	Verification buckling			Formule B							
111	Reduction factor for buckling										
112	λ _x	86,66	-	B6/B12							
113	λ _{rel,x}	1,47	-	(B112/PI()) * WORTEL(B23/B19)							
114	k _y	1,64	-	0,5 * (1 + (0,1 * (B113-0,3)) + B113^2)							
115	k _{c,y}	0,42	-	1 / (B114 + WORTEL(B114^2 - B113^2))							
116											
117	Load and verification										
118	Thickness vertical panels	30	mm								
119	σ _{c,0,d}	1,42	N/mm ²	B106/(2*B118)							
120											
121	γ _M	1,25									
122	Load duration	long-term									
123	Service class	1									
124	k _{mod}	0,7									
125	f _{c,0,d}	11,76	N/mm ²	B23*B124/B121							
126	UC	0,29	-	B119 / (B125*B115)							

Figure X.91: Structural verification CLT wall panel buckling max vertical load (2)_Excel

X.10.5.2 Including horizontal wind load

	A	B	C	D	E	F	G	H	I	J	K
1	Chosen wall panel										
2	Panel										
3	Number of layers	3	-								
4	Panel build-up	30-40-30	mm								
5	Thickness panel	100	mm								
6	Wall height (l_e)	2950	mm								
7	b_k	300	mm								
8	$I_{x,net}$	2,340E+07	mm ⁴								
9	$A_{x,net}$	1,80E+04	mm ²								
10	$W_{x,net}$	4,68E+05	mm ³								
11	$I_{x,eff}$ ($l_{ref} = 2950$ mm)	2,09E+07	mm ⁴								
12	$i_{x,eff}$	34,04	mm								
13	m	42	kg/m ²								
14											
15	Wood										
16	Strength class	C24									
17	$\rho_{xlam,mean}$	420	kg/m ³								
18	$\rho_{xlam,k}$	350	kg/m ³								
19	$E_{0,x,0.05}$	7400	MPa								
20	$E_{0,x,mean}$	11000	MPa								
21	$G_{90,0,xlay,mean}$	50	MPa								
22	$G_{90,xlay,mean}$	690	MPa								
23	$f_{c,0,x,k}$	21	N/mm ²								
24	$f_{m,x,k}$	24	N/mm ²								
25											
26	Structural scheme										
27	Floorspan	3,5	m								
28	Width column	0,3	m								
29	Width incl. half gallery	0,9	m								
30											
31											
32											
33											
34											
35											
36											
37											
38											
39											
40											
41											
42											
43											
44	Load definition above column (at gallery)										
45	Load combinations (NEN1990; CC2)	γ_G	γ_Q								
46											
47	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_0$							
48	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_{Q,1} + \sum Q_{k,i} \gamma_{Q,i} \psi_0$							
49											
50											
51											
52											
53											
54	Horizontal load										
55	p (22.4m) (Wind area II; Urban)	0,93	kN/m ²								
56	$C_s C_d$	1	-								
57	$C_{f,D}$	0,8	-								
58	$C_{f,E}$	0,7	-								
59	$C_{f,tot}$	1,5	-								
60	$q_{wind,SLS}$	1,26	kN/m								
61	$q_{wind,ULS}$	1,88	kN/m								
62	$M_{wind,ULS}$	2048622,89	Nmm								
63											
64	Vertical load										



		Formule B	
		B57+B58	
		B55*B56*B59*B29	
		B60*C48	
		(1/8)*B61*B6^2	

Figure X.92: Structural verification CLT wall panel buckling incl. wind load (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
65	Roofpanel topfloor	kN/m ²	kN/m				Formule C				
66	Permanent	0,786	4,13				B66*B27/2 * (B29/B28)				
67	Variable	1	5,25				(B67*B27/2) * (B29/B28)				
68		Eq 6.10b	4,95				(B48*C66) + (C48*B51*C67)				
69											
70	Floorpanel topfloor	kN/m ²	kN/m								
71	Permanent	0,964	5,06				B71*B27/2 * (B29/B28)				
72	Variable										
73	Residential loading	1,75	9,19				B73*B27/2 * (B29/B28)				
74	Non-structural walls	0,5	2,63				B74*B27/2 * (B29/B28)				
75		Eq 6.10b	13,16				(B48*C71) + (C48*B52*(C73+C74))				
76											
77	Roofpanel 1st floor	kN/m ²	kN/m								
78	Permanent	0,786	4,13				B78 * (B27/2) * (B29/B28)				
79	Variable	Not present	-								
80		Eq 6.10b	4,95				B48*C78				
81											
82	Floorpanel 1st floor	kN/m ²	kN/m								
83	Permanent	0,964	5,06				B83*B27/2 * (B29/B28)				
84	Variable										
85	Residential loading	1,75	9,19				B85*B27/2 * (B29/B28)				
86	Non-structural walls	0,5	2,63				B86*B27/2 * (B29/B28)				
87		Eq 6.10b	13,16				(B48*C83) + (C48 * B52*(C85+C86))				
88											
89	Wall panel itself										
90	h _{door}	2115	mm								
91	Number of doors	2	-				Formule B				
92	h _{reduced} ; height without openings	1670	mm				(2 * B6) - (B91*B90)				
93	A _{total}	2,77	m ²				((B28*(B6/1000)*2) + ((B29-B28) * (B92/1000)))				
94	h _{average2 floors}	9,24	m			(Includes at gallery)	B93/B28				
95											
96	Permanent load	kN/m ²	kN/m				Formule C				
97	Self-weight	0,42	3,88				B97*B94				
98	Facade	0,5	4,62				B98*B94				
99	Fire safety board	0,14	1,29				B99*B94				
100		Total	9,79				SOM(C97:C99)				
101		Eq 6.10b	11,75				B48*C100				
102											
103	Load due to frond facade	kN/m ²	kN/m								
104	Permanent	0,5	17,21			Devided over width column	B104*(B6*2/1000) * B27/2 / B28				
105		Eq 6.10b	20,65				B48*C104				
106											
107	Total load	Vertical		Horizontal			Formule B				
108	q _{UL5} Eq 6.10b	68,63	kN/m	2,05E+06	Nmm		C68+C75+C80+C87+C101+C105				
109											
110											
111											
112	Verification buckling										
113	Reduction factor for buckling						Formule B				
114	λ _x	86,66	-				B6/B12				
115	λ _{rel,x}	1,47	-				(B114/PI()) * WORTEL(B23/B19)				
116	k _y	1,64	-				0,5 * (1 + (0,1 * (B115-0,3)) + B115^2)				
117	k _{c,y}	0,42	-				1 / (B116 + WORTEL(B116^2 - B115^2))				
118											
119	Load and verification										
120	Thickness vertical panels	30	mm								
121	σ _{c,0,d}	1,14	N/mm ²				B108/(2*B120)				
122	σ _{M,d}	4,38	N/mm ²				D108/B10				
123											
124	γ _M	1,25	-								
125	Load duration	long-term	-								
126	Service class	1	-								
127	k _{mod}	0,7	-								
128	k _{th}	1,10	-				MIN((600/B5)^0,1; 1,1)				
129	f _{c,0,d}	11,76	N/mm ²				B23*B127/B124				
130	f _{M,d}	14,78	N/mm ²				B24*B127*B128/B124				
131	UC	0,53	-				(B121 / (B129*B117)) + (B122/B130)				

Figure X.93: Structural verification CLT wall panel buckling incl. wind load (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K
132											
133											
134	Verification deflection										
135	$U_{inst,Q}$	5,40	mm	$(5/384)*B60*B6^4 / (B20*B11)$							
136	Load duration	short-term									
137	Service class	1									
138	k_{def}	0,6									
139	ψ_2	0,3									
140	$U_{fin,Q}$	6,37	mm	$B135 * (1 + (B139*B138))$							
141	$W_{allowed}$	9,83	mm	$B6/300$							
142	UC	0,65		$B140/B141$							

Figure X.94: Structural verification CLT wall panel buckling incl. wind load (2)_Excel

Hand verification CLT Wall buckling both load combinations on next page.

X.10.5.3 Hand validation buckling

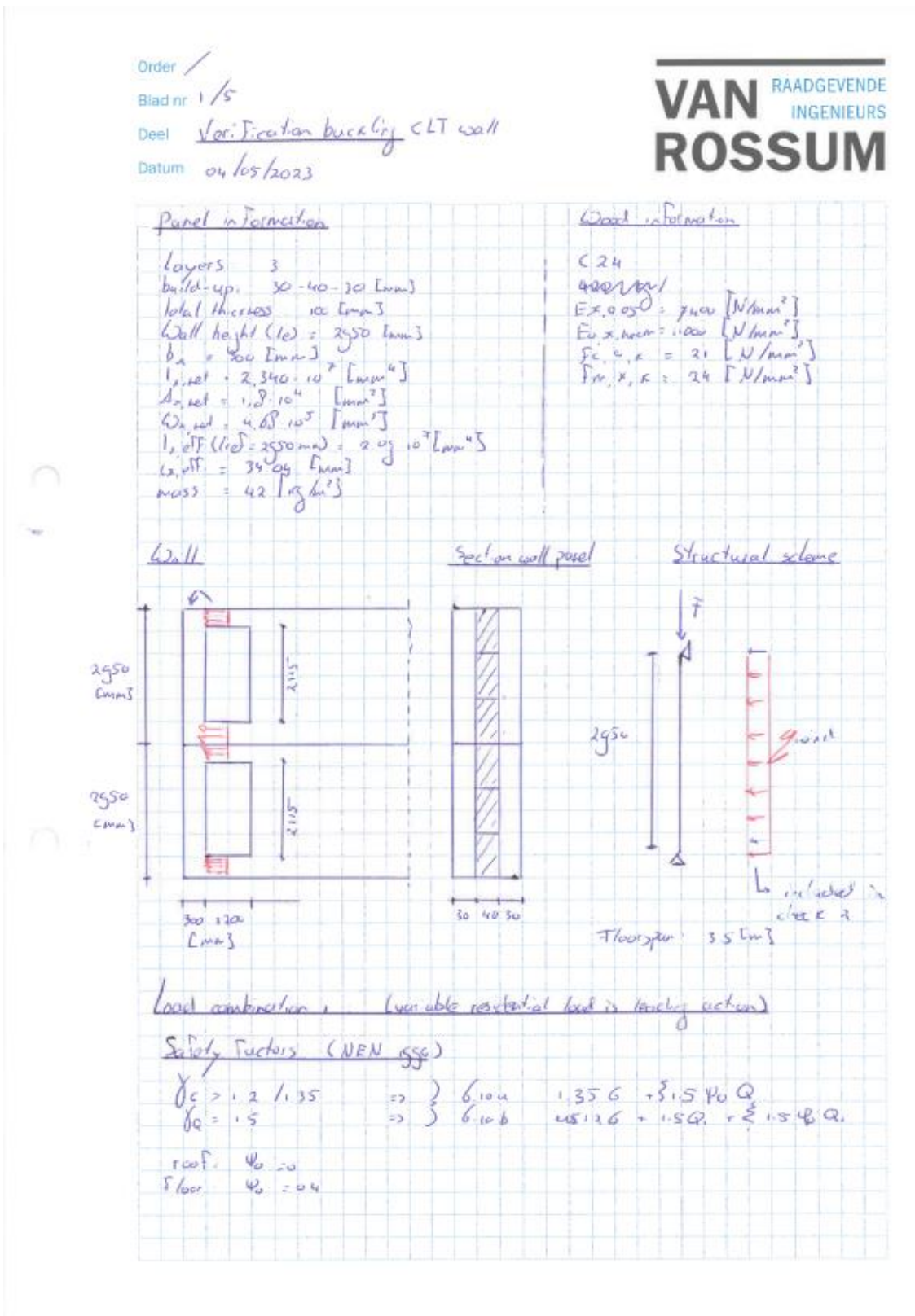


Figure X.95: Validation excel wall buckling (1)

Order /
 Blad nr 2/5
 Deel Verification buckling CLT wall
 Datum 04/05/2023



Loads

Roof panel top floor $[kN/m^2]$ $[m]$ ↗ g_{roof} only over column

permanent = $0.786 \times 3.5/2 \times 0.9/0.3 = 4.13 [kN/m]$

variable = $1 \times 3.5/2 \times 0.9/0.3 = 5.25 [kN/m]$

eg 6.10a = $(1.35 \cdot 4.13) + (1.5 \cdot 5.25) = 5.58 [kN/m]$ ↗

eg 6.10b = $(1.2 \cdot 4.13) + (1.5 \cdot 5.25) = 4.96 [kN/m]$ ↘

Floor panel top floor

permanent = $0.964 \times 3.5/2 \times 0.9/0.3 = 5.06 [kN/m]$ ↗ g_{fl}

variable = $1.75 \cdot 3.5/2 \cdot 0.9/0.3 = 9.19 [kN/m]$

non-str. loads = $0.5 \cdot 3.5/2 \cdot 0.9/0.3 = 2.63 [kN/m]$

eg 6.10a = $(1.35 \cdot 5.06) + (1.5 \cdot 0.4 \cdot (9.19 + 2.63)) = 13.92 [kN/m]$ ↗

eg 6.10b = $(1.2 \cdot 5.06) + (1.5 \cdot 9.19) + (1.5 \cdot 0.4 \cdot 2.63) = 21.43 [kN/m]$ ↘

↗ full load

Roof panel 1st floor

permanent = $0.786 [kN/m^2] \times 3.5/2 \times 0.9/0.3 = 4.13 [kN/m]$

Variable = not present

eg 6.10a = $1.35 \cdot 4.13 = 5.58 [kN/m]$ ↗

eg 6.10b = $1.2 \cdot 4.13 = 4.96 [kN/m]$ ↘

Floor panel 1st floor

↗ similar to top floor

eg 6.10a = $13.92 [kN/m]$ ↗

eg 6.10b = $21.43 [kN/m]$ ↘

Wall panel self

$h_{tot} = 2 \cdot h_{clear} = (2550 \times 2) - (215 \times 2) = 1670 [mm] = h_{reduced}$

$A_{tot} = (h_{tot} \cdot a_{ins}) + (4 \cdot h_{reduced} \cdot a_{brk} \cdot 0.06) = (2550 \times 2 \times 0.300) + (1670 \times 600) = 277000 [mm^2]$

↗ spread over column: $h_{column, req} = 277000 / 300 = 9240 [mm]$

permanent load $[kN/m^2]$ $[m]$

self weight = $(42 \cdot 10 / 1000) \cdot (9240 / 1000) = 3.88 [kN/m]$

Facade = $0.5 \cdot (9240 / 1000) = 4.62 [kN/m]$

Fine board = $0.4 \cdot (9240 / 1000) = 3.70 [kN/m]$

eg 6.10a = $1.35 \cdot 9.75 = 13.22 [kN/m]$ ↗

eg 6.10b = $1.2 \cdot 9.75 = 11.75 [kN/m]$ ↘

Figure X.96: Validation excel wall buckling (1)

Order /
 Blad nr 3/5
 Deel Verificatie buckling LET wall
 Datum 04/05/2023

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$$\text{Front facade permanent} = 0.5 \cdot (3.5/2) \cdot (2750 \text{ mm}) / 0.3 \cdot 17.21 \text{ [kN/m]}$$

\uparrow weight (kN/m²) \uparrow width span \uparrow total height \uparrow divided over column.

$$q_{g,10a} = 1.35 \cdot 17.21 = 23.23 \text{ [kN/m]} \quad \text{S}$$

$$q_{g,10b} = 1.2 \cdot 17.21 = 20.65 \text{ [kN/m]} \quad \text{S}$$

Total load

$$q_{g,10a} = 5.58 + 13.92 + 5.58 + 13.92 + 15.22 + 23.23 = 75.45 \text{ [kN/m]} \quad \text{S} \rightarrow 75.45 \text{ [kN/m]} \text{ (slat value)}$$

$$q_{g,10b} = 4.96 + 21.43 + 4.96 + 21.43 + 11.75 + 20.65 = 85.18 \text{ [kN/m]} \quad \text{S} \rightarrow 85.18 \text{ [kN/m]} \text{ (slat value)}$$

$$\hookrightarrow \text{Normative Guls} = 85.18 \text{ [kN/m]}$$

Buckling verification

$$\lambda_y = l_e / i_{x,eff} = 2750 / 34.04 = 80.66 \text{ [-]}$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{F_{00,r}}{E_{005}}} = \frac{80.66}{\pi} \sqrt{\frac{21}{7400}} = 1.47 \text{ [-]}$$

$$\eta_y = 0.5 (1 + 0.1(\lambda_{rel} - 0.3)) + \lambda_{rel,y} = 0.5(1 + 0.1(1.47 - 0.3)) + 1.47 = 1.64 \text{ [-]} \quad \text{S}$$

$$k_{c,y} = \frac{1}{\eta_y \cdot \sqrt{\eta_y^2 + \lambda_{rel}^2}} = \frac{1}{1.64 \cdot \sqrt{1.64^2 + 1.47^2}} = 0.42 \text{ [-]} \quad \text{S} \rightarrow 0.42 \text{ [-]} \text{ (slat value)}$$

$$\sigma_{c,ed} = \frac{\text{Guls}}{2 \text{ thickness without knells}} = \frac{85.18}{30730} = 1.42 \text{ [N/mm}^2\text{]} \quad \text{S}$$

$$\eta_M = 1.25 \quad \left. \begin{array}{l} \eta_{red} = 0.7 \\ \end{array} \right\} \sigma_{c,ed} = \frac{F_{00,r} \cdot \eta_{red}}{\eta_M} = \frac{21 \cdot 0.7}{1.25} = 11.76 \text{ [N/mm}^2\text{]}$$

$$\omega_c = \frac{\sigma_{c,ed}}{k_{c,y} \cdot F_{c,ed}} = \frac{1.42}{0.42 \cdot 11.76} = 0.29 \text{ [-]} \quad \text{S} \rightarrow 0.29 \text{ [-]} \text{ (slat value)}$$

Figure X.97: Validation excel wall buckling (3)

Order
 Blad nr 4/5
 Deel Verificatie buckling CLT panel
 Datum 04/05/2023



Load combination 2 - (Wind load is leading action)

Safety factors \rightarrow defined in K1

Max. zonal wind-load:

$s_p = 0.93$
 $s_{ct} = 1$
 $s_{ref} = 1.0$

$g_{0,ult} = 0.93 \cdot 1 \cdot 1.5 \cdot 0.8 = 1.26 \text{ [kN/m]} \quad \text{(sheet)}$

assume wind additional over half of height (side assumption)

$g_{0,ult} = 1.26 \cdot 1.5 = 1.89 \text{ [kN/m]} \quad \rightarrow 1.89 \text{ [kN/m]}$

$M_{wind,ult} = \frac{1}{8} g l^2 = \frac{1}{8} \cdot 1.89 \cdot 2950^2 = 2045087.5 \text{ [Nm]} \quad \rightarrow 2045087.5 \text{ [Nm]}$

Vertical loads

Roof panel top floor

permanent: $4.13 \text{ [kN/m]} \quad \rightarrow 6.10b = (1.2 \cdot 4.13) + (1.5 \cdot 0.5 \cdot 25) = 4.56 \text{ [kN/m]} \quad \rightarrow 4.56 \text{ [kN/m]}$

Variable: 5.25 [kN/m]

Floor panel top floor

permanent: 5.06 [kN/m]

Variable total: $5.19 + 2.63 = 11.82 \text{ [kN/m]}$

$6.10b = (1.2 \cdot 5.06) + (1.5 \cdot 0.4 \cdot 11.82) = 13.16 \text{ [kN/m]} \quad \rightarrow 13.16 \text{ [kN/m]}$

Roof panel 1st Floor (= same as top) $\rightarrow 4.56 \text{ [kN/m]}$

Floor panel 1st Floor (= same as top) $\rightarrow 13.16 \text{ [kN/m]}$

Wall panel (see K1) $\rightarrow 11.75 \text{ [kN/m]}$

Load Front Facade (See K1) $\rightarrow 20.65 \text{ [kN/m]}$

Total load $6.10b = (4.56 \cdot 2) + (13.16 \cdot 2) + 11.75 + 20.65 = 68.69 \text{ [kN/m]} \quad \rightarrow 68.69 \text{ [kN/m]} \quad \text{(sheet)}$

Verification buckling

$k_{c,y} = 0.42 \text{ (see K1)}$

$\sigma_{c,0,d} = \frac{g_{0,d}}{\sum E \text{ vertical lamella's}} = \frac{68.69}{1.30} = 1.14 \text{ [N/mm}^2] \quad \rightarrow 1.14 \text{ [N/mm}^2] \quad \text{(sheet)}$

$\sigma_{M,d} = \frac{M}{W_x} = \frac{2045087.5}{4.68 \cdot 10^7} = 4.37 \text{ [N/mm}^2] \quad \rightarrow 4.37 \text{ [N/mm}^2] \quad \text{(sheet)}$

$\delta M = 1.25$

$\delta_{swol} = 0.7$

$k_h = 1.10$

$f_{M,d} = 24.07 \cdot 1.1 \cdot 1.25 = 14.78 \text{ [N/mm}^2] \quad \rightarrow 14.78 \text{ [N/mm}^2] \quad \rightarrow 14.78 \text{ [N/mm}^2]$

$f_{c,0,d} = 21.07 / 1.25 = 11.76 \text{ [N/mm}^2] \quad \rightarrow 11.76 \text{ [N/mm}^2] \quad \rightarrow 11.76 \text{ [N/mm}^2]$

$UC = \frac{\sigma_{c,0,d}}{k_{ty} \cdot f_{c,0,d}} + \frac{\sigma_{M,d}}{f_{M,d}} = \frac{1.14}{0.42 \cdot 11.76} + \frac{4.37}{14.78} = 0.53 \text{ [-]} \quad \rightarrow 0.53 \text{ [-]} \quad \text{(sheet)}$

Figure X.98: Validation excel wall buckling (4)

Order /
 Blad nr 5/5
 Deel Verification buckling of CTF wall
 Datum 04/05/2023

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Verification deflection

$$w_{s1,q} = \frac{5}{384} \frac{q_{s1} L^4}{E_{masonry,eff}} = \frac{5}{384} \frac{126 \cdot 2750^4}{11000 \cdot 2 \cdot 10^7} = 5.40 \text{ [mm]} \quad \text{S}$$

$$k_{def} = 0.6 \quad \psi_2 = 0.5$$

$$w_{s1,q} = w_{s1,q} (1 + k_{def} \psi_2) = 5.40 (1 + 0.6 \cdot 0.5) = 6.37 \text{ [mm]}$$

$$UL = \frac{w_{s1,q}}{L/300} = \frac{6.37}{2750/300} = 0.69 \text{ L-3} \quad \text{S} \quad \leftrightarrow 0.65 \text{ (std)}$$

All excel's are correct for buckling

Figure X.99: Validation excel wall buckling (5)

X.10.6 CLT Wall Lintels

X.10.6.1 Lintel above gallery

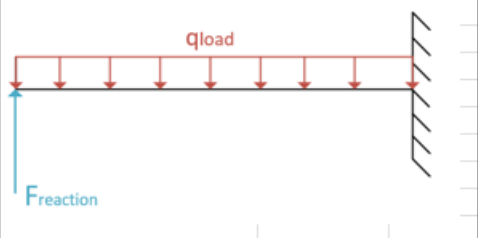
	A	B	C	D	E	F	G	H	I	J
1	Chosen wall panel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness panel	100	mm							
6	Wall height (l_w)	2950	mm							
7	m	42	kg/m ²							
8										
9	Wood									
10	Strength class	C24								
11	$\rho_{xlam,mean}$	420	kg/m ³							
12	$\rho_{xlam,k}$	350	kg/m ³							
13	$E_{0,x,0.05}$	7400	MPa							
14	$E_{0,x,mean}$	11000	MPa							
15	$G_{9090,xlay,mean}$	50	MPa							
16	$G_{90,xlay,mean}$	690	MPa							
17	$f_{c,0,x,k}$	21	N/mm ²							
18	$f_{m,x,k}$	24	N/mm ²							
19	$f_{V,090,k}$	4	N/mm ²							
20										
21	Structural scheme									
22	Span lintel	1,2	m							
23	Span floors	3,5	m							
24	Height lintel	485	mm							
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										
37	Load definition									
38	Load combinations (NEN1990; CC2)		<i>Variable residential load is leading action</i>							
39		γ_G	γ_Q							
40	6.10a	1,35	1,5	$q_{led} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_0$						
41	6.10b	1,2	1,5	$q_{led} = G_k \gamma_G + Q_{k,1} \gamma_{Q,1} + \sum Q_{k,i} \gamma_{Q,i} \psi_0$						
42										
43			ψ_0							
44	Roof	0								
45	Floor	0,4								
46										
47	Floorpanel topfloor	kN/m ²	kN/m							Formule C
48	Permanent	0,964	1,69							B48*B23/2
49	Variable									
50	Residential loading	1,75	3,06							B50*B23/2
51	Non-structural walls	0,5	0,88							B51*B23/2
52		Eq 6.10a	4,64							(B40*C48) + (C40*B45* (C50+C51))
53		Eq 6.10b	7,14							(B41*C48) + (C41*C50) + (C41*B45*C51)
54										
55	Roofpanel 1st floor	kN/m ²	kN/m							
56	Permanent	0,786	1,38							B56*B23/2
57	Variable	Not present	-							
58		Eq 6.10a	1,86							B40*C56
59		Eq 6.10b	1,65							B41*C56

Figure X.100: Structural verification CLT wall lintel above gallery (1)_Excel


	A	B	C	D	E	F	G	H	I	J
60										
61	Wall panel						Formule B			
62	Height lintel	485 mm					B24			
63	Height above lintel till opening	470 mm								
64	Total	955 mm					SOM(B62:B63)			
65										
66		kN/m ²	kN/m				Formule C			
67	Permanent	0,42	0,40				B67*B64/1000			
68		Eq 6.10a	0,54				B40*C67			
69		Eq 6.10b	0,48				B41*C67			
70										
71	Total load						Formule B			
72	Eq 6.10a	7,04 kN/m					C52+C58+C68			
73	Eq 6.10b	9,28 kN/m					C53+C59+C69			
74	q _{ULS}	9,28 kN/m					MAX(B72:B73)			
75	F _{reaction_ULS_Per lintel}	7,365 kN								
76										
77	Verification strength									
78	1. Bending moment						Formule B			
79										
80										
81										
82										
83	M _{ULS_matrixframe}	2920000 Nmm								
84	t _{horizontal layer}	40 mm								
85	W _z	1,57E+06 mm ³					(1/6)*B82*B24^2			
86	σ _{m,ed}	1,86 N/mm ²					B83/B85			
87										
88	γ _M	1,25								
89	Load duration	long-term								
90	Service class	1								
91	k _{mod}	0,7								
92	k _h	1,02					MIN((600/B24)^0,1; 1,1)			
93	f _{M,d}	13,73 N/mm ²					B18*B91*B92/B88			
94	UC	0,14 -					B86/B93			
95										
96	2. Shear force									
97	V _{d_net}	7365,00 N					B75*1000			
98	t _{horizontal layer}	40 mm								
99	τ _d	0,57 N/mm ²					(3/2)*B97/(B98*B24)			
100										
101	γ _M	1,25								
102	Load duration	long-term								
103	Service class	1								
104	k _{mod}	0,7								
105	f _{V,d}	2,24 N/mm ²					B19*B104/B101			
106	UC	0,25 -					B99/B105			

Figure X.101: Structural verification CLT wall lintel above gallery (2)_Excel

X.10.6.2 Lintel below gallery

	A	B	C	D	E	F	G	H	I	J
1	Chosen wall panel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness panel	100	mm							
6	Wall height (l_w)	2950	mm							
7	m	42	kg/m ²							
8										
9	Wood									
10	Strength class	C24								
11	$\rho_{xlam,mean}$	420	kg/m ³							
12	$\rho_{xlam,k}$	350	kg/m ³							
13	$E_{0,x,0,05}$	7400	MPa							
14	$E_{0,x,mean}$	11000	MPa							
15	$G_{9090,xlay,mean}$	50	MPa							
16	$G_{090,xlay,mean}$	690	MPa							
17	$f_{c,0,x,k}$	21	N/mm ²							
18	$f_{m,x,k}$	24	N/mm ²							
19	$f_{V,090,k}$	4	N/mm ²							
20										
21	Structural scheme									
22	Span lintel	1,2	m							
23	Span floors	3,5	m							
24	Height lintel	350	mm							
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										
37	Load definition									
38	Load combinations (NEN1990; CC2)			<i>Variable residential load is leading action</i>						
39		γ_G	γ_Q							
40	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_Q \psi_{0i}$						
41	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_Q + \sum Q_{k,i} \gamma_Q \psi_{0i}$						
42										
43			ψ_{0i}							
44	Roof	0								
45	Floor	0,4								
46										
47	Floorpanel topfloor	kN/m ²	kN/m							
48	Permanent	0,964	1,69							Formule C B48*B23/2
49	Variable									
50	Residential loading	1,75	3,06							B50*B23/2
51	Non-structural walls	0,5	0,88							B51*B23/2
52		Eq 6.10a	4,64							(B40*C48) + (C40*B45* (C50+C51))
53		Eq 6.10b	7,14							(B41*C48) + (C41*C50) + (C41*B45*C51)
54										
55	Wall panel									
56	Height lintel	350	mm							Formule B B24
57										
58		kN/m ²	kN/m							Formule C
59	Permanent	0,42	0,15							B59*B56/1000
60		Eq 6.10a	0,20							B40*C59
61		Eq 6.10b	0,18							B41*C59

Figure X.102: Structural verification CLT wall lintel below gallery (1)_Excel


	A	B	C	D	E	F	G	H	I	J
62										
63	Total load						Formule B			
64	Eq 6.10a		4,84 kN/m				C52+C60			
65	Eq 6.10b		7,32 kN/m				C53+C61			
66	Q _{ULS}		7,32 kN/m				MAX(B64:B65)			
67	F _{reaction_ULS_Per lintel}		7,365 kN							
68										
69	Verification strength									
70	1. Bending moment					Formule B				
71										
72										
73										
74										
75										
76	M _{ULS_resultant}		3710000 Nmm							
77	t _{horizontal layer}		40 mm							
78	W _z		8,17E+05 mm ³			(1/6)*B77*B24^2				
79	σ _{med}		4,54 N/mm ²			B76/B78				
80										
81	Y _M		1,25							
82	Load duration		long-term							
83	Service class		1							
84	k _{mod}		0,7							
85	k _h		1,06			MIN((600/B24)^0,1; 1,1)				
86	f _{M,d}		14,18 N/mm ²			B18*B84*B85/B81				
87	UC		0,32 -			B79/B86				
88										
89	2. Shear force									
90	V _d		7365,00 N			B67*1000				
91	t _{horizontal layer}		40 mm							
92	τ _d		0,79 N/mm ²			(3/2)*B90/(B91*B24)				
93										
94	Y _M		1,25							
95	Load duration		long-term							
96	Service class		1							
97	k _{mod}		0,7							
98	f _{V,d}		2,24 N/mm ²			B19*B97/B94				
99	UC		0,35 -			B92/B98				

Figure X.103: Structural verification CLT wall lintel below gallery (2)_Excel

X.10.6.3 Lintel above door

	A	B	C	D	E	F	G	H	I	J
1	Chosen wall panel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness panel	100	mm							
6	Wall height (l_w)	2950	mm							
7	m	42	kg/m ²							
8										
9	Wood									
10	Strength class	C24								
11	$\rho_{xlam,mean}$	420	kg/m ³							
12	$\rho_{xlam,k}$	350	kg/m ³							
13	$E_{0,x,0,05}$	7400	MPa							
14	$E_{0,x,mean}$	11000	MPa							
15	$G_{090,xlay,mean}$	50	MPa							
16	$G_{090,xlay,mean}$	690	MPa							
17	$f_{c,0,x,k}$	21	N/mm ²							
18	$f_{m,x,k}$	24	N/mm ²							
19	$f_{V,090,k}$	4	N/mm ²							
20										
21	Structural scheme									
22	Half span lintel	0,45	m							
23	Span floors	3,5	m							
24	Height lintel	485	mm							
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										
37	Load definition									
38	Load combinations (NEN1990; CC2)		<i>Variable residential load is leading action</i>							
39		γ_G	γ_Q							
40	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$						
41	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_{Q,1} + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$						
42										
43			ψ_0							
44	Roof	0								
45	Floor	0,4								
46										
47	Floorpanel topfloor	kN/m ²	kN/m							
48	Permanent	0,964	1,69							Formule C B48*B23/2
49	Variable									
50	Residential loading	1,75	3,06							B50*B23/2
51	Non-structural walls	0,5	0,88							B51*B23/2
52		Eq 6.10a	4,64							(B40*C48) + (C40*B45* (C50+C51))
53		Eq 6.10b	7,14							(B41*C48) + (C41*C50) + (C41*B45*C51)
54										
55	Roofpanel 1st floor	kN/m ²	kN/m							
56	Permanent	0,786	1,38							B56*B23/2
57	Variable	Not present	-							
58		Eq 6.10a	1,86							B40*C56
59		Eq 6.10b	1,65							B41*C56
60										

Figure X.104: Structural verification CLT wall lintel above door (1)_Excel



	A	B	C	D	E	F	G	H	I	J
61	Wall panel						Formule B			
62	Height lintel	485 mm					B24			
63	Height above lintel till opening	470 mm								
64	Total	955 mm					SOM(B62:B63)			
65										
66		kN/m ²	kN/m				Formule C			
67	Permanent	0,42	0,40				B67*B64/1000			
68		Eq 6.10a	0,54				B40*C67			
69		Eq 6.10b	0,48				B41*C67			
70										
71	Total load						Formule B			
72	Eq 6.10a	7,04 kN/m					C52+C58+C68			
73	Eq 6.10b	9,28 kN/m					C53+C59+C69			
74	q _{ULS}	9,28 kN/m					MAX(B72:B73)			
75	F _{reaction_ULS Per lintel}	3,6 kN								
76										
77	Verification strength									
78	1. Bending moment						Formule B			
79										
80										
81										
82										
83										
84	M _{ULS_resultant}	2560000	Nmm							
85	t _{horizontal layer}	40	mm							
86	W _z	1568166,667	mm ³				(1/6)*B85*B24^2			
87	σ _{m,ed}	1,63	N/mm ²				B84/B86			
88										
89	Y _M	1,25								
90	Load duration	long-term								
91	Service class	1								
92	k _{mod}	0,7								
93	k _{th}	1,02					MIN((600/B24)^0,1; 1,1)			
94	f _{M,d}	13,73	N/mm ²				B18*B92*B93/B89			
95	UC	0,12	-				B87/B94			
96										
97	2. Shear force									
98										
99										
100										
101										
102										
103	V _{d_net}	7773,78	N				(B75*1000) + (B74*B22*1000)			
104	t _{horizontal layer}	40	mm							
105	τ _d	0,60	N/mm ²				(3/2)*B103/(B104*B24)			
106										
107	Y _M	1,25								
108	Load duration	long-term								
109	Service class	1								
110	k _{mod}	0,7								
111	f _{V,d}	2,24	N/mm ²				B19*B110/B107			
112	UC	0,27	-				B105/B111			

Figure X.105: Structural verification CLT wall lintel above door (2)_Excel

X.10.6.4 Lintel below door

	A	B	C	D	E	F	G	H	I	J
1	Chosen wall panel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness panel	100	mm							
6	Wall height (l_w)	2950	mm							
7	m	42	kg/m ²							
8										
9	Wood									
10	Strength class	C24								
11	$\rho_{x,tem,mean}$	420	kg/m ³							
12	$\rho_{x,tem,k}$	350	kg/m ³							
13	$E_{0,x,0,05}$	7400	MPa							
14	$E_{0,x,mean}$	11000	MPa							
15	$G_{9090,x,lay,mean}$	50	MPa							
16	$G_{090,x,lay,mean}$	690	MPa							
17	$f_{c,0,x,k}$	21	N/mm ²							
18	$f_{m,x,k}$	24	N/mm ²							
19	$f_{V,090,k}$	4	N/mm ²							
20										
21	Structural scheme									
22	Half span lintel	0,45	m							
23	Span floors	3,5	m							
24	Height lintel	350	mm							
25										
26										
27										
28										
29										
30										
31										
32										
33										
34										
35										
36										
37	Load definition									
38	Load combinations (NEN1990; CC2)		<i>Variable residential load is leading action</i>							
39		γ_G	γ_Q							
40	6.10a	1,35	1,5	$q_{ed} = G_k \gamma_G + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$						
41	6.10b	1,2	1,5	$q_{ed} = G_k \gamma_G + Q_{k,1} \gamma_{Q,1} + \sum Q_{k,i} \gamma_{Q,i} \psi_{0,i}$						
42										
43			$\psi_{0,i}$							
44	Roof	0								
45	Floor	0,4								
46										
47	Floorpanel topfloor	kN/m ²	kN/m							
48	Permanent	0,964	1,69							Formule C B48*B23/2
49	Variable									
50	Residential loading	1,75	3,06							B50*B23/2
51	Non-structural walls	0,5	0,88							B51*B23/2
52		Eq 6.10a	4,64							(B40*C48) + (C40*B45* (C50+C51))
53		Eq 6.10b	7,14							(B41*C48) + (C41*C50) + (C41*B45*C51)
54										
55	Wall panel									
56	Height lintel	350	mm							Formule B B24
57										
58		kN/m ²	kN/m							Formule C
59	Permanent	0,42	0,15							B59*B56/1000
60		Eq 6.10a	0,20							B40*C59
61		Eq 6.10b	0,18							B41*C59

Figure X.106: Structural verification CLT wall lintel below door (1)_Excel

62					
63	Total load				Formule B
64	Eq 6.10a	4,84 kN/m			C52+C60
65	Eq 6.10b	7,32 kN/m			C53+C61
66	Q _{ULS}	7,32 kN/m			MAX(B64:B65)
67	F _{reaction_ULS_Per lintel}	3,6 kN			
68					
69	Verification strength				
70	1. Bending moment				Formule B
71					
72					
73					
74					
75	M _{ULS_resultant}	2360000 Nmm			
76	t _{horizontal layer}	40 mm			
77	W _z	816666,6667 mm ³	(1/6)*B76*B24^2		
78	σ _{m,ed}	2,89 N/mm ²	B75/B77		
79					
80	Y _M	1,25			
81	Load duration	long-term			
82	Service class	1			
83	k _{mod}	0,7			
84	k _{th}	1,06	MIN((600/B24)^0,1; 1,1)		
85	f _{M,d}	14,18 N/mm ²	B18*B83*B84/B80		
86	UC	0,20 -	B78/B85		
87					
88	2. Shear force				
89					
90					
91					
92					
93	V _d	6893,80 N	(B67*1000) + (B66*B22*1000)		
94	t _{horizontal layer}	40 mm			
95	τ _d	0,74 N/mm ²	(3/2)*B93/(B94*B24)		
96					
97	Y _M	1,25			
98	Load duration	long-term			
99	Service class	1			
100	k _{mod}	0,7			
101	f _{V,d}	2,24 N/mm ²	B19*B100/B97		
102	UC	0,33 -	B95/B101		

Figure X.107: Structural verification CLT wall lintel below door (2)_Excel

Hand validation CLT wall lintels next page.

X.10.6.5 Hand validation

Order

Blad nr 1/2

Deel Verificatie lintels CLT wall

Datum 04/05/2023

VAN ROSSUM RAADGEVENDE INGENIEURS

1. Above gallery

Above door: $g_{cls} = 9.20 \text{ [kN/m]}$
 $h_{lint} = 40.5 \text{ [mm]}$

Below door: $g_{cls} = 7.32 \text{ [kN/m]}$
 $h_{lint} = 330 \text{ [mm]}$

$F/4 = \frac{9.20 \cdot 4}{4} \approx 7365 \text{ [N]} \rightarrow 7365 \text{ [N]} \text{ (sluit)}$

Bending
check matrix frame

Moment at support = $\frac{F}{4} \cdot 1500 - \frac{1}{2} g_{cls} l^2 = 7365 \cdot 1500 - \frac{1}{2} \cdot 9.20 \cdot 1500^2 = 6077500 \text{ [Nmm]}$

$M_{ed} = 2.92 \cdot 10^6 \text{ [Nmm]}$ (Matrix frame)

$w_2 = \frac{1}{6} \cdot \frac{M_{ed}}{h_{lint}} = \frac{1}{6} \cdot \frac{2.92 \cdot 10^6}{40.5} = 156316.67 \text{ [mm]}$ Matrix frame: $0.6 \cdot 10^6 \text{ [Nmm]}$

$\sigma_{red} = \frac{M_{ed}}{w_2} = \frac{2.92 \cdot 10^6}{156316.67} = 186 \text{ [N/mm}^2]$ $\rightarrow 186 \text{ [N/mm}^2]$

$K_{inst} = 0.7$
 $K_1 = 1.02$
 $\gamma_{ar} = 1.25$

$F_{o,d} = \frac{20 \cdot 0.7 \cdot 1.02}{1.25} = 1373 \text{ [N/mm}^2]$

$UC = \frac{186}{1373} = 0.14 \rightarrow 0.14 \text{ (sluit value)}$

Shear

$V_{ed} = 7365 \text{ [N]}$

$\tau_{ed} = \frac{3}{2} \cdot \frac{V_{ed}}{h_{lint}} = \frac{3}{2} \cdot \frac{7365}{40.5} = 0.517 \text{ [N/mm}^2]$

$K_{inst} = 0.7$
 $K_{inst} = 0.7$

$F_{o,d} = \frac{4 \cdot 0.7}{1.25} = 2.24 \text{ [N/mm}^2]$

$UC = \frac{0.517}{2.24} = 0.23 \rightarrow 0.23 \text{ (sluit value)}$

Figure X.108: Validation excel wall lintels (1)

Order /
 Blad nr 2/2
 Deel Verificatie lintels CLT
 Datum 04/05/2023



2. Below gallery

Berekening

check Matrix Frame

Moment at support = $F/4 \cdot 1500 - \frac{1}{2} g l^2 = 17365 \cdot 10^2 \cdot 1500 - \frac{1}{2} \cdot 7,32 \cdot 1500^2$
 $= 2812500 \text{ [Nmm]}$

$M_{red} = 7,71 \cdot 10^6 \text{ [Nmm]}$

$W_e = \frac{1}{6} b h^2 = \frac{1}{6} \cdot 40 \cdot 550^2 = 8,6667 \text{ [mm}^3\text{]}$ \hookrightarrow Matrix frame: $2,31 \cdot 10^6 \text{ [Nmm]}$
 $\sigma_{M} = \frac{M_{red}}{W_e} = \frac{7,71 \cdot 10^6}{8,6667} = 4,54 \text{ [N/mm}^2\text{]}$ \hookrightarrow $\sigma \leq 4,54 \text{ [N/mm}^2\text{]}$ (ok)

$\gamma_M = 1,25$
 $k_{red} = 0,7$
 $k_h = 1,06$ } $F_{red} = \frac{24 \cdot 0,7 \cdot 1,06}{1,25} = 14,18 \text{ [N/mm}^2\text{]}$

$UC = \frac{4,54}{14,18} = 0,32 [-]$ \hookrightarrow $0,32 < 1$ (ok)

Slaar

$V_{ed} = 7365 \text{ [N]}$

$W_v = \frac{3}{2} \frac{V_{ed}}{b h} = \frac{3}{2} \cdot \frac{7365}{40 \cdot 550} = 0,79 \text{ [N/mm}^2\text{]}$ \hookrightarrow $0,79 \text{ [N/mm}^2\text{]}$

$\gamma_M = 1,25$
 $k_{red} = 0,7$ } $F_{v,ed} = \frac{0,7 \cdot 4}{1,25} = 2,24 \text{ [N/mm}^2\text{]}$

$UC = \frac{0,79}{2,24} = 0,35 [-]$ \hookrightarrow $0,35$ (slaat niet)

Both calculations are > 1, so for inner slabs, so exact is required.

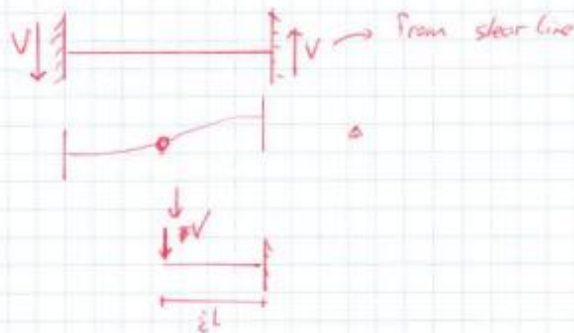


Figure X.109: Validation excel wall lintels (2)

X.10.7 CLT Stability wall

X.10.7.1 Placement on council estate

	A	B	C	D	E	F	G	H	I	J
1	STABILITY									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness outer panel	30	mm							
6	Thickness panel	100	mm							
7	Wall height	2950	mm							
8	$b_x (= L_{\text{stability wall}})$	2500	mm	Formule B						
9	$I_{z,x}$	7,81E+10	mm ⁴	(1/12)*(B5*2)*B8^3						
10	$W_{z,x}$	6,25E+07	mm ³	(1/6) * (B5*2)*B8^2						
11	m	42	kg/m ²							
12										
13	Wood									
14	Strength class	C24								
15	$\rho_{slam,mean}$	420	kg/m ³							
16	$\rho_{slam,k}$	350	kg/m ³							
17	$E_{0,x,0.05}$	7400	MPa							
18	$E_{0,x,mean}$	11000	MPa							
19	$G_{90,0,x,lay,mean}$	50	MPa							
20	$G_{90,0,x,lay,mean}$	690	MPa							
21	$f_{c,0,x,k}$	21	N/mm ²							
22	$f_{m,x,k}$	24	N/mm ²							
23	$f_{v,0,90,k}$	4	N/mm ²							
24										
25										
26	Horizontal load									
27	Length unit	9,45	m							
28	Units linked	4	-							
29	q_p (19.6m) (Wind area II; Urban)	0,9	kN/m ²							
30	$C_s C_d$	1	-							
31	$C_f D$	0,8	-							
32	$C_f E$	0,7	-							
33	$C_{f,tot}$	1,5	-	B31+B32						
34	$q_{wind,SLS}$	3,19	kN/m	B29*B30*B33*B27/B28						
35										
36	Deformation check									
37	Load duration	short-term								
38	Service class	1								
39	k_{def}	0,6								
40	ψ_2	0,3								
41										
42	u_{inst}	0,56	mm	(1/8)*B34*(B7*2)^4 / (B18*B9)						
43	u_{fin}	0,66	mm	B42*(1+(B39*B40))						
44	$W_{allowed}$	11,8	mm	(B7*2)/500						
45	UC	0,06		B43/B44						
46										
47										
48	Tension force due to wind load									
49	Tension due to wind									
50	Safety factor Q	1,5	-							
51	$q_{wind,Uls}$	4,78	kN/m	B34*B50						
52	M_{Uls}	8,33E+07	Nmm	(1/2)*B51*(B7*2)^2						
53	Tension force	33,31	kN	B52/B8/1000						
54										
55										
56	Normal force due to self-weight (Wall side)									
57	Safety factor G	0,9	-							
58	Span floorpanel	3,5	m							
59										
60	Side wall panel	kN/m ²	kN/m							Formule C

Figure X.110: Structural verification CLT stability wall council estate (1)_Excel

	A	B	C	D	E	F	G	H	I	J
61	Roofpanel topfloor	0,336	0,59					B61*B58/2		
62	Floorpanel topfloor	0,964	1,69					B62*B58/2		
63	Roofpanel first floor	0,336	0,59					B63*B58/2		
64	Floorpanel first floor	0,964	1,69					B64*B58/2		
65		q _k		4,55				SOM(C61:C64)	<Formule D	
66										
67	Self-weight	0,42	2,48					B67*B7*2/1000		
68	Fire board	0,14	0,83					B68*B7*2/1000		
69	Facade	Not considered as some units can have none						Formule D		
70				3,30				SOM(C67:C68)		
71			q _k	7,85 kN/m				D65+D70		
72			q _{ed}	7,07 kN/m				D71*B57		
73										
74	Stability wall panel itself	kN/m ²	kN/m					Formule C		
75	Self-weight	0,42	2,48					B75*B7*2/1000		
76	Fire board	0,14	0,826					B76*B7*2/1000		
77		q _k	3,30					SOM(C75:C76)		
78		q _{ed}	2,97 kN/m					C77*B57		
79								Formule B		
80	N _{stab.wall.ed}		3,72 kN					0,5*C78*B8/1000		
81	N _{needed from side wall}		29,59 kN					B53-B80		
82	Length side wall		4,19 m	> easily can be taken up by sidewall				B81/D72		

Figure X.111: Structural verification CLT stability wall council estate (2)_Excel

X.10.7.2 Placement on ground

	A	B	C	D	E	F	G	H	I	J
1	Chosen wall panel									
2	Panel									
3	Number of layers	3	-							
4	Panel build-up	30-40-30	mm							
5	Thickness outer panel	30	mm							
6	Thickness panel	100	mm							
7	Wall height	2950	mm							
8	b _x (= L _{stability wall})	2500	mm	Formule B						
9	I _{z,x}	7,81E+10	mm ⁴	(1/12)*(B5*2)*B8^3						
10	W _{z,x}	6,25E+07	mm ³	(1/6) * (B5*2)*B8^2						
11	m	42	kg/m ²							
12										
13	Wood									
14	Strength class	C24								
15	ρ _{slam,mean}	420	kg/m ³							
16	ρ _{slam,k}	350	kg/m ³							
17	E _{0,x,0.05}	7400	MPa							
18	E _{0,x,mean}	11000	MPa							
19	G _{9090,xlay,mean}	50	MPa							
20	G _{090,xlay,mean}	690	MPa							
21	f _{c,0,x,k}	21	N/mm ²							
22	f _{m,X,k}	24	N/mm ²							
23	f _{v,090,k}	4	N/mm ²							
24										
25										
26	Horizontal load									
27	Length unit	9,45	m							
28	Units linked	2	-							
29	q _p (6m) (Wind area II; Urban)	0,58	kN/m ²							
30	C _{sCd}	1	-							
31	C _{f,D}	0,8	-							
32	C _{f,E}	0,7	-							
33	C _{f,tot}	1,5	-	B31+B32						
34	q _{wind,SLS}	4,11	kN/m	B29*B30*B33*B27/B28						
35										
36	Deformation check									
37	Load duration	short-term								
38	Service class	1								
39	k _{def}	0,6								
40	ψ ₂	0,3								

Figure X.112: Structural verification CLT stability wall ground (1)_Excel

	A	B	C	D	E	F	G	H	I	J
41										
42		u_{inst}	0,72 mm	$(1/8)*B34*(B7*2)^4 / (B18*B9)$						
43		u_{fin}	0,85 mm	$B42*(1+(B39*B40))$						
44		$w_{allowed}$	11,8 mm	$(B7*2)/500$						
45		UC	0,07	$B43/B44$						
46										
47										
48	Tension force due to wind load									
49	Tension due to wind									
50	Safety factor Q	1,5 -								
51	q_{wind_ULS}	6,17 kN/m		$B34*B50$						
52	M_{ULS}	1,07E+08 Nmm		$(1/2)*B51*(B7*2)^2$			1,07E+02			
53	Tension force	42,93 kN		$B52/B8/1000$						
54										
55										
56	Normal force due to self-weight (Wall side)									
57	Safety factor G	0,9 -								
58	Span floorpanel	3,5 m								
59										
60	Side wall panel	kN/m ²	kN/m							
61	Roofpanel topfloor	0,336	0,59						Formule C	
62	Floorpanel topfloor	0,964	1,69						$B61*B58/2$	
63	Roofpanel first floor	0,336	0,59						$B62*B58/2$	
64	Floorpanel first floor	0,964	1,69						$B63*B58/2$	
65		q_k		4,55					$B64*B58/2$	
66									SOM(C61:C64)	<Formule D
67	Self-weight	0,42	2,48						$B67*B7*2/1000$	
68	Fire board	0,14	0,83						$B68*B7*2/1000$	
69	Facade	Not considered as some units can have none							Formule D	
70				3,30					SOM(C67:C68)	
71			q_k	7,85 kN/m					D65+D70	
72			q_{ed}	7,07 kN/m					D71*B57	
73										
74	Stability wall panel itself	kN/m ²	kN/m						Formule C	
75	Self-weight	0,42	2,48						$B75*B7*2/1000$	
76	Fire board	0,14	0,826						$B76*B7*2/1000$	
77		q_k	3,30						SOM(C75:C76)	
78		q_{ed}	2,97 kN/m						C77*B57	
79									Formule B	
80	$N_{stab.wall_ed}$	3,72 kN							$0,5*C78*B8/1000$	
81	$N_{needed\ from\ side\ wall}$	39,21 kN							B53-B80	
82	Length side wall	5,55 m		> easily can be taken up by sidewall					B81/D72	

Figure X.113: Structural verification CLT stability wall ground (2)_Excel


X.10.7.3 Hand validation stability wall sheet

Order /

Blad nr 1/2

Deel Verification stability wall CLT

Datum 09/05/2023



Chosen panel

Layers: $30 \cdot 40 \cdot 30 = 100 \text{ [mm]}$

Wall height: $2550 \cdot 2 \text{ [mm]}$

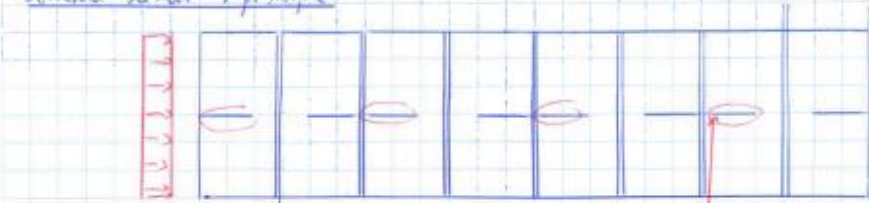
$I_{wall} = 2500 \text{ [mm}^3]$

$I_{2,x} = \frac{1}{12} l H^3 = \frac{1}{12} (2 \cdot 30) \cdot 2500^3 = 7,81 \cdot 10^{10} \text{ [mm}^4]$

$I_{2,y} = \frac{1}{12} l H^3 = \frac{1}{12} (2 \cdot 30) \cdot 2500^3 = 6,25 \cdot 10^9 \text{ [mm}^4]$


mass = $42 \text{ [kg/m}^3]$

Schematization / principle



per 2 blocks; 4 walls

Horizontal wind load



length unit = $9,45 \text{ [m]}$

walls = 4 (1-3)

$q_p = q_p(22,4) \cdot C_{pe} \cdot C_{pe} \cdot \text{length unit} / \text{walls}$

$= 0,93 \cdot 1,5 \cdot 9,45 / 4 = 3,30 \cdot 3,30 \text{ [kN/m]}$

$q_{p,ed} = q_p \cdot \gamma_a = 3,30 \cdot 1,5 = 4,95 \text{ [kN/m]}$

Note: in final sheet calculated with $q_p(19,6) = 0,9 \text{ [kN/m}^2]$. Still excel is validated, however values in screenshot differ a bit due to this change.

Deformation

$\kappa_{def} = 0,6$

$\gamma_c = 0,3$

$U_{inst} = \delta^2 \cdot q_{p,ed} \cdot l^4 / E_{max} \cdot I_{2,x} = \delta^2 \cdot 3,30 \cdot (2550 \cdot 2)^4 / (11000 \cdot 7,81 \cdot 10^{10}) = 0,50 \text{ [mm]}$

$U_{fin} = U_{inst} \cdot (1 + \kappa_{def} \cdot \gamma_c) = 0,50 \cdot (1 + 0,6 \cdot 0,3) = 0,60 \text{ [mm]}$

$U_{allow} = \frac{U}{\gamma_c} = \frac{0,50}{0,3} = 1,67 \text{ [mm]}$

$U_c = \frac{0,6}{1,67} = 0,36 \text{ [mm]}$

Figure X.114: Validation excel wall stability wall (1)

Order /
 Blad nr 2/2
 Deel Verification stability wall CLT
 Datum 07/05/2023



Verification for tension

$$M_{tuls} = \frac{1}{2} g_{ult} L^2 = \frac{1}{2} 4.54 \cdot (2.250)^2 = 11550.700 \text{ [Nm]} \\
 \text{Tension force} = \frac{M_{tuls}}{h_{wall}} = \frac{11550.700}{2500} = 4620.28 \text{ [N]} = 46.2 \text{ [kN]} \\
 S \Rightarrow 34.42 \text{ [kN]} \text{ (select)}$$

Normal force from self-weight side-walls and floors

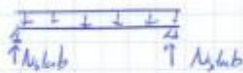
	[kN/m ²]	Floor span [m]	[kN/m]
Roof panel top floor	0.336	3.5/2	0.59
Roof panel 1st floor	0.59
Floor panel top floor	0.964	2.5/2	1.21
Floor panel 1st floor	1.21
			4.56 [kN/m] \Rightarrow 4.55 [kN] (select)

	mass [kN/m ²]	Height [m]	[kN/m]
Self-weight wall	$\frac{42 \cdot 100}{1000}$	$\frac{2.250}{1000}$	2.40 [kN/m]
Fire board	0.14	2.5/2	0.83 [kN/m]
			3.31 [kN/m] \Rightarrow 3.30 [kN] (select)

Total = 4.56 + 3.31 = 7.87 [kN/m]
 fact = g · γ_f = 7.87 · 1.35 = 10.62 [kN/m]
 S \Rightarrow 7.07 [kN/m] (select)

Stability wall itself

	mass [kN/m ²]	Height [m]	[kN/m]
Self-weight	$\frac{42 \cdot 100}{1000}$	2.25/2	2.40 [kN/m]
Fire board	0.14	2.5/2	0.83 [kN/m]
			3.31 [kN/m]
			fact = 0.5 · 3.31 = 1.65 [kN/m]
			\Rightarrow 2.57 [kN/m] (select)



$$N_{stab} = \frac{1}{2} g_{ult-stab} L = \frac{1}{2} 2.50 \cdot 2.5 = 3.13 \text{ [kN]} \\
 \text{Resulting tensile force} = 34.32 - 3.13 = 31.19 \text{ [kN]}$$

\hookrightarrow Needed wall length = $\frac{\text{Tensile force}}{N_{stab}} = \frac{31.19 \text{ [kN]}}{7.07 \text{ [kN/m]}} = 4.41 \text{ [m]} \cdot 1.35$
 \Rightarrow 4.34 [m] (select)
 4.3 < 9.45 · 1.35

Excel is validated.

Figure X.115: Validation excel wall stability wall (2)

X.10.8 Steel interlayer

X.10.8.1 Wind load is leading action (Variant 1a)

	A	B	C	D	E	F	G	H	I	J	K
Chosen wall panel											
1											
2	Applied combination	1a									
3	> Windload + downward load										
4	Wall height (l _w)	2950 mm									
5	b _y	2950 mm									
6	m	42 kg/m ²									
7											
8	Structural scheme										
9	Floorspan	3,5 m									
10	Span between steel beams	2,36 m									
11	Length stability wall	2500 mm									
12	Length unit	9,45 m									
13											
14											
15											
16											
17											
18											
19											
20											
21											
22											
23											
24	Load definition										
25	Load combinations (NEN1990; CC2)		<i>Wind load is leading action</i>								
26		Y _G	Y _Q								
27	6.10a	1,35	1,5	q _{red} = G _k Y _G + ΣQ _k Y _Q ψ ₀							
28	6.10b	1,2	1,5	q _{red} = G _k Y _G + Q _{k,1} Y _Q + ΣQ _{k,i} Y _Q ψ ₀							
29											
30		ψ ₀									
31	Roof	0									
32	Floor	0,4									
33											
34	Horizontal load										
35	Number of walls	4 -									
36	q _p (19.6m) (Wind area II; Urban)	0,9 kN/m ²									
37	C _s C _d	1 -									
38	C _{f,D}	0,8 -									
39	C _{r,E}	0,7 -									
40	C _{f,tot}	1,5 -									
41	q _{wind,SLS}	3,19 kN/m									
42	q _{wind,Uls}	4,78 kN/m									
43	M _{wind,Uls}	8,33E+07 Nmm									
44	F _{wind,hor}	112,90 kN									
45	F _{wind,ver}	33,31 kN									
46											
47	Roofpanel topfloor	kN/m ²	kN/m								
48	Permanent	0,786	1,38								
49	Variable	1	1,75	SLS ↓							
50		Eq 6.10b	1,65	1,38							
51											
52	Floorpanel topfloor	kN/m ²	kN/m								
53	Permanent	0,964	1,69								
54	Variable										
55	Residential loading	1,75	3,06								
56	Non-structural walls	0,5	0,88	SLS ↓							
57	Full loaded	Eq 6.10b	4,39	3,26							
58											
59	Roofpanel 1st floor	kN/m ²	kN/m								
60	Permanent	0,786	1,38								
61	Variable	Not present	-	SLS ↓							
62		Eq 6.10b	1,65	1,38							
63											
64	Floorpanel 1st floor	kN/m ²	kN/m								
65	Permanent	0,964	1,69								
66	Variable										
67	Residential loading	1,75	3,06								
68	Non-structural walls	0,5	0,88	SLS ↓							
69	Full loaded	Eq 6.10b	4,39	3,26							
70											
71	Wall panel itself										
72	Permanent load	kN/m ²	kN/m								
73	Self-weight	0,42	2,48	B73*B4*2/1000							

Figure X.116: Structural verification steel interlayer_loads var. 1a (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
74		Facade	0,5	2,95			B74*B4*2/1000				
75		Fire safety board	0,14	0,83			B75*B4*2/1000				
76			Total	6,25	SLS ↓		SOM(C73:C75)				
77			Eq 6.10b	7,50	6,25		B28*C76				
78											
79		q-load stability wall									
80		Permanent load	kN/m ²	kN/m							
81		Self-weight	0,42	2,48			B81*B4*2/1000				
82		Fire safety board	0,14	0,83			B82*B5*2/1000				
83			Total	3,30	SLS ↓		SOM(C81:C82)				
84			Eq 6.10b	3,96	3,30		C83*B28				
85											
86		Self-weight steel beam	kN/m								
87		Permanent load (HEB 260)	0,93				B87				
88			SLS	0,93	kN/m		B87*B28				
89			Eq 6.10b	1,12	kN/m						
90											
91											
92		Pointload facade	kN/m ²	kN							
93		Self-weight	0,5	5,16	SLS ↓		(B93*2*B4/1000)*B9/2				
94			Eq 6.10b	6,20	5,16		C93*B28				
95											
96											
97		Total load	kN/m		kN	kN/m (at stab. Wall)	Formule B				
98		Eq 6.10b _{ULS}	19,58	≈	46,26	3,96	C50+C57+C62+C69+C77				
99		Eq 6.10b _{SLS}	15,53	≈	36,69	3,30	D50+D57+D62+D69+D77				
100											
101		Overview loads_ULS									
102		Horizontal wind load _{ULS}		112,90	kN						
103		Vertical windloads _{ULS} +/-		33,31	kN						
104		q _{stability wall}		3,96	kN/m						
105		Permanent vertical loads		52,81	kN						
106		q _{profile}		1,12	kN/m						
107		Results ULS									
108											
109											
110											
111											
112											
113											
114											
115		Overview loads_SLS									
116		Horizontal wind load _{SLS}		75,27	kN						
117		Vertical windloads _{SLS} +/-		22,20	kN						
118		q _{stability wall}		3,30	kN/m						
119		Permanent vertical loads		41,89	kN						
120		q _{profile}		0,93	kN/m						
121		Results SLS									
122											
123											
124											
125											
126											

Figure X.117: Structural verification steel interlayer_loads var. 1a (2)_Excel

X.10.8.2 Variable load is leading action (Variant 1b)

	A	B	C	D	E	F	G	H	I	J	K
1	Properties										
2	Applied combination	1b									
3	Wall height (l _w)	2950	mm								
4	b _y	2950	mm								
5	b _x	1000	mm								
6	m	42	kg/m ²								
7											
8	Structural scheme										
9	Floorspan	3,5	m								
10	Span between steel beams	2,36	m								
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											
21											
22											
23											
24											
25											
26	Load definition										
27	Load combinations (NEN1990; CC2)			Variable residential load is leading action							
28		V _G	V _Q								
29	6.10a	1,35	1,5	q _{ed} = G _k V _G + ΣQ _k V _Q ψ ₀							
30	6.10b	1,2	1,5	q _{ed} = G _k V _G + Q _{k,1} V _Q + ΣQ _{k,i} V _Q ψ ₀							
31											
32			ψ ₀								
33	Roof	0									
34	Floor	0,4									
35											
36											
37	Roofpanel topfloor										
38	Permanent	0,786	1,38								Formule C
39	Variable	1	1,75	SLS ↓							B38*B9/2
40		Eq 6.10a	1,86	1,38							(B39*B9/2)
41		Eq 6.10b	1,65	1,38							(C38*B29) + (C39*B33*C29)
42											(B30*C38) + (C30*B33*C39)
43	Floorpanel topfloor										
44	Permanent	0,964	1,69								B44*B9/2
45	Variable										
46	Residential loading	1,75	3,06								B46*B9/2
47	Non-structural walls	0,5	0,88	SLS ↓							B47*B9/2
48		Eq 6.10a	4,64	3,26							(B29*C44) + (C29*(C46+C47)*B34)
49	Full loaded	Eq 6.10b	7,14	5,10							(B30*C44) + (C30*C46) + (C30*C47*B34)
50											
51	Roofpanel 1st floor										
52	Permanent	0,786	1,38								B52* (B9/2)
53	Variable	Not present	-	SLS ↓							
54		Eq 6.10a	1,86	1,38							B29*C52
55		Eq 6.10b	1,65	1,38							B30*C52
56											
57	Floorpanel 1st floor										
58	Permanent	0,964	1,69								B58*B9/2
59	Variable										
60	Residential loading	1,75	3,06								B60*B9/2
61	Non-structural walls	0,5	0,88	SLS ↓							B61*B9/2
62		Eq 6.10a	4,64	3,26							(B29*C58)+(C29*B34*(C60+C61))
63	Full loaded	Eq 6.10b	7,14	5,10							(B30*C58) + (C30*C60) + (C30*B34*C61)
64											
65	Wall panel itself										
66	Permanent load										
67	Self-weight	0,42	2,48								B67*B3*2/1000
68	Facade	0,5	2,95								B68*B3*2/1000
69	Fire safety board	0,14	0,83								B69*B3*2/1000
70		Total	6,25	SLS ↓							SOM(C67:C69)
71		Eq 6.10a	8,44	6,25							B29*C70
72		Eq 6.10b	7,50	6,25							B30*C70
73											

Figure X.118: Structural verification steel interlayer_loads var. 1b (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	
74	q-load stability wall											
75	Permanent load	kN/m ²	kN/m									
76	Self-weight	0,42	2,48				B76*B3*2/1000					
77	Fire safety board	0,14	0,83				B77*B4*2/1000					
78	Total		3,30	SLS ↓			SOM(C76:C77)					
79	Eq 6.10a		4,46	3,30			C78*B29					
80	Eq 6.10b		3,96	3,30			C78*B30					
81												
82	Self-weight steel beam											
83	Permanent load (HEB 260)	0,93										
84	SLS		0,93	kN/m			B83					
85	Eq 6.10a		1,26	kN/m			B83*B29					
86	Eq 6.10b		1,12	kN/m			B83*B30					
87												
88												
89	Pointload facade											
90	Self-weight	0,5	5,16	SLS ↓			(B90*2*B3/1000)*B9/2					
91	Eq 6.10a		6,97	5,16			C90*B29					
92	Eq 6.10b		6,20	5,16			C90*B30					
93												
94												
95	Total load											
96	Eq 6.10a _{ULS}	21,44	≈	50,64	4,46		Formule B					
97	Eq 6.10b _{ULS}	25,09	≈	59,28	3,96		C40+C48+C54+C62+C71					
98	Normative _{ULS}			59,28	3,96		C41+C49+C55+C63+C72					
99	Eq 6.10a _{SLS}	15,53	≈	36,69	3,30		MAX(D96:D97) < Formule D					
100	Eq 6.10b _{SLS}	19,20	≈	45,37	3,30		D40+D48+D54+D62+D71					
101	Normative _{SLS}			45,37	3,30		D41+D49+D55+D63+D72					
102							< Formule D					
103	Overview loads_ULS											
104	Vertical load _{S_ULS}	67,67 kN										
105	q _{stability wall}	3,96 kN/m										
106	q _{profile}	1,12 kN/m										
107	Results ULS											
108												
109												
110												
111												
112												
113												
114												
115	Overview loads_SLS											
116	Vertical load _{S_SLS}	51,79 kN										
117	q _{stability wall}	3,30 kN/m										
118	q _{profile}	0,93										
119	Results SLS											
120												
121												
122												
123												
124												

Figure X.119: Structural verification steel interlayer_loads var. 1b (2)_Excel

X.10.8.3 Permanent load low: 0.9G (Variant 2)

	A	B	C	D	E	F	G	H	I	J	K
1	Chosen wall panel										
2	Combination	2									
3	Wall height (l _w)	2950 mm									
4	b _y	2950 mm									
5	b _x	1000 mm									
6	m	42 kg/m ²									
7											
8											
9	Structural scheme										
10	Floorspan	3,5 m									
11	Span between steel beams	2,36 m									
12	Length stability wall	2500 mm									
13	Length unit	9,45 m									
14											
15											
16											
17											
18											
19											
20											
21											
22											
23											
24											
25											
26											
27	Load definition										
28	Load combinations (NEN1990; CC2)										
29		Y _G	Y _Q	<i>Wind load is leading action</i>							
30	6.10a	0,9	1,5	$q_{red} = G_k Y_G + \sum Q_{k,i} Y_{Q,i} \psi_{0,i}$							
31	6.10b	0,9	1,5	$q_{red} = G_k Y_G + Q_{k,1} Y_{Q,1} + \sum Q_{k,i} Y_{Q,i} \psi_{0,i}$							
32											
33											
34		Roof	ψ ₀								
35		Floor	0,4								
36											
37	Horizontal load										
38	Number of walls	4 -									
39	q _p (19.6m) (Wind area II; Urban)	0,9 kN/m ²									
40	C _s C _d	1 -									
41	C _{f,D}	0,8 -									
42	C _{f,E}	0,7 -									
43	C _{f,tot}	1,5 -									
44	Q _{wind,SLS}	3,19 kN/m									
45	Q _{wind,ULS}	4,78 kN/m									
46	M _{wind,ULS}	8,33E+07 Nmm									
47	F _{wind,hor}	112,90 kN									
48	F _{wind,ver}	33,31 kN									
49											
50	Roofpanel topfloor										
51	Permanent	0,786	1,38								
52	Variable	1	1,75	SLS ↓							
53		Eq 6.10b	1,24	1,38							
54											
55	Floorpanel topfloor										
56	Permanent	0,964	1,69								
57	Variable										
58	Residential loading	1,75	3,06								
59	Non-structural walls	0,5	0,88	SLS ↓							
60	Full loaded	Eq 6.10b	3,88	3,26							
61											
62	Roofpanel 1st floor										
63	Permanent	0,786	1,38								
64	Variable	Not present	-	SLS ↓							
65		Eq 6.10b	1,24	1,38							
66											
67	Floorpanel 1st floor										
68	Permanent	0,964	1,69								
69	Variable										
70	Residential loading	1,75	3,06								
71	Non-structural walls	0,5	0,88	SLS ↓							
72	Full loaded	Eq 6.10b	3,88	3,26							
73											

Figure X.120: Structural verification steel interlayer_loads var. 2 (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
74	Wall panel itself										
75	Permanent load	kN/m ²	kN/m								
76	Self-weight	0,42	2,48				B76*B3*2/1000				
77	Facade	0,5	2,95				B77*B3*2/1000				
78	Fire safety board	0,14	0,83				B78*B3*2/1000				
79		Total	6,25	SLS ↓			SOM(C76:C78)				
80		Eq 6.10b	5,63	6,25			B31*C79				
81	q-load stability wall										
83	Permanent load	kN/m ²	kN/m								
84	Self-weight	0,42	2,48				B84*B3*2/1000				
85	Fire safety board	0,14	0,83				B85*B4*2/1000				
86		Total	3,30	SLS ↓			SOM(C84:C85)				
87		Eq 6.10b	2,97	3,30			C86*B31				
88	Self-weight steel beam										
89	Permanent load (HEB 260)	kN/m	0,93								
91		SLS	0,93	kN/m			B90				
92		Eq 6.10b	0,84	kN/m			B90*B31				
93	Pointload facade										
94	Self-weight	kN/m ²	kN								
95		0,5	5,16	SLS ↓			(B95*2*B3/1000)*B10/2				
96		Eq 6.10b	4,65	5,16			C95*B31				
97	Total load										
99		kN/m		kN		kN/m (at stab. Wall)	Formule B				
100	Eq 6.10b _{ULS}	15,87	≈	37,48		2,97	C53+C60+C65+C72+C80				
101	Eq 6.10b _{SLS}	15,53	≈	36,69		3,30	D53+D60+D65+D72+D80				
102	Overview loads_ULS										
104	Horizontal wind load _{ULS}		112,90	kN							
105	Vertical windloads _{ULS}	+/-	33,31	kN							
106	q _{stability wall}		2,97	kN/m							
107	Permanent vertical loads		42,80	kN							
109	Results ULS										
110											
117	Overview loads_SLS										
118	Horizontal wind load _{SLS}		75,27	kN							
119	Vertical windloads _{SLS}	+/-	22,20	kN							
120	q _{stability wall}		3,30	kN/m							
121	Permanent vertical loads		41,89	kN							
123	Results SLS										
124											

Figure X.121: Structural verification steel interlayer_loads var. 2 (2)_Excel

X.10.8.4 Structural verification steel interlayer

	A	B	C	D	E	F	G
1	Verification steel beam bending						
2	M _{ed}	239,88	kNm				
3	N _{ed} (Note other load combination)	112,9	kN				
4	V _{ed}	80,47	kN				
5	w	21,9	mm				
6	Span beam	6600	mm				
7							
8	Factors						
9	Y _{M0}	1	-				
10							
11	Chosen profile						
12	HEB260						
13	f _{yd}	235	N/mm ²				
14	E _s	210000	N/mm ²				
15	G	81000	N/mm ²				
16	I _w	7,3628E+11	mm ⁶				
17	I _y	1257000	mm ⁴				
18	h	260	mm				
19	b	260	mm				
20	t _f	17,5	mm				
21	t _w	10	mm				
22	r	24	mm				
23	W _{ply}	1283000	mm ³				
24	A	11844	mm ²				
25	I _z	51350000	mm ⁴				
26							
27	Cross-section classification						
28	flange			Formule B			
29	c	101	mm	(B19/2) - (B21/2)-B22			
30	t	17,5	mm	B20			
31	α ^{fl}	5,77	-	B29/B30			
32	ξ	1	-	WORTEL(235/B13)			
33	Class 1 <	9	-	9*B32			
34	Class 2 <	10	-	10*B32			
35	Result	1					
36							
37	Web						
38	c	177	mm	B18 - (2*B20) - (2*B22)			
39	t	10	mm	B21			
40	c/t	17,7	-	B38/B39			
41	α	0,64	-	(1 / B38) * ((B18/2) + ((B3*1000) / (2*B21*B13)) - (B20+B22))			
42	Class 1 (α>0.5) <	54,51	-	396*B32 / ((13*B41) - 1)			
43	Class 2 (α>0.5) <	62,77	-	456*B32 / ((13*B41) - 1)			
44	Result	1					
45							
46	Overall class	1		MAX(B35;B44)			
47	> plastic analysis						
48							
49	Verification if axial force has to be included in analysis			Formule B			
50	N _{pl,Rd}	2783340	N	B13*B24			
51	0.25N _{pl,Rd}	695,84	kN	B50*0,25/1000			
52	N _{ed} < 0.25N _{pl,Rd}	WAAR	-	B3<B51			
53							
54	x = 0.5h _w t _w f _y / Y _{M0}	264,375	kN	0,5* (B18 - (2*B20)) * B21 * B13/B9 / 1000			
55	N _{ed} < x	WAAR	-	B3<B54			
56							
57	2 times 'WAAR' so axial force can be excluded from analysis						
58							
59	Verification if shearforce should be considered			Formule B			
60	A _v	3759	mm ²	MAX(B24 - (2 *B19*B20) + ((B21 + (2*B22)) * B20); (1,2 * (B18-(2*B20))*B21))			
61	V _{pl,Rd}	510	kN	B60*(B13 / WORTEL(3)) / B9 /1000			
62	0.5V _{pl,Rd}	255,01	kN	B61*0,5			
63	V _{ed} < 0.5V _{pl,Rd}	WAAR	-	B4<B62			
64							
65	True, so shear force can be neglected.						
66							
67	Cross-sectional check						
68	M _{pl,Rd}	301505000	Nmm	B23*B13			
69	M _{ed}	239880000	Nmm	B2*10^6			
70	UC	0,80	-	B69/B68			
71							


Figure X.122: Structural verification steel interlayer (1)_Excel

	A	B	C	D	E	F	G
72	Lateral torsional buckling						
73	L_{st}	6600 mm		B6			
74	L_{kip}	6600 mm		B6			
75	C_1	1,35 -					
76	C_2	-0,55 -					
77	S	1232 -		WORTEL((B14*B16)/(B15*B17))			
78	C	3,74		(PI()*B75*B73/B74) * (WORTEL(1 + ((PI())^2*B77^2/B74^2) * (B76^2 + 1))) + (PI())*B76*B77/B74))			
79	K_{red} (h/t _w < 75)	1					
80	M_{cr}	593055169 Nmm		B79*(B78/B73) * WORTEL(B14 * B25*B15*B17)			
81	λ_{Lt}	0,71		WORTEL(B23*B13/B80)			
82	Buckling curve	a					
83	α_{LT}	0,21 -					
84	Φ_{LT}	0,81 -		0,5*(1 + (B83*(B81-0,2)) + (B81^2))			
85	χ_{LT}	0,84 -		1/(B84 + WORTEL(B84^2 - B81^2))			
86							
87	$M_{pl,Ed}$	253729921 Nmm		B85*B68			
88	UC	0,95 -		B69/B87			
89	Safe, as not calculated with lateral supports, but these are there						
90							
91	Verification deflection						
92	$w_{allowed}$	26,4 mm		0,004*B6			
93	UC	0,83 -		B5/B91			
94							
95	Deflection for smaller profiles	w [mm]	UC	Formule C			
96	HEA260	31,3		1,2 B95/B91			
97	HEB240	29,1		1,1 B96/B91			

Figure X.123: Structural verification steel interlayer (2)_Excel

X.10.8.5 Hand validation steel interlayer

Order /
Blad nr 1/6
Deel Verification steel interlayer - loads
Datum 09/05/2023



• Variant 1a : 1.2 G + 1.5 Qw + Σ 1.5 Q_o φ

Horizontal wind load

$g_{w, s1s} = 3.30 \text{ kN/m}^2$
 $g_{w, s} = 3.30 \cdot 1.5 = 4.95 \text{ kN/m}^2$ (see stability verification)

Flexion - ULS = $\frac{M_w}{L_{wall}} \cdot \frac{4.95 \cdot (2.250)^2}{2 \cdot 2500 \cdot 1000} = 34.39 \text{ kNm}$

Fixed - Horizontal = $g_{w, L \cdot 4} = 4.95 \cdot 2.25 \cdot 4 = 116.58 \text{ kN} \leftrightarrow 116.6 \text{ kN}$

→ windload is divided over 4 walls, so multiply load with 4 to get total horizontal load

Natural loads

	permanent	variable	span	result
- Roof panel top floor	0.736	1	3.5/2	1.38 kN/m ²
8 1 st Floor			3.5/2	1.15 kN/m ²
$g_{ult} (6.rob) = (1.2 \cdot 1.38) + (1.5 \cdot 1.15) = 1.66 \text{ kN/m}^2$				
$g_{s1s} = 1.38 \text{ kN/m}^2$				
- Floor panel slab top	0.964	2.75	3.5/2	1.6 kN/m ²
€ 1 st Floor			3.5/2	3.54 kN/m ²
$g_{ult} (6.rob) = (1.2 \cdot 1.6) + (1.5 \cdot 3.54 \cdot 0.4) = 4.39 \text{ kN/m}^2$				
$g_{s1s} (6.rob) = (1.0 \cdot 1.6) + (1.0 \cdot 3.54 \cdot 0.4) = 3.27 \text{ kN/m}^2$				
- Wall panel	1.06	2.95	2	6.25 kN/m ²
$g_{ult} = 1.2 \cdot 6.25 = 7.50 \text{ kN/m}^2$				
$g_{s1s} = 6.25 \text{ kN/m}^2$				
(steel)				
→ $g_{ult, wall} = (2 \cdot 1.38) + (2 \cdot 3.27) + 6.25 = 15.55 \text{ kN/m}^2$				
$g_{ult, wall} = (2 \cdot 1.66) + (2 \cdot 4.39) + 7.50 = 15.60 \text{ kN/m}^2$				
↳ Point load ULS = $15.55 \cdot 2.36 = 46.21 \text{ kN}$				
↳ vertical frame: 52.8 kN				
Point load SLS = $15.55 \cdot 2.36 = 36.70 \text{ kN}$				
↳ vertical frame: 41.8 kN				
- Stability wall	permanent = $(0.42 + 0.14) \cdot 2.95 \cdot 2 = 3.30 \text{ kN/m}^2$			
↳ ULS : $3.30 \cdot 1.2 = 3.96 \text{ kN/m}^2$				
SLS : 3.30 kN/m^2				
- Self weight profile	$g_{ult} = \frac{0.93}{1.12} \cdot 1.2 = 2.02 \text{ kN/m}^2$			

Figure X.124: Validation excel steel interlayer (1)

Order /
 Blad nr 2/16
 Deel Verificatie steel interlayer - loads
 Datum 05/05/2023

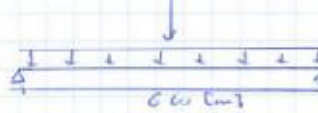
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• Simplified case multi-frame model

1. Moment

$F = (2 \cdot 52,0) + 31,30 = 136,5 \text{ kN}$

$g = 356 \cdot 1,12 = 398,72 \text{ kNm}$



$M_g = \frac{1}{8} g L^2 = \frac{1}{8} \cdot 398,72 \cdot 6,6^2 = 27,66 \text{ kNm}$

$M_F = \frac{1}{4} F L = \frac{1}{4} \cdot 136,5 \cdot 6,6 = 225,525 \text{ kNm}$

$252,88 \text{ kNm}$ $\rightarrow 225,43 \text{ kNm}$ (Multi-frame)

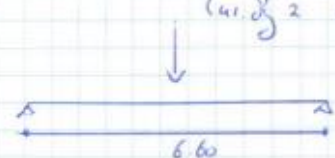
2. Support reactions

$\Sigma F_v = (3,56 \cdot 1,12) \cdot 6,6 - (3,56 \cdot 2) + (52,0 \cdot 2) = 131,31 = 164,24 \text{ kN}$

$\Sigma \text{support} = 73,21 + 85,33 = 164,24 \text{ kN}$ $\rightarrow 164,24 \text{ kN}$

3. Deflection

$(41,8 \cdot 2 + 22,20) = 105,5 \text{ kN}$



$w = \frac{1}{48} \frac{F L^3}{EI} = \frac{1}{48} \cdot \frac{105,5 \text{ kN} \cdot 6600^3}{210000 \cdot 14570 \cdot 10^4} = 20,3 \text{ mm}$

$\rightarrow 21,9 \text{ mm}$ (Multi-frame)

Figure X.125: Validation excel steel interlayer (2)

Order /
 Blad nr 3/6
 Deel Verificatie steel interlayer - loads
 Datum 05/05/2023



o Variant 1b

eg 6.10a: $1.35 \cdot 0.5 + 1.5 \cdot 0.4$
 eg 6.10b: $1.2 \cdot 0.5 + 1.5 \cdot 0.4$

roof ~~inter~~ panels (2x)

permanent = 1.38 [kN/m]
 variable = 1 [kN/m]

$g_{sls} = (1.0 \cdot 1.38) + (1.0 \cdot 0.0) = 1.38$ [kN/m] ✓
 $g_{sls} (6.10a) = (1.35 \cdot 1.38) + (1.5 \cdot 0.0) = 1.86$ [kN/m] ✓
 $g_{sls} (6.10b) = (1.2 \cdot 1.38) + (1.5 \cdot 0.0) = 1.66$ [kN/m] ✓

Floor panels (2x) → full loaded

permanent ~~eg~~ 1.63 [kN/m]
 variable:
 residential load: 3.06 [kN/m]
 Non-str. walls: 0.23 [kN/m]

6.10a $g_{sls} = (1.0 \cdot 1.63) + (1.0 \cdot 0.4 \cdot (3.06 + 0.23)) = 3.27$ [kN/m] ✓
 6.10b $g_{sls} = (1.0 \cdot 1.63) + (1.0 \cdot 3.06) + (1.0 \cdot 0.4 \cdot 0.23) = 5.10$ [kN/m] ✓

6.10a $g_{sls} = (1.35 \cdot 1.63) + (1.5 \cdot 0.4 \cdot (3.06 + 0.23)) = 4.65$ [kN/m] ✓
 6.10b $g_{sls} = (1.2 \cdot 1.63) + (1.5 \cdot 3.06) + (1.5 \cdot 0.4 \cdot 0.23) = 7.15$ [kN/m] ✓

Wall (slab)

permanent: 6.25 [kN/m]

6.10a $g_{sls} = 6.25$ [kN/m]
 $g_{sls} = 1.35 \cdot 6.25 = 8.44$ [kN/m]
 6.10b $g_{sls} = 1.2 \cdot 6.25 = 7.5$ [kN/m]

Total load for point loads

eg 6.10a (slab)
 $g_{sls} = (1.38 \cdot 2) + (3.27 \cdot 2) + 6.25 = 15.55$ [kN/m] ✓ ↔ 15.53 [kN/m]
 $g_{sls} = (1.86 \cdot 2) + (4.65 \cdot 2) + 8.44 = 21.46$ [kN/m] ✓ ↔ 21.44 [kN/m]

eg 6.10b → normative values (slab)
 $g_{sls} = (1.38 \cdot 2) + (5.10 \cdot 2) + 6.25 = 19.21$ [kN/m] ✓ ↔ 19.20 [kN/m]
 $g_{sls} = (1.66 \cdot 2) + (7.15 \cdot 2) + 7.5 = 25.12$ [kN/m] ✓ ↔ 25.09 [kN/m]

Point loads check matrix frame: → dist. between steel beams
 $F_{sls} = 25.12 \cdot 2.36 = 59.28$ [kN] ↔ 67.47 [kN] ✓
 $F_{sls} = 19.21 \cdot 2.36 = 45.33$ [kN] ↔ 51.75 [kN] ✓

Values matrix frame

Figure X.126: Validation excel steel interlayer (3)

Order /
 Blad nr 4/6
 Deel Verification steel interlayer - loads
 Datum 05/10/2023



Wood stability class

permanent = 3.30 [kN/m³]
 S1s = 3.30 [kN/m³]
 6.10a S1s = 3.30 · 1.35 = 4.46 [kN/m³]
 6.10b S1s = 3.30 · 1.20 = 3.96 [kN/m³]

Weight profile

S1s = 0.93 [kN/m³]
 6.10a S1s = 0.93 · 1.35 = 1.26 [kN/m³]
 6.10b S1s = 0.93 · 1.2 = 1.12 [kN/m³]

Verification matrix frame model

1. Moment

↓ 67.67 · 2 = 135.34 [kN]

q = (1.12 + 3.96) = 5.08 [kN]

6.60 [m]

$M_g = \frac{q \cdot L^2}{8} = \frac{5.08 \cdot 6.6^2}{8} = 27.66 [kNm]$
 $M_t = \frac{q \cdot L^2}{4 \cdot EI} = \frac{4 \cdot 135.34 \cdot 6.6}{250 \cdot 57 [kNm]}$
 Matrix frame 235 [kNm]

2. Support reactions

$\Sigma F_v = (5.08 \cdot 6.6) - (3.96 \cdot 2) + (67.67 \cdot 2) = 160.96 [kN]$
 $\Sigma_{support} = 80.47 \cdot 2 = 160.94$

3. Deflection

↓ 51.75 · 2 = 103.50 [kN]

6.60

$w = \frac{1}{48} \frac{F L^3}{EI} = \frac{1}{48} \frac{103.50 \cdot 1000 \cdot 6.6^3}{210000 \cdot 1420 \cdot 10^4} = 15.80 [mm]$
 Matrix frame: 2150 [mm]

Figure X.127: Validation excel steel interlayer (4)

Order /
 Blad nr 5/6
 Deel Verificatie steel interlayer - loads
 Datum 05/05/2023



• Variant 2 $0.5 G + 1.5 Q_{w} + \Sigma \cdot S Q_{w} \cdot \psi_0$

Horizontaal loads \rightarrow variant 1.0

Roofpanels (2x)
 permanent: 1.38 [kN/m]
 variabel: 1.75 [kN/m]
 $g_{s1s} = 1.38 + (1.17 \cdot 0.1) = 1.39 \text{ [kN/m]}$
 $g_{s1s} \cdot b_w = (1.39 \cdot 1.32) + (1.0 \cdot 0.17) = 1.94 + 0.17 = 2.11 \text{ [kN/m]}$

Floorpanels (2x) \rightarrow Full load
 permanent: 1.6 [kN/m]
 variabel:
 roo. load: 3.06 [kN/m]
 Mustr. walls: 0.89 [kN/m]
 $g_{s1s} = (1.0 \cdot 1.6) + (1.0 \cdot 3.06) = (1.0 \cdot 0.4 \cdot 0.89) = 3.27 \text{ [kN/m]}$
 $g_{s1s} = (0.5 \cdot 1.6) + (1.5 \cdot 0.4 \cdot (3.06 + 0.89)) = 2.89 \text{ [kN/m]}$

Wall panel itself
 permanent: 6.25 [kN/m]
 $g_{s1s} = 6.25 \text{ [kN/m]}$
 $g_{s1s} = 0.5 \cdot 6.25 = 3.13 \text{ [kN/m]}$

Total load for pointload
 $g_{s1s} = (2 \cdot 1.39) + (2 \cdot 3.27) + 6.25 = 15.55 \text{ [kN/m]}$
 $g_{s1s} = (2 \cdot 1.24) + (2 \cdot 3.89) + 5.63 = 15.87 \text{ [kN/m]}$
 (6 sheet)

pointloads check matrix frame (span = 2.26m)
 ULS: $F \approx 15.87 \cdot 2.36 = 37.45 \text{ [kN]}$
 SLS: $T \approx 15.55 \cdot 2.36 = 36.70 \text{ [kN]}$
 Matrix frame

Stability wall
 permanent: 3.30 [kN/m]
 $g_{s1s} = 3.30 \text{ [kN/m]}$
 $g_{s1s} = 0.5 \cdot 3.30 = 1.65 \text{ [kN/m]}$

Self-weight profile
 $g_{s1s} = 0.53 \text{ [kN/m]}$
 $g_{s1s} = 0.53 \cdot 0.5 = 0.26 \text{ [kN/m]}$

Figure X.128: Validation excel steel interlayer (5)

Order /
 Blad nr 6/6
 Deel Verificatie steel interlayer-loods
 Datum 09/05/2023

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Simple check Matrix Frame method

$$\Sigma F_v = (2597 + 0,04) \cdot 0,36 = (2597,2) + (422,004) + 33,31 - 31,31 = 157111$$

$$\Sigma_{\text{support}} = 14,50 + 111,07 + 41,54 = 157111 \text{ kN}$$

Simplified verification basis

ULS

$$M_{ed} = 239,08 \text{ [kNm]}$$

$$W_{pl,y} = 1203000 \text{ [mm}^3\text{]}$$

$$\sigma_m = \frac{M}{W} = \frac{239,08 \cdot 10^6}{1203000} = 198,69 < 235 \text{ [N/mm}^2\text{]}$$

SLS

$$w = 21,9 \text{ [mm]}$$

$$w_{\text{allowed}} = 0,004 \cdot 6600 = 26,4 \text{ [mm]}$$

$$21,9 < 26,4 \text{ [mm]}$$

Figure X.129: Validation excel steel interlayer (6)

X.10.9 Weight calculation CLT top up

X.10.9.1 1-storey top-up

	A	B	C	D	E	F	G	H	I	J	K
1	1-storey weight calculation										
2	Wall height (l_w)	2950	mm								
3	b_y	2950	mm								
4	b_x	1000	mm								
5	m	42	kg/m ²								
6											
7											
8	Structural scheme										
9	Floorspan	3,5	m								
10	Span between steel beams	2,36	m								
11	Length unit	9,45	m								
12											
13											
14	Load definition 1 storey										
15	Load combinations (NEN1990; CC2)			<i>Variable residential load is leading action</i>							
16		V_G	V_Q								
17	6.10a	1,35	1,5	$q_{ed} = G_k V_G + \sum Q_{k,i} V_{Q,i} \psi_{0,i}$							
18	6.10b	1,2	1,5	$q_{ed} = G_k V_G + Q_{k,1} V_Q + \sum Q_{k,i} V_{Q,i} \psi_{0,i}$							
19											
20			$\psi_{0,i}$								
21		Roof	0								
22		Floor	0,4								
23											
24											
25	Roofpanel	kN/m ²	kN/m								Formule C
26	Permanent (Solar panels excluded)	0,336	0,59								B26*B9/2
27		Eq 6.10a	0,79								(C26*B17)
28	Full loaded	Eq 6.10b	0,71								(B18*C26)
29											
30	Floorpanel topfloor	kN/m ²	kN/m								
31	Permanent	0,964	1,69								B31*B9/2
32	Variable										
33	Residential loading	1,75	3,06								B33*B9/2
34	Non-structural walls	0,5	0,88								B34*B9/2
35		Eq 6.10a	4,64								(B17*C31) + (C17*(C33+C34)*B22)
36	Full loaded	Eq 6.10b	7,14								(B18*C31) + (C18*C33) + (C18*C34*B22)
37											
38	Wall panel itself										
39	Permanent load	kN/m ²	kN/m								
40	Self-weight	0,42	1,24								B40*B2/1000
41	Fire safety board	0,14	0,41								B41*B2/1000
42	Total		1,65								SOM(C40:C41)
43		Eq 6.10a	2,23								B17*C42
44		Eq 6.10b	1,98								B18*C42
45											
46	q-load stability wall										
47	Permanent load	kN/m ²	kN/m								
48	Self-weight	0,42	1,24								B48*B2/1000
49	Fire safety board	0,14	0,41								B49*B3/1000
50	Total		1,65								SOM(C48:C49)
51		Eq 6.10a	2,23								C50*B17
52		Eq 6.10b	1,98								C50*B18
53											
54	Self-weight steel beam										
55	Permanent load (HEB 260)	0,93									
56		SLS	0,93								B55
57		Eq 6.10a	1,26								B55*B17
58		Eq 6.10b	1,12								B55*B18
59											
60											
61	Pointload facade										
62	Self-weight	0,5	2,58								(B62*B2/1000)*B9/2
63		Eq 6.10a	3,48								C62*B17
64		Eq 6.10b	3,10								C62*B18
65											
66											
67	Total load walls			kN/m		kN		kN/m (at stab. Wall)			Formule B
68	Eq 6.10a _{ULS}	7,66	≈	18,11	2,23						C27+C35+C43
69	Eq 6.10b _{ULS}	9,83	≈	23,23	1,98						C28+C36+C44
70	Normative _{ULS}			23,23	1,98						

Figure X.130: Weight calculation CLT top-up 1-storey (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
71											
72	Overview loads	Load [kN/m]	Amount [-]	Length [m]	Total load [kN]						
73	Floors + Walls	9,83	2	9,45	185,81						
74											
75		Load [kN/m]	Amount [-]	Length [m]	Total load [kN]						
76	Stability wall	1,98	1	2,5	4,96						
77											
78		Load [kN]	Amount [-]	Total load [kN]							
79	Fronnd + back facade	3,10	4	12,39							
80											
81		Load [kN/m]	Length [m]	Amount	Total load [kN]						
82	Steel beams	1,12	3,5	5	19,6						
83											
84	Total load	222,75 kN									
85	Area										
86		Width	3,5 m								
87		Length	9,45 m								
88		Area	33,08 m ²								
89											
90	Design load for 1 storey	6,7 kN/m ²									
91	Reduction 5%	6,4 kN/m ²									

Figure X.131: Weight calculation CLT top-up 1-storey (2)_Excel

X.10.9.2 2-storey top-up

	A	B	C	D	E	F	G	H	I	J	K
1	2-storey weight calculation										
2	Wall height (l _w)	2950 mm									
3	b _y	2950 mm									
4	b _x	1000 mm									
5	m	42 kg/m ²									
6											
7											
8	Structural scheme										
9	Floorspan	3,5 m									
10	Span between steel beams	2,36 m									
11	Length unit	9,45 m									
12											
13											
14	Load definition 2 stories										
15	Load combinations (NEN1990; CC2)		<i>Variable residential load is leading action</i>								
16		Y _G	Y _Q								
17	6.10a	1,35	1,5		q _{ed} = G _k Y _G + ΣQ _k Y _Q ψ ₀						
18	6.10b	1,2	1,5		q _{ed} = G _k Y _G + Q _{k,1} Y _Q + ΣQ _{k,i} Y _Q ψ ₀						
19											
20			ψ ₀								
21		Roof	0								
22		Floor	0,4								
23											
24											
25	Roofpanel topfloor		kN/m ²	kN/m							
26	Permanent (Solar panels excluded)	0,336	0,59								
27		Eq 6.10a	0,79								
28		Eq 6.10b	0,71								
29											
30	Floorpanel topfloor		kN/m ²	kN/m							
31	Permanent	0,964	1,69								
32	Variable										
33	Residential loading	1,75	3,06								
34	Non-structural walls	0,5	0,88								
35		Eq 6.10a	4,64								
36	Full loaded	Eq 6.10b	7,14								
37											
38	Roofpanel 1st floor		kN/m ²	kN/m							
39	Permanent (Solar panels excluded)	0,336	0,59								
40	Variable	Not present	-								
41		Eq 6.10a	0,79								
42		Eq 6.10b	0,71								
43											
44	Floorpanel 1st floor		kN/m ²	kN/m							
45	Permanent	0,964	1,69								
46	Variable										
47	Residential loading	1,75	3,06								
48	Non-structural walls	0,5	0,88								
49		Eq 6.10a	4,64								
50	Full loaded	Eq 6.10b	7,14								

Figure X.132: Weight calculation CLT top-up 2-storey (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
51											
52	Wall panel itself										
53	Permanent load	kN/m ²	kN/m								
54	Self-weight	0,42	2,48				B54*B2*2/1000				
55	Fire safety board	0,14	0,83				B55*B2*2/1000				
56		Total	3,30				SOM(C54:C55)				
57		Eq 6.10a	4,46				B17*C56				
58		Eq 6.10b	3,96				B18*C56				
59											
60	q-load stability wall										
61	Permanent load	kN/m ²	kN/m								
62	Self-weight	0,42	2,48				B62*B2*2/1000				
63	Fire safety board	0,14	0,83				B63*B3*2/1000				
64		Total	3,30				SOM(C62:C63)				
65		Eq 6.10a	4,46				C64*B17				
66		Eq 6.10b	3,96				C64*B18				
67											
68	Self-weight steel beam		kN/m								
69	Permanent load (HEB 260)	0,93									
70		SLS	0,93				B69				
71		Eq 6.10a	1,26				B69*B17				
72		Eq 6.10b	1,12				B69*B18				
73											
74											
75	Pointload facade		kN/m ²	kN							
76	Self-weight	0,5	5,16				(B76*2*B2/1000)*B9/2				
77		Eq 6.10a	6,97				C76*B17				
78		Eq 6.10b	6,20				C76*B18				
79											
80											
81	Total load walls		kN/m	kN	kN/m (at stab. Wall)		Formule B				
82	Eq 6.10a _{ULS}	15,33	≈	36,21	4,46		C27+C35+C41+C49+C57				
83	Eq 6.10b _{ULS}	19,66	≈	46,45	3,96		C28+C36+C42+C50+C58				
84	Normative _{ULS}			46,45	3,96						
85											
86	Overview loads		Load [kN/m]	Amount [-]	Length [m]	Total load [kN]					
87	Floors + Walls	19,66	2	9,45	371,62						
88											
89		Load [kN/m]	Amount [-]	Length [m]	Total load [kN]						
90	Stability wall	3,96	1	2,5	9,91						
91											
92		Load [kN]	Amount [-]	Total load [kN]							
93	Fronnd + back facade	6,20	4	24,78							
94											
95		Load [kN/m]	Length [m]	Amount	Total load [kN]						
96	Steel beams	1,12	3,5	5	19,6						
97											
98	Total load	425,91	kN								
99	Area										
100	Width	3,5	m								
101	Length	9,45	m								
102	Area	33,08	m ²								
103											
104	Design load for 2 stories	12,9	kN/m ²								
105	Reduction 5%	12,3	kN/m ²								

Figure X.133: Weight calculation CLT top-up 2-storey (2)_Excel

X.10.10 Environmental impact CLT top-up

X.10.10.1 1-layer top-up

	A	B	C	D	E	F
1	Environmental calculation					
2	CLT side walls			Formule B		
3	Standard dimensions					
4	Height	2950	mm			
5	Length	9450	mm			
6	Thickness	100	mm			
7	Opening at gallery					
8	Width	1200	mm			
9	Height	2115	mm			
10	Area (-)	2538000	mm ²	B8*B9		
11	Volumes					
12	Volume single wall	2,53	m ³	((B4*B5*B6) - (B10*B6)) * 10^-9		
13	Number of side walls per unit	2	-			
14	Volume CLT for side walls/ unit	5,07	m ³	B12*B13		
15	Gypsum board					
17	Wall area	25,34	m ²	((B4*B5)-B10) * 10^-6		
18	Thickness gypsum board	18	mm			
19	Volume gypsum board/wall	0,46	m ³	B17*(B18/1000)		
20	Volume gypsum board total for walls	0,91	m ³	B19*B13		
21	CLT stability wall					
22	Dimensions					
24	Height	2950	mm	B4		
25	Length	2500	mm			
26	Thickness	100	mm	B6		
27	Volume	0,74	m ³	B24*B25*B26*10^-9		
28	Gypsum board					
30	Wall area (2 sides)	14,75	m ²	B24*B25*10^-6 * 2		
31	Thickness gypsum board	18	mm	B18		
32	Volume gypsum board	0,27	m ³	B30*(B31/1000)		
33	CLT roof					
34	Dimensions					
36	Width	3500	mm			
37	Length	9450	mm	B5		
38	Thickness	80	mm			
39	Volume	2,65	m ³	B36*B37*B38*10^-9		
40	CLT floor					
41	Dimensions					
43	Width	3500	mm	B36		
44	Length	9450	mm	B5		
45	Thickness	120	mm			
46	Volume	3,97	m ³	B43*B44*B45*10^-9		
47	Steel interlayer					
48	Dimensions					
51	Area beam	11800	mm ²	118*10^2		
52	Length per unit	3500	mm	B36		
53	Amount of beams	5	-			
54	Volume per 2 units (above each other)	0,21	m ³	B51*B52*10^-9*B53		
55	Weight steel	7850	kg/m ³			
56	Weight steel per 2 units (above each other)	1621,03	kg	B54*B55		
57	Overview of materials					
59	Amount of units considered	1	-			
60	New floor area (2 units)	33,075	m ²	B36*B37*B59*10^-6		
61	Considered lifetime	75	y			
62	Material					
63		m ³	:10^-3 m ³ / m ² new floor area/ y	Formule C		
64	1. CLT					
65	CLT side walls	5,07		2,04	B65*1000/B60/B61	
66	CLT stability walls	0,74		0,30	B66*1000/B60/B61	
67	CLT floors	3,97		1,60	B67*1000/B60/B61	
68	CLT roofs	2,65		1,07	B68*1000/B60/B61	
69	Total CLT	12,42		5,01	B69*1000/B60/B61	
70	2. Steel interlayer					
71	Steel beams	0,21		0,08	B71*1000/B60/B61	
72	3. Gypsum board					
73	Gypsum plasterboard type F (Walls)	1,18		0,47	B73*1000/B60/B61	
74	Total					
75	Total material usage	13,80		5,56	C69+C71+C73	

Figure X.134: Impact calculation 1 layer CLT top-up (1)_Excel

	A	B	C	D	E	F
77						
78	Global warming potential of CLT units					
79	Impact materials	Unit	kg CO ₂ -eq/unit			
80	CLT	m ³	-714			
81	Steel	1000 kg	719			
82	Gypsum board (18 mm)	m ²	3,11			
83						
84	Material	Amount (2 units)	Unit	GWP [kg CO₂ -eq]		Formule D
85	CLT	12,42	m ³	-8868,17		B85*C80
86	Steel	1621,03	kg	1165,52		B86/1000*C81
87	Gypsum board	65,43	m ²	203,48		B87*C82
88				-7499,16		SOM(D85:D87)
89						
90	Total impact					
91	New floor area (2 units)	33,075	m ²			B60
92	Considered lifetime	75	y	Percentage of total		B61
93						
94	Material	Impact	Unit	% of total		Formule B
95	Impact CLT	-35,75	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	118,3		D85*10/B91/B92
96	Impact Steel	4,70	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	-15,5		D86*10/B91/B92
97	Impact Gypsum board	0,82	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	-2,7		D87*10/B91/B92
98	Total impact	-30,23	*10⁻¹ kg CO₂ -eq /m² new floor area / y			SOM(B95:B97)
99						
100						
101	Impact splitted per element					
102	Impact materials	Unit	kg CO ₂ -eq/unit			
103	CLT	m ³	-714			
104	Steel	1000 kg	719			
105	Gypsum board (18 mm)	m ²	3,11			
106						
107	New floor area (2 units)	33,075	m ²			B60
108	Considered lifetime	75	y			B61
109						
110	Element	Amount (2 units)	Unit	Impact		
111	Walls			[*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y]		Formule D
112	CLT	5,81	m ³	-16,71		10*B112*C103/(B108*B107)
113	Plasterboard	65,43	m ²	0,82		10*B113*C105/(B108*B107)
114				-15,89		SOM(D112:D113)
115	Floors					
116	CLT	3,97	m ³	-11,42		10*B116*C103/(B108*B107)
117						
118	Roofs					
119	CLT	2,65	m ³	-7,62		10*B119*C103/(B108*B107)
120						
121	Steel interlayer					
122	Steel	1621,03	kg	4,70		10*B122*C104/(1000*B108*B107)
123				Total impact	-30,23	SOM(D114+D116+D119+D122)

Figure X.135: Impact calculation 1 layer CLT top-up (2)_Excel

X.10.10.2 2-layer top-up

	A	B	C	D	E	F
1	Environmental calculation					
2	CLT side walls			Formule B		
3	Standard dimensions					
4	Height	2950	mm			
5	Length	9450	mm			
6	Thickness	100	mm			
7	Opening at gallery					
8	Width	1200	mm			
9	Height	2115	mm			
10	Area (-)	2538000	mm ²	B8*B9		
11	Volumes					
12	Volume single wall	2,53	m ³	((B4*B5*B6) - (B10*B6)) * 10^-9		
13	Number of side walls per unit	2	-			
14	Volume CLT for side walls/ unit	5,07	m ³	B12*B13		
15	Gypsum board					
17	Wall area	25,34	m ²	((B4*B5)-B10) * 10^-6		
18	Thickness gypsum board	18	mm			
19	Volume gypsum board/wall	0,46	m ³	B17*(B18/1000)		
20	Volume gypsum board total for walls	0,91	m ³	B19*B13		
22	CLT stability wall					
23	Dimensions					
24	Height	2950	mm	B4		
25	Length	2500	mm			
26	Thickness	100	mm	B6		
27	Volume	0,74	m ³	B24*B25*B26*10^-9		
29	Gypsum board					
30	Wall area (2 sides)	14,75	m ²	B24*B25*10^-6 * 2		
31	Thickness gypsum board	18	mm	B18		
32	Volume gypsum board	0,27	m ³	B30*(B31/1000)		
34	CLT roof					
35	Dimensions					
36	Width	3500	mm			
37	Length	9450	mm	B5		
38	Thickness	80	mm			
39	Volume	2,65	m ³	B36*B37*B38*10^-9		
41	CLT floor					
42	Dimensions					
43	Width	3500	mm	B36		
44	Length	9450	mm	B5		
45	Thickness	120	mm			
46	Volume	3,97	m ³	B43*B44*B45*10^-9		
49	Steel interlayer					
50	Dimensions					
51	Area beam	11800	mm ²	118*10^2		
52	Length per unit	3500	mm	B36		
53	Amount of beams	5	-			
54	Volume per 2 units (above each other)	0,21	m ³	B51*B52*10^-9*B53		
55	Weight steel	7850	kg/m ³			
56	Weight steel per 2 units (above each other)	1621,03	kg	B54*B55		
58	Overview of materials					
59	Amount of units considered	2	-			
60	New floor area (2 units)	66,15	m ²	B36*B37*B59*10^-6		
61	Considered lifetime	75	y			
63	Material	m³	:10^-3 m³/ m² new floor area/ y	Formule C		
64	1. CLT					
65	CLT side walls	10,14		2,04	B65*1000/B60/B61	
66	CLT stability walls	1,48		0,30	B66*1000/B60/B61	
67	CLT floors	7,94		1,60	B67*1000/B60/B61	
68	CLT roofs	5,29		1,07	B68*1000/B60/B61	
69	Total CLT	24,84		5,01	B69*1000/B60/B61	
70	2. Steel interlayer					
71	Steel beams	0,21		0,04	B71*1000/B60/B61	
72	3. Gypsum board					
73	Gypsum plasterboard type F (Walls)	2,36		0,47	B73*1000/B60/B61	
75	Total					
76	Total material usage	27,40		5,52	C69+C71+C73	

Figure X.136: Impact calculation 2-layer CLT top-up (1)_Excel

	A	B	C	D	E	F
77						
78	Global warming potential of CLT units					
79	Impact materials	Unit	kg CO ₂ -eq/unit			
80	CLT	m ³	-714			
81	Steel	1000 kg	719			
82	Gypsum board (18 mm)	m ²	3,11			
83						
84	Material	Amount (2 units)	Unit	GWP [kg CO₂ -eq]	Formule D	
85	CLT	24,84	m ³	-17736,33	B85*C80	
86	Steel	1621,03	kg	1165,52	B86/1000*C81	
87	Gypsum board	130,86	m ²	406,97	B87*C82	
88				-16163,85	SOM(D85:D87)	
89						
90	Total impact					
91	New floor area (2 units)	66,15	m ²		B60	
92	Considered lifetime	75	y	Percentage of total	B61	
93						
94	Material	Impact	Unit	% of total	Formule B	
95	Impact CLT	-35,75	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	109,7	D85*10/B91/B92	
96	Impact Steel	2,35	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	-7,2	D86*10/B91/B92	
97	Impact Gypsum board	0,82	*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y	-2,5	D87*10/B91/B92	
98	Total impact	-32,58	*10⁻¹ kg CO₂ -eq /m² new floor area / y		SOM(B95:B97)	
99						
100						
101	Impact splitted per element					
102	Impact materials	Unit	kg CO ₂ -eq/unit			
103	CLT	m ³	-714			
104	Steel	1000 kg	719			
105	Gypsum board (18 mm)	m ²	3,11			
106						
107	New floor area (2 units)	66,15	m ²		B60	
108	Considered lifetime	75	y		B61	
109						
110	Element	Amount (2 units)	Unit	Impact	Formule D	
111	Walls			[*10 ⁻¹ kg CO ₂ -eq /m ² new floor area / y]		
112	CLT	11,61	m ³	-16,71	10*B112*C103/(B108*B107)	
113	Plasterboard	130,86	m ²	0,82	10*B113*C105/(B108*B107)	
114				-15,89	SOM(D112:D113)	
115	Floors					
116	CLT	7,94	m ³	-11,42	10*B116*C103/(B108*B107)	
117						
118	Roofs					
119	CLT	5,29	m ³	-7,62	10*B119*C103/(B108*B107)	
120						
121	Steel interlayer					
122	Steel	1621,03	kg	2,35	10*B122*C104/(1000*B108*B107)	
123				-32,58	SOM(D114+D116+D119+D122)	
			Total impact			

Figure X.137: Impact calculation 2-layer CLT top-up (2)_Excel

X.10.10.3 Hand validation

Order ✓
 Blad nr 1/2
 Deel Verificatie environmental impact CLT
 Datum 01/06/2023

VAN ROSSUM
 RAADGEVENDE
 INGENIEURS

→ 2-story verification

Volume calculation

1. Side walls
 CLT → $V = 2 \left[\left[\frac{250}{1000} \cdot \frac{3450}{1000} \cdot \frac{1000}{1000} \right] - \left[\frac{1200}{1000} \cdot \frac{2115}{1000} \cdot \frac{1500}{1000} \right] \right] = 5.07 \text{ [m}^3\text{]} \checkmark$

↳ Gypsum board
 → $V_{wt} = \left[(2.95 \cdot 9.45) - (1.2 \cdot 2.115) \right] \cdot 2 = 50.68 \text{ [m}^2\text{]} \checkmark$
 → $V_{wt} = 50.68 \cdot 0.018 = 0.91 \text{ [m}^3\text{]} \checkmark$

2. CLT stability wall
 CLT → $V = 2.95 \cdot 2.5 \cdot 0.1 = 0.74 \text{ [m}^3\text{]} \checkmark$
 Gypsum board → $A_{wt} = 2 \cdot 2.95 \cdot 2.5 = 14.75 \text{ [m}^2\text{]}$
 $V_{wt} = 2 \cdot 14.75 \cdot 0.018 = 0.53 \text{ [m}^3\text{]} \checkmark$

3. CLT Roof
 ↳ $V = 3.5 \cdot 9.45 \cdot \frac{20}{1000} = 2.65 \text{ [m}^3\text{]} \checkmark$

4. CLT Floor
 ↳ $V = 3.5 \cdot 9.45 \cdot \frac{120}{1000} = 3.97 \text{ [m}^3\text{]} \checkmark$

5. Steel interlayer
 ↳ $V_{wt} = (11200 \cdot 10^{-6}) \cdot 7.5 \cdot 5 = 0.42 \text{ [m}^3\text{]} \checkmark$
 ↳ $W_{ght} = 0.21 \cdot 7250 = 1522.5 \text{ [kg]} \checkmark$

Calculation total material usage

CLT $V = (5.07 + 0.74 + 2.65 + 3.97) \cdot 2 = 24.86 \text{ [m}^3\text{]}$
 $RSL = 75y$
 $A = 3.5 \cdot 9.45 \cdot 2 = 66.15 \text{ [m}^2\text{]}$ } $mat. use = \frac{24.86 \cdot 1000}{75 \cdot 66.15} = 5.01 \cdot 10^{-3} \text{ m}^3/y/m^2 \checkmark$

Steel $V = 0.21 \text{ m}^3$
 $RSL = 75y$
 $A = 66.15 \text{ [m}^2\text{]}$ } $mat. use = \frac{0.21 \cdot 1000}{75 \cdot 66.15} = 0.04 \cdot 10^{-3} \text{ m}^3/y/m^2 \checkmark$

Gypsum board → $V = (0.91 + 0.53) \cdot 2 = 2.36 \text{ [m}^3\text{]}$
 $RSL = 75y$
 $A = 66.15 \text{ [m}^2\text{]}$ } $mat. use = \frac{2.36 \cdot 1000}{75 \cdot 66.15} = 0.48 \cdot 10^{-3} \text{ m}^3/y/m^2 \checkmark$

total → $V = 5.01 + 0.04 + 0.48 = 5.53 \cdot 10^{-3} \text{ m}^3/y/m^2 \text{ new floor} \checkmark$
 ↳ $5.52 \cdot 10^{-3} \text{ m}^3/y/m^2$

Figure X.138: Validation environmental impact CLT (1)

Order /
 Blad nr 2/2
 Deel Verification environmental impact CLT
 Datum 01/06/2023

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• Calculation GWP total

CLT → $V = 24.86 \text{ [m}^3\text{]}$

$$\left. \begin{array}{l} R_{SI} = 75 \text{ y} \\ A = 66.15 \text{ [m}^2\text{]} \end{array} \right\} GWP = \frac{24.86 \cdot 75 \cdot 10}{75 \cdot 66.15} = -35.178 \cdot 10^3 \text{ kg CO}_2\text{-eq/ly} \\ \text{↔ } -35.178 \cdot 10^3 \text{ kg CO}_2\text{-eq/ly}$$

Steel → $W = 162103 \text{ kg}$

$$\left. \begin{array}{l} R_{SI} = 75 \text{ y} \\ A = 66.15 \text{ [m}^2\text{]} \end{array} \right\} GWP = \frac{(162103 \text{ kg}) \cdot 75 \cdot 10}{75 \cdot 66.15} = 2.35 \cdot 10^4 \text{ kg CO}_2\text{-eq/ly} \\ \text{↔ } 2.35 \cdot 10^4 \text{ kg CO}_2\text{-eq/ly}$$

System board → $A = (50.68 + 14.75) \cdot 2 = 130.86 \text{ [m}^2\text{]}$

$$\left. \begin{array}{l} R_{SI} = 75 \text{ y} \\ A = 66.15 \text{ [m}^2\text{]} \end{array} \right\} GWP = \frac{130.86 \cdot 75 \cdot 10}{75 \cdot 66.15} = 0.82 \cdot 10^4 \text{ kg CO}_2\text{-eq/ly} \\ \text{↔ } 0.82 \cdot 10^4 \text{ kg CO}_2\text{-eq/ly}$$

Total : $-35.178 + 2.35 \cdot 10^4 = -32.610 \text{ kg CO}_2\text{-eq/ly} \\ \text{↔ } -32.610 \cdot 10^3 \text{ kg CO}_2\text{-eq/ly new floor}$

SHEET is validated, also holds for 1-storey top-up

Figure X.139: Validation environmental impact CLT (2)

X.11 Structural design of HSB top-up [Appendix D.4]

X.11.1 HSB roof structure

X.11.1.1 Excel sheet

	A	B	C	D	E	F	G	H	I	J	K
1	Calculation HSB Roof structure										
2	Safety factors (NEN-EN 1990)										
3	Eq 6.1	γ_G	γ_Q								
4	Eq 6.2	1,35	1,5								
5		1,2	1,5								
6	Chosen floor										
7	Timber beams										
8	E-modulus (C24)	11000	N/mm ²	Formule B							
9	Density	420	kg/m ³								
10	Height	170	mm								
11	Width	70	mm								
12	Span	4,6	mm								
13	Spacing	500	mm								
14	m_{beams}	10,00	kg/m ²	$(1/(B13/1000)) * 1 * (B10/1000)*(B11/1000) * B9$							
15	I_{beam}	28659166,67	mm ⁴	$(1/12)*B11*B10^3$							
16	W_{beam}	337166,6667	mm ³	$(1/6)*B11*B10^2$							
17	Multiplex										
19	E-modulus (C18)	9000	N/mm ²								
20	Density	550	kg/m ³								
21	Thickness	18	mm								
22	$m_{multiplex}$	9,90	kg/m ²	$(B21/1000)*B20$							
23	Loads										
24	Permanent										
25	Beam	0,10	kN/m ²	Formule B							
26	Multiplex	0,10	kN/m ²	$B14*10/1000$							
27	Ceiling	0,20	kN/m ²	$B22*10/1000$							
28	Roof finishing	0,20	kN/m ²								
29	Total_characteristic	0,60	kN/m²								
30	Variable										
31	Imposed loading	1	kN/m ²	$\psi_{0,j}$	$\psi_{2,j}$						
32	Snowload:										
33	s	0,7	kN/m ²								
34	μ_j	0,8	-								
35	C_e	1	-								
36	C_f	1	-								
37	Value snowload	0,56	kN/m ²	$B35*B36*B37*B38$							
38	Windload										
39	$q_{0,(22.4)}$	0,93	kN/m ²								
40	$C_{f,zone F}$	-1,8	-								
41	$C_{f,zone G}$	-1,2	-								
42	$C_{f,zone H}$	-0,7	-								
43	$C_{f,zone I}$	-0,2	-								
44	$C_s C_d$	1	-								
45	$q_{w,k}$	-0,186	kN/m ²	$B42*MAX(B43:B46)*B47$							
46	Design value load										
47	Eq 6.1	0,81	kN/m ²	SLS	0,60	kN/m ²	$(B3*B30)+(C3*D33*B48)$				
48	Eq 6.2	2,22	kN/m ²		1,60	kN/m ²	$(B4*B30) + (C4*B33) + (C4*D33*B39)$				
49	Normative design value	2,22	kN/m²		1,60	kN/m²	MAX(B51:B52)				
50	Verification bending (ULS)										
51	Material properties (C24)										
52	Climate class	I	-	Formule B							
53	Load duration	long-term	-								
54	k_{mod}	0,7	-								
55	$f_{m,k}$	24	N/mm ²								
56	γ_M	1,3	-								
57	k_b	1	-	$ALS(B10<150; MIN((150/B10)^{0,2}; 1,3); 1)$							
58	$f_{m,d}$	12,92	N/mm ²	$B60*B59*B62/B61$							
59	Forces_ULS and verification										
60	q_{Ed}	1,11	N/mm	$B53*B13/1000$							
61	Span	4600	mm	$B12*1000$							
62	Moment	2934300	Nmm	$(1/8)*B66*B67^2$							
63	σ_M	8,70	N/mm ²	$B68/B16$							
64	UC1	0,67	-	$B69/B63$							

Figure X.140: Structural verification HSB roof structure (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K
71											
72	Verification shear (ULS)										
73	Material properties (C24)										
74	Climate class	I	-								
75	Load duration	long-term	-								
76	k_{mod}	0,7	-								
77	$f_{v,k}$	4	N/mm ²								
78	γ_M	1,3	-								
79	$f_{m,d}$	2,15	N/mm ²	B77*B76/B78							
80											
81	Forces_ULS and verification										
82	q_{Ed}	1,11	N/mm	B53*B13/1000							
83	Span	4600	mm	B12*1000							
84	V	2552	N	(1/2)*B82*B83							
85	τ_v	0,32	N/mm ²	(3/2)*B84/(B11*B10)							
86	UC2	0,15	-	B85/B79							
87											
88	Verification deflections (SLS)										
89	Values										
90	$q_{g,k}$	0,30	N/mm	B30*B13/1000							
91	$q_{Q1,k}$	0,5	N/mm	B33*B13/1000							
92	$q_{Q1,s,k}$	0,28	N/mm	B39*B13/1000							
93	$q_{Q1,w,k}$	-0,093	N/mm	B48*B13/1000							
94	Instantaneous deflections										
95	$u_{inst,G}$	5,54	mm	(5/384)*B90*(B12*1000)^4/(B8*B15)							
96	$u_{inst,Q1}$	9,25	mm	(5/384)*B91*(B12*1000)^4 / (B8*B15)							
97	$u_{inst,Q1,s}$	5,18	mm	(5/384)*B92*(B12*1000)^4/(B8*B15)							
98	$u_{inst,Q1,w}$	-1,72	mm	(5/384)*B93*(B12*1000)^4/(B8*B15)							
99	Final deflections										
100	k_{def} (Climate class I; Sawn wood)	0,6									
101	$u_{fin,G}$	8,86	mm	B95*(1+B100)							
102	$u_{fin,Q1}$	9,25	mm	B96*(1 + (E33*B100))							
103	$u_{fin,Q1,s}$	0,00	mm								
104	$u_{fin,Q1,w}$	0,00	mm								
105	u_{fin}	18,11	mm	SOM(B101:B104)							
106	Verification										
107	$w_{allowed}$	18,4	mm	0,004*B12*1000							
108	UC3	0,98	-	B105/B107							
109											
110	Small verification multiplex itself										
111				Formule B							
112											
113	Verification ULS (Bending)										
114	Material properties (C18)										
115	Climate class	I	-								
116	Load duration	long-term	-								
117	k_{mod}	0,7	-								
118	$f_{m,k}$	18	N/mm ²								
119	γ_M	1,2	-								
120	$f_{m,d}$	10,50	N/mm ²	B117*B118/B119							
121											
122	Forces_ULS and verification										
123	q_{ed} (Considering 1 m)	2,22	kN/m	B53*1							
124	L	500	mm	B13							
125	M_{ed}	69336	Nmm	(1/8)*B123*B124^2							
126	W_{floor}	54000	mm ³	(1/6)*1000*B21^2							
127	σ_M	1,28	N/mm ²	B125/B126							
128	UC4	0,12	-	B127/B120							
129											
130	Verification deflections (SLS)										
131	Values										
132	$q_{g,k}$	0,60	N/mm	B30*1							
133	$q_{Q1,k}$	1	N/mm	B33*1							
134	$q_{Q1,s,k}$	0,56	N/mm	B39*1							
135	$q_{Q1,w,k}$	-0,186	N/mm	B48*1							
136	Instantaneous deflections										
137	$u_{inst,G}$	0,11	mm	(5/384)*B132*(B124)^4/(B19*(1/12)*1000*B21^3)							
138	$u_{inst,Q1}$	0,19	mm	(5/384)*B133*(B124)^4/(B19*(1/12)*1000*B21^3)							
139	$u_{inst,Q1,s}$	0,10	mm	(5/384)*B134*(B124)^4/(B19*(1/12)*1000*B21^3)							
140	$u_{inst,Q1,w}$	-0,03	mm	(5/384)*B135*(B124)^4/(B19*(1/12)*1000*B21^3)							
141	Final deflections										
142	k_{def} (Climate class I; multiplex)	0,8									
143	$u_{fin,G}$	0,20	mm	B137*(1+B142)							
144	$u_{fin,Q1}$	0,19	mm	B138*(1 + (E33*B142))							
145	$u_{fin,Q1,s}$	0,00	mm	B139*(D33+(E33*B142))							
146	$u_{fin,Q1,w}$	0,00	mm	B140*(D33+(E33*B142))							
147	u_{fin}	0,39	mm	SOM(B143:B146)							
148	Verification										
149	$w_{allowed}$	2	mm	0,004*B124							
150	UC5	0,19	-	B147/B149							
151											
152											
153											
154	Results				Beams			Plywood		m_{permanent}	
155	Span [m]	Applied beam (C24)	Spacing [mm]	Multiplex (C18)	UC1	UC2	UC3	UC4	UC5	kN/m ²	
156	3.9 - 4.6	70x170	500	18	0,67	0,15	0,98	0,12	0,19	0,60	
157	4.7 - 5.4	70x195	400	18	0,58	0,13	0,87	0,08	0,10	0,64	
158	5.5 - 6.6	70x245	400	18	0,56	0,12	0,83	0,08	0,11	0,68	

Figure X.141: Structural verification HSB roof structure (2)_Excel

X.11.1.2 Excel hand validation

Order ✓
 Blad nr 1 / 3
 Deel Verificatie HSB ~~W~~ Roof
 Datum 17/05/2023



o Scheme



o Beam properties

E-modulus = 11000 [N/mm ²]	Multiplex properties
p = 420 [kg/m ³]	E-modulus = 9000 [N/mm ²]
H = 170 [mm]	p = 550 [kg/m ³]
b = 170 [mm]	d = 18 [mm]
Span = 4.6 [m]	
Span _{eff} = 5.2 [m]	
I_{DIN} = $(170/100) \cdot (170/100) \cdot 420 \cdot \frac{1000}{5000} = 10.00$ [kg/m ²]	
I_{DIN} = $\frac{1}{12} b h^3 = \frac{1}{12} \cdot 170 \cdot 170^3 = 2.86 \cdot 10^7$ [mm ⁴]	
W = $\frac{1}{6} b h^2 = \frac{1}{6} \cdot 170 \cdot 170^2 = 337166.67$ [mm ³]	

o Loads (defined in excel)

permanent = 0.6 [kN/m²]

Variable

1. wind = -0.86 [kN/m²]
2. snow = 0.56 [kN/m²]
3. imposed = 1 [kN/m²]

$\psi_0 = 0$

by 6.10a: $1.35 \cdot 0 = 1.35 \cdot \psi_0 = 1.35 \cdot 0 = 0.0$ [kN/m²]

by 6.10b: $1.2 \cdot 0 + 1.5 \cdot 0 + 2.15 \cdot 0.0 = (1.2 \cdot 0) + (1.5 \cdot 1) = 2.22$ [kN/m²]

$q_{ed} = 2.22$ [kN/m²] $\rightarrow 2.22$ [kN/m²] (select)

$q_{ed} = 2.22 \cdot \frac{5000}{1000} = 1.11$ [kN/m] = 1 [kN/m]

1. Beam

o Verification bending beam

$M_{ed} = \frac{1}{8} q_{ed} l^2 = \frac{1}{8} \cdot 1.11 \cdot 4600^2 = 2935950$ [Nmm] (select)

$\sigma_{M} = \frac{M}{W} = \frac{2935950}{337166} \cdot 8.170$ [N/mm²] $\rightarrow 8.170$ [N/mm²]

$k_{red} = 0.7$
 $\beta_M = 1.3$
 $k_{M2} = 24$
 $k_{M1} = 1$

$f_{M2} = \frac{24 \cdot 0.7}{1.3} = 12.52$ [N/mm²]

$u_c = \frac{\sigma_M}{f_{M2}} = \frac{8.170}{12.52} = 0.67$ [-] $\rightarrow 0.67$ [-]

Figure X.142: Validation HSB roof structure (1)

Order ✓
 Blad nr 2 / 3
 Deel Ver. Fictian HSB roof
 Datum 17/05/2023



• Ver. Fictian - above beams

$$V_{act} = \frac{1}{2} \cdot l = \frac{1}{2} \cdot 11 \cdot 600 = 3300 \text{ [N]}$$

$$c_v = \frac{3}{2} \frac{V_{act}}{b \cdot l} = \frac{3}{2} \frac{3300}{170 \cdot 170} = 0.32 \text{ [N/mm}^2\text{]} \quad \text{Staal} \rightarrow 0.32 \text{ [N/mm}^2\text{]}$$

$$\left. \begin{matrix} k_{red} = 0.7 \\ f_{yk} = 4 \\ \gamma_M = 1.3 \end{matrix} \right\} f_{red} = \frac{0.7 \cdot 4}{1.3} = 2.15 \text{ [N/mm}^2\text{]}$$

$$L_b UC = \frac{0.32}{2.15} = 0.15 \text{ [-]} \quad \text{Staal} \rightarrow 0.15 \text{ (steel value)}$$

• Ver. Fictian - deflections ($\alpha_{rel} = 0.6$) $\psi_2 = 0$

$$u_{fin, ai} = \left(\frac{5}{384} \left(\frac{q \cdot s_{span}}{1000} \right) \frac{s^4}{EI_{beam}} \right) \cdot (1 + \alpha_{rel}) \cdot I \cdot I_g \text{ [mm]}$$

$$= \left(\frac{5}{384} \cdot (0.6 \frac{span}{1000}) \cdot \frac{span^4}{(1000 \cdot 286 \cdot 10^9}) \right) \cdot (1 + 0.6) \cdot I \cdot I_g \text{ [mm]}$$

$$u_{fin, ai} = \left(\frac{5}{384} \left(\frac{q \cdot s_{span}}{1000} \right) \frac{s^4}{EI_{beam}} \right) \cdot (1 + \psi_2 \alpha_{rel})$$

$$= \left(\frac{5}{384} \cdot (1 \cdot \frac{span}{1000}) \cdot \frac{span^4}{(1000 \cdot 286 \cdot 10^9}) \right) \cdot (1 + 0.6) \cdot I \cdot I_g \text{ [mm]}$$

$$u_{fin, ai} = 0 \quad (\psi_0 = \psi_n = 0)$$

$$u_{fin} = I \cdot I_g = 9.27 = 18.16 \text{ [mm]} \quad \text{Staal} \rightarrow 18.16 \text{ [mm]}$$

$$u = \frac{u_{fin}}{0.001} = \frac{18.16}{0.001} = 0.59 \text{ [-]} \quad \text{Staal} \rightarrow 0.59 \text{ (steel value)}$$

2. Multi-plate

• Ver. Fictian bending

$$L_b \text{ consideration } 1 \text{ [m]} \rightarrow q_{act} = 2.22 \cdot 1 = 2.22 \text{ [N/mm]}$$

$$s_{span} = 3000 \text{ [mm]}$$

$$M_{act} = \frac{q \cdot l^2}{8} = \frac{2.22 \cdot 3000^2}{8} = 65375 \text{ [Nmm]}$$

$$W = \frac{b \cdot h^3}{12} = \frac{1000 \cdot 18^3}{12} = 54000 \text{ [mm}^3\text{]}$$

$$\sigma_{act} = \frac{M}{W} = \frac{65375}{54000} = 1.21 \text{ [N/mm}^2\text{]}$$

$$\left. \begin{matrix} k_{red} = 0.7 \\ \gamma_M = 1.3 \end{matrix} \right\} f_{red} = \frac{0.7 \cdot 4}{1.3} = 2.15 \text{ [N/mm}^2\text{]}$$

$$L_b UC = \frac{1.21}{2.15} = 0.12 \text{ [-]} \quad \text{Staal} \rightarrow 0.12 \text{ (steel value)}$$

Figure X.143: Validation HSB roof structure (2)

Order ✓
 Blad nr 3/3
 Deel Verificatie HSB roof
 Datum 17/05/2023

VAN RAADGEVENDE
ROSSUM INGENIEURS

deflection parallel plate $k_{def} = 0,3$

$$u_{p,c} = \left(\frac{5}{384} \frac{q_c L^4}{EI} \right) (1 + k_{def})$$

$$= \left[\frac{5}{384} \cdot (0,6 \cdot 1) \frac{500^4}{(9000 \cdot (12 \cdot 1000 \cdot 10^9))} \right] (1 + 0,3) = 0,20 \text{ [mm]}$$

$$u_{p,q,p} = \left(\frac{5}{384} \frac{q_c L^4}{EI} \right) (1 + k_{def})$$

$$= \left[\frac{5}{384} \cdot (1 \cdot 1) \frac{500^4}{(9000 \cdot (12 \cdot 1000 \cdot 10^9))} \right] (1 + 0,3) = 0,35 \text{ [mm]}$$

$u_{in} = 0,35 \text{ [mm]} \quad \& \leftrightarrow 0,35 \text{ [mm]}$

$MC = \frac{u_{in}}{0,004} = \frac{0,35}{0,004} = 0,20 \text{ [-]} \quad \& \leftrightarrow 0,35 \text{ (steel value)}$

SHEET Roof = Verleiderlaaf

Figure X.144: Validation HSB roof structure (3)

X.11.2 HSB floor structure

X.11.2.1 Calculation normal floor

	A	B	C	D	E	F	G	H	I	J	K	L	M									
1	Calculation residential floor																					
2	Safety factors (NEN-EN 1990)																					
3	Eq 6.1	V _G	V _Q																			
4	Eq 6.2		1,35	1,5																		
5			1,2	1,5																		
6	Chosen floor																					
7	Timber beams																					
8	E-modulus (C24)		11000	N/mm ²	Formule B																	
9	Density		420	kg/m ³																		
10	H _{beam}		270	mm																		
11	b _{beam}		70	mm																		
12	Span (L)		4,6	m																		
13	Spacing		400	mm																		
14	I _{beam}		1,15E+08	mm ⁴																		
15	W _{beam}		8,51E+05	mm ³																		
16	(EI) _{single beam}		1,26E+12	Nmm ²																		
17	(EI) _{floor}		3,16E+12	Nmm ² /m																		
18																						
19	Multiplex																					
20	E-modulus (C18)		9000	N/mm ²																		
21	Density		550	kg/m ³																		
22	Thickness		18	mm																		
23	Floor width (B)		10	m																		
24	(EI) _b		4,37E+09	Nmm ² /m																		
25																						
26	Mass permanent																					
27	m _{beams,d}		19,85	kg/m ²																		
28	m _{multiplex}		9,90	kg/m ²																		
29	m _{add;fermacel}		26,00	kg/m ²																		
30	m _{total}		55,75	kg/m ²																		
31																						
32	Structural scheme																					
33																						
34																						
35																						
36																						
37																						
38	Loads																					
39	Permanent																					
40	Beam		0,20	kN/m ²	Formule B																	
41	Multiplex		0,10	kN/m ²																		
42	Ceiling		0,20	kN/m ²																		
43	Finishing (22 mm fermacell)		0,26	kN/m ²																		
44	Total_characteristic		0,76	kN/m ²																		
45					SOM(B40:B43)																	
46	Variable																					
47	Imposed loading		1,75	kN/m ²	Formule B																	
48	Non-structural walls		0,5	kN/m ²																		
49	Total_characteristic		2,25	kN/m ²																		
50					ψ _{0,j}	ψ _{2,j}																
51						0,4	0,3															
52					SOM(B47:B48)																	
53																						
54	Design value load																					
55	Eq 6.1		2,37	kN/m ²	Formule B																	
56	Eq 6.2		3,83	kN/m ²																		
57	Normative design value		3,83	kN/m ²																		
58					SOM(B52:B53)																	
59	Verification bending (ULS)																					
60	Material properties (C24)																					
61	Climate class		I	-	Formule B																	
62	Load duration		long-term	-																		
63	k _{mod}		0,7	-																		
64	k _h		1	-																		
65	f _{m,k}		24	N/mm ²																		
66	γ _M		1,3	-																		
67	f _{m,d}		12,92	N/mm ²																		
68														ALS(B10<150; MIN((150/B10)^0,2; 1,3); 1)								
69					B62*B60*B61/B63																	
70	Forces_ULS and verification																					
71	q _{Ed}		1,53	N/mm	Formule B																	
72	Span		4600	mm																		
73	Moment		4,06E+06	Nmm																		
74	σ _M		4,77	N/mm ²																		
75	UC1		0,37	-																		
76					B54*B13/1000																	
77					B12*1000																	
78					(1/8)*B67*B68^2																	
79					B69/B15																	
80					B70/B64																	

Figure X.145: Structural verification HSB floor structure (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M
72													
73	Verification shear (ULS)												
74	Material properties (C24)												
75	Climate class	I	-										
76	Load duration	long-term	-										
77	k_{mod}	0,7	-										
78	$f_{v,k}$	4	N/mm ²										
79	γ_M	1,3	-										
80	$f_{v,d}$	2,15	N/mm ²	B78*B77/B79									
81													
82	Forces_ULS and verification												
83	q_{Ed}	1,53	N/mm	B54*B13/1000									
84	Span	4600	mm	B12*1000									
85	V	3527	N	(1/2)*B83*B84									
86	τ_v	0,28	N/mm ²	(3/2)*B85/(B11*B10)									
87	UC2	0,13	-	B86/B80									
88													
89	Verification deflections (SLS)												
90	Values												
91	q_{gk}	0,30	N/mm	B44*B13/1000									
92	$q_{Q1,k}$	0,7	N/mm	B47*B13/1000									
93	$q_{Q2,k}$	0,2	N/mm	B48*B13/1000									
94	Instantaneous deflections												
95	$u_{inst,G}$	1,40	mm	(5/384)*B91*(B12*1000)^4/(B16)									
96	$u_{inst,Q1}$	3,23	mm	(5/384)*B92*(B12*1000)^4 / (B16)									
97	$u_{inst,Q2}$	0,92	mm	(5/384)*B93*(B12*1000)^4/(B16)									
98	Final deflections												
99	k_{def} (Climate class I; Sawn wood)	0,6	-										
100	$u_{fin,G}$	2,24	mm	B95*(1+B99)									
101	$u_{fin,Q1}$	3,81	mm	B96*(1 + (E49*B99))									
102	$u_{fin,Q2}$	0,54	mm	B97*(D49+(E49*B99))									
103	u_{fin}	6,59	mm	SOM(B100:B102)									
104	Verification												
105	$w_{allowed}$	18,4	mm	0,004*B12*1000									
106	UC3	0,36	-	B103/B105									
107	Verification vibrations												
108	1. Fundamental frequency												
109													
110	L	4,6	m										
111	$(EI)_L$	3,16E+06	Nm ² /m										
112	m	55,75	kg/m ²										
113	f1	17,67	Hz	>									
114	UC4	0,45	-										
115													
116	2. Pointload deflection												
117													
118	a	0,4	m	(=spacing)									
119	a1	1	m										
120	$EI_{0,5\sigma_{exp}}$	4,37E+03	Nm ² /m										
121	k_r	0,60	-										
122	Pointload F	3	kN										
123	$EI_{single\ beam}$	1,26E+06	Nm ²										
124	L	4,6	m										
125	w	2,90	mm										
126	$w_{per\ kN}$	0,97	<										
127	UC5	0,97	-										
128													
129	3. Impulse velocity response												
130													
131	b	120	m/Ns ²										
132	ξ	0,01	-										
133	f1	17,67	Hz										
134	v_{max}	0,019	m/s										
135													
136	B	10	m										
137	L	4,6	m										
138	$(EI)_L$	3,16E+06	Nm ² /m										
139	$(EI)_b$	4,37E+03	Nm ² /m										
140	n_{40}	16,06	-										
141													
142	m	55,75	kg/m ²										
143	v	0,015	m/s										
144	UC6	0,75	-										
145													
146													
147	Small verification multiplex itself												
148	> Mark, the supplier states that 18 mm could be used for both 400 & 600 mm spacing												
149													
150	Verification ULS (Bending)												
151	Material properties (C18)												
152	Climate class	I	-										
153	Load duration	long-term	-										
154	k_{mod}	0,7	-										

Figure X.146: Structural verification HSB floor structure (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	
155		$f_{m,k}$	18 N/mm ²											
156		γ_M	1,2 -											
157		$f_{m,d}$	10,50 N/mm ²	B155*B60/B156										
158														
159	Forces_ULS and verification													
160		q_{ed} (Considering 1 m)	3,83 kN/m	B54*1										
161		L	400 mm	B13										
162		M_{ed}	7,67E+04 Nmm	(1/8)*B160*B161^2										
163		W_{floor}	54000 mm ³	(1/6)*1000*B22^2										
164		σ_M	1,42 N/mm ²	B162/B163										
165		UC7	0,14	B164/B157										
166														
167	Verification deflections (SLS)													
168	Values													
169		$q_{g,k}$	0,76 N/mm	B44*1										
170		$q_{Q1,k}$	1,75 N/mm	B47*1										
171		$q_{Q2,k}$	0,5 N/mm	B48*1										
172	Instantaneous deflections													
173		$u_{inst,G}$	0,06 mm	(5/384)*B169*(B13)^4/(B20*(1/12)*1000*B22^3)										
174		$u_{inst,Q1}$	0,13 mm	(5/384)*B170*(B13)^4/(B20*(1/12)*1000*B22^3)										
175		$u_{inst,Q2}$	0,04 mm	(5/384)*B171*(B13)^4/(B20*(1/12)*1000*B22^3)										
176	Final deflections													
177		k_{def} (Climate class I; multiplex)	0,8											
178		$u_{fin,G}$	0,10 mm	B173*(1+B177)										
179		$u_{fin,Q1}$	0,17 mm	B174*(1 + (E49*B177))										
180		$u_{fin,Q2}$	0,02 mm	B175*(D49+(E49*B177))										
181		u_{fm}	0,29 mm	SOM(B178:B180)										
182	Verification													
183		$W_{allowed}$	1,6 mm	0,004*B13										
184		UC8	0,18 -	B181/B183										
185														
186														
187														
188	Results			Beams						Multiplex		$m_{permanent}$		
189		Span [m]	Applied beam (C24)	Spacing [mm]	Multiplex (C18)	UC1	UC2	UC3	UC4	UC5	UC6	UC7	UC8	kn/m^2
190		3.9 - 4.6	70x270	400	18	0,37	0,13	0,36	0,45	0,97	0,75	0,14	0,18	0,76
191		4.7 - 5.4	95x295	500	21	0,4	0,13	0,42	0,55	0,93	0,64	0,16	0,23	0,81
192		5.5 - 5.7	95x295	400	21	0,36	0,11	0,4	0,57	0,95	0,59	0,1	0,12	0,87
193		5.8 - 6.6*	95x295	170	21	0,23	0,06	0,31	0,63	0,98	0,39	0,02	0,01	
194		5.8 - 6.6	70 x 245	400	18	0,34	0,12	0,31	0,4	0,78	0,66	0,1	0,12	0,98

* As not efficient, the main steel beams span from wall to wall (spaced 4 [m]), and secondary beams (70x245 - 400mm) are applied.

Figure X.147: Structural verification HSB floor structure (3)_Excel

X.11.2.2 Verification normal beams below façade

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Calculation residential floor_Below facade												
2	Safety factors (NEN-EN 1990) γ_G γ_Q												
3	Eq 6.1	1,35	1,5										
4	Eq 6.2	1,2	1,5										
5	Chosen floor												
6	Timber beams												
8	E-modulus (C24)	11000	N/mm ²										
9	Density	420	kg/m ³										
10	H _{beam}	270	mm										
11	b _{beam}	70	mm										
12	Span (L)	4,6	m										
13	Spacing	400	mm										
14	I _{beam}	1,15E+08	mm ⁴	(1/12)*B11*B10^3									
15	W _{beam}	8,51E+05	mm ³	(1/6)*B11*B10^2									
16	(EI) _{single beam}	1,26E+12	Nmm ²	B8*B14									
17	(EI) _{floor}	3,16E+12	Nmm ² /m	B16*1/(B13/1000)									
18	Multiplex												
20	E-modulus (C18)	9000	N/mm ²										
21	Density	550	kg/m ³										
22	Thickness	18	mm										
23	Floor width (B)	10	m										
24	(EI) _b	4,37E+09	Nmm ² /m	B20*(1/12)*1000*B22^3									
25	Mass permanent												
27	m _{beams,j}	19,85	kg/m ²	(1/(B13/1000)) * 1 * (B10/1000)*(B11/1000) * B9									
28	m _{multiplex}	9,90	kg/m ²	(B22/1000)*B21									
29	m _{add,Fermacell}	26,00	kg/m ²										
30	m _{total}	55,75	kg/m ²	SOM(B27:B29)									
31	Structural scheme												
33													
37	Loads												
38	Permanent kN/m ² kN/m Formule C												
40	Beam	0,198	0,08	B40*B13/1000									
41	Multiplex	0,099	0,04	B41*B13/1000									
42	Facade	0,5	1,40	B42*2,8									
43	Ceiling	0,20	0,08	B43*B13/1000									
44	Finishing (22 mm fermacell)	0,26	0,10	B44*B13/1000									
45	Total_characteristic	1,26	1,70	SOM(C40:C44)									
46	Variable kN/m ² kN/m												
48	Imposed loading	1,75	0,7	B48*B13/1000									
49	Non-structural walls	0,5	0,2	B49*B13/1000	$\psi_{0,j}$	$\psi_{2,j}$							
50	Total_characteristic	2,25	0,9	0,4	0,3								
51	Design value load												
53	Eq 6.1	2,84	kN/m	(B3*C45) + (C3*D50*C50)									
54	Eq 6.2	3,21	kN/m	(B4*C45) + (C4*C48) + (C4*D50*C49)									
55	Normative design value	3,21	kN/m	MAX(B53:B54)									
56	Verification bending (ULS)												
57	Material properties (C24)												
59	Climate class	I	-										
60	Load duration	long-term	-										
61	k _{mod}	0,7	-										
62	k _h	1	-	ALS(B10<150; MIN((150/B10)^0,2; 1,3); 1)									
63	f _{m,k}	24	N/mm ²										
64	γ _M	1,3	-										
65	f _{m,d}	12,92	N/mm ²	B63*B61*B62/B64									
66	Forces_ULS and verification												
68	q _{Ed}	3,21	N/mm	B55									
69	Span	4600	mm	B12*1000									
70	Moment	8,50E+06	Nmm	(1/8)*B68*B69^2									
71	σ _M	9,99	N/mm ²	B70/B15									
72	UC1	0,77	-	B71/B65									

Figure X.148: Structural verification HSB floor structure (Below façade) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M
72		UC1	0,77 -		B71/B65								
73													
74		Verification shear (ULS)											
75		Material properties (C24)											
76		Climate class	I	-									
77		Load duration	long-term	-									
78		k_{mod}	0,7	-									
79		$f_{v,k}$	4	N/mm ²									
80		V_M	1,3	-									
81		$f_{V,d}$	2,15	N/mm ²	B79*B78/B80								
82													
83		Forces_ULS and verification											
84		q_{Ed}	3,21	N/mm	B55								
85		Span	4600	mm	B12*1000								
86		V	7391	N	(1/2)*B84*B85								
87		τ_v	0,59	N/mm ²	(3/2)*B86/(B11*B10)								
88		UC2	0,27 -		B87/B81								
89													
90		Verification deflections (SLS)											
91		Values											
92		$q_{g,k}$	1,70	N/mm	C45								
93		$q_{Q1,k}$	0,7	N/mm	C48								
94		$q_{Q2,k}$	0,2	N/mm	C49								
95		Instantaneous deflections											
96		$u_{inst,G}$	7,86	mm	(5/384)*B92*(B12*1000)^4/(B16)								
97		$u_{inst,Q1}$	3,23	mm	(5/384)*B93*(B12*1000)^4 / (B16)								
98		$u_{inst,Q2}$	0,92	mm	(5/384)*B94*(B12*1000)^4/(B16)								
99		Final deflections											
100		k_{def} (Climate class I; Sawn wood)	0,6										
101		$u_{fin,G}$	12,58	mm	B96*(1+B100)								
102		$u_{fin,Q1}$	3,81	mm	B97*(1+(E50*B100))								
103		$u_{fin,Q2}$	0,54	mm	B98*(D50+(E50*B100))								
104		u_{fin}	16,93	mm	SOM(B101:B103)								
105		Verification											
106		$W_{allowed}$	18,4	mm	0,004*B12*1000								
107		UC3	0,92 -		B104/B106								
108													
109													
110		Results				Beams							
111		Span [m]	Applied beam (C24)	Spacing [mm]	Multiplex (C18)	UC1	UC2	UC3					
112		3.9 - 4.6	70x270	400	18	0,77	0,27	0,92					
113		4.7 - 5.4	95x295	500	21	0,74	0,24	0,93					
114		5.5 - 5.7	95x295 (2 beams)	400	21	0,37	0,12	0,5					

Figure X.149: Structural verification HSB floor structure (Below façade) (2)_Excel

X.11.2.3 Steel beams for span (5.8-6.6 m)

	A	B	C	D	E	F	G
1	Structural calculation main beams						
2	Safety factors (NEN-EN 1990)	V _G	V _Q				
3	Eq 6.1	1,35	1,5				
4	Eq 6.2	1,2	1,5				
5							
6	Loads from timber floor						
7	Permanent					Formule B	
8	Beam	0,20	kN/m ²				
9	Multiplex	0,10	kN/m ²				
10	Ceiling	0,20	kN/m ²				
11	Finishing (22 mm fermacell)	0,26	kN/m ²				
12	Total_characteristic	0,76	kN/m ²			SOM(B8:B11)	
13							
14	Variable						
15	Imposed loading	1,75	kN/m ²				
16	Non-structural walls	0,5	kN/m ²	ψ _{0,j}	ψ _{2,j}		
17	Total_characteristic	2,25	kN/m ²	0,4	0,3	SOM(B15:B16)	
18							
19	Design value load	ULS		SLS			
20	Eq 6.1	2,37	kN/m ²	1,66	kN/m ²	(B3*B12)+(C3*B17*D17)	
21	Eq 6.2	3,83	kN/m ²	2,71	kN/m ²	(B4*B12) + (C4*B15) + (C4*B16*D17)	
22	Normative design value	3,83	kN/m ²	2,71	kN/m ²	MAX(B20:B21)	
23							
24	Span (transfers load)	4000	mm				
25	Spacing secondary beams	400	mm	SLS ↓			
26	F _{ed}	6,13	kN	4,33	kN	(B24/1000)*(B25/1000)*B22	
27	q _{ed}	15,33	kN/m	10,83	kN/m	B26*(1000/B25)	
28							
29							
30	Total load						
31	Chosen beam HEA220					Formule B	
32	f _{yd}	235	N/mm ²				
33	b _{beam}	220	mm				
34	h _{beam}	210	mm				
35	A	6430	mm ²				
36	W _{y,pl}	5,69E+05	mm ³				
37	I _y	5,41E+07	mm ⁴				
38	ρ	7850	kg/m ³				
39	E _s	210000	N/mm ²				
40	t _f	11	mm				
41	t _w	7	mm				
42	r	18	mm				
43	γ _{M0}	1					
44							
45	Load						
46	q _{self-weight}	0,51	kN/m				
47	q _{self-weight_Ed}	0,61	kN/m	B46*B4			
48	q _{total}	15,94	kN/m	B47+B27			
49							
50	Cross-section classification						
51	flange						
52	c	88,5	mm	(B33/2) - (B41/2)-B42			
53	t	11	mm	B40			
54	c/t	8,05	-	B52/B53			
55	ξ	1	-	WORTEL(235/B32)			
56		Class 1 <	9	9*B55			
57		Class 2 <	10	10*B55			
58	Result	1					
59							

Figure X.150: Structural verification HSB floor structure (Steel beam) (1)_Excel

60	Web			
61	c	152 mm	B34-(2*B40) - (2*B42)	
62	t	7 mm	B41	
63	z^*	21,71 -	B61/B62	
64		Class 1 < 72 -	72*B55	
65		Class 2 < 83 -	83*B55	
66	Result	1		
67	Both class 1, so plastic analysis			
68				
69	Verification ULS_Bending			
70	Loads			
71	Span main beam	6600 mm		
72	M_{ed}	8,68E+07 Nmm	(1/8)*B48*B71^2	
73	$\sigma_{M,Ed}$	1,53E+02 N/mm ²	B72/B36	
74				
75	Material properties and verification			
76	f_{yd}	235,00 N/mm ²	B32	
77	UC	0,65 -	B73/B76	
78				
79	Verification ULS_Shear			
80	Loads			
81	Span main beam	6600 mm	B71	
82	V_{ed}	5,26E+04 N	0,5*B48*B81	
83	$A_{V,z}$	2063 mm ²	MAX(B35 - (2*B33*B40) + ((B41 + (2*B42))*B40); 1,2*(B34-(2*B40))*B41)	
84	$V_{pl,Rd}$	279902 N	B83*(B32/WORTEL(3))/B43	
85	UC	0,19	B82/B84	
86				
87	Verification deflections (SLS)			
88	w	24,65 mm	(5/384)*(D27+B46)*(B71^4) / (B39*B37)	
89	$w_{allowed}$	26,40 mm	B71*0,004	
90	UC	0,93 -	B88/B89	
91				
92	Optimization profile	UC_{bending}	UC_{shear}	UC_w
93	HEA200	0,85	0,21	1,36
94	HEA220	0,65	0,19	0,93
95	HEA240	0,5	0,16	0,66

Figure X.151: Structural verification HSB floor structure (Steel beam) (2)_Excel

X.11.2.4 Steel beams for span (check below façade)

	A	B	C	D	E	F	G
1	Structural calculation main beams (Check below façade)						
2	Safety factors (NEN-EN 1990)		V _G	V _Q			
3	Eq 6.1	1,35	1,5				
4	Eq 6.2	1,2	1,5				
5							
6	Loads from timber floor						
7	Permanent					Formule B	
8	Beam	0,20	kN/m ²				
9	Multiplex	0,10	kN/m ²				
10	Ceiling	0,20	kN/m ²				
11	Finishing (22 mm fermacell)	0,26	kN/m ²				
12	Total_characteristic	0,76	kN/m ²			SOM(B8:B11)	
13							
14	Variable						
15	Imposed loading	1,75	kN/m ²				
16	Non-structural walls	0,5	kN/m ²	ψ _{0,j}	ψ _{2,j}		
17	Total_characteristic	2,25	kN/m ²	0,4	0,3	SOM(B15:B16)	
18							
19	Design value load		ULS	SLS			
20	Eq 6.1	2,37	kN/m ²	1,66	kN/m ²	(B3*B12)+(C3*B17*D17)	
21	Eq 6.2	3,83	kN/m ²	2,71	kN/m ²	(B4*B12) + (C4*B15) + (C4*B16*D17)	
22	Normative design value	3,83	kN/m ²	2,71	kN/m ²	MAX(B20:B21)	
23							
24	Span (that transfers load)	2600	mm	* first beams are closer at each other			
25	Spacing secondary beams	400	mm	SLS ↓			
26	F _{ed}	3,99	kN	2,82	kN	(B24/1000)*(B25/1000)*B22	
27	Q _{ed}	9,97	kN/m	7,04	kN/m	B26*(1000/B25)	
28	Q _{ed + facade}	11,65	kN/m	8,44	kN/m	B27+(0,5*2,8*B4)	
29							
30	Total load						
31	Chosen beam HEA220					Formule B	
32	f _{yd}	235	N/mm ²				
33	b _{beam}	220	mm				
34	h _{beam}	210	mm				
35	A	6434	mm ²				
36	W _{y,pl}	5,69E+05	mm ³				
37	I _y	5,41E+07	mm ⁴				
38	ρ	7850	kg/m ³				
39	E _s	210000	N/mm ²				
40	t _f	11	mm				
41	t _w	7	mm				
42	r	18	mm				
43	γ _{MO}	1					
44							
45	Load						
46	Q _{self-weight}	0,51	kN/m				
47	Q _{self-weight_Ed}	0,61	kN/m	B46*B4			
48	Q _{total}	12,25	kN/m	B47+B28			
49							
50	Cross-section classification						
51	flange						
52	c	88,5	mm	(B33/2) - (B41/2)-B42			
53	t	11	mm	B40			
54	c _r	8,05	-	B52/B53			
55	ε	1	-	WORTEL(235/B32)			
56	Class 1 <	9	-	9*B55			
57	Class 2 <	10	-	10*B55			
58	Result	1					
59							
60	Web						
61	c	152	mm	B34-(2*B40) - (2*B42)			
62	t	7	mm	B41			
63	c _r	21,71	-	B61/B62			
64	Class 1 <	72	-	72*B55			
65	Class 2 <	83	-	83*B55			
66	Result	1					
67	Both class 1, so plastic analysis						

Figure X.152: Structural verification HSB floor structure (Steel beam below façade) (1)_Excel

68				
69	Verification ULS_Bending			
70	Loads			
71	Span main beam	6600	mm	
72	M_{ed}	6,67E+07	Nmm	(1/8)*B48*B71^2
73	$\sigma_{M,Ed}$	1,17E+02	N/mm ²	B72/B36
74				
75	Material properties and verification			
76	f_{yd}	235,00	N/mm ²	B32
77	UC	0,50	-	B73/B76
78				
79	Verification ULS_Shear			
80	Loads			
81	Span main beam	6600	mm	B71
82	V_{ed}	4,04E+04	N	0,5*B48*B81
83	$A_{V,z}$	2067	mm ²	MAX(B35 - (2*B33*B40) + ((B41 + (2*B42))*B40); 1,2*(B34-(2*B40))*B41)
84	$V_{pl,Rd}$	280445	N	B83*(B32/WORTEL(3))/B43
85	UC	0,14	-	B82/B84
86				
87	Verification deflections (SLS)			
88	w	19,45	mm	(5/384)*(D27+B46)*(B71^4) / (B39*B37)
89	$w_{allowed}$	26,40	mm	B71*0,004
90	UC	0,74	-	B88/B89
91				
92	Optimization profile	UC_{bending}	UC_{shear}	UC_w
93	HEA220	0,5	0,14	0,74

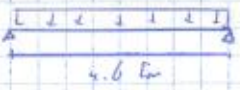
Figure X.153: Structural verification HSB floor structure (Steel beam below facade) (2)_Excel

X.11.2.5 Hand validation

Order ✓
 Blad nr 1/4
 Deel Verificatie HSB Floor
 Datum 17/05/2023

VAN ROSSUM RAADGEVENDE INGENIEURS

Scheme



4.6 m

Beam properties	Multiplex properties
E-modulus = 11000 [N/mm ²]	E-modulus = 9000 [N/mm ²]
ρ = 420 [kg/m ³]	ρ = 550 [kg/m ³]
h _{beam} = 270 [mm]	b = 18 [mm]
b _{beam} = 70 [mm]	Floor width B = 10 [m]
Span = 4.6 [m]	(EI) _h = 9000 · 12 · 1000 · 18 ³ = 4.57 · 10 ⁹ [Nmm ² /m] 8
spacing = 400 [mm]	
I _{beam} = 1/12 · bh ³ = 1/12 · 70 · 270 ³ = 1.15 · 10 ⁸ [mm ⁴]	
I _{multiplex} = 1/12 · bh ³ = 1/12 · 70 · 270 ³ · 8.51 · 10 ² [mm ⁴]	
(EI) _{beam} = 11000 · 1.15 · 10 ⁸ = 1.26 · 10 ¹² [Nmm ²]	
(EI) _{L floor} = 11000 · 1.15 · 10 ⁸ · $\frac{10000}{400}$ = 3.16 · 10 ¹² [Nmm ² /m]	

Mass

beams = (270/1000) (70/1000) 420 $\frac{1000}{400}$ = 14.85 [kg/m³]

multiplex = (18/1000) 550 = 9.90 [kg/m³]

in Achiel. Formcell = $\frac{2.6 \cdot 1000}{400}$ = 6.5 [kg/m³]

55.75 [kg/m³]

Loads (exact obtained in excel)

permanent: org 0.76 [kN/m²]

Variable:

- imposed = 1.75 [kN/m²]
- walls = $\frac{1.40}{2.25}$ [kN/m²]

ψ₀ = 0.4, ψ₂ = 0.5

og 6.10a = (1.35 · 0.76) + (1.5 · 0.4 · 2.75) = 2.38 [kN/m²]

og 6.10b = (1.2 · 0.76) + (1.5 · 1.75) + (1.5 · 0.4 · 0.5) = 3.84 [kN/m²]

↳ fact = 3.84 [kN/m²] = 2.84 · 0.4 = 1.14 [kN/m²]

Verificatie HSB Beam

1 Bending

$$\sigma = \frac{M}{W} = \frac{\beta \gamma L^2}{\delta \cdot 51 \cdot 10^7} = \frac{3 \cdot 1.57 \cdot 4600^2}{\delta \cdot 51 \cdot 10^7} = 4.76 [N/mm^2]$$

End = $\frac{0.7 \cdot 24}{1.3} = 1.29 [N/mm^2]$ → uc = $\frac{4.76}{1.29} = 3.69 [-]$ 5

Figure X.154: Validation HSB floor structure (1)

Order ✓
 Blad nr 2/4
 Deel Verificatie HSB floor
 Datum 17/05/2023.



2. Slab

$$I_{sl} = \frac{3}{2} \frac{V_{sl}}{b \cdot h} = \frac{3}{2} \frac{2 \cdot 4 \cdot L}{b \cdot h} = \frac{3}{2} \frac{2 \cdot 158.4600}{70 \cdot 270} = 0.20 \text{ [m}^4\text{]}$$

$$F_{u,d} = \frac{0.17 \cdot 4}{1.3} = 2.15 \text{ [N/mm}^2\text{]} \rightarrow UC = \frac{0.20}{2.15} = 0.09 \text{ [-]} \quad 5 \rightarrow 0.09 \text{ (slab)}$$

Verificatie SLS beam

1. Deflection beam (Rd=0.6, $\psi_2=0.3$) 0.76 m

$$U_{fin, q} = \left(\frac{5}{384} \frac{q L^4}{EI} \right) (1 + \kappa_{def}) = \left[\frac{5}{384} \frac{158.4600 \cdot 0.76^4}{(11000 \cdot 15 \cdot 10^8)} \right] (1 + 0.6) = 2.24 \text{ [mm]}$$

$$U_{fin, q_1} = \left(\frac{5}{384} \frac{q L^4}{EI} \right) (1 + \kappa_{def}) = \left[\frac{5}{384} \frac{1.75 \cdot 0.4 \cdot 0.76^4}{(11000 \cdot 15 \cdot 10^8)} \right] (1 + 0.3 \cdot 0.6) = 3.81 \text{ [mm]}$$

$$U_{fin, q_2} = \left(\frac{5}{384} \frac{q L^4}{EI} \right) (1 + \kappa_{def}) = \left[\frac{5}{384} \frac{0.5 \cdot 0.4 \cdot 0.76^4}{(11000 \cdot 15 \cdot 10^8)} \right] (0.4 + 0.3 \cdot 0.6) = 0.53 \text{ [mm]}$$

$$U_{fin} = 2.24 + 3.81 + 0.53 = 6.58 \text{ [mm]}, 5 \leftrightarrow 6.58 \text{ [mm]}$$

$$UC = \frac{U_{fin}}{0.004 L} = \frac{6.58}{0.004 \cdot 0.76} = 0.36 \text{ [-]} \quad 8 \leftrightarrow 0.36 \text{ (slab value)}$$

2. Verificatie vibrations

=> Fundamental frequency

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{EI_1}{m}} = \frac{\pi}{2 \cdot 0.76^2} \sqrt{\frac{316 \cdot 10^6 \cdot 10^{-6}}{55.75}} = 17.67 \text{ Hz} < 8 \text{ Hz} \quad 8$$

↑ slab = 17.67 Hz

=> Point deflection

$$X_r = 0.37 + 0.8 \frac{\text{spacing}}{[m]} \frac{E_{comp}}{E_{steel}} = 0.37 + 0.8 \frac{0.4}{1} = 0.69 \text{ [mm]} \quad 8$$

$$\omega = \frac{X_r F L^3}{48 EI} = \frac{0.6 \cdot 3000 \cdot 0.76^3}{48 \cdot 1.26 \cdot 10^{12}} = 250 \text{ [mm]} = \frac{250}{3} = 0.97 \text{ [mm/10]} \quad 8$$

$$UC = \frac{0.97}{1} = 0.97 \text{ [-]} \quad 8 \leftrightarrow 0.97 \text{ (slab value)}$$

Figure X.155: Validation HSB floor structure (2)

Order ✓
 Blad nr 3/4
 Deel Verificatie HSB Floor
 Datum 17/05/2023

VAN ROSSUM RAADGEVENDE INGENIEURS

⇒ Impulse velocity response

$$M_{40} = \left[\left(\left(\frac{40}{F_1} \right)^2 - 1 \right) \left(\frac{13}{L} \right)^4 \left(\frac{E I_b}{\rho A} \right) \right]^{0.25} = \left[\left(\left(\frac{40}{17.67} \right)^2 - 1 \right) \left(\frac{10}{4.6} \right)^4 \left(\frac{2.16 \cdot 10^6}{4.37 \cdot 10^3} \right) \right]^{0.25} = 1.606 [-]$$

$$v = \frac{4 (0.4 + 0.6 M_{40})}{m_{Bl} \cdot 200} = \frac{4 (0.4 + 0.6 \cdot 1.606)}{(55.75 + 10 \cdot 4.6) + 200} = 0.015 \text{ s}$$

$$v \leq b^{1.3-1} = \frac{17.67 \cdot 0.01}{120} = 0.015 \text{ s}$$

$$UC = \frac{0.015}{0.011} = 0.70 \text{ s} \text{ } \checkmark \Leftrightarrow 0.75 \text{ (steel)}$$

Verification plywood

1. Bending

$$q_{ed} = 3.24 \cdot 1 \text{ km} = 3.24 \text{ kNm/m}$$

$$Max \sigma_m = \frac{M}{W} = \frac{\frac{1}{8} q L^2}{\frac{1}{6} b h^2} = \frac{\frac{1}{8} \cdot 3.24 \cdot 4000^2}{\frac{1}{6} \cdot 1000 \cdot 10^3} = 1.42 \text{ [N/mm}^2]$$

$$f_{md} = \frac{0.7 \cdot 10.5}{1.2} = 10.5 \text{ [N/mm}^2] \rightarrow UC = \frac{1.42}{10.5} = 0.14 [-] \checkmark$$

2. deflection

$$u_{fin, G} = \left(\frac{5}{384} q L^4 / EI \right) (1 + \nu \delta^2) = \left[\frac{5}{384} \cdot 0.76 \cdot 4000^4 / (4.37 \cdot 10^4) \right] (1 + 0.3) = 0.06 \text{ [m]}$$

$$u_{fin, Q_1} = \left(\frac{5}{384} q L^4 / EI \right) (1 + \nu \delta^2) = \left[\frac{5}{384} \cdot 1.75 \cdot 4000^4 / (4.37 \cdot 10^4) \right] (1 + 0.3) = 0.17$$

$$u_{fin, Q_2} = \left(\frac{5}{384} q L^4 / EI \right) (1 + \nu \delta^2) = \left[\frac{5}{384} \cdot 0.5 \cdot 4000^4 / (4.37 \cdot 10^4) \right] (1 + 0.3) = 0.02 \text{ [m]}$$

$$u_{fin} = \frac{0.10}{0.014} = 0.17 + 0.02 = 0.19 \text{ [m]} \checkmark$$

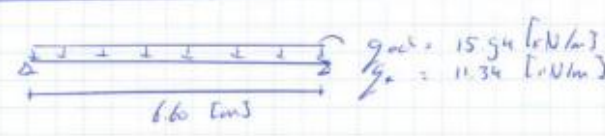
$$UC = \frac{u_{fin}}{0.014} = \frac{0.19}{0.014} = 0.18 [-] \checkmark \rightarrow 0.18 [-] \checkmark$$

Figure X.156: Validation HSB floor structure (3)

Order ✓
 Blad nr 4/4
 Deel Verificatie HSB Floor
 Datum 17/05/2023

VAN ROSSUM RAADGEVENDE
INGENIEURS

Verificatie steel beam



$g_{d1} = 15.54 \text{ [kN/m]}$
 $q_k = 11.34 \text{ [kN/m]}$
 6.6 [m]

Bending

$$\sigma_M = \frac{M}{W} = \frac{\frac{1}{8} q L^2}{W_{pl}} = \frac{\frac{1}{8} \cdot 15.54 \cdot (6.6 \cdot 1000)^2}{5.6 \cdot 10^7} = 152.5 \text{ [N/mm}^2\text{]} < 235 \text{ , } f_{yEd} \uparrow$$

$$UC = \frac{153}{235} = 0.65 \text{ [-]} \quad \text{OK} \rightarrow 0.65 \text{ [-]} \text{ (steel)}$$

Deflection

$$w = \frac{5}{384} \frac{q L^4}{EI} = \frac{5}{384} \cdot \frac{11.34 \cdot 6600^4}{210000 \cdot 54.10 \cdot 10^6} = 24.66 \text{ [mm]}$$

$$UC = \frac{w}{0.0046} = \frac{24.66}{0.004666} = 0.53 \text{ [-]} \quad \text{OK} \rightarrow 0.53 \text{ [-]} \text{ (steel)}$$

Excels for floors are validated

Figure X.157: Validation HSB floor structure (4)

X.11.3 HSB wall

X.11.3.1 Single wall (combination 1)

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (1-storey; Single wall; Combination 1)													
2	Safety factors (NEN-EN 1990)			Formule B										
3	Eq 6.1	γ_G	γ_Q	1,35	1,5									
4	Eq 6.2			1,2	1,5									
6	Chosen column													
7	Timber column													
8	E-modulus (C24)		11000	N/mm ²										
9	E _{0,05}		7400	N/mm ²										
10	Density		420	kg/m ³										
11	Width (b)		58	mm										
12	Length (h)		89	mm										
13	A		3382	mm ²										
14	Spacing columns		0,6	m										
15	CFL		6,7	m										
16	m _{column}		2,37	kg/m ²										
17	Height floor		2,80	m										
18	Number of stories		1	-										
19	I _{column}		2232401,833	mm ⁴										
20	W _{column}		50166,33333	mm ³										
21	Half span gallery beam		0,6	m										
22	Number of columns at start/end wall		2	-										
23	Opening factor		0,7	-										
24	Plywood													
25	Thickness		18	mm										
26	Density		550	kg/m ³										
27	m _{plywood}		9,9	kg/m ²										
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														
43														
44														
45	Load (Imposed load is leading action)													
46	Roof (Full loaded)			kN/m ²	kN _{normal}	kN _{facade}								
47	Permanent load		0,7	2,01	2,11									
48	Variable load													
49	Imposed load		1	2,87	3,02									
50	eq 6.1			2,71	2,85									
51	eq 6.2			6,72	7,06									
52														
53	Self-weight wall			kN/m ²	kN _{normal}	kN _{facade}								
54	Permanent load													
55	Columns		0,02	0,04	0,04									
56	Gallery beam				0,07									
57	Gypsum board (Type F)		0,14	0,47	0,24									
58	Plywood (18 mm)		0,10	0,33	0,17									
59					0,84									
60	eq 6.1			1,14	0,69									
61	eq 6.2			1,01	0,61									
62														
63	Pointload frond facade			kN/m ²	kN _{facade}									
64	Permanent load		0,5	4,69										
65	eq 6.1			6,33										
66	eq 6.2			5,63										
67														
68														
69														
70														
71	Total load			kN (incl. facade)*	kN (excl. facade)									
72	eq 6.1			4,94	3,85									
73	eq 6.2			6,65	7,73									
74	F _{red}			6,65	7,73									
75	* Value is divided by factor 2 at facade. At that location 2 columns must be placed													
76														
77														
78	Verification strength													
79	Buckling analysis > Supported at weak axis													
80	Reduction factor													
81	I _y		2,23E+06	mm ³										
82	I _y		25,69	mm										
83	L _{buc}		2800	mm										
84	λ _y		108,98	-										
85	f _{e,0,k}		21	N/mm ²										
86	λ _{rel}		1,85	-										
87	k _φ		2,36	-										
88	k _c		0,26	-										
89														
90	Material properties													
91	Climate class		I	-										
92	Load duration		long-term	-										
93	k _{mod}		0,7	-										
94	f _{e,0,k}		21	N/mm ²										
95	Y _M		1,3	-										
96	f _{e,0,d}		11,31	N/mm ²										
97														

Figure X.158: Structural verification HSB wall (1-storey; Single wall; combination 1) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M
98	Forces and unity check (Incl. facade)												
99		$\sigma_{N, facade}$	1,97 N/mm ²	B74*1000/B13									
100		UC_{facade}	0,67 -	B99/(B96*B88)									
101	Forces and unity check (Excl. facade)												
102		σ_N	2,29 N/mm ²	C74*1000/B13									
103		UC	0,78 -	B102/(B88*B96)									
104													
105													
106	Compression perpendicular to the grain (Beam below column)												
107		$k_{c,90}$	1,25										
108		x_i	30,00	MIN(30; B14*1000/2; B11)									
109		l_{eff}	98,00 mm	B11+(2*B108)									
110		A_{eff}	8722,00 mm ²	B12*B109									
111		$\sigma_{N, Normal}$	0,89 N/mm ²	C74*1000/B110									
112		$f_{c,90,d}$	2,50 N/mm ²										
113		$f_{c,90,d}$	1,35 N/mm ²	B112*B93/B95									
114		$UC_{normal//}$	0,53 -	B111/(B107*B113)									
115													
116													
117	Results 1-storey flat (Single-wall)												
118	CFL	Column	UC_{facade} *	UC_{normal}	$UC_{normal} //$								
119	6.70	SLS 38 x 89 - sp. 600 mm	0.67 (2)	0.78	0.53								
120	8.49	SLS 38 x 89 - sp. 600 mm	0.84 (2)	0.95	0.65								
121	10.28	SLS 38 x 120 - sp. 600 mm	0.44 (2)	0.50	0.57								
122	* placement of 2 columns at begin and end of wall												

Figure X.159: Structural verification HSB wall (1-storey; Single wall; combination 1) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (2-storey; Single wall; Combination 1)													
2	Safety factors (NEN-EN 1990)			Formule B										
3	Eq 6.1	γ_G	1,35	γ_Q	1,5									
4	Eq 6.2		1,2		1,5									
5														
6	Chosen column													
7	Timber column													
8		E-modulus (C24)	11000 N/mm ²											
9		$E_{0,05}$	7400 N/mm ²											
10		Density	420 kg/m ³											
11		Width (b)	38 mm											
12		Length (h)	140 mm											
13		A	5320 mm ²											
14		Spacing columns	0,6 m											
15		CFL	6,7 m											
16		m_{column}	3,72 kg/m ²											
17		Height floor	2,80 m											
18		Number of stories	2 -											
19		I_{column}	8689333,333 mm ⁴											
20		W_{column}	124133,3333 mm ³											
21		Half span gallery beam	0,6 m											
22		Number of columns at start/end wall	1 -											
23		Opening factor	0,7 -											
24	Plywood													
25		Thickness	18 mm											
26		Density	550 kg/m ³											
27		$m_{plywood}$	9,9 kg/m ²											
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														
43														
44														
45	Load (Imposed load is leading action)													
46	Roof (Full loaded)			Formule C										
47	Permanent load	kN/m ²	0,7	kN _{normal}	2,01	kN _{facade}	2,11	ψ_0	B47*(B15/2) * B14 * (1/B23)	B47*(B15/2) * (B21+(B14/2))				
48	Variable load							0						
49	Imposed load		1		2,87		3,02		B49*(B15/2)*B14*(1/B23)	B49 * (B15/2) * ((B14/2)+B21)				
50	eq 6.1				2,71		2,85		(B3*C47) + (C3*E48*C49)	(B3*D47) + (C3*E48*D49)				
51	eq 6.2				6,72		7,06		(B4*C47) + (C4*C49)	(B4*D47) + (C4*D49)				
52														
53	2nd floor (Full loaded)			Formule D										
54	Permanent load	kN/m ²	1	kN _{normal}	2,87	kN _{facade}	3,02	ψ_0	B54*(B15/2) * B14 * (1/B23)	B54*(B15/2) * ((B14/2)+B21)				
55	Variable							0,4						
56	Imposed load		1,75		5,03		5,28		B56*(B15/2) * B14*(1/B23)	B56*(B15/2)*((B14/2)+B21)				
57	Non-structural walls		0,5		1,44		1,51		B57*(B15/2) * B14 * (1/B23)	B57*(B15/2)*((B14/2)+B21)				
58	eq 6.1				7,75		8,14		(B3*C54) + (C3*E55*(C56+C57))	(B3*D54)+(C3*E55*(D56+D57))				
59	eq 6.2				11,84		12,44		(B4*C54) + (C4*C56) + (C4*E55*C57)	(B4*D54) + (C4*D56) + (C4*E55*D57)				
60														
61	Self-weight wall			Formule E										
62	Permanent load	kN/m ²		kN _{normal}		kN _{facade}								
63	Columns		0,04		0,13		0,06		B63*B14*B18*B17	B63*(B14/2)*(B17*B18)*B22				
64	Gypsum board (Type F)		0,14		0,94		0,47		B65*B14*B18*B17*2	B65*(B14/2)*B17*B18*2				
65	Plywood (18 mm)		0,10		0,67		0,33		B66*B14*B18*B17*2	B66*(B14/2)*B17*B18*2				
66					1,73		0,94		SOM(C63:C66)	SOM(D63:D66)				
67														
68	eq 6.1				2,34		1,26		B3*C67	B3*D67				
69	eq 6.2				2,08		1,12		B4*C67	B4*D67				
70														

Figure X.160: Structural verification HSB wall (2-storey; Single wall; combination 1) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
71	Pointload frond facade	kN/m ²		kN _{facade}										
72	Permanent load		0,5	9,38		B72*(B15/2)*B17*B18								
73	eq 6.1			12,66		B3*C72								
74	eq 6.2			11,26		B4*C72								
75														
76														
77														
78														
79	Total load	kN (incl. facade)*		kN (excl. facade)		Formule B					Formule C			
80	eq 6.1		24,92	12,80		(D50+D58+D68+C73)/B22					C50+C58+C68			
81	eq 6.2		31,87	20,64		(D51+D59+D69+C74)/B22					C51+C59+C69			
82	F _{tot}		31,87	20,64		MAX(B80:B81)					MAX(C80:C81)			
83	* Value is divided by factor 1 or 2 at facade. At that location, 1 or 2 columns must be placed													
84														
85														
86	Verification strength													
87	Buckling analysis > Supported at weak axis													
88	Reduction factor													
89	I _y		8,69E+06 mm ³			B19								
90	I _y		40,41 mm			WORTEL(B89/B13)								
91	l _{buck}		2800 mm			B17*1000								
92	λ _y		69,28 -			B91/B90								
93	f _{c,0,k}		21 N/mm ²											
94	λ _{rel,y}		1,17 -			(B92/PI()) * WORTEL(B93/B9)								
95	k _y		1,28 -			0,5 * (1 + (0,2 * (B94-0,3)) + B94^2)								
96	k _c		0,56 -			1/(B95+WORTEL(B95^2 - B94^2))								
97														
98	Material properties													
99	Climate class	I												
100	Load duration	long-term												
101	k _{mod}		0,7 -											
102	f _{c,0,k}		21 N/mm ²			B93								
103	γ _M		1,3 -											
104	f _{c,0,d}		11,31 N/mm ²			B102*B101/B103								
105														
106	Forces and unity check (Incl. facade)													
107	σ _{N,normal}		5,99 N/mm ²			B82*1000/B13								
108	UC _{facade}		0,94 -			B107/(B104*B96)								
109	Forces and unity check (Excl. facade)													
110	σ _N		3,88 N/mm ²			C82*1000/B13								
111	UC		0,61 -			B110/(B96*B104)								
112														
113														
114	Compression perpendicular to the grain (Beam below column)													
115	k _{c,90}		1,25											
116	x _i		30,00			MIN(30; B14*1000/2; B11)								
117	l _{eff}		98,00 mm			B11+(B116^2)								
118	A _{eff}		13720,00 mm ²			B12*B117								
119	σ _{N,Normal}		1,50 N/mm ²			C82*1000/B118								
120	f _{c,90,k}		2,50 N/mm ²											
121	f _{c,90,d}		1,35 N/mm ²			B120*B101/B103								
122	UC _{normal//}		0,89 -			B119/(B121*B115)								
123														
124														
125	Results 2-storey flat (Single wall)													
126	CFL		Column	UC _{facade} *	UC _{normal}	UC _{normal} //								
127	6.70		SLS 38 x 140 - sp. 600 mm	0.94 (1)	0.61	0.89								
128	8.49		SLS 38 x 140 - sp. 400 mm	0.55 (2)	0.51	0.74								
129	10.28		SLS 38 x 140 - sp. 400 mm	0.66 (2)	0.60	0.88								
130	* placement of 1 or 2 columns at begin and end of wall													
131														

Figure X.161: Structural verification HSB wall (2-storey; Single wall; combination 1) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (3-storey; Single wall; Combination 1)													
2	Safety factors (NEN-EN 1990)													
3	Eq 6.1	V ₀	1,35	1,5		Formule B								
4	Eq 6.2	V ₀	1,2	1,5										
5	Chooosen column													
6	Timber column													
7	E-modulus (C24)		11000 N/mm ²											
8	E _{0,05}		7400 N/mm ²											
9	Density		420 kg/m ³											
10	Width (b)		38 mm											
11	Length (h)		140 mm											
12	A		5320 mm ²			B11*B12								
13	Spacing columns		0,4 m											
14	CFL		6,7 m											
15	m _{column}		5,59 kg/m ²											
16	Height floor		2,80 m			(B11/1000) * (B12/1000) * B10 * (1/B14)								
17	Number of stories		3 -											
18	I _{column}		8,69E+06 mm ⁴			(1/12)*B11*B12^3								
19	W _{column}		124133,3333 mm ³			(1/6)*B11*B12^2								
20	Half span gallery beam		0,6 m											
21	Number of columns at start/end wall		2 -											
22	Opening factor		0,7 -											
23														
24	Plywood													
25	Thickness		18 mm											
26	Density		550 kg/m ³											
27	m _{plywood}		9,9 kg/m ²			(B25/1000)*B26								
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														

Figure X.162: Structural verification HSB wall (3-storey; Single wall; combination 1) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
43														
44														
45	Load (Imposed load is leading action)						Formule C				Formule D			
46	Roof	kN/m ²	kN _{normal}	kN _{facade}										
47	Permanent load	0,7	1,34	1,88		ψ_0	B47*(B15/2) * B14 * (1/B23)				B47*(B15/2) * ((B21+(B14/2))			
48	Variable load					0								
49	Imposed load	1	1,91	2,68			B49*(B15/2)*B14*(1/B23)				B49 * (B15/2) * ((B14/2)+B21)			
50	eq 6.1		1,81	2,53			(B3*C47) + (C3*E48*C49)				(B3*D47) + (C3*E48*D49)			
51	eq 6.2		1,61	2,25			(B4*C47) + (C4*E48*C49)				(B4*D47) + (C4*E48*D49)			
52														
53	3rd floor (Full loaded)	kN/m ²	kN _{normal}	kN _{facade}										
54	Permanent load	1	1,91	2,68		ψ_0	B54*(B15/2) * B14 * (1/B23)				B54*(B15/2) * ((B14/2)+B21)			
55	Variable					0,4								
56	Imposed load	1,75	3,35	4,69			B56*(B15/2) * B14*(1/B23)				B56 * (B15/2) * ((B14/2) + B21)			
57	Non-structural walls	0,5	0,96	1,34			B57*(B15/2) * B14*(1/B23)				B57*(B15/2) * ((B14/2)+B21)			
58	eq 6.1		5,17	7,24			(B3*C54) + (C3*E55*(C56+C57))				(B3*D54)+(C3*E55*(D56+D57))			
59	eq 6.2		7,90	11,06			(B4*C54) + (C4*E55*(C56+C57))				(B4*D54) + (C4*E55*(D56+D57))			
60														
61	2nd floor (Full loaded)	kN/m ²	kN _{normal}	kN _{facade}										
62	Permanent load	1	1,91	2,68		ψ_0	B62*(B15/2) * B14 * (1/B23)				B62*(B15/2) * ((B14/2)+B21)			
63	Variable					0,4								
64	Imposed load	1,75	3,35	4,69			B64*(B15/2) * B14*(1/B23)				B64*(B15/2)*((B14/2)+B21)			
65	Non-structural walls	0,5	0,96	1,34			B65*(B15/2) * B14 * (1/B23)				B65*(B15/2)*((B14/2)+B21)			
66	eq 6.1		5,17	7,24			(B3*C62) + (C3*E63*(C64+C65))				(B3*D62)+(C3*E63*(D64+D65))			
67	eq 6.2		7,90	11,06			(B4*C62) + (C4*E63*(C64+C65))				(B4*D62) + (C4*E63*(D64+D65))			
68														
69	Self-weight wall	kN/m ²	kN _{normal}	kN _{facade}										
70	Permanent load													
71	Columns		0,06	0,19	0,19		B71*B14*B18*B17				B71*(B14/2)*(B17*B18)*B22			
72	Gallery beam				0,07									
73	Gypsum board (Type F)	0,14	0,94	0,47			B73*B14*B18*B17*2				B73*(B14/2)*B17*B18*2			
74	Plywood (18 mm)	0,10	0,67	0,33			B74*B14*B18*B17*2				B74*(B14/2)*B17*B18*2			
75			1,79	1,06			SOM(C71:C74)				SOM(D71:D74)			
76	eq 6.1		2,42	1,43			B3*C75				B3*D75			
77	eq 6.2		2,15	1,27			B4*C75				B4*D75			
78														
79	Pointload frond facade	kN/m ²	kN _{facade}											
80	Permanent load	0,5	14,07				B80*(B15/2)*B17*B18							
81	eq 6.1		18,99				B3*C80							
82	eq 6.2		16,88				B4*C80							
83														
84														
85														
86														
87	Total load	kN (incl. facade)*	kN (excl. facade)				Formule B				Formule C			
88	eq 6.1	18,72	14,57				(D50+D58+D66+D76+C81)/B22				C50+C58+C66+C76			
89	eq 6.2	21,26	19,55				(D51+D59+D67+D77+C82)/B22				C51+C59+C67+C77			
90	F _{red}	21,26	19,55				MAX(B88:B89)				MAX(C88:C89)			
91	* Value is divided by factor 2 at facade. At that location, 2 columns must be placed													
92														
93														
94	Verification strength													
95	Buckling analysis > Supported at weak axis													
96	Reduction factor													
97			8,69E+06 mm ³				B19							
98	I _y		40,41 mm				WORTEL(B97/B13)							
99	I _z		2800 mm				B17*1000							
100	I _{bucl}		69,28 -				B99/B98							
101	λ _y													
102	f _{c,0,k}		21 N/mm ²				(B100/PI()) * WORTEL(B101/B9)							
103	λ _{rel,y}		1,17 -				0,5 * (1 + (0,2 * (B102-0,3)) + B102^2)							
104	k _y		1,28 -				1/(B103+WORTEL(B103^2 - B102^2))							
105	k _z		0,56 -											
106	Material properties													
107	Climate class	I												
108	Load duration	long-term												
109	k _{mod}		0,7 -											
110	f _{c,0,k}		21 N/mm ²				B101							
111	γ _M		1,3 -											
112	f _{c,0,d}		11,31 N/mm ²				B110*B109/B111							
113														
114	Forces and unity check (Incl. facade)													
115	σ _{N, facade}		4,00 N/mm ²				B90*1000/B13							
116	UC _{facade}		0,63 -				B115/(B112*B104)							
117	Forces and unity check (Excl. facade)													
118	σ _N		3,68 N/mm ²				C90*1000/B13							
119	UC		0,58 -				B118/(B104*B112)							
120														
121														
122	Compression perpendicular to the grain (Beam below column)													
123	k _{c,90}		1,25											
124	x _i		30,00				MIN(30; B14*1000/2; B11)							
125	l _{eff}		98,00 mm				B11+(B124*2)							
126	A _{eff}		13720,00 mm ²				B125*B12							
127	σ _{N, Normal}		1,43 N/mm ²				C90*1000/B126							
128	f _{c,90,k}		2,50 N/mm ²											
129	f _{c,90,d}		1,35 N/mm ²				B128*B109/B111							
130	UC _{normal//}		0,85 -				B127/(B129*B123)							
131														
132														
133	Results 3-storey flat (Single wall)													
134	CFL		Column	UC _{facade} *	UC _{normal}	UC _{normal} //								
135	6.70		SLS 38 x 140 - sp. 400 mm	0.63 (2)	0.58	0.85								
136	8.49		SLS 38 x 170 - sp. 400 mm	0.51 (2)	0.47	0.87								
137	10.28		SLS 38 x 184 - sp. 400 mm	0.53 (2)	0.48	0.95								
138	* placement of 2 columns at begin and end of wall													

Figure X.163: Structural verification HSB wall (3-storey; Single wall; combination 1) (2)_Excel

X.11.3.2 Single wall (combination 2)

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (3-storey; Single wall Combination 2)													
2	Safety factors (NEN-EN 1990)		V _g	V _o	Formule B									
3	Eq 6.1		1,35	1,5										
4	Eq 6.2		1,2	1,5										
5	Chosen column													
6	Timber column													
8	E-modulus (C24)		11000	N/mm ²										
9	E _{0,05}		7400	N/mm ²										
10	Density		420	kg/m ³										
11	Width (b)		38	mm										
12	Length (h)		120	mm										
13	A		4560	mm ²										
14	Spacing columns		0,6	m										
15	CFL		6,7	m										
16	m _{column}		3,19	kg/m ²										
17	Height floor		2,80	m										
18	Number of stories		1	-										
19	I _{column}		5472000	mm ⁴										
20	W _{column}		91200	mm ³										
21	Half span gallery beam		0,6	m										
22	Number of columns at start/end wall		1	-										
23	Opening factor		0,7	-										
24	Plywood													
25	Thickness		18	mm										
26	Density		550	kg/m ³										
27	m _{plywood}		9,9	kg/m ²										
28	Structural scheme													
29														
44	Load (wind load is leading action)													
45	Roof													
46	Permanent load	kN/m ²	0,7	kN _{normal}	1,01	kN _{facade}	1,06							
47	Variable load													
48	Imposed load		1	1,44	1,51									
49	eq 6.2			1,21	1,27									
50	Self-weight wall													
51	Permanent load	kN/m ²		kN _{normal}		kN _{facade}								
52	Columns		0,03	0,05	0,03									
53	Gallery beam				0,07									
54	Facade		0,50	0,84	0,42									
55	Gypsum board (Type F) (single side)		0,14	0,24	0,12									
56	Plywood (18 mm)		0,10	0,33	0,17									
57	eq 6.2			1,46	0,80									
58	eq 6.2			1,75	0,96									
59	Pointload frond facade													
60	Permanent load	kN/m ²	0,5	kN _{facade}	2,35									
61	eq 6.2			2,81										
62	Total load													
63	eq 6.2	kN (incl. facade)*		kN (excl. facade)	5,04	2,96								
64	F _{tot}			5,04	2,96									
72	Wind Load													
73	q _w (19.6m) (Wind area II; Urban)		0,84	kN/m ²										
74	C _s C _d		1	-										
75	C _{f,D}		0,8	-										
76	C _{f,E}		0,7	-										
77	C _{f,tot}		0,8	-										
78	Q _{wind,SLS,Normal}		0,40	kN/m										
79	Q _{wind,SLS,Facade}		0,20	kN/m										
80	Formule B													
81	Nmm _{normal}			Nmm _{facade}										
82	M _{red}		592704	296352										
83	Formule B													
84	Formule C													
85	Verification strength													
86	Buckling analysis > Supported at weak axis													
87	Reduction factor													
88	I _y		5,47E+06	mm ³										
89	I _y		34,64	mm										
90	L _{buck}		2800	mm										
91	λ _y		80,83	-										
92	f _{c,Dk}		21	N/mm2										
93	λ _{rel}		1,37	-										
94	k _y		1,55	-										
95	k _c		0,44	-										
96	Formule B													
97	Formule C													

Figure X.164: Structural verification HSB wall (1-storey; Single wall; combination 2) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
97	Material properties													
98	Climate class	I	-											
99	Load duration	long-term	-											
100	k_{mod}	0,7	-											
101	$f_{c,0,k}$	21	N/mm ²	B92										
102	f_m	24	N/mm ²											
103	γ_{M1}	1,3	-											
104	k_{01}	1,05	-	ALS(B12<150; MIN((150/B12)^0,2; 1,3); 1)										
105	$f_{c,0,d}$	11,31	N/mm ²	B101*B100/B103										
106	$f_{m,d}$	13,51	N/mm ²	B104*B100*B102/B103										
107														
108	Forces and unity check (Incl. facade)													
109	$\sigma_{N, facade}$	1,11	N/mm ²	B68*1000/B13										
110	$\sigma_{M, facade}$	3,25	N/mm ²	C82/B20										
111	UC _{facade}	0,46	-	(B109/(B95*B105)) + (B110/B106)										
112	Forces and unity check (Excl. facade)													
113	σ_H	0,65	N/mm ²	C68*1000/B13										
114	σ_{H1}	6,50	N/mm ²	B82/B20										
115	UC	0,61	-	(B113/(B105*B95)) + (B114/B106)										
116														
117														
118	Compression perpendicular to the grain (Beam below column)													
119	$k_{c,90}$	1,25	-											
120	x_i	30,00	-	MIN(30; B14*1000/2; B11)										
121	l_{eff}	98,00	mm	B11+(B120*2)										
122	A_{eff}	11760,00	mm ²	B121*B12										
123	$\sigma_{N, Normal}$	0,25	N/mm ²	C68*1000/B122										
124	$f_{c,90,k}$	2,50	N/mm ²											
125	$f_{c,90,d}$	1,35	N/mm ²	B124*B100/B103										
126	UC _{normal//}	0,15	-	B123/(B125*B119)										
127														
128														
129	Results 1-storey flat (Single wall;wind)													
130	CFL		Column	UC _{facade}	UC _{normal}	UC _{normal //}								
131	6.70		SLS 38 x 120 - sp. 600 mm	0.46 (1)	0.61	0.15								
132	8.49		SLS 38 x 120 - sp. 600 mm	0.51 (1)	0.62	0.17								
133	10.28		SLS 38 x 120 - sp. 600 mm	0.56 (1)	0.64	0.18								

Figure X.165: Structural verification HSB wall (1-storey; Single wall; combination 2) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (2-storey; Single wall; Combination 2)													
2	Safety factors (NEN-EN 1990)													
3	Eq 6.1	γ_G	1,35	1,5	Formule B									
4	Eq 6.2	γ_Q	1,2	1,5										
5	Chosen column													
6	Timber column													
7	E-modulus (C24)	11000	N/mm ²											
8	$E_{0,05}$	7400	N/mm ²											
9	Density	420	kg/m ³											
10	Width (b)	38	mm											
11	Length (h)	140	mm											
12	A	5320	mm ²	B11*B12										
13	Spacing columns	0,6	m											
14	CFL	6,7	m											
15	m_{column}	3,72	kg/m ²	(B11/1000) * (B12/1000) * B10 * (1/B14)										
16	Height floor	2,80	m											
17	Number of stories	2	-											
18	V_{column}	8689333,333	mm ⁴	(1/12)*B11*B12*3										
19	W_{column}	124133,3333	mm ³	(1/6)*B11*B12*2										
20	Half span gallery beam	0,6	m											
21	Number of columns at start/end wall	1	-											
22	Opening factor	0,7	-											
23														
24	Plywood													
25	Thickness	18	mm											
26	Density	550	kg/m ³											
27	$m_{plywood}$	9,9	kg/m ²	(B25/1000)*B26										
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														
43														
44														
45	Load (wind load is leading action)													
46	Roof	kN/m ²	kN _{normal}	kN _{facade}										
47	Permanent load	0,7	1,01	1,06	Formule C	Formule D								
48	Variable load				ψ_0 B47*(B15/4) * B14 * (1/B25)	B47*(B15/4) * (B21+(B14/2))								
49	Imposed load	1	1,44	1,51	B49*(B15/4)*B14*(1/B23)	B49 * (B15/4) * ((B14/2)+B21)								
50	eq 6.2		1,21	1,27	(B4*C47) + (C4*E48*C49)	(B4*D47) + (C4*E48*D49)								
51														
52	2nd floor	kN/m ²	kN _{normal}	kN _{facade}										
53	Permanent load	1	1,44	1,51	ψ_0 B53*(B15/4) * B14 * (1/B25)	B53*(B15/4) * ((B14/2)+B21)								
54	Variable													
55	Imposed load	1,75	2,51	2,64	B55*(B15/4) * B14*(1/B23)	B55*(B15/4)*((B14/2)+B21)								
56	Non-structural walls	0,5	0,72	0,75	B56*(B15/4) * B14 * (1/B23)	B56*(B15/4)*((B14/2)+B21)								
57	eq 6.2		3,66	3,84	(B4*C53) + (C4*E54*(C55+C56))	(B4*D53) + (C4*E54*(D55+D56))								
58														
59	Self-weight wall													
60	Permanent load													
61	Columns	0,04	0,13	0,06	B61*B14*B18*B17	B61*(B14/2)*(B17*B18)*B22								
62	Gallery beam			0,07										
63	Facade	0,50	1,68	0,84	B63 * B14 *B17*B18									
64	Gypsum board (Type F) (single side)	0,14	0,47	0,24	B64*B14*B18*B17	B64*(B14/2)*B17*B18								
65	Plywood (18 mm)	0,10	0,67	0,33	B65*B14*B18*B17*2	B65*(B14/2)*B17*B18*2								
66			2,94	1,54	SOM(D61.C65)	SOM(D61.D65)								
67	eq 6.2		3,53	1,85	B4*C66	B4*D66								

Figure X.166: Structural verification HSB wall (2-storey; Single wall; combination 2) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
69	Pointload frond facade		kN/m ²		kN _{facade}									
70	Permanent load		0,5		4,69									
71		eq 6.2			5,63									
72														
73	Total load		kN (incl. facade)*		kN (excl. facade)									
74		eq 6.2			12,59									
75		F _{tot}			12,59									
76		* Value is divided by factor 1 or 2 at facade. At that location, 1 or 2 columns must be placed												
77														
78														
79	Wind Load													
80		q _p (19.6m) (Wind area II; Urban)			0,9									
81		C _s C _d			1									
82		C _{f,D}			0,8									
83		C _{f,E}			0,7									
84		C _{f,tot}			0,8									
85	q _{wind,SLS_Normal}				0,43									
86	q _{wind,SLS_facade}				0,22									
87														
88		N _{mm,normal}			N _{mm,facade}									
89		M _{red}		635040	817520									
90														
91														
92	Verification strength													
93	Buckling analysis > Supported at weak axis													
94	Reduction factor													
95		I _y		8,69E+06	mm ⁴									
96		I _z		40,41	mm									
97		l _{buck}		2800	mm									
98		λ _y		69,28	-									
99		f _{c,0,k}		21	N/mm ²									
100		λ _{rel,y}		1,17	-									
101		k _y		1,28	-									
102		k _z		0,56	-									
103														
104	Material properties													
105		Climate class		I	-									
106		Load duration		long-term	-									
107		k _{mod}		0,7	-									
108		f _{c,0,k}		21	N/mm ²									
109		f _m		24	N/mm ²									
110		V _{sk}		1,3	-									
111		k _α		1,01	-									
112		f _{c,0,d}		11,31	N/mm ²									
113		f _{m,d}		13,10	N/mm ²									
114														
115	Forces and unity check (Incl. facade)													
116		σ _{N, facade}		2,37	N/mm ²									
117		σ _{M, facade}		2,56	N/mm ²									
118		UC _{facade}		0,57	-									
119	Forces and unity check (Excl. facade)													
120		σ _N		1,58	N/mm ²									
121		σ _M		5,12	N/mm ²									
122		UC		0,64	-									
123														
124														
125	Compression perpendicular to the grain (Beam below column)													
126		k _{c,90}		1,25	-									
127		x _i		30,00	mm									
128		I _{eff}		98,00	mm ⁴									
129		A _{eff}		13720,00	mm ²									
130		σ _{N, normal}		0,61	N/mm ²									
131		f _{c,90,k}		2,50	N/mm ²									
132		f _{c,90,d}		1,35	N/mm ²									
133		UC _{compression}		0,36	-									
134														
135														
136	Results 2-storey flat (Single wall;wind)													
137		CFL		Column	UC _{facade} *	UC _{normal}	UC _{normal} //							
138		6.70		SLS 38 x 140 - sp. 600 mm	0.57 (1)	0.64	0.36							
139		8.49		SLS 38 x 140 - sp. 400 mm	0.28 (2)	0.45	0.28							
140		10.28		SLS 38 x 140 - sp. 400 mm	0.32 (2)	0.48	0.32							
141		* placement of 1 or 2 columns at begin and end of wall												
142														

Figure X.167: Structural verification HSB wall (2-storey; Single wall; combination 2) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (3-storey; Single wall; Combination 2)													
2	Safety factors (NEN-EN 1990)	V _G	V _Q											
3	Eq 6.1		1,35	1,5										
4	Eq 6.2		1,2	1,5										
5														
6	Chosen column													
7	Timber column													
8		E-modulus (C24)		11000	N/mm ²									
9		E _{0,05}		7400	N/mm ²									
10		Density		420	kg/m ³									
11		Width (b)		38	mm									
12		Length (h)		140	mm									
13		A		5320	mm ²									
14		Spacing columns		0,4	m									
15		CFL		6,7	m									
16		m _{column}		5,59	kg/m ²									
17		Height floor		2,80	m									
18		Number of stories		3	-									
19		I _{column}		8689333,333	mm ⁴									
20		W _{column}		1,24E+05	mm ³									
21		Half span gallery beam		0,6	m									
22		Number of columns at start/end wall		2	-									
23		Opening factor		0,7	-									
24	Plywood													
25		Thickness		18	mm									
26		Density		550	kg/m ³									
27		ρ _{plywood}		9,9	kg/m ²									
28														

Figure X.168: Structural verification HSB wall (3-storey; Single wall; combination 2) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
29	Structural scheme													
30														
45	Load (wind load is leading action)													
46	Roof	kn/m ²	kn _{normal}	kn _{facade}		Formule C								
47	Permanent load	0,7	0,67	0,94		ψ ₀ B47*(B15/4) * B14 * (1/B23)					Formule D			
48	Variable load										B47*(B15/4) * ((B21+B14/2))			
49	Imposed load	1	0,96	1,34		B49*(B15/4)*B14*(1/B23)					B49 * (B15/4) * ((B14/2)+B21)			
50	eq 6.2		0,80	1,13		(B4*C47) + (C4*E48*C49)					(B4*D47) + (C4*E48*D49)			
52	3rd floor	kn/m ²	kn _{normal}	kn _{facade}										
53	Permanent load	1	0,96	1,34		ψ ₀ B53*(B15/4) * B14 * (1/B23)					B53*(B15/4) * ((B14/2)+B21)			
54	Variable													
55	Imposed load	1,75	1,68	2,35		B55*(B15/4) * B14*(1/B23)					B55 * (B15/4) * ((B14/2) + B21)			
56	Non-structural walls	0,5	0,48	0,67		B56*(B15/4) * B14*(1/B23)					B56*(B15/4) * ((B14/2)+B21)			
57	eq 6.2		2,44	3,42		(B4*C53) + (C4*E54*(C55+C56))					(B4*D53) + (C4*E54*(D55+D56))			
59	2nd floor	kn/m ²	kn _{normal}	kn _{facade}										
60	Permanent load	1	0,96	1,34		ψ ₀ B60*(B15/4) * B14 * (1/B23)					B60*(B15/4) * ((B14/2)+B21)			
61	Variable													
62	Imposed load	1,75	1,68	2,35		B62*(B15/4) * B14*(1/B23)					B62*(B15/4)*((B14/2)+B21)			
63	Non-structural walls	0,5	0,48	0,67		B63*(B15/4) * B14 * (1/B23)					B63*(B15/4)*((B14/2)+B21)			
64	eq 6.2		2,44	3,42		(B4*C60) + (C4*E61*(C62+C63))					(B4*D60) + (C4*E61*(D62+D63))			
66	Self-weight wall	kn/m ²	kn _{normal}	kn _{facade}										
67	Permanent load													
68	Columns		0,06	0,19		B68*B14*B18*B17					B68*(B14/2)*((B17*B18)*B22)			
69	Gallery beam			0,07										
70	Facade	0,50	1,68	0,84		B70 * B14 * B17*B18					B70*((B14/2))*B18*B17			
71	Gypsum board (Type F) (single side)	0,14	0,47	0,24		B71*B14*B18*B17					B71*(B14/2)*B17*B18			
72	Plywood (18 mm)	0,10	0,67	0,33		B72*B14*B18*B17*2					B72*(B14/2)*B17*B18*2			
73			3,00	1,67		SOM(C68:C72)					SOM(D68:D72)			
74	eq 6.2		3,60	2,00		B4*C73					B4*D73			
76	Pointload frond facade	kn/m ²	kn _{facade}											
77	Permanent load	0,5	7,04			B77*(B15/4)*B17*B18								
78	eq 6.2		8,44			B4*C77								
80	Total load	kn (incl. facade)*	kn (excl. facade)			Formule B					Formule C			
81	eq 6.2		9,20	9,29		(D50+D57+D64+D74+C78)/B22					C50+C57+C64+C74			
82	F _{ed}		9,20	9,29		MAX(B81:B81)					MAX(C81:C81)			
83			* Value is divided by factor 2 at facade. At that location, 2 columns must be placed											
86	Wind Load					Formule B								
87	q _p (22.4m) (Wind area II; Urban)	0,93	kn/m ²											
88	C _s C _d	1												
89	C _t D	0,8												
90	C _t E	0,7												
91	C _t tot	0,8				MAX(B89;B90)								
92	q _{wind_SLS_Normal}	0,30	kn/m			B87*B88*B91*B14								
93	q _{wind_SLS_facade}	0,07	kn/m			B87*B88*B91*(B14/2)/B22								
95	N _{normal}		N _{normal}	N _{facade}							Formule C			
96	M _{ed}	4,37E+05	1,09E+05			(1/8)*B92*C4*(B17*1000)^2					(1/8)*B93*C4*(B17*1000)^2			
99	Verification strength													
100	Buckling analysis > Supported at weak axis													
101	Reduction factor													
102	I _y	8,69E+06	mm ³			B19								
103	I _y	40,41	mm			WORTEL(B102/B13)								
104	L ₀ 00	2800	mm			B17*1000								
105	λ _y	69,28	-			B104/B103								
106	f _{c,0,k}	21	N/mm2											
107	λ _{rel}	1,17	-			(B105/P1()) * WORTEL(B106/B9)								
108	k _φ	1,28	-			0,5 * (1 + (0,2 * (B107-0,3)) + B107^2)								
109	k _φ	0,56	-			1/(B108+WORTEL(B108^2 - B107^2))								
111	Material properties													
112	Climate class	II												
113	Load duration	long-term												
114	k _{mod}	0,7												
115	f _{c,0,k}	21	N/mm2			B106								
116	f _m	24	N/mm2											
117	Y _{M1}	1,3												
118	k _φ	1,01				ALS(B12<150; MIN((150/B12)^0,2; 1,3); 1)								
119	f _{c,0,d}	11,31	N/mm2			B115*B114/B117								
120	f _{m,d}	13,10	N/mm2			B118*B114*B116/B117								
122	Forces and unity check (Incl. facade)													
123	σ _{N, facade}	1,73	N/mm2			B82*1000/B13								
124	σ _{M, facade}	0,88	N/mm2			C96/B20								
125	U _{C, facade}	0,34				(B123/(B109*B119)) + (B124/B120)								
126	Forces and unity check (Excl. facade)													
127	σ _N	1,75	N/mm2			C82*1000/B13								
128	σ _M	3,52	N/mm2			B96/B20								

Figure X.169: Structural verification HSB wall (3-storey; Single wall; combination 2) (2)_Excel

129	UC	0,54														
130																
131																
132	Compression perpendicular to the grain (Beam below column)															
133	$k_{c,90}$	1,25														
134	x_i	30,00														
135	l_{eff}	98,00 mm														
136	A_{eff}	13720,00 mm ²														
137	$\sigma_{N,Normal}$	0,68 N/mm ²														
138	$f_{c,90,k}$	2,50 N/mm ²														
139	$f_{c,90,d}$	1,35 N/mm ²														
140	$UC_{normal//}$	0,40														
141																
142																
143	Results 3-storey flat (Single wall;wind)															
144	CFL		Column	UC_{facade}	UC_{normal}	$UC_{normal //}$										
145	6.70		SLS 38 x 140 - sp. 400 mm	0.34 (2)	0.54	0.40										
146	8.49		SLS 38 x 170 - sp. 400 mm	0.27 (2)	0.39	0.39										
147	10.28		SLS 38 x 184 - sp. 400 mm	0.26 (2)	0.36	0.41										
148	* placement of 2 columns at begin and end of wall															
149																

Figure X.170: Structural verification HSB wall (3-storey; Single wall; combination 2) (3)_Excel

X.11.3.3 Housing separation wall

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (1-storey; Housing separation wall)													
2	Safety factors (NEN-EN 1990)		V _G	V _d	Formule B									
3	Eq 6.1		1,35	1,5										
4	Eq 6.2		1,2	1,5										
5	Chosen column													
6	Timber column													
8	E-modulus (C24)		11000	N/mm ²										
9	E _{0,05}		7400	N/mm ²										
10	Density		420	kg/m ³										
11	Width (b)		38	mm										
12	Length (h)		89	mm										
13	A		3382	mm ²	B11*B12									
14	Spacing columns		0,6	m										
15	CFL		6,7	m										
16	m _{column}		2,37	kg/m ²	(B11/1000) * (B12/1000) * B10 * (1/B14)									
17	Height floor		2,80	m										
18	Number of stories		1	-										
19	I _{column}		2232401,833	mm ⁴	(1/12)*B11*B12^3									
20	W _{column}		50166,33333	mm ³	(1/6)*B11*B12^2									
21	Half span gallery beam		0,6	m										
22	Number of columns at start/end wall		1	-										
23	Factor load transfer		0,35	-										
24	Plywood													
25	Thickness		18	mm										
26	Density		550	kg/m ³										
27	m _{plywood}		9,9	kg/m ²	(B25/1000)*B26									
28	Structural scheme													
30														
44	Load (Imposed load is leading action)													
46	Roof (Full loaded)													
47	Permanent load	kN/m ²	0,7	kN _{normal}	0,98	kN _{facade}	1,48	ψ ₀		Formule C B47*B15*B23*B14				
48	Variable load							0		Formule D B47*B15*B23*((B14/2)+B21)				
49	Imposed load		1		1,41		2,11			B49*B15*B23*B14				
50	eq 6.1				1,33		1,99			(B3*C47) + (C3*E48*C49)				
51	eq 6.2				3,29		4,94			(B4*C47) + (C4*C49)				
52	Self-weight wall													
54	Permanent load	kN/m ²		kN _{normal}		kN _{facade}								
55	Columns		0,02		0,04		0,02			B55*B14*B18*B17				
56	Gallery beam						0,07			B55*(B14/2)*B17*B18*B22				
57	Gypsum board (Type F)		0,14		0,24		0,12			B57*B14*B18*B17				
58	Plywood (18 mm)		0,10		0,17		0,08			B58*B14*B18*B17				
59					0,44		0,29			SOM(D55-D58)				
60	eq 6.1				0,60		0,39			B3*C59				
61	eq 6.2				0,53		0,35			B4*C59				
62	Pointload frond facade													
64	Permanent load	kN/m ²	0,5	kN _{facade}			3,28			B64*B23*B15*B18*B17				
65	eq 6.1						4,43			B3*C64				
66	eq 6.2						3,94			B4*C64				
71	Total load													
72	eq 6.1	kN (incl. facade)*		kN (excl. facade)			1,93			Formule B (D50+D60+C65)/B22				
73	eq 6.2		6,82		9,23		3,82			(D51+D61+C66)/B22				
74	F _{red}		9,23		9,23		3,82			MAX(B72:B73)				
75	* Value is divided by factor 1 or 2 at facade. At that location, 1 or 2 column(s) must be placed													
77	Verification strength													
78	Buckling analysis > Supported at weak axis													
80	Reduction factor													
81	I _y		2,23E+06	mm ³	B19									
82	i _y		25,69	mm	WORTEL(B81/B13)									
83	l _{buck}		2800	mm	B17*1000									
84	λ _y		108,98	-	B83/B82									
85	f _{e,0,k}		21	N/mm ²										
86	λ _{rel}		1,85	-	(B84/Pi()) * WORTEL(B85/B9)									
87	k _y		2,36	-	0,5 * (1 + (0,2 * (B86-0,3)) + B86^2)									
88	k _c		0,26	-	1/(B87+WORTEL(B87^2 - B86^2))									
90	Material properties													
91	Climate class													
92	Load duration	long-term												
93	k _{mod}		0,7											
94	f _{e,0,k}		21	N/mm ²	B85									
95	γ _M		1,3											
96	f _{e,0,d}		11,31	N/mm ²	B94*B93/B95									

Figure X.171: Structural verification HSB wall (1-storey; Housing separation wall) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
97	Forces and unity check (incl. facade)													
98		$\sigma_{N, facade}$	2,73 N/mm ²	B74*1000/B13										
99		UC_{facade}	0,93	B99/(B96*B88)										
100	Forces and unity check (Excl. facade)													
101		σ_H	1,13 N/mm ²	C74*1000/B13										
102		UC	0,38	B102/(B88*B96)										
103														
104														
105														
106	Compression perpendicular to the grain (Beam below column)													
107		$k_{c,90}$	1,25											
108		x_i	30,00	MIN(30; B14*1000/2; B11)										
109		l_{eff}	98,00 mm	B11+(2*B108)										
110		A_{eff}	8722,00 mm ²	B12*B109										
111		$\sigma_{H, Normal}$	0,44 N/mm ²	C74*1000/B110										
112		$f_{c,90,d}$	2,50 N/mm ²											
113		$f_{t,90,d}$	1,35 N/mm ²	B112*B93/B95										
114		$UC_{normal//}$	0,26	B111/(B107*B113)										
115														
116														
117	Results 1-storey flat (sep-wall)													
118		CFL	Column	UC_{facade}^*	UC_{normal}	$UC_{normal} //$								
119		6.70	SLS 38 x 89 - sp. 600 mm	0.93 (1)	0.38	0.26								
120		8.49	SLS 38 x 89 - sp. 600 mm	0.58 (2)	0.47	0.32								
121		10.28	SLS 38 x 89 - sp. 600 mm	0.70 (2)	0.56	0.38								
122	* placement of 1 or 2 columns at begin and end of wall													

Figure X.172: Structural verification HSB wall (1-storey; Housing separation wall) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (2-storey; Housing separation wall)													
2	Safety factors (NEN-EN 1990)													
3	Eq 6.1	γ_G	1,35	1,5	Formule B									
4	Eq 6.2		1,2	1,5										
5	Chosen column													
6	Timber column													
7		E-modulus (C24)	11000 N/mm ²											
8		$E_{0,05}$	7400 N/mm ²											
9		Density	420 kg/m ³											
10		Width (b)	38 mm											
11		Length (h)	120 mm											
12		A	4560 mm ²	B11*B12										
13		Spacing columns	0,6 m											
14		CFL	6,7 m											
15		m_{column}	3,19 kg/m ²	(B11/1000) * (B12/1000) * B10 * (1/B14)										
16		Height floor	2,80 m											
17		Number of stories	2											
18		I_{column}	5472000 mm ⁴	(1/12)*B11*B12^3										
19		W_{column}	91200 mm ³	(1/6)*B11*B12^2										
20		Half span gallery beam	0,6 m											
21		Number of columns at start/end wall	2											
22		Factor load transfer	0,35											
23	Plywood													
24		Thickness	18 mm											
25		Density	550 kg/m ³											
26		$m_{plywood}$	9,9 kg/m ²	(B25/1000)*B26										
27														
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														
43														
44														
45	Load (Imposed load is leading action)													
46	Roof (Full loaded)													
47	Permanent load	kn/m ²	0,7	kn _{normal}	0,98	kn _{facade}	1,48	ψ ₀	0	Formule C	B47*B15*B23*B14	Formule D	B47*B15*B23*((B14/2)+B21)	
48	Variable load													
49	Imposed load		1		1,41	2,11				B49*B15*B23*B14	B49*B15*B23*((B14/2)+B21)			
50	eq 6.1				1,33	1,99				(B3*C47) + (C3*E48*C49)	(B3*D47) + (C3*E48*D49)			
51	eq 6.2				3,29	4,94				(B4*C47) + (C4*C49)	(B4*D47) + (C4*D49)			
52														
53	2nd floor (Full loaded)													
54	Permanent load	kn/m ²	1	kn _{normal}	1,41	kn _{facade}	2,11	ψ ₀	0,4	Formule C	B54*B15*B14*B23	Formule D	B54*B15*B23*((B14/2)+B21)	
55	Variable													
56	Imposed load		1,75		2,46	3,69				B56*B15*B14*B23	B56*B15*B23*((B14/2)+B21)			
57	Non-structural walls		0,5		0,70	1,06				B57*B15*B14*B23	B57*B15*B23*((B14/2)+B21)			
58	eq 6.1				3,80	5,70				(B3*C54) + (C3*E55*(C56+C57))	(B3*D54)+(C3*E55*(D56+D57))			
59	eq 6.2				5,80	8,71				(B4*C54) + (C4*C56) + (C4*E55*C57)	(B4*D54) + (C4*D56) + (C4*E55*D57)			
60														
61	Self-weight wall													
62	Permanent load	kn/m ²		kn _{normal}		kn _{facade}								
63		Columns	0,03		0,11	0,05				B63*B14*B18*B17	B63*(B14/2)*B17*B18*B22			
64		Gallery beam				0,07								
65		Gypsum board (Type F)	0,14		0,47	0,24				B65*B14*B18*B17	B65*(B14/2)*B18*B17			
66		Plywood (18 mm)	0,10		0,33	0,17				B66*B14*B18*B17	B66*(B14/2)*B18*B17			
67					0,91	0,59				SOM(C63.C66)	SOM(D63.D66)			
68	eq 6.1				1,23	0,71				B3*C67	B3*D67			
69	eq 6.2				1,09	0,63				B4*C67	B4*D67			
70														

Figure X.173: Structural verification HSB wall (2-storey; Housing separation wall) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
70														
71	Pointload frond facade	kN/m ²		kN _{facade}										
72	Permanent load	0,5		6,57		B72*B23*B15*B18*B17								
73		eq 6.1		8,86		B3*C72								
74		eq 6.2		7,88		B4*C72								
75														
76														
77														
78	Total load	kN (incl. facade)*		kN (excl. facade)										
80		eq 6.1		6,36		Formule B				Formule C				
81		eq 6.2		10,19		(D50+D58+D68+C73)/B22				C50+C58+C68				
82		F _{ed}		10,19		(D51+D59+D69+C74)/B22				C51+C59+C69				
83						MAX(B80:B81)				MAX(C80:C81)				
84		* Value is divided by factor 1 or 2 at facade. At that location, 1 or 2 column(s) must be placed												
85														
86	Verification strength													
87	Buckling analysis > Supported at weak axis					Formule B								
88	Reduction factor													
89		I _y	5,47E+06	mm ³		B19								
90		I _y	34,64	mm		WORTEL(B89/B13)								
91		I _{bus}	2800	mm		B17*1000								
92		λ _y	80,83	-		B91/B90								
93		f _{c,0,k}	21	N/mm ²										
94		λ _{rel}	1,37	-		(B92/Pi()) * WORTEL(B93/B9)								
95		k _y	1,55	-		0,5 * (1 + (0,2 * (B94-0,3)) + B94^2)								
96		k _c	0,44	-		1/(B95+WORTEL(B95^2 - B94^2))								
97														
98	Material properties													
99		Climate class	1	-										
100		Load duration	long-term	-										
101		k _{road}	0,7	-										
102		f _{c,0,k}	21	N/mm ²		B93								
103		γ _M	1,3	-										
104		f _{c,0,d}	11,31	N/mm ²		B102*B101/B103								
105														
106	Forces and unity check (Incl. facade)													
107		σ _{N,1+ext}	4,86	N/mm ²		B82*1000/B13								
108		U _{C,ext}	0,97	-		B107/(B104*B96)								
109	Forces and unity check (Excl. facade)													
110		σ _N	2,23	N/mm ²		C82*1000/B13								
111		U _C	0,45	-		B110/(B96*B104)								
112														
113														
114	Compression perpendicular to the grain (Beam below column)													
115		k _{c,90}	1,25	-										
116		κ _c	30,00	-		MIN(50; B14*1000/2; B11)								
117		I _{eff}	98,00	mm		B11+(B116^2)								
118		A _{eff}	11760,00	mm ²		B12*B117								
119		σ _{N,Normal}	0,87	N/mm ²		C82*1000/B118								
120		f _{c,90,k}	2,50	N/mm ²										
121		f _{c,90,d}	1,35	N/mm ²		B120*B101/B103								
122		U _{C,normal//}	0,51	-		B119/(B121*B115)								
123														
124														
125	Results 2-storey flat (sep-wall)													
126		CFL		Column	U _{C,ext} *	U _{C,normal}	U _{C,normal} //							
127		6.70		SLS 38 x 120 - sp. 600 mm	0.97 (1)	0.45	0.51							
128		8.49		SLS 38 x 120 - sp. 600 mm	0.61 (2)	0.55	0.64							
129		10.28		SLS 38 x 120 - sp. 600 mm	0.74 (2)	0.66	0.76							
130		* placement of 1 or 2 columns at begin and end of wall												
131														

Figure X.174: Structural verification HSB wall (2-storey; Housing separation wall) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation HSB wall (3 storey; Housing separation wall)													
2	Safety factors (NEN-EN 1990)	γ _G	γ _Q			Formule B								
3		Eq 6.1	1,35	1,5										
4		Eq 6.2	1,2	1,5										
5														
6	Chooosen column													
7	Timber column													
8		E-modulus (C24)	11000	N/mm ²										
9		E _{0,05}	7400	N/mm ²										
10		Density	420	kg/m ³										
11		Width (b)	38	mm										
12		Length (h)	120	mm										
13		A	4560	mm ²		B11*B12								
14		Spacing columns	0,6	m										
15		CFL	6,7	m										
16		m _{column}	3,19	kg/m ³		(B11/1000) * (B12/1000) * B10 * (1/B14)								
17		Height floor	2,80	m										
18		Number of stories	3	-										
19		I _{column}	5472000	mm ⁴		(1/12)*B11*B12^3								
20		W _{column}	91200	mm ³		(1/6)*B11*B12^2								
21		Half span gallery beam	0,6	m										
22		Number of columns at start/end wall	2	-										
23		Factor load transfer	0,35	-										
24	Plywood													
25		Thickness	18	mm										
26		Density	550	kg/m ³										
27		m _{plywood}	9,9	kg/m ²		(B25/1000)*B26								
28														
29	Structural scheme													
30														
31														
32														
33														
34														
35														
36														
37														
38														
39														
40														
41														
42														

Figure X.175: Structural verification HSB wall (3-storey; Housing separation wall) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
43														
44														
45	Load (Imposed load is leading action)													
46	Roof	kN/m ²	kN _{normal}	kN _{facade}										
47	Permanent load	0,7	0,98	1,48	ψ ₀	Formule C			Formule D					
48	Variable load				0	B47*B15*B23*B14			B47*B15*B23*((B14/2)+B21)					
49	Imposed load	1	1,41	2,11		B49*B15*B23*B14			B49*B15*B23*((B14/2)+B21)					
50	eq 6.1		1,33	1,99		(B3*C47) + (C3*E48*C49)			(B3*D47) + (C3*E48*D49)					
51	eq 6.2		1,18	1,77		(B4*C47) + (C4*E48*C49)			(B4*D47) + (C4*E48*D49)					
52														
53	3rd floor (Full loaded)	kN/m ²	kN _{normal}	kN _{facade}										
54	Permanent load	1	1,41	2,11	ψ ₀	B54*B15*B14*B23			B54*B15*B23*((B14/2)+B21)					
55	Variable				0,4									
56	Imposed load	1,75	2,46	3,69		B56*B15*B14*B23			B56*B15*B23*((B14/2)+B21)					
57	Non-structural walls	0,5	0,70	1,06		B57*B15*B14*B23			B57*B15*B23*((B14/2)+B21)					
58	eq 6.1		3,80	5,70		(B3*C54) + (C3*E55*(C56+C57))			(B3*D54)+(C3*E55*(D56+D57))					
59	eq 6.2		5,80	8,71		(B4*C54) + (C4*E55*(C56+C57))			(B4*D54) + (C4*E55*(D56+D57))					
60														
61	2nd floor (Full loaded)	kN/m ²	kN _{normal}	kN _{facade}										
62	Permanent load	1	1,41	2,11	ψ ₀	B62*B15*B14*B23			B62*B15*B23*((B14/2)+B21)					
63	Variable				0,4									
64	Imposed load	1,75	2,46	3,69		B64*B15*B14*B23			B64*B15*B23*((B14/2)+B21)					
65	Non-structural walls	0,5	0,70	1,06		B65*B15*B14*B23			B65*B15*B23*((B14/2)+B21)					
66	eq 6.1		3,80	5,70		(B3*C62) + (C3*E63*(C64+C65))			(B3*D62)+(C3*E63*(D64+D65))					
67	eq 6.2		5,80	8,71		(B4*C62) + (C4*E63*(C64+C65))			(B4*D62) + (C4*E63*(D64+D65))					
68														
69	Self-weight wall	kN/m ²	kN _{normal}	kN _{facade}										
70	Permanent load													
71	Columns		0,03	0,16	0,16	B71*B14*B18*B17			B71*(B14/2)*B17*B18*B22					
72	Gallery beam				0,07									
73	Gypsum board (Type F)		0,14	0,71	0,35	B73*B14*B18*B17			B73*(B14/2)*B18*B17					
74	Plywood (18 mm)		0,10	0,50	0,25	B74*B14*B18*B17			B74*(B14/2)*B18*B17					
75					0,83	SOM(C71-C74)			SOM(D71-D74)					
76	eq 6.1			1,84	1,12	B3*C75			B3*D75					
77	eq 6.2			1,64	1,00	B4*C75			B4*D75					
78														
79	Pointload frond facade	kN/m ²	kN _{facade}											
80	Permanent load	0,5		9,85		B80*B23*B15*B18*B17			B80*B23*B15*B18*B17					
81	eq 6.1			13,30		B3*C80			B3*C80					
82	eq 6.2			11,82		B4*C80			B4*C80					
83														
84														
85														
86														
87	Total load	kN (incl. facade)*	kN (excl. facade)											
88	eq 6.1		13,91	10,77		Formule B			Formule C					
89	eq 6.2		16,00	14,43		[D50+D58+D66+D76+C81]/B22			C50+C58+C66+C76					
90	F _{ed}		16,00	14,43		[D51+D59+D67+D77+C82]/B22			C51+C59+C67+C77					
91						MAX(B88-B89)			MAX(C88-C89)					
92														
93														
94	Verification strength													
95	Buckling analysis - Supported at weak axis													
96	Reduction factor													
97	I _y		5,47E+06 mm ³		B19									
98	I _y		34,64 mm		WORTEL(B97/B13)									
99	l _{bucc}		2800 mm		B17*1000									
100	λ _y		80,83 -		B99/B98									
101	f _{c,0,k}		21 N/mm ²											
102	λ _{rel}		1,37 -		(B100/PI()) * WORTEL(B101/B9)									
103	k _y		1,55 -		0,5 * (1 + (0,2 * (B102-0,3)) + B102^2)									
104	k _c		0,44 -		1/(B103+WORTEL(B103^2 - B102^2))									
105														
106	Material properties													
107	Climate class	I												
108	Load duration	long-term												
109	k _{mod}		0,7 -											
110	f _{c,0,k}		21 N/mm ²		B101									
111	γ _{Ma}		1,3 -											
112	f _{c,0,d}		11,31 N/mm ²		B110*B109/B111									
113														
114	Forces and unity check (Incl. facade)													
115	σ _{N, facade}		3,51 N/mm ²		B90*1000/B13									
116	UC _{facade}		0,70 -		B115/(B112*B104)									
117	Forces and unity check (Excl. facade)													
118	σ _N		3,16 N/mm ²		C90*1000/B13									
119	UC		0,63 -		B118/(B104*B112)									
120														
121														
122	Compression perpendicular to the grain (Beam below column)													
123	k _{c,90}		1,25											
124	x _i		30,00		MIN(30; B14*1000/2; B11)									
125	l _{eff}		98,00 mm		B11+(B124*2)									
126	A _{eff}		11760,00 mm ²		B125*B12									
127	σ _{N, normal}		1,23 N/mm ²		C90*1000/B126									
128	f _{c,90,k}		2,50 N/mm ²											
129	f _{c,90,d}		1,35 N/mm ²		B128*B109/B111									
130	UC _{normal/7}		0,73 -		B127/(B129*B123)									
131														
132														
133	Results 3-storey flat (sep-wall)													
134	CFL	Column	UC _{facade} *	UC _{normal}	UC _{normal} //									
135	6.70	SLS 38 x 120 - sp. 600 mm	0.70 (2)	0.63	0.73									
136	8.49	SLS 38 x 120 - sp. 600 mm	0.88 (2)	0.78	0.90									
137	10.28	SLS 38 x 140 - sp. 600 mm	0.72 (2)	0.63	0.92									
138	* placement of 2 columns at begin and end of wall													
139														

Figure X.176: Structural verification HSB wall (3-storey; Housing separation wall) (2)_Excel

X.11.3.4 Hand validation

Note: later during design, compression perpendicular to grain is added (beam), which appeared to be a normative design parameter for the 3-storey walls. Resultingly, the size of the columns is changed. Note that still the validation holds, as the excel is not changed.

Order /
 Blad nr 1 / 4
 Deel Verificatie inner-walls HSB
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3-storey top-up (inner-walls)

• Properties column

$b \times h = 300 \times 140 \text{ [mm]}$ $\rho = 420 \text{ [kg/m}^3\text{]}$
 $s_{max} = 140 \text{ [mm]}$ $E_{col} = 7400 \text{ [N/mm}^2\text{]}$
 $CFI = 10 \cdot 20 \text{ [mm]}$ plywood: $10 \text{ [mm]} \rightarrow 350 \text{ [kg/m}^3\text{]}$
 $F_{k,rot} = 2 \text{ [mm]}$
 Opening Factor = $0.7 \cdot 1.3$
 $s_{pan} \text{ half gallery} = 0.6 \text{ [m]}$

• Load calculation (imposed load & loading column)

(1) Roof [kN/m²]

(a) permanent - normal = $0.7 \cdot \frac{CFI}{2} \cdot s_{max} \cdot \left(\frac{1}{s_{max}}\right) \cdot 0.7 \cdot \frac{10 \cdot 20}{2} \cdot 0.4 \cdot \frac{1}{0.7} = 2.06 \text{ X}$
 (b) permanent - facade = $0.7 \cdot \frac{CFI}{2} \cdot (s_{max}/2 + s_{pan} \cdot \text{gallery}) = 0.7 \cdot \frac{10 \cdot 20}{2} \cdot (0.4/2 + 0.6) = 2.9$

(a) Variable - normal = $1.0 \cdot \frac{CFI}{2} \cdot s_{max} \cdot \left(\frac{1}{s_{max}}\right) \cdot 1 \cdot \frac{10 \cdot 20}{2} \cdot 0.4 \cdot \frac{1}{0.7} = 2.94 \text{ S}$
 (b) Variable - facade = $1.0 \cdot (CFI/2) \cdot (s_{max}/2 + \text{gallery}) = 1 \cdot (10 \cdot 20/2) \cdot (0.4/2 + 0.6) = 4.11 \text{ S}$

Normal column

$q_{k,1} = 1.35 \cdot G + 1.5 \cdot q_k = (1.35 \cdot 2.06) + (1.5 \cdot 0.294) = 2.78 \text{ [kN]} \text{ X}$
 $q_{k,2} = 1.2 \cdot G + 1.5 \cdot q_k = (1.2 \cdot 2.06) + (1.5 \cdot 0.294) = 2.47 \text{ [kN]} \text{ X}$

Facade column

$q_{k,1} = 1.35 \cdot G + 1.5 \cdot q_k = (1.35 \cdot 2.9) + (1.5 \cdot 0.411) = 3.8 \text{ [kN]} \text{ S}$
 $q_{k,2} = 1.2 \cdot G + 1.5 \cdot q_k = (1.2 \cdot 2.9) + (1.5 \cdot 0.411) = 3.46 \text{ [kN]} \text{ S}$

(2) 1st & 2nd floor [kN/m²]

(a) permanent - normal = $1.0 \cdot \frac{CFI}{2} \cdot s_{max} \cdot \left(\frac{1}{s_{max}}\right) \cdot 1 \cdot \frac{10 \cdot 20}{2} \cdot 0.4 \cdot \left(\frac{1}{0.7}\right) = 2.94 \text{ [kN]} \text{ S}$
 (b) permanent - facade = $1.0 \cdot CFI/2 \cdot (s_{max}/2 + \text{gallery}) = 1 \cdot \frac{10 \cdot 20}{2} \cdot (0.4/2 + 0.6) = 4.11 \text{ [kN]} \text{ S}$

(a) Variable - normal = [kN/m²]
 - imposed load = $1.75 \cdot (CFI/2) \cdot s_{max} \cdot \left(\frac{1}{s_{max}}\right) = 1.75 \cdot \frac{10 \cdot 20}{2} \cdot 0.4 \cdot \left(\frac{1}{0.7}\right) = 5.14$
 - Non-st. walls = $0.5 \cdot \frac{10 \cdot 20}{2} \cdot 0.4 \cdot \left(\frac{1}{0.7}\right) = 1.47$

Figure X.177: Validation HSB Wall (Single wall; combination 1) (1)

Order ✓
 Blad nr 2/4
 Deel Verificatie inner-walls HSB
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(b) Variable facade $\frac{c_{FE}}{2}$ (3000/2 1500) = 1.75 $\frac{10^2}{2}$ (0.4/2 + 0.6) = 17.20
 - imposed load = 1.75
 - Non-stb walls = 0.5 $\frac{10^2}{2}$ (0.4/2 + 0.6) = 2.06

eg 6.1 = 1.35 G + 1.5 Q₀ Q = (1.35 · 2.54) + (1.5 · 0 + (5.14 + 1.47)) = 7.94 [kN] ✓
 eg 6.2 = 1.2 G + 1.5 Q_{imp} + 1.5 Q₀ Q = (1.2 · 2.54) + (1.5 · 5.14) + (1.5 · 0 + 1.47) = 12.12 [kN]

Normal columns
 eg 6.1 = 1.35 G + 1.5 Q₀ Q = (1.35 · 4.11) + (1.5 · 0 + (7.20 + 2.06)) = 11.10 [kN]
 eg 6.2 = 1.2 G + 1.5 Q_{imp} + 1.5 Q₀ Q = (1.2 · 4.11) + (1.5 · 7.20) + (1.5 · 0 + 2.06) = 16.97 [kN]

(c) Self-weight wall $\frac{1}{0.1} \cdot 0.4$
 permanent - normal $\frac{1}{0.1} \cdot 0.4$
 columns = (38/1000) (140/1000) 3 (story) 2.8 (h_{st}) 420 [kg/m³] · 10/1000 = 0.15
 gypsum board = 0.14 [kN/m²] · 0.4 (space) · 3 · 2.8 · 2 = 0.94 (2 · 2.8) ✓
 plywood (skin) = (18/1000) 550 [kg/m³] · 0.4 (space) · 3 · 2.8 · 2 = 0.67 $\frac{10^3}{1000}$ ✓
 1.06 [kN] ✓

(b) permanent - facade
 columns = (38/1000) (140/1000) 0.4 · 0.2 · 3 · 2.8 · 420 10/1000 · 2 = 0.15
 gallery beam = 0.67
 gypsum board = 0.14 [kN/m²] · 0.4 (space) · 3 · 2.8 · 2 = 0.47
 plywood = (18/1000) 550 [kg/m³] (0.4/2) 3 · 2.8 · 2 = 0.33 $\frac{10^3}{1000}$ ✓
 1.06 [kN] ✓

Normal columns
 eg 6.1 = 1.35 G + 1.5 Q₀ Q = 1.35 · 1.80 = 2.43 [kN] ✓
 eg 6.2 = 1.2 G = 1.2 · 1.20 = 1.44 [kN] ✓

Facade columns
 eg 6.1 = 1.35 G = 1.35 · 1.06 = 1.43 [kN] ✓
 eg 6.2 = 1.2 G = 1.2 · 1.06 = 1.27 [kN] ✓

(4) Paint load facade - facade $\frac{c_{FE}}{2} = 10^2$
 permanent facade = 0.5 [kN/m²] $\frac{c_{FE}}{2}$ · 3 (6.0m) 2.8 (h_{st}) · 21.57 [kN]
 eg 6.1 = 1.35 G = 1.35 · 21.57 = 29.11 [kN] ✓
 eg 6.2 = 1.2 G = 1.2 · 21.57 = 25.88 [kN] ✓

Figure X.178: Validation HSB Wall (Single wall; combination 1) (2)

Order /
 Blad nr 3 / 4
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(5) Total load

(a) Normal columns

eg 6.1 = $2.78 + (2.7.54) + 2.43 = 21.09 \text{ [kN]}$
 eg 6.2 = $2.47 + (2.12.12) + 2.16 = 28.87 \text{ [kN]}$
 $F_{tot} = 28.87 \text{ [kN]} \rightarrow 28.87 \text{ (sheet value)}$

(b) Facade columns

eg 6.1 = $(3.89 + (2.11.10) + 1.43 + 25.14) / 2 = 28.33 \text{ [kN]}$
 eg 6.2 = $(3.48 + (2.16.77) + 1.27 + 25.51) / 2 = 22.25 \text{ [kN]}$
 $F_{tot} = 32.25 \text{ [kN]} \rightarrow 32.25 \text{ (sheet value)}$
division by factor 2 due to 2 columns, etc

• Calculation buckling factor

\rightarrow supported around weak axis.

$I_y = \frac{1}{12}bh^3 = \frac{1}{12} \cdot 30 \cdot 140^3 = 86925000 \text{ [mm}^4]$
 $i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{86925000}{30 \cdot 140}} = 40.410 \text{ [mm]}$

$\lambda_y = \frac{L_{eff}}{i_y} = \frac{2600}{40.41} = 64.28 \text{ [-]}$

$\lambda_{rel} = \frac{\lambda_y}{\pi} \sqrt{\frac{F_{crit}}{E_{mod}}} = \frac{64.28}{\pi} \sqrt{\frac{21}{7400}} = 1.17 \text{ [-]}$

$k_y = 0.5(1 + \beta(\lambda_{rel} - 0.3) + \lambda_{rel}^2) = 0.5(1 + 0.2(1.17 - 0.3) + 1.17^2) = 1.27$
 $k_{eff} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel}^2}} = \frac{1}{1.27 + \sqrt{1.27^2 - 1.17^2}} = 0.56 \text{ [-]} \rightarrow 0.56 \text{ (sheet value)}$

• Verification

$F_{mod} = 0.7$
 $F_{crit} = 21 \text{ [N/mm}^2]$
 $\gamma_M = 1.3$
 $F_{red} = 11.31 \text{ [N/mm}^2]$

Normal columns

$\sigma_N = \frac{F_{tot}}{A} = \frac{28.87 \cdot 10^3}{30 \cdot 140} = 5.43 \text{ N/mm}^2$

$\eta \xi = \frac{\sigma_N}{F_{red} \cdot \xi} = \frac{5.43}{11.31 \cdot 0.56} = 0.86 \text{ [-]} \rightarrow 0.85 \text{ (sheet value)}$

Figure X.179: Validation HSB Wall (Single wall; combination 1) (3)

Order ✓
 Blad nr 4/4
 Deel Verification inner-walls HSB
 Datum 24/05/2023

VAN ROSSUM RAADGEVENDE
 INGENIEURS

Facade columns

$$\sigma_u = \frac{F_{ed}}{A} = \frac{3224 \text{ kN}}{53.72 \text{ m}^2} = 607 \text{ [N/mm}^2\text{]}$$

$$u_1 = \frac{\sigma_u}{\text{Fact. } K_c} = \frac{607}{11.36} = 0.55 \text{ [} \rightarrow \text{ } \delta \text{ as } 0.55 \text{ [} \rightarrow \text{ (steel value)}\text{]}$$

SHEET VALIDATED

Note: sheet holds also for:

- SHEET inner-walls (2-storey) → small adaptation; 1-storey deleted from excel and roof eq 6.2 = 0.20 → 1.5

~~SHEET VALIDATED BY ROSSUM~~

~~24/05/2023~~
~~CPK - multi - fact 0.95 0.55 instead of 0.20 (1.5)~~

Figure X.180: Validation HSB Wall (Single wall; combination 1) (4)

Order /
 Blad nr 1/3
 Deel Verifi.cation Facade walls - HSB
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Note: later during design, compression perpendicular to grain is added (beam), which appeared to be a normative design parameter for the 3-storey walls. Resultingly, the size of the columns is changed. Note that still the validation holds, as the excel is not changed.

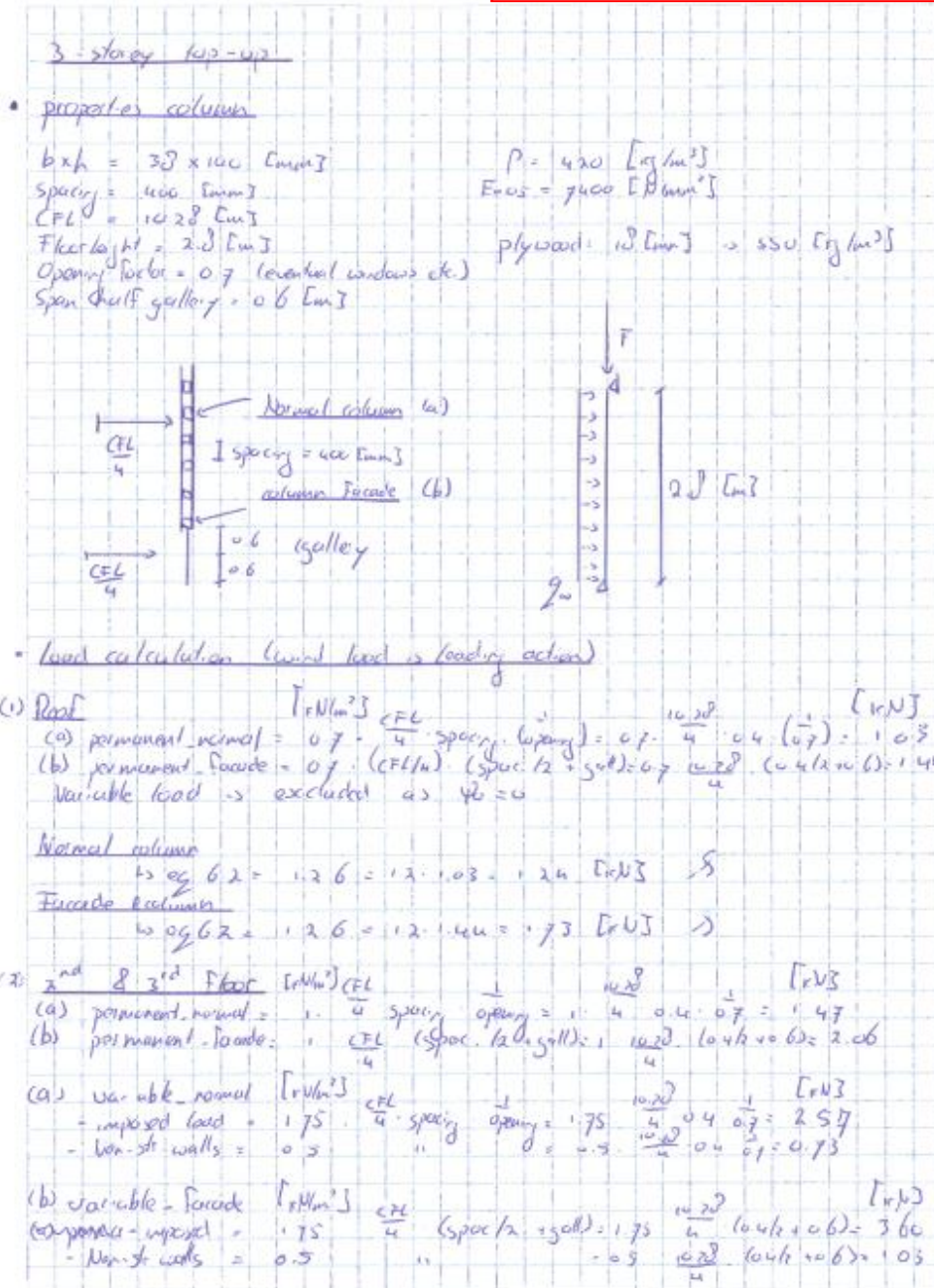


Figure X.181: Validation HSB Wall (Single wall; combination 2) (1)

Order /
 Blad nr 2/3
 Deel Verification Façade walls - HSB
 Datum 24/05/2023



Normal column
 $eg\ 6.2 = 1.2 G + 1.5 Q_{Ed} = (1.2 \cdot 1.47) + (1.5 \cdot 0 + (2.57 + 0.73)) = 3.74\ [kN]$

Façade column
 $eg\ 6.2 = 1.2 G + 1.5 Q_{Ed} = (1.2 \cdot 2.06) + (1.5 \cdot 0 + (3.60 + 1.03)) = 5.25\ [kN]$

(3) Self-weight wall

(a) permanent - normal
 Columns = $(38/1000) (140/1000) 0.4 \cdot 0.4 \cdot 3 \cdot 2 \cdot 420 \cdot 10/1000 = 0.15\ [kN]$
 Gypsum board = $0.14\ [kN/m^2] 0.4 \cdot 3 \cdot 2 \cdot 2 = 0.47\ [kN]$
 Plywood = $(12/1000) 550\ [kN/m^2] 10/1000 \cdot 0.4 \cdot 3 \cdot 2 \cdot 2 = 0.67\ [kN]$
 Façade = $0.5\ [kN/m^2] 0.4 \cdot 3 \cdot 2 \cdot 2 = 1.67\ [kN]$
3.01 [kN]

(b) permanent - Façade
 Columns = $(38/1000) (140/1000) 0.4 \cdot 0.2 \cdot 3 \cdot 2 \cdot 420 \cdot 10/1000 \cdot 2 = 0.11\ [kN]$
 Gypsum board = $0.14\ [kN/m^2] (0.4/2) \cdot 3 \cdot 2 \cdot 2 = 0.28\ [kN]$
 Plywood = $(12/1000) 550\ [kN/m^2] 10/1000 \cdot \frac{0.4}{2} \cdot 3 \cdot 2 \cdot 2 \cdot 2 = 0.33\ [kN]$
 Façade = $0.5\ [kN/m^2] \cdot \frac{0.4}{2} \cdot 3 \cdot 2 \cdot 2 = 0.30\ [kN]$
 Gallery beam = 0.07 [kN]
1.07 [kN]

Normal column
 $eg\ 6.2 = 1.2 G = 1.2 \cdot 3.01 = 3.61\ [kN]$

Façade column
 $eg\ 6.2 = 1.2 G = 1.2 \cdot 1.07 = 1.28\ [kN]$

(4) Point load Façade $(\frac{F_{Ed}}{4}) = 10.25$
 (a) permanent - Façade = $0.5 \cdot \frac{F_{Ed}}{4} \cdot 3 (261mm) \cdot 2 \cdot 2 = 10.75\ [kN]$
 b) $eg\ 6.2 = 1.2 G = 1.2 \cdot 10.75 = 12.90\ [kN]$

(5) Total load (Vertical)

Normal column (eg 6.2) = $1.24 + (2 \cdot 3.74) + 3.61 = 12.33\ [kN]$
 Façade column (eg 6.2) = $(1.73 + (2 \cdot 5.25) + 2.00 + 12.95) / 2 = 13.58\ [kN]$

(6) Wind load

(a) $g_{w0} = g_p$ (s.d. Cf - spacing = 0.95) $\cdot 1 \cdot 0.8 \cdot 0.4 = 0.31\ [kN/m^2]$ (Normal)
 (b) $g_{w0} = \frac{spacing}{2} / \text{number of columns} \cdot 0.95 \cdot 1 \cdot 0.8 \cdot 0.2 = 0.07\ [kN/m^2]$ (Façade)
 Med. normal = $\frac{1}{2} \cdot 0.31 \cdot 2800^2 = 441000\ [Nmm]$
 Med. façade = $\frac{1}{2} \cdot 0.07 \cdot 2800^2 = 109360\ [Nmm]$

Figure X.182: Validation HSB Wall (Single wall; combination 2) (2)

Order /
 Blad nr 3/3
 Deel Verification Facade walls - HSB
 Datum 24/05/2023



o Verification

$$\left. \begin{array}{l} K_{red} = 0.7 \\ K_k = 1.01 \\ \gamma_M = 1.3 \\ f_{red} = 21 \\ F_m = 24 \end{array} \right\} \begin{array}{l} F_{red} = 11.31 \text{ [N/mm}^2\text{]} \\ F_{red} = 13.10 \text{ [N/mm}^2\text{]} \end{array}$$

Normal column

$$UC = \frac{\sigma_N}{F_{red} \cdot \kappa_c} + \frac{\sigma_M}{F_{red}} \leq 1.0 = \frac{F_{ed}/A}{F_{red} \cdot \kappa_c} + \frac{M/W_y}{F_{red}} \leq 1.0$$

$$\rightarrow \frac{12337}{11.31 \cdot 0.56} + \frac{441000 / (\sigma^2 \cdot 38140)}{13.10} = 0.64 \text{ []}$$

Facade column

$$UC = \frac{F_{ed}/A}{F_{red} \cdot \kappa_c} + \frac{M/W_y}{F_{red}} = \frac{(13.59 \cdot 1000) / (38140)}{11.31 \cdot 0.56} + \frac{108368 / (\sigma^2 \cdot 38140)}{13.10} = 0.4$$

0.7 → calculated for 38140 in steel - "Verification in facade walls HSB"

SHEET is Validated

Validation holds also for facade-walls: 2-story
 - 3rd story is deleted from excel
 - Reason

Figure X.183: Validation HSB Wall (Single wall; combination 2) (3)

Order ✓
 Blad nr 1/3
 Deel Verification Housing separation walls
 Datum 24/05/2023

Note: later during design, compression perpendicular to grain is added (beam), which appeared to be a normative design parameter for the 3-storey walls. Resultingly, the size of the columns is changed. Note that still the validation holds, as the excel is not changed.

3-storey top-up (sep walls)

- Properties column (C24)

$b \times h = 300 \times 120 \text{ [mm]}$ $\rho = 420 \text{ [kg/m}^3\text{]}$
 $spac = 600 \text{ [mm]}$ $E_{0.05} = 7400 \text{ [N/mm}^2\text{]}$
 $CFL = 10.28 \text{ [m]}$ plywood 18 mm \rightarrow ssw $\text{[kg/m}^3\text{]}$
 $\text{Floor height} = 2.8 \text{ [m]}$ Load transp. = 0.35 CFL
 $\text{Span height gallery} = 0.6 \text{ [m]}$
-
- Load calculation (imposed load is leading action)

(1) Floor $\text{[N/m}^2\text{]}$ $\text{[m}^3\text{]}$ [m] [kN]

(a) permanent - normal = $0.7 \cdot 0.35 \text{ CFL} \cdot \text{spacing} = 0.7 \cdot 0.35 \cdot 10.28 \cdot 0.6 = 1.51$

(b) permanent - facade = $\text{exp } 0.7 \cdot 0.35 \text{ CFL} \cdot (\text{spacing} + \text{gallery}) = 0.7 \cdot 0.35 \cdot 10.28 \cdot (0.6 + 0.6) = 2.27$

a \hookrightarrow Variable load is excluded as $\psi_0 = 0$ for roof.

Normal column

eg 6.1 = $1.35 \cdot 0 = 1.35 \cdot 1.51 = 2.04 \text{ [kN]} \quad \delta$

eg 6.2 = $1.2 \cdot 0 = 1.2 \cdot 1.51 = 1.81 \text{ [kN]} \quad \delta$

Facade column

eg 6.1 = $1.35 \cdot 0 = 1.35 \cdot 2.27 = 2.06 \text{ [kN]} \quad \delta$

eg 6.2 = $1.2 \cdot 0 = 1.2 \cdot 2.27 = 2.72 \text{ [kN]} \quad \delta$

(2) 2nd and 3rd floor $\text{[kN/m}^2\text{]}$ [kN]

(a) permanent - normal = $1 \cdot 0.35 \text{ CFL} \cdot \text{spacing} = 1 \cdot 0.35 \cdot 10.28 \cdot 0.6 = 2.16$

(b) permanent - facade = $1 \cdot 0.35 \text{ CFL} \cdot (\text{spacing} + \text{gallery}) = 1 \cdot 0.35 \cdot 10.28 \cdot (0.6 + 0.6) = 3.24$

(a) Variable - normal [kN]

- imposed load = $1.75 \cdot 0.35 \text{ CFL} \cdot \text{spacing} = 1.75 \cdot 0.35 \cdot 10.28 \cdot 0.6 = 3.70$

- Non st. walls = $0.5 \cdot \dots = 0.5 \cdot 0.35 \cdot 10.28 \cdot 0.6 = 1.02$

Figure X.184: Validation HSB Wall (Housing separation wall) (1)

Order ✓
 Blad nr 2/3
 Deel Verification housing separation wall
 Datum 24/05/2023



b) Variable - Facade
 - imposed load = $1.75 \cdot 0.35 \cdot C_{FE} \cdot (\frac{2.00z}{2} + g_{all}) = 1.75 \cdot 0.35 \cdot 10.20 \cdot (\frac{0.6}{2} + 0.6) = 5.1$
 - Non-str walls = $0.5 \cdot \dots = 0.5 \cdot 0.35 \cdot 10.20 \cdot (\frac{0.6}{2} + 0.6) = 1.6$

Normal columns

eg b.1 = $1.35G + 1.5Q = (1.35 \cdot 2.16) + (1.5 \cdot 0.4 \cdot (3.70 + 1.00)) = 5.83 [kN]$
 eg b.2 = $1.2G + 1.5Q = (1.2 \cdot 2.16) + (1.5 \cdot 0.4 \cdot (3.70 + 1.00)) = 5.83 [kN]$

Facade columns

eg b.1 = $1.35G + 1.5Q = (1.35 \cdot 3.24) + (1.5 \cdot 0.4 \cdot (5.67 + 1.62)) = 8.75 [kN]$
 eg b.2 = $1.2G + 1.5Q = (1.2 \cdot 3.24) + (1.5 \cdot 0.4 \cdot (5.67 + 1.62)) = 8.75 [kN]$

3) Self-weight wall

a) permanent - normal

- columns = $(3.0/1000) \cdot (120/1000) \cdot 0.6 \cdot 3 \cdot 2.0 \cdot 420 \cdot \frac{10}{1000} = 0.16 [kN]$
 - Gypsum board = $0.14 \cdot 0.6 \cdot 3 \cdot 2.0 = 0.5 [kN]$
 - Plywood = $(1.0/1000) \cdot 550 \cdot \frac{10}{1000} \cdot 0.6 \cdot 3 \cdot 2.0 = 0.20 [kN]$
 = $0.37 [kN]$

b) permanent - Facade

- columns = $(3.0/1000) \cdot (120/1000) \cdot 0.6 \cdot 0.3 \cdot 3 \cdot 2.0 \cdot 420 \cdot \frac{10}{1000} = 0.24 [kN]$
 - Gypsum board = $0.14 \cdot (0.6/2) \cdot 3 \cdot 2.0 = 0.35 [kN]$
 - Plywood = $(1.0/1000) \cdot 550 \cdot \frac{10}{1000} \cdot \frac{0.6}{2} \cdot 3 \cdot 2.0 = 0.25 [kN]$
 - Gully beam = $0.07 [kN]$
 = $0.91 [kN]$

Normal columns

eg b.1 = $1.35G = 1.35 \cdot 1.37 = 1.85 [kN]$
 eg b.2 = $1.2G = 1.2 \cdot 1.37 = 1.64 [kN]$

Facade columns

eg b.1 = $1.35G = 1.35 \cdot 0.9 = 1.23 [kN]$
 eg b.2 = $1.2G = 1.2 \cdot 0.9 = 1.10 [kN]$

4) Front-facade

permanent load = $0.5 \cdot 0.35 \cdot C_{FE} \cdot 3 \cdot 2.0 = 1.5 [kN]$
 eg b.1 = $1.35G = 1.35 \cdot 15.11 = 20.40 [kN]$
 eg b.2 = $1.2G = 1.2 \cdot 15.11 = 18.13 [kN]$

Figure X.185: Validation HSB Wall (Housing separation wall) (2)

Order ✓
 Blad nr 3 / 3
 Deel Verification housing separation wall
 Datum 24/05/2023

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5) Total load

Normal columns

$$\text{eg } b_1 = 2.04 + (5.83 \cdot 2) + 1.85 = 15.55 \text{ kN}$$

$$\text{eg } b_2 = 1.81 + (8.51 \cdot 2) + 1.64 = 21.27 \text{ kN}$$

$$F_{tot} = 21.27 \text{ kN} \quad \rightarrow 21.26 \text{ kN} \quad (\text{slat value})$$

Facade columns

$$\text{eg } b_1 = (3.06 + (8.75 \cdot 2) + 1.23 + 30.40) / 3 = 14.06 \text{ kN}$$

$$\text{eg } b_2 = (2.72 + (13.47 \cdot 2) + 1.10 + 18.13) / 3 = 16.22 \text{ kN}$$

$$F_{tot} = 16.22 \text{ kN} \quad \rightarrow 16.22 \text{ kN}$$

• Calculation buckling factor

↳ supported around weak axis

$$I_y = \frac{1}{12} b h^3 = \frac{1}{12} \cdot 30 \cdot 120^3 = 5472000 \text{ mm}^4 \quad \checkmark$$

$$i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{5472000}{30 \cdot 120}} = 34.64 \text{ mm} \quad \checkmark$$

$$\lambda_y = \frac{l_{buck}}{i_y} = \frac{2800}{34.64} = 80.83 \quad \checkmark$$

$$\lambda_{rel} = \frac{\lambda_y}{\pi} \sqrt{\frac{F_{crit}}{E_{crit}}} = \frac{80.83}{\pi} \sqrt{\frac{21}{17400}} = 1.37 \text{ [-]} \quad \checkmark$$

$$k_y = 0.5(1 + \beta_1(\lambda_{rel} - 0.3) + \lambda_{rel}^2) = 0.5(1 + 0.2(1.37 - 0.3) + 1.37^2) = 1.55 \text{ [-]}$$

$$k_c = \frac{1}{k_y - \sqrt{k_y^2 - \lambda_{rel}^2}} = \frac{1}{1.55 - \sqrt{1.55^2 - 1.37^2}} = 0.44 \text{ [-]} \quad \checkmark \rightarrow 0.44 \text{ (slat)}$$

• Verification

$$\left. \begin{array}{l} k_{mod} = 0.7 \\ \gamma_M = 1.3 \\ F_{crit} = 21 \text{ N/mm}^2 \end{array} \right\} F_{red} = \frac{21 \cdot 0.7}{1.3} = 11.31 \text{ [N/mm}^2]$$

Normal columns

$$\sigma_{N} = F_{tot} / A = \frac{21.26}{2000 \cdot 10^3 / (30 \cdot 120)} = 4.66 \text{ [N/mm}^2]$$

$$UC = \frac{\sigma_N}{k_c \cdot F_{red}} = \frac{4.66}{0.44 \cdot 11.31} = 0.94 \text{ [-]} < 1.0 \quad \checkmark \rightarrow \text{slat } 0.94$$

Facade columns

$$UC = \frac{\sigma_N}{k_c \cdot F_{red}} = \frac{F_{tot} / A}{k_c \cdot F_{red}} = \frac{16.22 \cdot 10^3 / (30 \cdot 120)}{0.44 \cdot 11.31} = 0.71 < 1.0 \quad \checkmark \rightarrow \text{slat } 0.71$$

SHEET = validated ; Also holds for 2-story (roof: 1.26 + 1.50)

Figure X.186: Validation HSB Wall (Housing separation wall) (3)

X.11.4 Gallery beam

X.11.4.1 Floor level

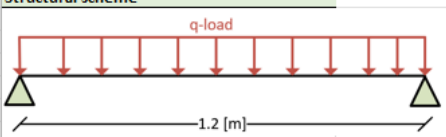
	A	B	C	D	E	F
1	Calculation gallery beam (floors)					
2	Safety factors (NEN-EN 1990)	V _G	V _Q	Formule B		
3	Eq 6.1	1,35	1,5			
4	Eq 6.2	1,2	1,5			
5						
6	Chosen beam					
7	Timber gallery beams					
8	E-modulus (C24)	11000	N/mm ²			
9	Density	420	kg/m ³			
10	Height	270	mm			
11	Width	70	mm			
12	Span	1,2	m			
13	CFL	10,28	m			
14	m _{beams}	7,94	kg/m	(B10/1000) * (B11/1000) * B9		
15	I _{beam}	1,15E+08	mm ⁴	(1/12)*B11*B10^3		
16	W _{beam}	8,51E+05	mm ³	(1/6)*B11*B10^2		
17	Number of beams	1	-			
18						
19	Structural scheme					
20						
21						
22						
23						
24						
25						
26						
27						
28	Loads					
29	Floors	kN/m ²	kN/m		Formule C	
30	Permanent	1		5,14	ψ ₀	B30*B13/2/B17
31	Variable				0,4	
32	Imposed loading	1,75		9,00		B32*B13/2/B17
33	Non-structural walls (safe consideration)	0,5		2,57		B33*B13/2/B17
34						
35			ULS [kN/m]		SLS [kN/m]	
36	eq 6.1			13,88		(B3*C30) + (C3*D31* (C32+C33))
37	eq 6.2			21,20		(B4*C30) + (C4*C32) + (C4*D31*C33)
38						
39	Self-weight beam					
40	Characteristic self-weight		0,08			Formule B
41						
42			ULS [kN/m]		SLS [kN/m]	
43	eq 6.1		0,11		0,08	B40*B3
44	eq 6.2		0,10		0,08	B40*B4
45						
46	Total					
47			ULS [kN/m]		SLS [kN/m]	
48	eq 6.1		13,99		9,85	C36+B43
49	eq 6.2		21,30		15,24	C37+B44
50	q _{ed}		21,30		15,24	MAX(B48:B49)
51						
52	Verification strength					
53	1. Bending moment					
54	Material properties (C24)					
55	Climate class	I/II	-			
56	Load duration	long-term	-			
57	k _{mod}	0,7	-			
58	k _{th}	1	-	ALS(B10<150; MIN((150/B10)^0,2; 1,3); 1)		
59	f _{m,k}	24	N/mm ²			
60	Y _M	1,3	-			
61	f _{m,d}	12,92	N/mm ²	B59*B57*B58/B60		
62						
63	Forces_ULS and verification					
64	M _{ed}	3,83E+06	Nmm	(1/8)*B50*(B12*1000)^2		
65	σ _M	4,51	N/mm ²	B64/B16		
66	UC1	0,35	-	B65/B61		
67						

Figure X.187: Validation Gallery beam (Floor level) (1)_Excel

	A	B	C	D	E	F
68	2. Shear force					
69	Material properties (C24)					
70	Climate class	I/II	-			
71	Load duration	long-term	-			
72	k_{mod}	0,7	-			
73	$f_{v,k}$	4	N/mm ²			
74	Y_M	1,3	-			
75	$f_{v,d}$	2,15	N/mm ²	B73*B72/B74		
76						
77	Forces_ULS and verification					
78	V	12779	N	0,5*B50*B12*1000		
79	T_V	1,01	N/mm ²	(3/2)*B78/(B10*B11)		
80	UC2	0,47	-	B79/B75		
81						
82	3. Load perpendicular to the grain					
83	V_{ed}	12779	N	B78		
84	Column width	76	(2 columns 38 x 89)			
85	Column height	89				
86	x_i	30	> 30 mm on both sides needed!	MIN(30; B84; (B12*1000))		
87	l_{eff}	136	mm	B84+(2*B86)		
88	A_{eff}	9520	mm ²	B87*MIN(B85;B11)		
89	$k_{c,90}$	1,25	-			
90	$\sigma_{c,90,d}$	1,34	N/mm ²	B83/B88		
91						
92	Material properties and verification					
93	Climate class	I/II	-			
94	Load duration	long-term	-			
95	k_{mod}	0,7	-			
96	$f_{c,90,k}$	2,5	N/mm ²			
97	Y_M	1,3	-			
98	$f_{c,90,d}$	1,35	N/mm ²	B96*B95/B97		
99	UC3	0,80	-	(B90)/(B98*B89))		
100						
101	Verification deflections					
102	Instantaneous deflections					
103	$u_{inst,G}$	0,11	mm	(5/384)*(C30+B40)*(B12*1000)^4 / (B8*B15)		
104	$u_{inst,Q1}$	0,19	mm	(5/384)*C32*(B12*1000)^4 / (B8*B15)		
105	$u_{inst,Qi}$	0,05	mm	(5/384)*C33*(B12*1000)^4 / (B8*B15)		
106	Final deflections					
107	k_{def} (Climate class I/II; Sawn wood)	0,8				
108	ψ_2	0,3				
109	$u_{fin,G}$	0,20	mm	B103*(1+B107)		
110	$u_{fin,Q1}$	0,24	mm	B104*(1+(B108*B107))		
111	$u_{fin,Qi}$	0,04	mm	B105*(D31 + (B108*B107))		
112	u_{fin}	0,47	mm	SOM(B109:B111)		
113	Verification					
114	$W_{allowed}$	4,8	mm	0,004*B12*1000		
115	UC4	0,10	-	B112/B114		
116						
117						
118	Most worst case scenario					
119	Properties		ULS			SLS
120	CFL	Beams	UC1	UC2	UC3	UC4
121	10,28	70 x 270	0,35	0,47	0,8	0,1

Figure X.188: Validation Gallery beam (Floor level) (2)_Excel

X.11.4.2 Roof level

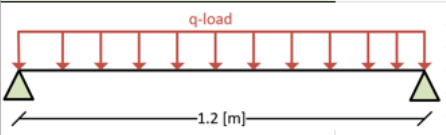
	A	B	C	D	E	F
1	Calculation gallery beam (roof)					
2	Safety factors (NEN-EN 1990)		V_G	V_Q	Formule B	
3	Eq 6.1	1,35		1,5		
4	Eq 6.2	1,2		1,5		
5						
6	Chosen beam					
7	Timber gallery beams					
8	E-modulus (C24)	11000	N/mm ²			
9	Density	420	kg/m ³			
10	Height	170	mm			
11	Width	70	mm			
12	Span	1,2	m			
13	CFL	10,28	m			
14	m_{beams}	5,00	kg/m	$(B10/1000) * (B11/1000) * B9$		
15	I_{beam}	2,87E+07	mm ⁴	$(1/12)*B11*B10^3$		
16	W_{beam}	3,37E+05	mm ³	$(1/6)*B11*B10^2$		
17	Number of beams	1	-			
18						
19	Structural scheme					
20						
21						
22						
23						
24						
25						
26						
27						
28	Loads					
29	Floors	kn/m ²	kn/m	Formule C		
30	Permanent	0,7	3,60	ψ_0	$B30*B13/2/B17$	
31	Variable			0		
32	Imposed loading	1	5,14		$B32*B13/2/B17$	
33						
34			ULS [kn/m]	SLS [kn/m]		
35	eq 6.1		4,86	3,60	$(B3*C30) + (C3*D31*C32)$	
36	eq 6.2		12,03	8,74	$(B4*C30) + (C4*C32)$	
37						
38	Self-weight beam	kn/m	Formule B			
39	Characteristic self-weight	0,05	$B14*10/1000$			
40		ULS [kn/m]	SLS [kn/m]			
41	eq 6.1	0,07	0,05	$B39*B3$		
42	eq 6.2	0,06	0,05	$B39*B4$		
43						
44						
45	Total		ULS [kn/m]	SLS [kn/m]		
46	eq 6.1	4,92	3,65	$C35+B42$		
47	eq 6.2	12,09	8,79	$C36+B43$		
48	q_{ed}	12,09	8,79	$MAX(B47:B48)$		
49						
50						
51	Verification strength					
52	1. Bending moment					
53	Material properties (C24)					
54	Climate class	I/II	-	$ALS(B10<150; MIN((150/B10)^0,2; 1,3); 1)$		
55	Load duration	long-term	-			
56	k_{mod}	0,7	-			
57	k_{th}	1	-			
58	$f_{m,k}$	24	N/mm ²			
59	γ_M	1,3	-	$B58*B56*B57/B59$		
60	$f_{m,d}$	12,92	N/mm ²			
61	Forces_ULS and verification					
62	M_{ed}	2,18E+06	Nmm	$(1/8)*B49*(B12*1000)^2$		
63	σ_M	6,45	N/mm ²	$B63/B16$		
64	UC1	0,50	-	$B64/B60$		
65						
66						
67	2. Shear force					
68	Material properties (C24)					
69	Climate class	I/II	-	$B72*B71/B73$		
70	Load duration	long-term	-			
71	k_{mod}	0,7	-			
72	$f_{v,k}$	4	N/mm ²			
73	γ_M	1,3	-			
74	$f_{v,d}$	2,15	N/mm ²			
75						

Figure X.189: Validation Gallery beam (Roof level) (1)_Excel

	A	B	C	D	E	F
75						
76	Forces_ULS and verification					
77	V	7253 N		0,5*B49*B12*1000		
78	T _v	0,91 N/mm ²		(3/2)*B77/(B10*B11)		
79	UC2	0,42	-	B78/B74		
80						
81	3. Load perpendicular to the grain					
82	V _{ed}	7253 N		B77		
83	Column width	76 (2 columns 38 x 89)				
84	Column height	89 (2 columns 38 x 89)				
85	x _i	30 > 30 mm on both sides needed!		MIN(30; B83; (B12*1000))		
86	l _{eff}	136 mm		B83+(B85*2)		
87	A _{eff}	9520 mm ²		B86*MIN(B84;B11)		
88	k _{c,90}	1,25	-			
89	σ _{c,90,d}	0,76 N/mm ²		B82/B87		
90						
91	Material properties and verification					
92	Climate class	I/II	-			
93	Load duration	long-term	-			
94	k _{mod}	0,7	-			
95	f _{c,90,k}	2,5 N/mm ²				
96	γ _M	1,3	-			
97	f _{c,90,d}	1,35 N/mm ²		B95*B94/B96		
98	UC3	0,45	-	(B89)/(B97*B88))		
99						
100	Verification deflections					
101	Instantaneous deflections					
102	u _{inst,G}	0,31 mm		(5/384)*(C30+B39)*(B12*1000) ⁴ / (B8*B15)		
103	u _{inst,Q1}	0,44 mm		(5/384)*C32*(B12*1000) ⁴ / (B8*B15)		
104	Final deflections					
105	k _{def} (Climate class I/II; Sawn wood)	0,8				
106	ψ ₂	0				
107	u _{fin,G}	0,56 mm		B102*(1+B105)		
108	u _{fin,Q1}	0,44 mm		B103*(1+(B106*B105))		
109	u _{fin}	1,00 mm		SOM(B107:B108)		
110	Verification					
111	w _{allowed}	4,8 mm		0,004*B12*1000		
112	UC4	0,21	-	B109/B111		
113						
114						
115	Most worst case scenario					
116	Properties		ULS		SLS	
117	CFL	Beams	UC1	UC2	UC3	UC4
118	10,28	70 x 170	0,5	0,42	0,45	0,21

Figure X.190: Validation Gallery beam (Roof level) (2)_Excel

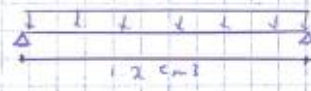
X.11.4.3 Hand validation

Order ✓
 Blad nr 1/2
 Deel Verificatie gallery beam (Afor)
 Datum 24/05/2023

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- Properties beam (24)

E -modulus = 11000 [N/mm²]
 ρ = 420 [kg/m³]
 $b \times h$ = 70 x 270 [mm²]
 Span = 1.2 [m]
 CFL = 0.28 [m]



→ Mass beams = (70/1000) (270/1000) 420 = 7.94 [kg/m] ✓
 → I beams = $\frac{1}{12} bh^3 = \frac{1}{12} \cdot 70 \cdot 270^3 = 114817500$ [mm⁴] ✓
 → W beams = $\frac{1}{6} bh^2 = \frac{1}{6} \cdot 70 \cdot 270^2 = 850500$ [mm³] ✓
- Loads [N/m]

permanent: $1 \cdot \frac{CFL}{2} = 1 \cdot \frac{0.28}{2} = 0.14$ [kN/m]
 Variable:
 → imposed $1.75 \cdot \frac{CFL}{2} = 1.75 \cdot \frac{0.28}{2} = 0.245$ [kN/m]
 → Non-st. ult. $0.5 \cdot \frac{CFL}{2} = 0.5 \cdot \frac{0.28}{2} = 0.07$ [kN/m]

eg G1 = $1.35 \cdot 0.14 + 1.5 \cdot 0.245 = (1.35 \cdot 0.14) + (1.5 \cdot 0.4) \cdot (1 + 2.57) = 13.8$ [kN/m] ✓
 eg G2 = $1.2 \cdot 0.14 + 1.5 \cdot 0.245 = (1.2 \cdot 0.14) + (1.5 \cdot 0.4) \cdot (1 + 2.57) = 21.21$ ✓

Self-weight beam

→ channel st.c = 0.08 [kN/m]

eg G1 ⇒ $1.35 \cdot 0.08 = 0.11$ [kN/m]
 eg G2 ⇒ $1.2 \cdot 0.08 = 0.10$ [kN/m]

Total loads

eg G1 ⇒ $13.8 + 0.11 = 13.91$ [kN/m] ✓
 eg G2 ⇒ $21.21 + 0.10 = 21.31$ [kN/m] ✓
 G_{ed} = 21.31 [kN/m] ✓ ← 21.30 [kN/m] (last value)
- Verification strength

1. Bending
 $K_{red} = 1$
 $M_{ed} = \frac{G_{ed} l^2}{8} = \frac{21.31 \cdot 1200^2}{8} = 3835200$ [Nmm]
 $W = 850500$ [mm³]
 $\sigma_{M} = \frac{M}{W} = \frac{3835200}{850500} = 4.51$ [N/mm²] ✓

$K_{red} = 0.7$
 $K_A = 1$
 $\gamma_M = 1.1$
 $f_{yEd} = 24$

$f_{yEd} = 12.92$ [N/mm²]
 $\hookrightarrow UC = \frac{4.51}{12.92} = 0.35$ [-] ✓ (↔ 0.35 > 0.1)

Figure X.191: Validation Gallery beam (1)

Order /
 Blad nr 2/2
 Deel Verification gallery beam (floor)
 Datum 24/05/2023



2 Slab force

$$V_{ed} = \frac{1}{2} g L = \frac{1}{2} \cdot 21.31 \cdot 1200 = 12786 \text{ [N]}$$

$$Z_{ed} = \frac{1}{2} \frac{V_{ed}}{b h} = \frac{1}{2} \cdot \frac{12786}{70 \cdot 270} = 1.01 \text{ [N/mm}^2\text{]}$$

$$\left. \begin{matrix} \gamma_{mod} = 0.7 \\ \gamma_M = 1.3 \\ \gamma_{FR} = 4 \end{matrix} \right\} F_{ed} = 2.15 \text{ [N/mm}^2\text{]}$$

$$L_{> UC} = \frac{1.01}{2.15} = 0.47 \text{ [-]} \quad \text{OK} \rightarrow 0.47 \text{ (slab)}$$

3 load per perpendicular to the gully

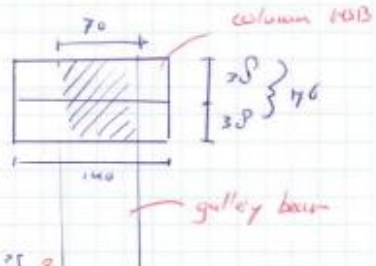
$$V_{ed} = 12786 \text{ [N]}$$

$$x_i = \min(30; 76; 1200) = 30 \text{ [mm]} \quad \text{span}$$

$$l_{eff} = 176 + 2x_i = 136 \text{ [mm]}$$

$$A_{dF} = l_{eff} \cdot \text{width } 70 = 136 \cdot 70 = 9520 \text{ [mm}^2\text{]}$$

$$k_{c,90} = 1.75$$



$$L_{> \sigma_{c,90}} = \frac{V_{ed}}{A_{dF}} = \frac{12786}{9520} = 1.34 \text{ [N/mm}^2\text{]} \quad \text{OK}$$

$$\left. \begin{matrix} \gamma_{mod} = 0.7 \\ F_{c,90,d} = 2.5 \text{ [N/mm}^2\text{]} \\ \gamma_M = 1.3 \end{matrix} \right\} F_{c,90,d} = 1.35 \text{ [N/mm}^2\text{]}$$

$$UC = \frac{\sigma_{c,90}}{F_{c,90,d}} = \frac{1.34}{1.35 \cdot 1.75} = 0.80 \text{ [-]} \quad \text{OK} \rightarrow \text{slab value} = 0.80 \text{ [-]}$$

Verification deflection

$$u_{fin, G} = \left(\frac{1}{384} g L^4 / EI \right) \cdot (1 + \mu \beta^2) = \left[\frac{1}{384} \cdot (5.14 + 0.08) \cdot 1200^4 / (11000 \cdot 1.15 \cdot 10^8) \right] \cdot (1 + 0.9) = 0.20 \text{ [mm]} \quad \text{OK}$$

$$u_{fin, Q_1} = \left(\frac{1}{192} g L^4 / EI \right) \cdot (1 + 4\mu \beta^2) = \left[\frac{1}{192} \cdot (9.00 + 1200^4 / (11000 \cdot 1.15 \cdot 10^8)) \right] \cdot (1 + 0.3 \cdot 0.9) = 0.24 \text{ [mm]} \quad \text{OK}$$

$$u_{fin, Q_2} = \left(\frac{1}{384} g L^4 / EI \right) \cdot (1 + 4\mu \beta^2) = \left[\frac{1}{384} \cdot 2.57 \cdot 1200^4 / (11000 \cdot 1.15 \cdot 10^8) \right] \cdot (1 + 0.4 \cdot 0.7 \cdot 0.9) = 0.04 \text{ [mm]} \quad \text{OK}$$

$$u_{fin} = 0.20 + 0.24 + 0.04 = 0.48 \text{ [mm]} \quad \text{OK} \rightarrow 0.47 \text{ [mm]} \text{ (slab)}$$

$$UC = \frac{u_{fin}}{0.004 \cdot 1200} = \frac{0.47}{0.004 \cdot 1200} = 0.1 \text{ [-]} \quad \text{OK} \rightarrow 0.01 \text{ (slab)}$$

SHEET = validated; → holds also for gully beam roof

Figure X.192: Validation Gallery beam (2)

X.11.5 Gallery column

X.11.5.1 1-storey


	A	B	C	D	E	F	G	H	I	J	K	L
1	Calculation gallery column_1 storey											
2	Safety factors (NEN-EN 1990)											
3	Eq 6.1	Y _G	Y _Q									
4	Eq 6.2		1,35	1,5								
5			1,2	1,5								
6	Chosen column											
7	Timber column											
8	E-modulus (C24)	11000	N/mm ²	Formule B								
9	E _{0,05}	7400	N/mm ²									
10	Density	420	kg/m ³									
11	f _{c,0,k}	21	N/mm ²									
12	Width (Short axis)	70	mm									
13	Length (Long axis)	70	mm									
14	A	4900	mm ²	B12*B13								
15	Span gallery beam	1,2	m									
16	CFL	10,28	m									
17	m _{column}	2,06	kg/m	(B12/1000) * (B13/1000) * B10								
18	Height floor	2,80	m									
19	Number of stories	1,00	-									
20	I _{column}	2000833,33	mm ⁴	(1/12)*B13*B12^3								
21	W _{column}	57166,6667	mm ³	(1/6)*B13*B12^2								
22												
23	Structural scheme											
24												
25												
26												
27												
28												
29												
30												
31												
32												
33												
34												
35												
36												
37												
38	Load (normative = lowest column)											
39	Roof (Full loaded)											
40	Permanent load	kN/m ²	kN					Formule C				
41	Variable load							B40*(B16/2)*(B15/2)				
42	Imposed load							B42*(B16/2)*(B15/2)				
43	eq 6.1	0,7	2,16					(B3*C40) + (C3*D41*C42)				
44	eq 6.2	1	3,08					(B4*C40) + (C4*C42)				
45												
46	Self-weight column											
47	Permanent load		kN					B19*B18*B17*10/1000				
48	eq 6.1	0,06	0,08					B3*C47				
49	eq 6.2	0,07	0,07					B4*C47				
50												
51	Total load											
52	eq 6.1	2,99						C43+C48				
53	eq 6.2	7,29						C44+C49				
54	F _{ed}	7,29						MAX(B52:B53)				
55												
56	Verification strength											
57	Buckling analysis											
58	Reduction factor (I_y = I_x)											
59	I _y	2,00E+06	mm ³	(1/12)*B13*B12^3								
60	i _y	20,21	mm	WORTEL(B59/B14)								
61	L _{buc}	2800	mm	B18*1000								
62	λ _y	138,56	-	B61/B60								
63	λ _{rel}	2,35	-	(B62/PI())*WORTEL(B11/B9)								
64	k _y	3,47	-	0,5*(1 + (0,2*(B63-0,3)) + B63^2)								
65	k _c	0,17	-	1/(B64+WORTEL(B64^2 - B63^2))								
66												
67	Material properties											
68	Climate class	I/II	-									
69	Load duration	long-term	-									
70	k _{mod}	0,7	-									
71	f _{c,0,k}	21	N/mm ²	B11								
72	f _{m,k}	24	N/mm ²									
73	γ _M	1,3	-									
74	f _{c,d}	11,31	N/mm ²	B71*B70/B73								
75	f _{m,d}	12,92	N/mm ²	B72*B70/B73								
76												
77	Forces and unity check											
78	σ _N	1,49	N/mm ²	B54*1000/B14								
79	UC	0,79	-	(B78/(B65*B74))								
80												
81	Results 1 storey											
82	CFL	Column	UC	UC (44x70)								
83	6.70 - 8.49	70 x 70	0,65	2,54								
84	8.49 - 10.28	70 x 70	0,79	3,07								

Figure X.193: Validation Gallery column (1-storey) _Excel

X.11.5.2 2-storey

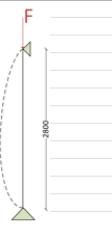
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	
1	Calculation gallery column_2-storey														
2	Safety factors (NEN-EN 1990)														
3	Eq 6.1	V _g	V _d												
4	Eq 6.2	1,35	1,5												
5		1,2	1,5												
6	Chosen column														
7	Timber column														
8	E-modulus (C24)	11000	N/mm ²	Formule B											
9	E _{0,05}	7400	N/mm ²												
10	Density	420	kg/m ³												
11	f _{c,0,k}	21	N/mm ²												
12	Width (Short axis)	95	mm												
13	Length (Long axis)	95	mm												
14	A	9025	mm ²												
15	Span gallery beam	1,2	m												
16	CFL	10,28	m												
17	m _{column}	3,79	kg/m		(B12/1000) * (B13/1000) * B10										
18	Height floor	2,80	m												
19	Number of stories	2,00	-												
20	I _{column}	6787552,08	mm ⁴	(1/12)*B13*B12^3											
21	W _{column}	142895,833	mm ³	(1/6)*B13*B12^2											
22															
23	Structural scheme														
24															
25															
26															
27															
28															
29															
30															
31															
32															
33															
34															
35															
36															
37															
38	Load (normative = lowest column)														
39	Roof (Full loaded)														
40	Permanent load	0,7	2,16		Formule C										
41	Variable load			ψ ₀		B40*(B16/2)*(B15/2)									
42	Imposed load	1	3,08	0		B42*(B16/2)*(B15/2)									
43	eq 6.1		2,91		(B3*C40) + (C3*D41*C42)										
44	eq 6.2		7,22		(B4*C40) + (C4*C42)										
45															
46	2nd gallery floor (Full loaded)														
47	Permanent load	1	3,08		ψ ₀	B47*(B16/2)*(B15/2)									
48	Variable			0,4		B49*(B16/2)*(B15/2)									
49	Imposed load	1,75	5,40			B49*(B16/2)*(B15/2)									
50	Non-structural walls	0,5	1,54		(B3*C47)+(C3*D48*(C49+C50))										
51	eq 6.1		8,33		(B4*C47) + (C4*C49) + (C4*C50*D48)										
52	eq 6.2		12,72												
53															
54	Self-weight column														
55	Permanent load		0,21		B19*B18*B17*10/1000										
56	eq 6.1		0,29			B3*C55									
57	eq 6.2		0,25			B4*C55									
58															
59	Total load														
60	eq 6.1	11,53			C43+C51+C56										
61	eq 6.2	20,19				C44+C52+C57									
62	F _{ed}	20,19				MAX(B60;B61)									
63															
64	Verification strength														
65	Buckling analysis														
66	Reduction factor (I_y = I_x)														
67	I _y	6,79E+06	mm ⁴		Formule B	(1/12)*B13*B12^3									
68	I _x	27,42	mm			WORTEL(B67/B14)									
69	L _{0,UC}	2800	mm			B18*1000									
70	λ _y	102,10	-			B69/B68									
71	λ _{rel}	1,73	-			(B70/PI())*WORTEL(B11/B9)									
72	k _y	2,14	-			0,5*(1 + (0,2*(B71-0,3)) + B71^2)									
73	k _c	0,29	-			1/(B72+WORTEL(B72^2 - B71^2))									
74															
75	Material properties														
76	Climate class	I/II	-			B11									
77	Load duration	long-term	-												
78	k _{mod}	0,7	-												
79	f _{c,0,k}	21	N/mm ²												
80	f _{tr,k}	24	N/mm ²												
81	γ _{Mt}	1,3	-												
82	f _{c,0,d}	11,31	N/mm ²												
83	f _{tr,d}	12,92	N/mm ²												
84															
85	Forces and unity check														
86	σ _M	2,24	N/mm ²		B62*1000/B14										
87	UC	0,67	-			(B86/(B73*B82))									
88															
89	Results 2 stories														
90	CFL	Column	UC	UC (70 x 70)											
91	6.70 - 8.49	95 x 95	0,56	1,8											
92	8.49 - 10.28	95 x 95	0,67	2,18											

Figure X.194: Validation Gallery column (2-storey) _Excel

X.11.5.3 3-storey


	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Calculation gallery column 3_storey													
2	Safety factors (NEN-EN 1990)													
3	Eq 6.1	V ₀	1,35	V ₀	1,5									
4	Eq 6.2		1,2		1,5									
5														
6	Chosen column													
7	Timber column													
8	E-modulus (C24)	11000	N/mm ²											
9	E _{0,05}	7400	N/mm ²											
10	Density	420	kg/m ³											
11	f _{c,0,k}	21	N/mm ²											
12	Width (Short axis)	95	mm											
13	Length (Long axis)	95	mm											
14	A	9025	mm ²	B12*B13										
15	Span gallery beam	1,2	m											
16	CFL	10,28	m	Formule B										
17	m _{column}	3,79	kg/m	(B12/1000) * (B13/1000) * B10										
18	Height floor	2,80	m											
19	Number of stories	3,00	-											
20	I _{column}	6787552,08	mm ⁴	(1/12)*B13*B12^3										
21	W _{column}	142895,833	mm ³	(1/6)*B13*B12^2										
22														
23	Structural scheme													
24														
25														
26														
27														
28														
29														
30														
31														
32														
33														
34														
35														
36														
37														
38	Load (normative = lowest column)													
39	Roof													
40	Permanent load	0,7	kN/m ²	2,16	ψ ₀	B40*(B16/2)*(B15/2)								
41	Variable load				0									
42	Imposed load	1	kN/m ²	3,08		B42*(B16/2) *(B15/2)								
43	eq 6.1			2,91		(B3*C40) + (C3*D41*C42)								
44	eq 6.2			2,59		(B4*C40) + (C4*C42*D41)								
45														
46	3rd gallery floor (Full loaded)													
47	Permanent load	1	kN/m ²	3,08	ψ ₀	B47*(B16/2) *(B15/2)								
48	Variable				0,4									
49	Imposed load	1,75	kN/m ²	5,40		B49*(B16/2) *(B15/2)								
50	Non-structural walls	0,5	kN/m ²	1,54		B50*(B16/2) *(B15/2)								
51	eq 6.1			8,33		(B3*C47) + (C3*D48*(C49+C50))								
52	eq 6.2			12,72		(B4*C47) + (C4*C49) + (C4*D48*C50)								
53														
54	2nd gallery floor (Full loaded)													
55	Permanent load	1	kN/m ²	3,08	ψ ₀	B55*(B16/2)*(B15/2)								
56	Variable				0,4									
57	Imposed load	1,75	kN/m ²	5,40		B57*(B16/2)*(B15/2)								
58	Non-structural walls	0,5	kN/m ²	1,54		B58*(B16/2)*(B15/2)								
59	eq 6.1			8,33		(B3*C55)+(C3*D56*(C57+C58))								
60	eq 6.2			12,72		(B4*C55) +(C4*C57) +(C4*C58*D56)								
61														
62	Self-weight column													
63	Permanent load			0,32		B19*B18*B17*10/1000								
64	eq 6.1			0,43		B3*C63								
65	eq 6.2			0,38		B4*C63								
66														
67	Total load													
68	eq 6.1			20,00		C43+C51+C59+C64								
69	eq 6.2			28,42		C44+C52+C60+C65								
70	F _{tot}			28,42		MAX(B68:B69)								
71														
72	Verification strength													
73	Buckling analysis													
74	Reduction factor (I_y = I_x)													
75	I _y	6,79E+06	mm ³			(1/12)*B13*B12^3								
76	i _y	27,42	mm			WORTEL(B75/B14)								
77	L _{0,02}	2800	mm			B18*1000								
78	λ _y	102,10	-			B77/B76								
79	λ _{rel}	1,73	-			(B78/PI())*WORTEL(B11/B9)								
80	k _y	2,14	-			0,5*(1 + (0,2*(B79-0,3)) + B79^2)								
81	k _c	0,29	-			1/(B80+WORTEL(B80^2 - B79^2))								
82														
83	Material properties													
84	Climate class	I/II	-											
85	Load duration	long-term	-											
86	k _{mod}	0,7	-											

Figure X.195: Validation Gallery column (3-storey) (1) _Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
86		k_{mod}	0,7 -											
87		$f_{c,0,k}$	21 N/mm ²	B11										
88		$f_{t,k}$	24 N/mm ²											
89		γ_M	1,3 -											
90		$f_{c,0,d}$	11,31 N/mm ²	B87*B86/B89										
91		$f_{t,d}$	12,92 N/mm ²	B88*B86/B89										
92														
93	Forces and unity check													
94		σ_N	3,15 N/mm ²	B70*1000/B14										
95		UC	0,95 -	(B94/(B81*B90))										
96														
97	Results 3 stories													
98		CFL	Column	UC	UC (70 x 70)									
99		6.70 - 8.49	95 x 95	0,78	2,53									
100		8.49 - 10.28	95 x 95	0,95	3,06									

Figure X.196: Validation Gallery column (3-storey) (2) _Excel

X.11.5.4 Hand validation

Order /
 Blad nr 1/2
 Deel Verification column gallery
 Datum 24/05/2023



Verification for 3-20103

- Properties column (C24)
 E -modulus = $11000 \text{ [N/mm}^2\text{]}$
 $E_{0.05}$ = $71000 \text{ [N/mm}^2\text{]}$
 ρ = $420 \text{ [kg/m}^3\text{]}$
 $b \times h$ = $95 \times 95 \text{ [mm}^2\text{]}$
 A = $95 \times 95 = 9025 \text{ [mm}^2\text{]}$
 Span gallery = 10.28 [m]
 CFL = 10.28 [m]
 mass-column = $(95/1000) \cdot (95/1000) \cdot 420 = 3.75 \text{ [kg/m]}$
 Height floor = 2.8 [m]
- Load definition

1) Roof

	$\text{[N/m}^2\text{]}$	[m]	[m]	[kN]
permanent	0.7	$CFL/2$	$Span/2$	$0.7 \cdot 10.28/2 \cdot 10.28/2 = 2.16$
Variable	1	$CFL/2$	$Span/2$	$1 \cdot 10.28/2 \cdot 10.28/2 = 2.60$

eg 6.1 = $1.35G + 1.5\psi_0Q = (1.35 \cdot 2.16) + (1.5 \cdot 0.3 \cdot 2.60) = 2.91 \text{ [kN]}$
 eg 6.2 = $1.2G + 1.5\psi_0Q = (1.2 \cdot 2.16) + (1.5 \cdot 0.3 \cdot 2.60) = 2.95 \text{ [kN]}$

2) 2nd and 3rd floor

	$\text{[N/m}^2\text{]}$	[m]	[m]	[kN]
permanent	1	$CFL/2$	$Span/2$	$1 \cdot 10.28/2 \cdot 10.28/2 = 2.60$
Variable				
→ imposed	1.75			$1.75 \cdot 10.28/2 \cdot 10.28/2 = 5.40$
→ Non-structural	0.5			$0.5 \cdot 10.28/2 \cdot 10.28/2 = 1.54$

eg 6.1 = $1.35G + 1.5\psi_0Q = (1.35 \cdot 2.60) + (1.5 \cdot 0.4 \cdot (5.40 + 1.54)) = 8.32 \text{ [kN]}$
 eg 6.2 = $1.2G + 1.5\psi_0Q + 1.5\psi_0Q = (1.2 \cdot 2.60) + (1.5 \cdot 0.4 \cdot 5.40) + (1.5 \cdot 0.4 \cdot 1.54) = 12.72 \text{ [kN]}$

3) Self-weight column

permanent load = $mass \cdot height = 3.75 \frac{\text{kg}}{\text{m}} \cdot 2.8 = 0.32 \text{ [kN]}$

eg 6.1 = $1.35G = 1.35 \cdot 0.32 = 0.43 \text{ [kN]}$
 eg 6.2 = $1.2G = 1.2 \cdot 0.32 = 0.38 \text{ [kN]}$

4) Total load (dead)

eg 6.1 = $2.91 + (2.8 \cdot 3.2) + 0.43 = 19.98 \text{ [kN]}$
 eg 6.2 = $2.95 + (2.8 \cdot 12.72) + 0.38 = 28.41 \text{ [kN]}$

→ Fact = 28.41 [kN] → 28.42 [kN] (start)

Figure X.197: Validation Gallery column (1)

Order ✓
 Blad nr 2 / 2
 Deel Verification column gallery
 Datum 24/05/2023

VAN ROSSUM RAADGEVENDE
 INGENIEURS

Verification strength

→ Reduction factor buckling

$$I_y = I_z = \frac{1}{12} b h^3 = \frac{1}{12} \cdot 45^4 = 61787552 \text{ [mm}^4\text{]} \quad \&$$

$$I_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{61787552}{9025}} = 27.42 \text{ [mm]} \quad \&$$

$$\lambda_y = \frac{L_{buck}}{I_y} = \frac{2800}{27.42} = 102.1 \text{ [-]} \quad \&$$

$$\lambda_{rel} = \frac{\lambda_y}{\pi} \sqrt{\frac{F_{E05}}{E_{005}}} = \frac{102.1}{\pi} \sqrt{\frac{21}{7000}} = 1.73 \text{ [-]} \quad \&$$

$$k_y = 0.5 \cdot (1 + \beta (\lambda_{rel} - 0.3) + \lambda_{rel}^2) = 0.5 \cdot (1 + 0.2(1.73 - 0.3) + 0.3^2) = 0.14 \text{ [-]}$$

$$k_y = \frac{1}{\sqrt{1 + \lambda_{rel}^2}} = \frac{1}{\sqrt{1 + 1.73^2}} = 0.25 \text{ [-]} \quad \& \text{ See 0.25 (Steel)}$$

→ Material properties

$$\left. \begin{array}{l} k_{red} = 0.7 \\ k_{0.2} = 21 \text{ N/mm}^2 \\ \gamma_M = 1.3 \end{array} \right\} F_{red} = \frac{21 \cdot 0.7}{1.3} = 11.31 \text{ [N/mm}^2\text{]}$$

$$UL = \frac{\sigma_w}{k_y \cdot F_{red}} = \frac{F_{act}/A}{k_y \cdot F_{red}} = \frac{2800 \cdot 10^3 / 9025}{0.25 \cdot 11.31} = 0.96 \text{ [-]} < 1.0$$

$\&$ Steel value = 0.95

SHEET is Validated Works also for 2-story column gallery
 (same steel used, only 1 story deleted and roof 1.26 + 1.5 Q)

Figure X.198: Validation Gallery column (2)

X.11.6 HSB stability wall

X.11.6.1 Tension in wall

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	
1	Verification tension in stability wall_option 2				Formule B											
2	Parameters geometry															
3	Ratio: Depth / length stability wall	4,5	-													
4	Depth building	10,85	m													
5	Width unit building	18	m													
6	Storey height	2,8	m													
7	Number of floors	3	-													
8																
9	Contr. width 2nd floor	0	m													
10	Contr. Width other floors	0	m													
11																
12	Wall parameters															
13	Length stability wall	2,41	m	B4/B3												
14	Longitudinal direction															
15	NFL	2,67	m													
16	Floorlength a	2,47	m													
17	Floorlength b	1,44	m	B15-(0,5*B16)												
18																
19	Transversal direction															
20	Half floor span	1,96	m													
21																
22	Structural applied weights															
23	Timber roof structure (Sheet roof)	0,7	kN/m ²													
24	Timber floor (Sheet floor)	1	kN/m ²													
25																
26	Timber wall (double) (Sheet walls)															
27	Columns	0,06	kN/m ²													
28	Gypsum board	0,28	kN/m ²													
29	Plywood (18 mm)	0,20	kN/m ²													
30		0,54	kN/m ²	SOM(B27-B29)												
31	Weight per meter wall length	1,52	kN/m/storey	B30*B6												
32																
33	Safety factors (NEN1990)															
34	Permanent load	0,9	-													
35	Variable load	1,5	-													
36																
37																
38	Verification in longitudinal direction															
39	Loads caused by self-weight (assume no contributing width)				Formule C			Formule D			Formule E					
40		kN/m ²	Pointload left/right [kN]	Pointload middle [kN]	kN/m											
41	Roof structure	0,7	0,00	0,00		B41*B17*B10		B41*B16*B10								
42	3rd top-up floor	1	0,00	0,00		B42*B17*B10		B42*B16*B10								
43	2nd top-up floor	1	0,00	0,00		B43*B17*B9		B43*B16*B9								
44	1st top-up floor	> does not transfer load to stability wall														
45	Walls															
46	Stability wall				4,55											
47	wall 3rd floor			0,00	0,00	B31*B10		B31*B10						B31*B7		
48	wall 2nd floor			0,00	0,00	B31*B9		B31*B9								
49	Total			0,00	0,00	4,55		C41+C42+C43+C47+C48		D41+D42+D43+D47+D48				E46		
50																
51	Compressive force SLS	5,49	kN													
52	Compressive force ULS	4,94	kN	Formule B C49+(D49/2)+(0,5*E49*B13)												
53					B34*B51											
54	Tension caused by windload															
55	Number of porches considered (Horizontal linking of	4	-													
56	q _p (22.4m) (Wind area II; Urban)	0,93	kN/m ²													
57	C _s C _d	1	-													
58	C _{f,D}	0,8	-													
59	C _{f,E}	0,7	-													
60	C _{f,tot}	0,8	> only suction or pressure	MAX(B58;B59)												
61	q _{wind,SLS}	2,02	kN/m	B56*B57*B60*B4/B55												
62	q _{wind,ULS}	3,03	kN/m	B61*B35												
63	M _{wind,ULS}	1,07E+08	Nmm	(1/2)*B62*(B7*B6*1000)^2												
64	T _{ed}	44,29	kN	B63/(B13*1000)/1000												
65	Tension anchor	39,36	kN	B64-B52												
66																
67	Tension anchorage															
68	Tension per wall side (Housing sep. wall)	19,68	kN	B65/2												
69	Capacity per Anchor (WHT440)	21,3	kN													
70	Number of anchors	1	-	AFRONDEN BOVEN(B68/B69;1)												
71																
72																
73	Verification in transversal direction															
74	Loads caused by self-weight															
75		kN/m ²	kN/m		Formule C											
76	Roof	0,7	1,372		B76*B20											
77	3rd top-up floor	1	1,96		B77*B20											
78	2nd top-up floor	1	1,96		B78*B20											
79	Wall		4,55		B31*B7											
80			9,84		SOM(C76;C79)											
81	Compression force per side	53,40	kN		C80*B4/2											
82	Compression force_ed	48,06	kN		B81*B34											
83																
84	Tension caused by windload				Formule B											
85	q _p (22.4m) (Wind area II; Urban)	0,93	kN/m ²													
86	C _s C _d	1	-													
87	C _{f,D}	0,8	-													
88	C _{f,E}	0,7	-													
89	C _{f,tot}	1,5	-	SOM(B87;B88)												
90	q _{wind,SLS}	4,10	kN/m	B85*B86*B89*2*B20*0,75												
91	q _{wind,ULS}	6,15	kN/m	B90*B35												
92	M _{wind,ULS}	2,17E+08	Nmm	(1/2)*B91*(B7*B6*1000)^2												
93	T _{ed}	20,00	kN	B92/(B4*1000)/1000												
94	Compression (> No tension)	28,06	kN	B82-B93												

Figure X.199: Stability (Tension in wall) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
95															
96															
97	Maximal compression due to longitudinal wind load														
98	Compressive force														
99	Compression due to wind	44,29 kN		B64											
100	Compression due to self-weight	4,94 kN		B52											
101	Total	49,23 kN		B99+B52											
102	Number of columns	4	-	((B13/2/0,6)*2)											
103	Force per column	12,25 kN		B101/B102											
104															
105	Column properties														
106	k_c	0,44	-												
107	Area	4560 mm ²													
108	σ_N	2,69 N/mm ²		B103*1000/B107											
109															
110	Climate class	I	-												
111	Load duration	long-term	-												
112	k_{mod}	0,7	-												
113	$f_{c,d,k}$	21 N/mm ²													
114	γ_M	1,3	-												
115	$f_{c,d,d}$	11,31 N/mm ²		B113*B112/B114											
116	UC	0,54	-	B108/(B106*B115)											
117															
118	Compression perpendicular to grain (Beam below wall)														
119	$k_{c,90}$	1,25	-												
120	χ_i	30,00		MIN(50; 600/2; 38)											
121	l_{eff}	98,00 mm		38*(2*B120)											
122	A_{eff}	11760,00 mm ²		120*B121											
123	σ_N	1,04 N/mm ²		B103*1000/B122											
124	$f_{c,90,k}$	2,50 N/mm ²													
125	$f_{c,90,d}$	1,35 N/mm ²		B124*B112/B114											
126	UC	0,62	-	B123/(B119*B125)											
127															
128	Maximal compression due to transversal wind load														
129	Compressive force														
130	Compression due to wind	20,00 kN		B93											
131	Compression due to self-weight	48,06 kN		B82											
132	Total	68,06 kN		SOM(B130:B131)											
133	Number of columns considered	9	-	(B4/2/0,6)											
134	Force per column	7,53 kN		B132/B133											
135															
136	Column properties														
137	k_c	0,44	-												
138	Area	4560 mm ²													
139	σ_N	1,65 N/mm ²		B134*1000/B138											
140															
141	Climate class	I	-												
142	Load duration	long-term	-												
143	k_{mod}	0,7	-												
144	$f_{c,d,k}$	21 N/mm ²													
145	γ_M	1,3	-												
146	$f_{c,d,d}$	11,31 N/mm ²		B144*B143/B145											
147	UC	0,33	-	B139/(B137*B146)											

Figure X.200: Stability (Tension in wall) (2)_Excel

X.11.6.2 Capacity of nails

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1	Calculation capacity nails (3.1x50)															
2	Fastener properties			Formule B					*							
3	d	3,1 mm							Round nails	15 %						
4	Length	50 mm							Squared/profiled nails	25 %						
5	$f_{t,k}$	400 N/mm ²							Other nails	50 %						
6	Type of fastener limit	15 % of Johansen part*							Screws	100 %						
7	$M_{y,Rk}$	2273,64		0,3*B5*(B3^2,6)					Bolds	25 %						
8									Dowels	0 %						
9	Multiplex properties															
10	t_1	18 mm														
11	ρ_k	470 kg/m ³														
12	$f_{t,1,k}$	36,82 N/mm ²		0,11*B11*(B3^0,3)												
13																
14	Softwood properties			Formule B												
15	t_2	32 mm	<wood thickness	B4-B10												
16	ρ_k	350 kg/m ³														
17	$f_{t,2,k}$	27,81 N/mm ²		0,082*(1-(0,01*B3))*B16												
18	β	0,76		B17/B12												
19																
20	Estimation $F_{t,Rk}$															
21	$0,25F_{t,Rk}^*$	754,77 N		(1/4)*Pi*(B3^2 * B5^0,25)												
22	<i>*Most presumable it is a bit different, but capacity is limited to 15% of Johansen part (below)</i>															
23	Equations $F_{t,Rk}$		Johansen part	MAX: 0.25F_{t,Rk}	F_{t,Rk}											
24	a	2054,55 N		0	2054,55	B12*B10*B3										
25	b	2758,78 N		0	2758,78	B17*B15*B3										
26	c	1034,66 N		155,20	1189,86	((B12*B10*B3)/(1+B18)) * (WORTEL((B18+(2*B18^2*(1+(B15/B10) + (B15/B10)^2)) + (B18^3*(B15/B10)^2))) - (B18*(1+(B15/B10))))										
27	d	1214,98 N		182,25	1397,23	((1,05*B12*B10*B3)/(2+B18)) * (WORTEL((2*B18*(1+B18)) + ((4*B18*(2+B18)*B7)/(B12*B3*B10^2)) - B18))										
28	e	1804,23 N		270,63	2074,86	((1,05*B12*B15*B3)/(1+(2*B18))) * (WORTEL((2*B18^2*(1+B18)) + ((4*B18*(1+(2*B18)*B7)/(B12*B3*B15^2)) - B18)))										
29	f	768,59 N		115,29	883,88	(1,15 * WORTEL((2*B18)/(1+B18)) * WORTEL(2*B7*B12*B3))										
30																
31	$F_{t,Rk}$	883,88 N				MIN(E24:E29)										
32																
33	Design strength			Formule b												
34	k_{mod}	0,7	-													
35	γ_M	1,3	-													
36	$F_{t,Rd}$	475,94 N		B31*B34/B35												
37																
38	Minimal spacing															
39	a_1	31 mm		ALS(B3<5; (5 + (5*COS(0))*B3; (5 + (7*COS(0))*B3)												

Figure X.201: Capacity of nails_Excel

X.11.6.3 Needed nails

	A	B	C	D	E	F	G
1	Verification shear force Option 2						
2	Stability wall properties			Formule B			
3	Ratio depth/ length stab. Wall	4,5	-				
4	Depth building	10,85	m				
5	Length stability wall	2,41	m	B4/B3			
6	H _{wall}	2,8	m				
7	Number of stories	3	-				
8	Thickness plywood (C18)	18	mm				
9	Column width (C24)	38	mm				
10	Column depth (C24)	120	mm				
11	Spacing	600	mm				
12							
13	Definition wind load						
14	q _p (22.4m) (Wind area II; Urban)	0,93	kN/m ²				
15	CsCd	1	-				
16	Cf _D	0,8	-				
17	Cf _E	0,7	-				
18	Cf _{tot}	0,8	-	MAX(B16:B17)			
19	q _{wind_SLS}	8,07	kN/m	B14*B15*B18*B4			
20	Safety factor wind	1,5	-				
21	q _{wind_ULT}	12,11	kN/m	B19*B20			
22	Q _{n,tot}	84,76	kN	(B21*(B7-0,5)*B6)			
23							
24	Number of walls	4	-				
25	Number of sides (Housing sep. wall)	2	-				
26	Q _n ; per wall_side	10,60	kN	B22/B24/B25			
27							
28	Calculation shear						
29	F1	12,30	kN	B26*B6/B5			
30	τ ₁	4,39	kN/m	B29/B6			
31	F2	10,60	kN	B26			
32	τ ₂	4,39	kN/m	B31/B5			
33	τ ₁ = τ ₂	WAAR	-	B30=B32			
34							
35	Number of nails needed						
36	Capacity nail (3.1*50)	475,94	N				
37	Needed screws per meter	10	-	AFRONDEN.BOVEN((B32/(B36/1000)); 1)			
38	Maximal spacing	100,0	mm	AFRONDEN.BENEDEN.WISK(1000/B37;1)			
39	Applied spacing	100	mm				

Figure X.202: Needed nails_Excel

X.11.6.4 Hand validation

Order /
Blad nr 1/3
Deel Verificatie tension in stability wall (opt 2)
Datum 26/05/2023

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Building parameters

Ratio = 4.5 [-]
 depth build_y = 10.85 [m] ⇒ k_{slab wall} = 10.85/4.5 = 2.41 [m]
 width build_x = 13.00 [m]
 storey height = 2.8 [m]
 Floors = 3 [-]
 NFL = 2.67 [-] ⇒ Only weight slab wall considered, so no
 linking to slabs

Used weights (From other sheets)

timber roof = 0.7 [kN/m²]
 timber floor = 1.0 [kN/m²]
 timber wall = 1.52 [kN/m /storey]

Safety factors

$\gamma_G = 0.9$ [-]
 $\gamma_Q = 1.3$ [-]

1. Verification in structural elements

a) Compression caused by wall = 1.52 · 3 = 4.56 [kN/m] = 4.56 [kN/m]
 $F_{c,ed} = 0.9 \cdot (1.3) \cdot 4.56 \cdot 2.41 = 4.54$ [kN]

b) tension by wind
 ↳ 4 panes considered ⇒ linking + dilatation
 $g_{w,slab} = (s_{ci} (z - g_{p(2.4)})) \cdot \text{width} / 4$
 $= 1 \cdot 0.7 \cdot 0.53 \cdot 10.85 / 4 = 2.02$ [kN/m]
 $g_{w,wls} = 1.5 \cdot 2.02 = 3.03$ [kN/m]
 $M_{wind,wls} = \frac{1}{2} g l^2 = \frac{1}{2} \cdot 3.03 \cdot (3 \cdot 2.8)^2 = 106.90$ [kNm]
 $T_{ed} = \frac{M_{wind,wls}}{k_{wall}} = \frac{106.90}{2.41} = 44.3$ [kN] ⇒ 44.3 kN

c) Force to anchor
 Force per wall (2, using sep. wall) = $\frac{T_{ed} - F_{c,ed}}{2} = \frac{44.3 - 4.54}{2} = 19.88$
 Anchor capacity = 21.3 [kN]
 ↳ Number of anchors (per wall) = 1

Figure X.203: Validation stability (Tension in wall) (1)

Order ✓

Blad nr 2 / 3

Deel Verification tension in stability wall (option 2)

Datum 26/05/2023

VAN ROSSUM RAADGEVENDE
INGENIEURS
d) Verification maximum compression

$$\begin{aligned} F_{c, \text{ wall}} &= 4.54 \text{ (kN)} \\ F_{c, \text{ wind}} &= \frac{44.25 \text{ (kN)}}{45.25 \text{ (kN)}} \end{aligned}$$

$$\text{Number of columns} = \left(\frac{1.56 \text{ (m)}}{\text{spacing}} \right)^2 = \left(\frac{2.4 \text{ (m)}}{0.6} \right)^2 = 4 \text{ (3)}$$

$$N_{c, \text{ column}} = 45.25 / 4 = 12.3 \text{ (kN)}$$

$$K_c = 0.44 \text{ (buckling reduction; steel walls)}$$

$$A = 30^2 \cdot 120 = 4560 \text{ (mm}^2\text{)}$$

$$\sigma_N = \frac{F_c}{A} = \frac{12.3 \cdot 1000}{4560} = 2.69 \text{ (N/mm}^2\text{)}$$

$$\left. \begin{aligned} \gamma_{\text{mod}} &= 0.7 \\ \gamma_{\text{M}} &= 1.3 \end{aligned} \right\} F_{c, \text{ red}} = 11.31 \text{ (N/mm}^2\text{)}$$

$$\hookrightarrow \text{UC} = \frac{\sigma_N}{K_c \cdot F_{c, \text{ red}}} = \frac{2.69}{0.44 \cdot 11.31} = 0.54 \text{ (3)} \quad \rightarrow \text{OK (3)}$$

e) Compression at grain

$K_{c,90}$ in final excel changed to 1.25

$$K_{c,90} = 1.0$$

$$x' = \min \left(30, \frac{\text{spacing}}{2}, \text{width column} \right) = \min \left(30, \frac{600}{2}, 300 \right) = 30 \text{ (mm)}$$

$$A_{eff} = 30^2 + 2 \cdot 30 \cdot 50 = 5700 \text{ (mm}^2\text{)}$$

$$A_{eff} = 90^2 \cdot 120 = 11760 \text{ (mm}^2\text{)}$$

$$\sigma_N = \frac{F_c}{A_{eff}} = \frac{12.3 \cdot 1000}{11760} = 1.04 \text{ (N/mm}^2\text{)}$$

$$F_{c,90,d} = \frac{2.5 \cdot 0.7}{1.3} = 1.35 \text{ (N/mm}^2\text{)} \Rightarrow \text{UC} = \frac{1.04}{1.35} = 0.77 \text{ (3)} \quad \rightarrow \text{OK (3)}$$

Figure X.204: Validation stability (Tension in wall) (2)

Order /
 Blad nr 3/3
 Deel Verification tension in stability wall (option 2)
 Datum 26/05/2023

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2. Verification in transversal direction
 ↳ considered span/2 = 1.56 [m] (betreft de breedte van de muur)

a) Compressive load

Roof = $0.7 \cdot 1.56 = 1.092$ [kN/m]
 3rd Fl = $1 \cdot 1.56 = 1.56$ [kN/m]
 2nd Fl = $1 \cdot 1.56 = 1.56$ [kN/m]
 Wall = $1.52 \cdot 3 = 4.56$ [kN/m]
 $g_{ps} = 9.85$ [kN/m³]

$F_c = \frac{1}{2} g l = \frac{1}{2} g_{ps} \cdot 10.85 = 53.40$ [kN]
 $F_{red} = 0.9 \cdot 53.40 = 48.06$ [kN]

b) Wind load

$q_w = q_p(12.4) (s.c.) \cdot c_f \cdot l \cdot width = 0.53 \cdot 1.15 \cdot (2 \cdot 1.56 \cdot 0.75) = 1.10$ [kN/m]
 $q_{wind} = 1.5 \cdot 1.10 = 1.65$ [kN/m]

Moment = $\frac{1}{2} q l^2 = \frac{1}{2} \cdot 1.65 \cdot (3.12)^2 = 2.17$ [kNm]
 $T_{red} = \frac{Moment}{l_{wall}} = \frac{2.17}{10.85} = 0.20$ [kN] < Compressive force, so no tension

c) Maximum compression

$F_{c, max} = F_{c, wall} + F_{c, wind} = 48.06 + 20 = 68.06$ [kN]
 $Columns = \frac{F_{c, max}}{spacing} = \frac{68.06}{0.6} \cdot g [-]$
 Force per column = $68.06 / 6 = 11.34$ [kN]

$\chi_e = 0.99$ (stet walls)
 $k_{red} = 0.7$ } $f_{red} = 11.34$ [kN/m²]
 $\gamma_M = 1.3$ }

$\sigma_N = \frac{F_c}{A} = \frac{17.56 \cdot 1000}{6500} = 2.69$ [kN/m²]

↳ $UC = \frac{2.69}{11.34} = 0.24$ [-] < 0.33

> IFFT > Validation!
 ↳ 4th basis op. correct

Figure X.205: Validation stability (Tension in wall) (3)

Order ✓
 Blad nr 1/1
 Deel Verification capacity nails
 Datum 26/05/2023



Ver. Capacity nails

Fastener

nail = 3.1 x 50 [mm]
 d = 3.1 [mm]
 L = 50 [mm]
 $F_{u,k} = 400$ [N/mm²]
 $M_{y,Re} = 0.3 F_{u,k} d^{1.6} = 0.3 \cdot 400 \cdot 3.1^{1.6} = 2273.64$ [Nmm] $\rightarrow 2273.64$ [Nmm]

Multiplier

$k_1 = 1.0$ [mm]
 $P_k = 470$ [kg/m²]
 $F_{h,1,k} = 0.11 P_k d^{-0.3} = 0.11 \cdot 470 \cdot 3.1^{-0.3} = 36.82$ [N/mm²] $\rightarrow 36.82$ [N/mm²]

Softwood

$t_2 = \min(t_{fast}; L - t_1) = \min(120; 72) = 72$ [mm]
 $P_k = 350$ [kg/m²]
 $F_{h,2,k} = 0.082 (1 - 0.01 d) P_k = 0.082 (1 - 0.01 \cdot 3.1) 350 = 27.81$ [N/mm²] $\rightarrow 27.81$ [N/mm²]
 $\beta = \frac{F_{h,1,k}}{F_{h,2,k}} = \frac{36.82}{27.81} = 1.32$ [] $\rightarrow 1.32$

Estimation $F_{u,k}$

$F_{u,k} = (1.2 \pi d^2) F_{u,k} = 1.2 \pi \cdot 3.1^2 \cdot 400 = 3019$ N
 $1.05 F_{u,k} = 1.05 \cdot 3019 = 3170$ [N] \rightarrow Note is a high value, but $F_{u,k}$ is limited to 15% of Johnson part, so not realistic in design this value.

Calculation $F_{u,Re}$

$F_{u,Re} = \text{normal } 15\% \text{ of Johnson part.}$
 (a) $F_{h,1,k} / d = 36.82 / 3.1 = 11.88$ [N/mm] $\rightarrow 11.88$ [N/mm]
 (b) $F_{h,2,k} / d = 27.81 / 3.1 = 8.97$ [N/mm] $\rightarrow 8.97$ [N/mm]
 (c) $\frac{F_{h,1,k} / d}{1 + \beta} \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{1}{\beta} + \left(\frac{1}{\beta}\right)^2 \right] + \beta^2 \left(\frac{1}{\beta}\right)^2} - \beta \left(1 + \frac{1}{\beta}\right) \right] \cdot \frac{F_{u,k}}{4}$
 $= \frac{36.82 / 3.1}{1 + 1.32} \left[\sqrt{1.32 + 2 \cdot 1.32^2 \left[1 + \frac{1}{1.32} + \left(\frac{1}{1.32}\right)^2 \right] + 1.32^2 \left(\frac{1}{1.32}\right)^2} - 1.32 \left(1 + \frac{1}{1.32}\right) \right] \cdot \frac{3019}{4}$
 $= 1194.93$ [N] $\rightarrow 1195$ [N]
 (d) $1.05 \frac{F_{u,k} / d}{2 + \beta} \left[\sqrt{2\beta(1 + \beta) + \frac{4\beta(1 + \beta) M_{y,Re}}{F_{h,1,k} d t_1} - \beta} \right] \cdot \frac{F_{u,k}}{4}$
 $= 1.05 \frac{36.82 / 3.1}{2 + 1.32} \left[\sqrt{2 \cdot 1.32 \cdot (1 + 1.32) + \frac{4 \cdot 1.32 \cdot (200 \cdot 76) \cdot 2273.64}{36.82 \cdot 3.1 \cdot 18} - 1.32} \right] \cdot \frac{3019}{4}$
 $= 1401.50$ [N] $\rightarrow 1397$ [N]

Figure X.206: Validation stability (Capacity of nails) (1)

Order ✓
 Blad nr 2/2
 Deel Verification capacity nails
 Datum 26/05/2023

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(e)
$$\frac{1,05 \cdot F_{k,1} \cdot e \cdot l_{ed}}{1 + 2\beta} \left[\sqrt{2 \cdot P^2(1, \beta) + \frac{4 \cdot \beta \cdot (1 + 2\beta) \cdot M_{y, Rc} - \beta}{F_{k,1} \cdot e \cdot l_{ed}^2}} \right] \cdot \frac{F_{t,n}}{4}$$

$$= \frac{1,05 \cdot 36 \cdot 82 \cdot 32 \cdot 31}{1 + 2 \cdot 0,76} \left[\sqrt{2 \cdot 0,76^2 \cdot (1 + 0,76) + \frac{4 \cdot 0,76 \cdot (1 + 2 \cdot 0,76) \cdot 2273 \cdot 64 - 0,76}{36 \cdot 82 \cdot 31 \cdot 32^2}} \right] \cdot 1,15$$

$$= 20877,17 \text{ [N]} \quad \rightarrow \quad 20877,17 \cdot 8 = 167017,36 \text{ [N]}$$

(f)
$$1,5 \cdot \sqrt{\frac{2\beta}{1+\beta}} \cdot \sqrt{2 \cdot M_{y, Rc} \cdot l_{ed}} \cdot \frac{F_{t,n}}{2}$$

$$= 1,5 \cdot \sqrt{\frac{2 \cdot 0,76}{1 + 0,76}} \cdot \sqrt{2 \cdot 2273 \cdot 64 \cdot 36 \cdot 82 \cdot 31} \cdot 1,15 = 88544 \text{ [N]} \quad \rightarrow \quad 88544 \cdot 8 = 708352 \text{ [N]}$$

$$F_{v, Rc} = \min \{c; d; e; d; e; f\} = 88544 \text{ [N]}$$

$$F_{v, d} = \frac{\text{small } F_{v, Rc}}{8n} = \frac{0,7 \cdot 88544}{1,3} = 47670 \text{ [N]} \quad \rightarrow \quad 47670 \cdot 8 = 381360 \text{ [N]}$$

minimal spacing

$$d \leq s \text{ [mm]} \rightarrow a_1 = (s + s \cdot \cos(\alpha)) \cdot d = 10 \cdot 31 = 31 \text{ [mm]} \quad \rightarrow \quad 31 \text{ [mm]}$$

SHEET = Vertical
 ↳ holds also for steel screws

Figure X.207: Validation stability (Capacity of nails) (2)

Order ✓
 Blad nr 1/1
 Deel Verification steel needed nails (option 2)
 Datum 26/05/2023



1. Definition wind load

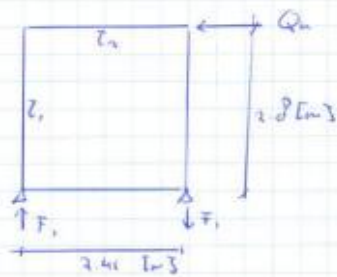
$$q_{w,sts} = q_p(z, w) \cdot C_{pe} \cdot C_{s1} \text{ with building} \\ = 0.53 \cdot 1.08 \cdot 10.05 = 5.87 \text{ [kN/m}^2\text{]}$$

$$q_{w,ult} = \gamma_w \cdot q_{w,sts} = 1.5 \cdot 5.87 = 8.81 \text{ [kN/m}^2\text{]}$$

$$Q_{w, tot} = q_{w,ult} \cdot 2.5 \cdot 1.5 \cdot 2.5 = 8.81 \cdot 2.5 \cdot 2.5 = 55.06 \text{ [kN]}$$

$$\left. \begin{array}{l} \text{Number of panels} = 144 \\ \text{Nails / panel} = 2 \text{ (having sep walls)} \end{array} \right\} Q_{n, tot} = \frac{55.06}{144} = 0.382 \text{ [kN]} \\ \rightarrow 10.60 \text{ [kN]}$$

2. Shear stress and needed nails



$$F_1 = \frac{Q_n \cdot z_1}{2.41} = 12.31 \text{ [kN]} \\ \rightarrow 12.30 \text{ [kN]}$$

$$z_1 = z_2 = \frac{F_1}{k} = \frac{12.30}{1.8} = 6.83 \text{ [m]}$$

Screws / Nails (3.1 x 50) $\Rightarrow T_{v,Rd} = 0.475 \text{ [kN]}$

$$\text{Needed screws} = \frac{6.35}{0.475} = 13.37 \Rightarrow 14 \text{ screws / meter wall}$$

$$\text{Spacing} = \frac{1000}{14} = 71.4 \text{ [mm]} \rightarrow 100 \text{ [mm]}$$

SHEET = Validated

\rightarrow Holds also for option 1

Figure X.208: Validation stability (Needed nails)

X.11.7 Weight calculation HSB top-up

X.11.7.1 1-storey

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Weight calculation HSB top-up (1 added layer)														
2	Building information														
3	Depth	10,85	m												
4	Unit length	18	m												
5	Stories	1	-												
6	Storey height	2,8	m												
7	Load calculation														
9	Load combinations (NEN1990; CC2) Variable residential load is leading action														
10		V_G	V_Q												
11	6.10a	1,35	1,5	$q_{red} = G_k V_G + \sum Q_{k,i} V_{Q,i} \psi_{0,i}$											
12	6.10b	1,2	1,5	$q_{red} = G_k V_G + Q_{k,1} V_Q + \sum Q_{k,i} V_{Q,i} \psi_{0,i}$											
14	Roof kN/m ² Formule B														
15	Permanent (incl. finishings)	0,7	ψ_0												
16	Variable	-	0												
17	Eq 6.10a	0,95		B11*B15											
18	Eq 6.10b	0,84		B12*B15											
20	1st floor (Full loaded) kN/m ²														
21	Permanent (incl. finishings)	1	ψ_0												
22	Variable		0,4												
23	Residential load	1,75													
24	Non-str. walls	0,5													
25	Eq 6.10a	2,70		(B11*B21)+(C11*C22*(B23+B24))											
26	Eq 6.10b	4,13		(B12*B21)+(C12*B23)+(C12*C22*B24)											
28	Housing separation walls														
29	Columns														
30	Width	38	mm												
31	Height	89	mm												
32	Spacing	600	mm												
33	Density	420	kg/m ³												
34	Sides	2	-												
35	Weight	0,05	kN/m ² wall												
36				(B30/1000)*(B31/1000)*(1000/B32)*B34*B33*10/1000											
37	Plywood														
38	Thickness	18	mm												
39	Sides	2	-												
40	Density	550	kg/m ³												
41	Weight	0,20	kN/m ² wall												
42				(B38/1000)*B39*B40*10/1000											
43	Fire safety board														
44	Weight	0,14	kN/m ²												
45	Weight	0,28	kN/m ² wall												
46	Total weight per m2 wall	0,53	kN/m ² wall												
47	Total length sep. Wall	21,71	m												
48	Total wall area	60,79	m ²												
49	Weight separation walls wall	31,93	kN												
50	Weight per m2 flat area	0,16	kN/m ²												
51	Eq 6.10a	0,22	kN/m ²	B50*B11											
52	Eq 6.10b	0,20	kN/m ²	B50*B12											
54	Inner walls														
55	Columns														
56	Width	38	mm												
57	Height	120	mm												
58	Spacing	600	mm												
59	Density	420	kg/m ³												
60	Weight	0,03	kN/m ² wall												
61				(B56/1000)*(B57/1000)*(1000/B58)*B59*10/1000											
62	Plywood														
63	Thickness	18	mm												
64	Sides	2	-												
65	Density	550	kg/m ³												
66	Weight	0,20	kN/m ² wall												
67				(B63/1000)*B64*B65*10/1000											
68	Fire safety board														
69	Weight	0,14	kN/m ²												
70	Weight	0,28	kN/m ² wall												
71	Total weight per m2 wall	0,51	kN/m ² wall												
72	Total length sep. Wall	28,95	m												
73	Total wall area	81,06	m ²												
74	Weight separation walls wall	41,33	kN												
75	Weight per m2 flat area	0,21	kN/m ²												
76	Eq 6.10a	0,29	kN/m ²	B75*B11											
77	Eq 6.10b	0,25	kN/m ²	B75*B12											

Figure X.209: Weight calculation HSB top-up (1-storey) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
78															
79	Columns gallery														
80	Width	70	mm												
81	Height	70	mm												
82	Amount	6	-												
83	Density	420	kg/m ³												
84	Weight total	0,35	kN	(B80/1000)*(B81/1000)*B82*B5*B6*B83*10/1000											
85	Weight per m2 flat area	0,002	kN/m ²	B84/(B3*B4)											
86	Eq 6.10a	0,002	kN/m ²	B85*B11											
87	Eq 6.10b	0,002	kN/m ²	B85*B12											
88															
89	Beams gallery floor														
90	Height	295	mm												
91	Width	95	mm												
92	Length	1200	mm												
93	Density	420	kg/m ³												
94	Amount	6	-												
95	Weight total	0,85	kN	(B90/1000)*(B91/1000)*(B92/1000)*B94*B5*B93*10/1000											
96	Weight per m2 flat area	0,00	kN/m ²	B95/(B3*B4)											
97	Eq 6.10a	0,01	kN/m ²	B96*B11											
98	Eq 6.10b	0,01	kN/m ²	B96*B12											
99															
100	Beams gallery roof														
101	Height	245	mm												
102	Width	70	mm												
103	Length	1200	mm												
104	Density	420	kg/m ³												
105	Amount	6	-												
106	Weight total	0,52	kN	(B101/1000)*(B102/1000) * (B103/1000) * B105* (B104*10/1000)											
107	Weight per m2 flat area	0,003	kN/m ²	B106/(B3*B4)											
108	Eq 6.10a	0,004	kN/m ²	B107*B11											
109	Eq 6.10b	0,003	kN/m ²	B107*B12											
110															
111	Fronnd and back facade														
112	Weighth	0,5	kN/m ² facade												
113	Area frond + back facade	100,8	m ² facade	B4*B5*B6*2											
114	Total weight	50,4	kN	B113*B112											
115	Weight per m2 flat area	0,26	kN/m ²	B114/(B3*B4)											
116	Eq 6.10a	0,35	kN/m ²	B115*B11											
117	Eq 6.10b	0,31	kN/m ²	B115*B12											
118															
119	Total weight 2 stories														
120	Eq 6.10a	4,51	kN/m ²	B17+B25+B51+B76+B86+B97+B108+B116											
121	Eq 6.10b	5,74	kN/m ²	B18+B26+B52+B77+B87+B98+B109+B117											
122	Normative weight	5,74	kN/m ²	MAX(B120:B121)											
123	5% exceeding	5,5	kN/m ²	B122*0,95											

Figure X.210: Weight calculation HSB top-up (1-storey) (2)_Excel

X.11.7.2 2-storey

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Weight calculation HSB top-up (2 added layers)														
2	Building information														
3	Depth	10,85	m												
4	Unit length	18	m												
5	Stories	2	-												
6	Storey heighth	2,8	m												
7															
8	Load calculation														
9	Load combinations (NEN1990; CC2)			<i>Variable residential load is leading action</i>											
10		Y _G	Y _Q												
11	6.10a	1,35	1,5	q _{red} = G _k Y _G + ΣQ _k Y _Q ψ ₀											
12	6.10b	1,2	1,5	q _{red} = G _k Y _G + Q _{k,1} Y _Q + ΣQ _{k,i} Y _Q ψ ₀											
13															
14	Roof			Formule B											
15	Permanent (incl. finishings)	0,7	ψ ₀												
16	Variable	-	0												
17	Eq 6.10a	0,95		B11*B15											
18	Eq 6.10b	0,84		B12*B15											
19															
20	2nd floor (Full loaded)														
21	Permanent (incl. finishings)	1	ψ ₀												
22	Variable		0,4												
23	Residential load	1,75													
24	Non-str. walls	0,5													
25	Eq 6.10a	2,70		(B11*B21)+(C11*C22*(B23+B24))											
26	Eq 6.10b	4,13		(B12*B21)+(C12*B23)+(C12*C22*B24)											
27															
28	1st floor (Full loaded)														
29	Permanent (incl. finishings)	1	ψ ₀												
30	Variable		0,4												
31	Residential load	1,75													
32	Non-str. walls	0,5													

Figure X.211: Weight calculation HSB top-up (2-storey) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
33		Eq 6.10a	2,70	(B11*B29)+(C11*C30*(B31+B32))											
34		Eq 6.10b	4,13	(B12*B29)+(C12*B31)+(C12*C30*B32)											
35															
36	Housing seperation walls														
37	Columns														
38		Width	38 mm												
39		Height	120 mm												
40		Spacing	600 mm												
41		Density	420 kg/m ³												
42		Sides	2 -												
43		Weight	0,06 kN/m ² wall	(B38/1000)*(B39/1000)*(1000/B40)*B42*B41*10/1000											
44															
45	Plywood														
46		Thickness	18 mm												
47		Sides	2 -												
48		Density	550 kg/m ³												
49		Weight	0,20 kN/m ² wall	(B46/1000)*B47*B48*10/1000											
50															
51	Fire safety board														
52		Weigth	0,28 kN/m ² wall	B51*B47											
53															
54		Total weight per m2 wall	0,54 kN/m ² wall	B43+B49+B52											
55		Total length sep. Wall	21,71 m												
56		Total wall area	121,58 m ²	B55*B6*B5											
57		Weight seperation walls wall	65,87 kN	B54*B56											
58		Weight per m2 flat area	0,34 kN/m ²	B57/(B3*B4)											
59		Eq 6.10a	0,46 kN/m ²	B58*B11											
60		Eq 6.10b	0,40 kN/m ²	B58*B12											
61															
62	Inner walls														
63	Columns														
64		Width	38 mm												
65		Height	140 mm												
66		Spacing	400 mm												
67		Density	420 kg/m ³												
68		Weight	0,06 kN/m ² wall	(B64/1000)*(B65/1000)*(1000/B66)*B67*10/1000											
69															
70	Plywood														
71		Thickness	18 mm												
72		Sides	2 -												
73		Density	550 kg/m ³												
74		Weight	0,20 kN/m ² wall	(B71/1000)*B72*B73*10/1000											
75															
76	Fire safety board														
77		Weigth	0,28 kN/m ² wall	B76*B72											
78															
79		Total weight per m2 wall	0,53 kN/m ² wall	B68+B74+B77											
80		Total length sep. Wall	28,95 m												
81		Total wall area	162,12 m ²	B80*B5*B6											
82		Weight seperation walls wall	86,55 kN	B79*B81											
83		Weight per m2 flat area	0,44 kN/m ²	B82/(B3*B4)											
84		Eq 6.10a	0,60 kN/m ²	B83*B11											
85		Eq 6.10b	0,53 kN/m ²	B83*B12											
86															
87	Columns gallery														
88		Width	95 mm												
89		Height	95 mm												
90		Amount	6 -												
91		Density	420 kg/m ³												
92		Weight total	1,27 kN	(B88/1000)*(B89/1000)*B90*B5*B6*B91*10/1000											
93		Weight per m2 flat area	0,01 kN/m ²	B92/(B3*B4)											
94		Eq 6.10a	0,01 kN/m ²	B93*B11											
95		Eq 6.10b	0,01 kN/m ²	B93*B12											
96															
97	Beams gallery floor														
98		Height	295 mm												
99		Width	95 mm												
100		Length	1200 mm												
101		Density	420 kg/m ³												
102		Amount	6 -												
103		Weight total	1,69 kN	(B98/1000)*(B99/1000)*(B100/1000)*B102*B5*B101*10/1000											
104		Weight per m2 flat area	0,01 kN/m ²	B103/(B3*B4)											
105		Eq 6.10a	0,01 kN/m ²	B104*B11											
106		Eq 6.10b	0,01 kN/m ²	B104*B12											
107															
108	Beams gallery roof														
109		Height	245 mm												
110		Width	70 mm												
111		Length	1200 mm												
112		Density	420 kg/m ³												
113		Amount	6 -												
114		Weight total	0,52 kN	(B109/1000)*(B110/1000) * (B111/1000) * B113* (B112*10/1000)											
115		Weight per m2 flat area	0,003 kN/m ²	B114/(B3*B4)											
116		Eq 6.10a	0,004 kN/m ²	B115*B11											
117		Eq 6.10b	0,003 kN/m ²	B115*B12											

Figure X.212: Weight calculation HSB top-up (2-storey) (2)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
111	Length	1200	mm												
112	Density	420	kg/m ³												
113	Amount	6	-												
114	Weight total	0,52	kN	(B109/1000)*(B110/1000) * (B111/1000) * B113* (B112*10/1000)											
115	Weight per m2 flat area	0,003	kN/m ²	B114/(B3*B4)											
116	Eq 6.10a	0,004	kN/m ²	B115*B11											
117	Eq 6.10b	0,003	kN/m ²	B115*B12											
118															
119	Front and back facade														
120	Weight	0,5	kN/m ² facade												
121	Area front + back facade	201,6	m ² facade	B4*B5*B6*2											
122	Total weight	100,8	kN	B121*B120											
123	Weight per m2 flat area	0,52	kN/m ²	B122/(B3*B4)											
124	Eq 6.10a	0,70	kN/m ²	B123*B11											
125	Eq 6.10b	0,62	kN/m ²	B123*B12											
126															
127	total weight 2 stories														
128	Eq 6.10a	8,12	kN/m ²	B17+B25+B33+B59+B84+B94+B105+B116+B124											
129	Eq 6.10b	10,67	kN/m ²	B18+B26+B34+B60+B85+B95+B106+B117+B125											
130	Normative weight	10,67	kN/m ²	MAX(B128-B129)											
131	5% exceeding	10,1	kN/m ²	B130*0,95											

Figure X.213: Weight calculation HSB top-up (2-storey) (3)_Excel

X.11.7.3 3-storey

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
1	Weight calculation HSB top-up (3 added layers)														
2	Building information														
3	Depth	10,85	m												
4	Unit length	18	m												
5	Stories	3	-												
6	Storey height	2,8	m												
7															
8	Load calculation														
9	Load combinations (NEN1990; CC2)			Variable residential load is leading action											
10		V_G	V_Q												
11	6.10a	1,35	1,5	$q_{red} = G_k V_G + \sum Q_{k,i} V_{Q,i}$											
12	6.10b	1,2	1,5	$q_{red} = G_k V_G + Q_{k,1} V_Q + \sum Q_{k,i} V_{Q,i}$											
13															
14	Roof			Formule B											
15	Permanent (incl. finishings)	0,7	ψ_0												
16	Variable	-	0												
17	Eq 6.10a	0,95		B11*B15											
18	Eq 6.10b	0,84		B12*B15											
19															
20	3rd floor														
21	Permanent (incl. finishings)	1	ψ_0												
22	Variable		0,4												
23	Residential load	1,75													
24	Non-str. walls	0,5													
25	Eq 6.10a	2,70		(B11*B21)+(C11*C22*(B23+B24))											
26	Eq 6.10b	2,55		(B12*B21)+(C12*C22*(B23+B24))											
27															
28	2nd floor (Full loaded)														
29	Permanent (incl. finishings)	1	ψ_0												
30	Variable		0,4												
31	Residential load	1,75													
32	Non-str. walls	0,5													
33	Eq 6.10a	2,70		(B11*B29)+(C11*C30*(B31+B32))											
34	Eq 6.10b	4,13		(B12*B29)+(C12*B31)+(C12*C30*B32)											
35															
36	1st floor (Full loaded)														
37	Permanent (incl. finishings)	1	ψ_0												
38	Variable		0,4												
39	Residential load	1,75													
40	Non-str. walls	0,5													
41	Eq 6.10a	2,70		(B11*B37)+(C11*C38*(B39+B40))											
42	Eq 6.10b	4,13		(B12*B37)+(C12*B39)+(C12*C38*B40)											
43															
44	Housing seperation walls														
45	Columns														
46	Width	38	mm												
47	Height	140	mm												
48	Spacing	400	mm												
49	Density	420	kg/m ³												
50	Sides	2	-												

Figure X.214: Weight calculation HSB top-up (3-storey) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O
51	Weight	0,11	kN/m ² wall	(B46/1000)*(B47/1000)*(1000/B48)*B50*B49*10/1000											
52															
53	Plywood														
54	Thickness	18	mm												
55	Sides	2	-												
56	Density	550	kg/m ³												
57	Weight	0,20	kN/m ² wall	(B54/1000)*B55*B56*10/1000											
58															
59	Fire safety board	0,14	kN/m ²												
60	Weigth	0,28	kN/m ² wall	B59*B55											
61															
62	Total weight per m2 wall	0,59	kN/m ² wall	B51+B57+B60											
63	Total length sep. Wall	21,71	m												
64	Total wall area	182,36	m ²	B63*B6*B5											
65	Weight seperation walls wall	107,54	kN	B62*B64											
66	Weight per m2 flat area	0,55	kN/m ²	B65/(B3*B4)											
67	Eq 6.10a	0,74	kN/m ²	B66*B11											
68	Eq 6.10b	0,66	kN/m ²	B66*B12											
69															
70	Inner walls														
71	Columns														
72	Width	38	mm												
73	Height	184	mm												
74	Spacing	400	mm												
75	Density	420	kg/m ³												
76	Weight	0,07	kN/m ² wall	(B72/1000)*(B73/1000)*(1000/B74)*B75*10/1000											
77															
78	Plywood														
79	Thickness	18	mm												
80	Sides	2	-												
81	Density	550	kg/m ³												
82	Weight	0,20	kN/m ² wall	(B79/1000)*B80*B81*10/1000											
83															
84	Fire safety board	0,14	kN/m ²												
85	Weigth	0,28	kN/m ² wall	B84*B80											
86															
87	Total weight per m2 wall	0,55	kN/m ² wall	B76+B82+B85											
88	Total length sep. Wall	28,95	m												
89	Total wall area	243,18	m ²	B88*B5*B6											
90	Weight seperation walls wall	134,09	kN	B87*B89											
91	Weight per m2 flat area	0,69	kN/m ²	B90/(B3*B4)											
92	Eq 6.10a	0,93	kN/m ²	B91*B11											
93	Eq 6.10b	0,82	kN/m ²	B91*B12											
94															
95	Columns gallery														
96	Width	95	mm												
97	Height	95	mm												
98	Amount	6	-												
99	Density	420	kg/m ³												
100	Weight total	1,91	kN	(B96/1000)*(B97/1000)*B98*B5*B6*B99*10/1000											
101	Weight per m2 flat area	0,01	kN/m ²	B100/(B3*B4)											
102	Eq 6.10a	0,01	kN/m ²	B101*B11											
103	Eq 6.10b	0,01	kN/m ²	B101*B12											
104															
105	Beams gallery floor														
106	Height	295	mm												
107	Width	95	mm												
108	Length	1200	mm												
109	Density	420	kg/m ³												
110	Amount	6	-												
111	Weight total	2,54	kN	(B106/1000)*(B107/1000)*(B108/1000)*B110*B5*B109*10/1000											
112	Weight per m2 flat area	0,01	kN/m ²	B111/(B3*B4)											
113	Eq 6.10a	0,02	kN/m ²	B112*B11											
114	Eq 6.10b	0,02	kN/m ²	B112*B12											
115															
116	Beams gallery roof														
117	Height	245	mm												
118	Width	70	mm												
119	Length	1200	mm												
120	Density	420	kg/m ³												
121	Amount	6	-												
122	Weight total	0,52	kN	(B117/1000)*(B118/1000) * (B119/1000) * B121* (B120*10/1000)											
123	Weight per m2 flat area	0,003	kN/m ²	B122/(B3*B4)											
124	Eq 6.10a	0,004	kN/m ²	B123*B11											
125	Eq 6.10b	0,003	kN/m ²	B123*B12											

Figure X.215: Weight calculation HSB top-up (3-storey) (2)_Excel

126				
127	Fronnd and back facade			
128	Weighth	0,5	kN/m ² facade	
129	Area frond + back facade	302,4	m ² facade	B4*B5*B6*2
130	Total weight	151,2	kN	B129*B128
131	Weight per m2 flat area	0,77	kN/m ²	B130/(B3*B4)
132	Eq 6.10a	1,05	kN/m ²	B131*B11
133	Eq 6.10b	0,93	kN/m ²	B131*B12
134				
135	Total weight 3 stories			
136	Eq 6.10a	11,79	kN/m ²	B17+B25+B33+B41+B67+B92+B102+B113+B124+B132
137	Eq 6.10b	14,08	kN/m ²	B18+B26+B34+B42+B68+B93+B103+B114+B125+B133
138	Normative weight	14,08	kN/m ²	MAX(B136:B137)
139	5% exceeding	13,4	kN/m ²	B138*0,95

Figure X.216: Weight calculation HSB top-up (3-storey) (3)_Excel

X.11.8 Environmental impact HSB top-up

X.11.8.1 No additional room

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1	Type of porch flat			Formule B												
2	Variant	Excludes an additional room														
3	Unit width	18	m													
4	Depth building	10,85	m													
5	Number of stories	2														
6	Storey height	2,8	m													
7	Total new floor area	390,6	m ²	B3*B4*B5												
8	Floor span	3,9-4,6	m													
9																
10	Wall lengths															
11	L _{gallery}	1,2	m													
12	L _{wall}	9,65	m	B4-B11												
13	L _{wallmiddle} single	5,43	m	B4/2												
14	L _{wallmiddle} double	4,23	m	B12-B13												
15	L _{ability wall}	2,41	m													
16																
17																
18	Material usage															
19	Floors															
20	1. Beams (Sawn wood)															
21	B _{beam}	70	mm													
22	H _{beam}	270	mm													
23	Spacing	400	mm													
24	Volume	4,73E-02	m ³ /m ² floor	(B21/1000)*(B22/1000)*(1000/B23) * 1												
25	Volume total	1,85E+01	m ³	B24*B3*B4*B5												
26																
27	Steel beams (eventual)															
28	Area	0	mm ²													
29	Amount of beams	5														
30	Volume	0,00E+00	m ³ /m ² floor	B28*10^-6*B29*B3/(B4*B3)												
31	Volume total	0,00E+00	m ³	B30*B3*B4*B5												
32																
33	2. Plywood															
34	Thickness	18	mm													
35	Volume	1,80E-02	m ³ /m ² floor	(B34/1000)*1^3												
36	Volume total	7,03E+00	m ³	B35*B3*B4*B5												
37																
38																
39	Roof															
40	1. Beams (Sawn wood)															
41	B _{beam}	70	mm													
42	H _{beam}	170	mm													
43	Spacing	500	mm													
44	Volume	2,38E-02	m ³ /m ² flat	(B41/1000)*(B42/1000)*(1000/B43)												
45	Volume total	4,65E+00	m ³	B44*B3*B4												
46																
47	2. Plywood															
48	Thickness	18	mm													
49	Volume	1,80E-02	m ³ /m ² flat	(B48/1000)												
50	Volume total	3,52E+00	m ³	B49*B3*B4												
51																
52																
53	Gallery beams (Sawn wood)															
54	1. Floors															
55	B _{beam}	70	mm	B21												
56	H _{beam}	270	mm	B22												
57	L _{beam}	1,2	m	B11												
58	Number of beams	6														
59	Volume	6,97E-04	m ³ /m ² floor	((B55/1000)*(B56/1000)*B57*B58)/(B3*B4)												
60	Volume total	2,72E-01	m ³	B59*B3*B4*B5												
61																
62	2. Roof															
63	B _{beam}	70	mm	B41												
64	H _{beam}	170	mm	B42												
65	L _{beam}	1,2	m	B11												
66	Number of beams	6														
67	Volume	4,39E-04	m ³ /m ² floor	((B63/1000)*(B64/1000)*B65*B66)/(B3*B4)												
68	Volume total	8,57E-02	m ³	B67*B3*B4												
69																
70																
71	Housing seperation wall															
72	1. Columns (Sawn wood)															
73	B _{column}	38	mm													
74	H _{column}	120	mm													
75	Spacing	600	mm													
76	Rows	2														
77	Total wall length (includes stories)	43,42	m	((B12*2/2)+(B14*1)+B13+B15)*B5												
78	Volume total	1,85E+00	m ³	(B73/1000)*(B74/1000)*(1000/B75)*B76*B77*B6												
79																
80	2. Beams (Sawn wood)															
81	Amount per wall	4														
82	Total length	173,68	m	B77*B81												
83	Volume total	7,92E-01	m ³	(B73/1000)*(B74/1000)*B82												
84																
85	3. Plywood															
86	Thickness plywood	18	mm													
87	Total length panel	86,84	m	((B12*2)+(B14*2)+(B13*2)+(B15*2))*B5												
88	Volume total	4,38E+00	m ³	(B86/1000)*B87*B6												
89																
90	4. Gypsum board															
91	Thickness	18	mm													
92	Total length	86,84	m	B87												
93	Volume total	4,38E+00	m ³	(B91/1000)*B92*B6												
94																
95																
96	Inner wall															
97	1. Columns (Sawn wood)															
98	B _{column}	38	mm													
99	H _{column}	140	mm													

Figure X.217: Environmental impact HSB top-up (No additional room) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	N	O	P
100	Spacing	400 mm													
101	Rows	1													
102	Total wall length (includes stories)	47,05 m		((B12*2)+B14)*B5											
103	Volume total	1,75E+00 m ³		(B98/1000)*(B99/1000)*(1000/B100)*B101*B102*B6											
104															
105	2. Beams (Sawn wood)														
106	Amount per wall	2													
107	Total length	94,1 m		B102*B106											
108	Volume total	5,01E-01 m ³		(B98/1000)*(B99/1000)*B107											
109															
110	3. Plywood														
111	Thickness plywood	18 mm													
112	Total length panel	94,1 m		B102*2											
113	Volume total	4,74E+00 m ³		B112*(B111/1000)*B6											
114															
115	4. Gypsum board														
116	Thickness	18 mm													
117	Total length	94,1 m		B112											
118	Volume total	4,74E+00 m ³		B117*(B116/1000)*B6											
119															
120															
121	Columns gallery														
122	B _{column}	95 mm													
123	H _{column}	95 mm													
124	Amount	6													
125	Volume total	3,03E-01 m ³		(B122/1000)*(B123/1000)*B124*B5*B6											
126															
127															
128															
129	Total material use														
129	RSL range	23	42	y											
130	New floor area	390,6	m ²												
131															
132	Element														
132															
133	Floors														
133															
134	Sawn wood	2,08E+00		10 ⁻³ m ³ / y / m ² new floor area	10 ⁻³ m ³ / y / m ² new floor area	(B25+B60)*1000/(B129*B130)					(B25+B60)*1000/(C129*B130)				
135	Plywood	7,83E-01				B36*1000/(B129*B130)					B36*1000/(C129*B130)				
136	Steel	0,00E+00				B31*1000/(B129*B130)					B31*1000/(C129*B130)				
137	Total	2,87E+00				SOM(B134:C136)					SOM(D134:E136)				
138															
139	Roof														
140	Sawn wood	5,27E-01				(B45+B68)*1000/(B129*B130)					(B45+B68)*1000/(C129*B130)				
141	Plywood	3,91E-01				B50*1000/(B129*B130)					B50*1000/(C129*B130)				
142	Total	9,18E-01				SOM(B140:C141)					SOM(D140:E141)				
143															
144	Walls														
145	Sawn wood	5,78E-01				(B78+B103+B125+B88+B108)*1000/(B129*B130)					(B78+B103+B125+B88+B108)*1000/(C129*B130)				
146	Plywood	1,02E+00				(B88+B113)*1000/(B129*B130)					(B88+B113)*1000/(C129*B130)				
147	Gypsum board	1,02E+00				(B93+B118)*1000/(B129*B130)					(B93+B118)*1000/(C129*B130)				
148	Total	2,61E+00				SOM(B145:C147)					SOM(D145:E147)				
149															
150	Total of all elements														
151	Sawn wood	3,19E+00				B134+B140+B145					D134+D140+D145				
152	Plywood	2,19E+00				B135+B141+B146					D135+D141+D146				
153	Gypsum board	1,02E+00				B147					D147				
154	Steel	0,00E+00				B136					D136				
155	Total	6,39E+00				SOM(B151:C154)					SOM(D151:E154)				
156															
157															
158															
159	Environmental impact														
159	RSL range	23	42	y											
160	New floor area	390,6	m ²												
161															
162	Impact of materials														
162															
163	Sawn wood	1 m ³		Unit	kg CO ₂ -eq / unit										
163															
164	Plywood	1 m ³													
164															
165	Gypsum board	1 m ³													
165															
166	Steel	1000 kg													
166															
167															
168															
169	Element														
169															
170	Floors														
170															
171	Sawn wood	1,87E+01 m ³		Amount	Unit	10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area	0 ⁻¹ kg CO ₂ -eq/y/m ² new floor area				(B171*D163/B163*10)/(B159*B160)				(B171*D163/B163*10)/(C159*B160)
172	Plywood	7,03E+00 m ³									(B172*D164/B164*10)/(B159*B160)				(B172*D164/B164*10)/(C159*B160)
173	Steel	0,00E+00 kg									B173*10*D166/B166 / (B159*B160)				B173*10*D166/B166 / (C159*B160)
174	Total	-									SOM(D171:E173)				SOM(F171:G173)
175															
176	Roof														
177	Sawn wood	4,73E+00 m ³									(B177*D163/B163*10)/(B159*B160)				(B177*D163/B163*10)/(C159*B160)
178	Plywood	3,52E+00 m ³									(B178*D164/B164*10)/(B159*B160)				(B178*D164/B164*10)/(C159*B160)
179	Total	-									SOM(D177:E178)				SOM(F177:G178)
180															
181	Walls														
182	Sawn wood	5,20E+00 m ³									(B182*D163/B163*10)/(B159*B160)				(B182*D163/B163*10)/(C159*B160)
183	Plywood	9,12E+00 m ³									(B183*D164/B164*10)/(B159*B160)				(B183*D164/B164*10)/(C159*B160)
184	Gypsum board	506,63 m ²									B184*D165*10/B165/(B159*B160)				B184*D165*10/B165/(C159*B160)
185	Total	-									SOM(D182:E184)				SOM(F182:G184)
186															
187	Total of all elements														
188	Sawn wood	2,87E+01 m ³									D171+D177+D182				F171+F177+F182
189	Plywood	1,97E+01 m ³									D172+D178+D183				F172+F178+F183
190	Gypsum board	506,63 m ²									D184				F184
191	Steel	0,00E+00 kg									D173				F173
192	Total	-									SOM(D188:E191)				SOM(F188:G191)

Figure X.218: Environmental impact HSB top-up (No additional room) (2)_Excel

X.11.8.2 Additional room

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
1	Type of porch flat			Formule B												
2	Variant	Includes an additional room														
3	Unit width	18	m													
4	Depth building	10,85	m													
5	Number of stories	2														
6	Storey height	2,8	m													
7	Total new floor area	390,6	m ²													
8	Floor span	3,9-4,6	m	B3*B4*B5												
9																
10	Wall lengths															
11	L _{gallery}	1,2	m													
12	L _{wall}	9,65	m	B4-B11												
13	L _{wall/middle/single}	5,43	m	B4/2												
14	L _{wall/middle/double}	4,23	m	B12-B13												
15	L _{stability wall}	2,41	m													
16																
17																
18	Material usage															
19	Floors															
20	1. Beams (Sawn wood)															
21	B _{beam}	70	mm													
22	H _{beam}	270	mm													
23	Spacing	400	mm													
24	Volume	4,73E-02	m ³ /m ² floor	(B21/1000)*(B22/1000)*(1000/B23) * 1												
25	Volume total	1,85E+01	m ³	B24*B3*B4*B5												
26																
27	Steel beams (eventual)															
28	Area	0	mm ²													
29	Amount of beams	5														
30	Volume	0,00E+00	m ³ /m ² floor	B28*10^-6*B29*B3/(B4*B3)												
31	Volume total	0,00E+00	m ³	B30*B3*B4*B5												
32																
33	2. Plywood															
34	Thickness	18	mm													
35	Volume	1,80E-02	m ³ /m ² floor	(B34/1000*1*1)												
36	Volume total	7,03E+00	m ³	B35*B3*B4*B5												
37																
38																
39	Roof															
40	1. Beams (Sawn wood)															
41	B _{beam}	70	mm													
42	H _{beam}	170	mm													
43	Spacing	500	mm													
44	Volume	2,38E-02	m ³ /m ² flat	(B41/1000)*(B42/1000)*(1000/B43)												
45	Volume total	4,65E+00	m ³	B44*B3*B4												
46																
47	2. Plywood															
48	Thickness	18	mm													
49	Volume	1,80E-02	m ³ /m ² flat	(B48/1000)												
50	Volume total	3,52E+00	m ³	B49*B3*B4												
51																
52																
53	Gallery beams (Sawn wood)															
54	1. Floors															
55	B _{beam}	70	mm	B21												
56	H _{beam}	270	mm	B22												
57	L _{beam}	1,2	m	B11												
58	Number of beams	6														
59	Volume	6,97E-04	m ³ /m ² floor	((B55/1000)*(B56/1000)*B57*B58)/(B3*B4)												
60	Volume total	2,72E-01	m ³	B59*B3*B4*B5												
61																
62	2. Roof															
63	B _{beam}	70	mm	B41												
64	H _{beam}	170	mm	B42												
65	L _{beam}	1,2	m	B11												
66	Number of beams	6														
67	Volume	4,39E-04	m ³ /m ² floor	((B63/1000)*(B64/1000)*B65*B66)/(B3*B4)												
68	Volume total	8,57E-02	m ³	B67*B3*B4												
69																
70																
71	Housing separation wall															
72	1. Columns (Sawn wood)															
73	B _{column}	38	mm													
74	H _{column}	120	mm													
75	Spacing	600	mm													
76	Rows	2														
77	Total wall length (includes stories)	43,42	m	((B12*2/2)+(B14*1)+B13+B15)*B5												
78	Volume total	1,85E+00	m ³	(B73/1000)*(B74/1000)*(1000/B75)*B76*B77*B6												
79																
80	2. Beams (Sawn wood)															
81	Amount per wall	4														
82	Total length	173,68	m	B77*B81												
83	Volume total	7,92E-01	m ³	(B73/1000)*(B74/1000)*B82												
84																
85	3. Plywood															
86	Thickness plywood	18	mm													
87	Total length panel	86,84	m	((B12*2)+(B14*2)+(B13*2)+(B15*2))*B5												
88	Volume total	4,38E+00	m ³	(B86/1000)*B87*B6												
89																
90	4. Gypsum board															
91	Thickness	18	mm													
92	Total length	86,84	m	B87												
93	Volume total	4,38E+00	m ³	(B91/1000)*B92*B6												
94																
95																
96	Inner wall															
97	1. Columns (Sawn wood)															
98	B _{column}	38	mm													
99	H _{column}	140	mm													

Figure X.219: Environmental impact HSB top-up (Additional room) (1)_Excel

	A	B	C	D	E	F	G	H	I	J	K	L	N	O	P
100	Spacing	400 mm													
101	Rows	1													
102	Total wall length (includes stories)	57,90 m		((B12*2)+B14+B13)*B5											
103	Volume total	2,16E+00 m ³		(B98/1000)*(B99/1000)*(1000/B100)*B101*B102*B6											
104															
105	2. Beams (Sawn wood)														
106	Amount per wall	2													
107	Total length	115,8 m		B102*B106											
108	Volume total	6,16E-01 m ³		(B98/1000)*(B99/1000)*B107											
109															
110	3. Plywood														
111	Thickness plywood	18 mm													
112	Total length panel	115,8 m		B102*2											
113	Volume total	5,84E+00 m ³		B112*(B111/1000)*B6											
114															
115	4. Gypsum board														
116	Thickness	18 mm													
117	Total length	115,8 m		B112											
118	Volume total	5,84E+00 m ³		B117*(B116/1000)*B6											
119															
120															
121	Columns gallery														
122	B _{column}	95 mm													
123	H _{column}	95 mm													
124	Amount	6													
125	Volume total	3,03E-01 m ³		(B122/1000)*(B123/1000)*B124*B5*B6											
126															
127															
128															
129	Total material use														
130	RSL range	23	42	y											
131	New floor area	390,6	m ²												
132	Element														
133	Floors														
134															
135	Sawn wood	2,08E+00		10 ⁻³ m ³ /y/ m ² new floor area	1,14E+00		(B25+B60)*1000/(B129*B130)				(B25+B60)*1000/(C129*B130)				
136	Plywood	7,83E-01			4,29E-01		B36*1000/(B129*B130)				B36*1000/(C129*B130)				
137	Steel	0,00E+00			0,00E+00		B31*1000/(B129*B130)				B31*1000/(C129*B130)				
138	Total	2,87E+00			1,57E+00		SOM(B134:C136)				SOM(D134:E136)				
139	Roof														
140	Sawn wood	5,27E-01			2,89E-01		(B45+B68)*1000/(B129*B130)				(B45+B68)*1000/(C129*B130)				
141	Plywood	3,91E-01			2,14E-01		B50*1000/(B129*B130)				B50*1000/(C129*B130)				
142	Total	9,18E-01			5,03E-01		SOM(B140:C141)				SOM(D140:E141)				
143															
144	Walls														
145	Sawn wood	6,36E-01			3,48E-01		(B78+B83+B103+B108+B125)*1000/(B129*B130)				(B78+B108+B103+B83+B125)*1000/(C129*B130)				
146	Plywood	1,14E+00			6,23E-01		(B88+B113)*1000/(B129*B130)				(B88+B113)*1000/(C129*B130)				
147	Gypsum board	1,14E+00			6,23E-01		(B93+B118)*1000/(B129*B130)				(B93+B118)*1000/(C129*B130)				
148	Total	2,91E+00			1,59E+00		SOM(B145:C147)				SOM(D145:E147)				
149															
150	Total of all elements														
151	Sawn wood	3,25E+00			1,78E+00		B134+B140+B145				D134+D140+D145				
152	Plywood	2,31E+00			1,27E+00		B135+B141+B146				D135+D141+D146				
153	Gypsum board	1,14E+00			6,23E-01		B147				D147				
154	Steel	0,00E+00			0,00E+00		B136				D136				
155	Total	6,70E+00			3,67E+00		SOM(B151:C154)				SOM(D151:E154)				
156															
157															
158															
159	Environmental impact														
160	RSL range	23	42	y											
161	New floor area	390,6	m ²												
162	Impact of materials														
163	Sawn wood	1 m ³		Unit	kg CO ₂ -eq / unit										
164	Plywood	1 m ³			-1460										
165	Gypsum board	1 m ³			-734										
166	Steel	1000 kg			3,11										
167					719										
168															
169	Element														
170	Floors														
171															
172	Sawn wood	1,87E+01 m ³			10 ⁻¹ kg CO ₂ -eq/y/m ² new floor area	0 ⁻¹ kg CO ₂ -eq/y/m ² new floor area					(B171*D163/B163*10)/(B159*B160)			(B171*D163/B163*10)/(C159*B160)	
173	Plywood	7,03E+00 m ³			-3,04E+01	-1,67E+01					(B172*D164/B164*10)/(B159*B160)			(B172*D164/B164*10)/(C159*B160)	
174	Steel	0,00E+00 kg			-5,74E+00	-3,15E+00					B173*10*D166/B166 / (B159*B160)			B173*10*D166/B166 / (C159*B160)	
175	Total				0,00E+00	0,00E+00					SOM(D171:E173)			SOM(F171:G173)	
176	Roof														
177	Sawn wood	4,73E+00 m ³			-3,62E+01	-1,98E+01					(B177*D163/B163*10)/(B159*B160)			(B177*D163/B163*10)/(C159*B160)	
178	Plywood	3,52E+00 m ³			-7,69E+00	-4,21E+00					(B178*D164/B164*10)/(B159*B160)			(B178*D164/B164*10)/(C159*B160)	
179	Total				-2,87E+00	-1,57E+00					SOM(D177:E178)			SOM(F177:G178)	
180					-1,06E+01	-5,79E+00									
181	Walls														
182	Sawn wood	5,72E+00 m ³			-9,29E+00	-5,09E+00					(B182*D163/B163*10)/(B159*B160)			(B182*D163/B163*10)/(C159*B160)	
183	Plywood	1,02E+01 m ³			-8,34E+00	-4,57E+00					(B183*D164/B164*10)/(B159*B160)			(B183*D164/B164*10)/(C159*B160)	
184	Gypsum board	567,39 m ²			1,96E+00	1,08E+00					B184*D165*10/B165/(B159*B160)			B184*D165*10/B165/(C159*B160)	
185	Total				-1,57E+01	-8,58E+00					SOM(D182:E184)			SOM(F182:G184)	
186															
187	Total of all elements														
188	Sawn wood	2,92E+01 m ³			-4,74E+01	-2,60E+01					D171+D177+D182			F171+F177+F182	
189	Plywood	2,08E+01 m ³			-1,70E+01	-9,29E+00					D172+D178+D183			F172+F178+F183	
190	Gypsum board	567,39 m ²			1,96E+00	1,08E+00					D184			F184	
191	Steel	0,00E+00 kg			0,00E+00	0,00E+00					D173			F173	
192	Total				-6,24E+01	-3,42E+01					SOM(D188:E191)			SOM(F188:G191)	

Figure X.220: Environmental impact HSB top-up (Additional room) (2)_Excel

X.11.8.3 Hand validation

Order ✓
 Blad nr 1/3
 Deel verification environmental impact HSB
 Datum 01/06/2023

Validation for floorspan 3.9 - 4.6 / 2 story / no odd room

Building information ↳ Put holes for all slabs

No odd room
 Unit width = 10 Ps
 depth = 10.85 m
 stories = 2
 height story = 2.8 m
 Total new floor area = 2 · 10 · 10.85 = 350.6 m² 8 → 350.6
 Floor span = 3.9 - 4.6 m

Load wall length:

L_{gallery} = 1.2 m
 L_{side walls} = 10.85 · 1.2 = 13.02 m
 L_{wall middle} = 10.85 / 2 = 5.43 m
 L_{wall middle double} = 9.65 - 5.43 = 4.22 m
 L_{sub wall} = 2.41 m

o Calculation volumes

1. Floors

→ beams: $V = \frac{170}{1000} \cdot \frac{270}{1000} \cdot \frac{1000}{400} \cdot 1 \cdot 2 \cdot \underbrace{10 \cdot 10.85}_{\text{total m}^2} = 18.5 \text{ m}^3$ 8

→ plywood: $V = \frac{10}{1000} \cdot 2 \cdot 10 \cdot 10.85 = 7.03 \text{ m}^3$ 8

2. Roof

→ beams: $V = \frac{70}{1000} \cdot \frac{170}{1000} \cdot \frac{1000}{500} \cdot \underbrace{10 \cdot 10.85}_{\text{total volume roof}} = 4.65 \text{ m}^3$ 8

→ plywood: $V = \frac{10}{1000} \cdot 10 \cdot 10.85 = 3.52 \text{ m}^3$ 8

3. Gallery beams

→ Floors: $V = \frac{70}{1000} \cdot \frac{270}{1000} \cdot 1.2 \cdot \underbrace{6 \cdot 2}_{\text{number of beams}} = 0.24 \text{ m}^3$ 8

→ Roof: $V = \frac{70}{1000} \cdot \frac{170}{1000} \cdot 1.2 \cdot 6 \cdot 1 = 0.074 \text{ m}^3$ 8

Figure X.221: Validation environmental impact (1)

Order ✓
 Blad nr 2/3
 Deel Ver. Validation environmental impact HSB
 Datum 01/06/2027



4. Massing separation wall 36.2m

→ columns $L_{tot} = \left(\frac{9.65 \cdot 2}{2} + 5.42 + 4.22 + 2.41 \right) \cdot 2 = 43.44 \text{ [m]}$

$V = \frac{30}{1000} \cdot \frac{20}{1000} \cdot \frac{1000}{600} \cdot 2 \cdot 43.44 \cdot 2.0 = 1.05 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

$\underbrace{\hspace{10em}}_{V / \text{m}^2 \text{ wall}} \quad \underbrace{\hspace{10em}}_{\text{m}^2 \text{ wall}}$

→ plywood: $V = \frac{10}{1000} \cdot 43.44 \cdot 2 \cdot 2.0 = 1.74 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

\uparrow
2 sheets

→ Gypsum board: $V = \frac{10}{1000} \cdot 43.44 \cdot 2 \cdot 2.0 = 1.74 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

5. Inner wall 5 stories

$L_{tot} = \left(\frac{9.65 \cdot 2}{2} + 4.22 \right) \cdot 2 = 47.06 \text{ [m]}$

→ columns: $V = \frac{20}{1000} \cdot \frac{140}{1000} \cdot \frac{1000}{600} \cdot 47.06 \cdot 2.0 = 1.17 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

→ plywood: $V = \frac{10}{1000} \cdot 47.06 \cdot 2 \cdot 2.0 = 1.88 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

→ Gypsum board: $V = \frac{10}{1000} \cdot 47.06 \cdot 2 \cdot 2.0 = 1.88 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

6. Columns gallery

$V = \frac{95}{1000} \cdot \frac{95}{1000} \cdot 6 \cdot 2.0 \cdot 2 = 0.203 \text{ [m}^3\text{]} \text{ } \mathcal{R}$

• Ver. Fraction total material use

1. Struct wood = $1.05 + 1.74 + 1.74 + 1.17 + 0.203 = 5.86 \text{ m}^3$

$\left. \begin{array}{l} \text{PSL 23 y} \\ \text{Area: } 350.6 \text{ [m}^2\text{]} \end{array} \right\} \text{ mat. use} = \frac{5.86 \cdot 1000}{23 \cdot 350.6} = 0.75 \cdot 10^{-3} \text{ m}^3/\text{m}^2 \text{ } \mathcal{R}$

$\left. \begin{array}{l} \text{PSL 42 y} \\ \text{Area: } 350.6 \text{ [m}^2\text{]} \end{array} \right\} \text{ mat. use} = \frac{5.86 \cdot 1000}{42 \cdot 350.6} = 0.39 \cdot 10^{-3} \text{ m}^3/\text{m}^2 \text{ } \mathcal{R}$

Figure X.222: Validation environmental impact (2)

Order ✓
 Blad nr 3/3
 Deel Verification environmental impact HSB
 Datum 01/06/2023



2. plywood = $7.03 + 7.52 + 0.027 + 4.30 + 4.74 = 19.76 \text{ } [m^3]$

$RSL = 23y$
 $A = 350.6 [m^2]$ } mat. use = $\frac{19.76 \cdot 1000}{23 \cdot 350.6} = 2.20 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 2.20 \cdot 10^{-3}$

$RSL = 42y$
 $A = 350.6 [m^2]$ } mat. use = $\frac{19.76 \cdot 1000}{42 \cdot 350.6} = 1.20 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 1.20 \cdot 10^{-3}$

3. Gypsum board = $4.30 + 4.74 = 9.12 \text{ } [m^3]$

$RSL = 23y$
 $A = 350.6 [m^2]$ } mat. use = $\frac{9.12 \cdot 1000}{23 \cdot 350.6} = 1.02 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 1.02 \cdot 10^{-3}$

$RSL = 42y$
 $A = 350.6 [m^2]$ } mat. use = $\frac{9.12 \cdot 1000}{42 \cdot 350.6} = 0.556 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 0.556 \cdot 10^{-3}$

4. Total $23y = 2.20 + 1.02 + 0.556 = 3.776 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 3.776 \cdot 10^{-3}$

Total $42y = 1.20 + 0.556 = 1.756 \cdot 10^{-3} \text{ } m^3/y/m^2$ $\rightarrow 1.756 \cdot 10^{-3}$

• Verification environmental impact total

1. Steel wood $\rightarrow V = 26.74 \text{ } [m^3]$

$RSL = 23y$
 $A = 350.6 [m^2]$ } GWP = $\frac{26.74 \cdot 1460 \cdot 10}{23 \cdot 350.6} = -43.5 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -43.5 \cdot 10^{-1}$

$RSL = 42y$ \rightarrow GWP = $\frac{26.74 \cdot 1460 \cdot 10}{42 \cdot 350.6} = -23.8 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -23.8 \cdot 10^{-1}$

2. Plywood $\rightarrow V = 19.76 \text{ } [m^3]$

$RSL = 23y$
 $A = 350.6 [m^2]$ } GWP = $\frac{19.76 \cdot 734 \cdot 10}{23 \cdot 350.6} = -16.1 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -16.1 \cdot 10^{-1}$

$RSL = 42y$ \rightarrow GWP = $\frac{19.76 \cdot 734 \cdot 10}{42 \cdot 350.6} = -8.94 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -8.94 \cdot 10^{-1}$

3. Gypsum board $\rightarrow V = 9.12 \text{ } [m^3]$; Area = $446 (43 \cdot 44 + 2 \cdot 2 \cdot 8) + (47 \cdot 06 + 2 \cdot 2 \cdot 8) = 506.8 \text{ } [m^2]$

$RSL = 23y$
 $A = 350.6 [m^2]$ } GWP = $\frac{9.12 \cdot 311 \cdot 10}{23 \cdot 350.6} = 1.75 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow 1.75 \cdot 10^{-1}$

$RSL = 42y$ \rightarrow GWP = $\frac{9.12 \cdot 311 \cdot 10}{42 \cdot 350.6} = 0.91 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow 0.91 \cdot 10^{-1}$

4. total

$23y \rightarrow -43.5 - 16.1 + 1.75 = -57.85 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -57.8 \cdot 10^{-1}$

$42y \rightarrow -23.8 - 8.94 + 0.91 = -31.83 \cdot 10^{-1} \text{ } \text{ } [CO_2\text{-eq}/y/m^2]$ $\rightarrow -31.7 \cdot 10^{-1}$

Excel steel environmental impact HSB validated

Figure X.223: Validation environmental impact (3)