Stabilisering av fleravåningsbyggnader i trä

Stability design of multi-storey timber structures

Thomas Brajerski
Abstract

Stability design of buildings is an extremely important discipline within the field of structural engineering.

This report demonstrates, with help of a worked example, the theory behind the stabilisation of multi-storey timber structures. The worked example is an existing 6-storey building, primarily made with cross laminated timber (CLT). The construction project is located in central Umeå and is designed and managed by Martinsons Byggsystem. The building uses an array of CLT wall panels as a stabilising system.

A number of key points that will be looked at in the worked example; firstly, analysing the loads acting on the building, specifically the calculation of lateral wind loads and their effect on stabilising CLT panels; secondly, a deeper look into the methods of stabilisation using CLT walls. This will include the stability checks for overturning, sliding and shear and counter measures taken against any potential instability. Finally, a look into how CLT walls can be designed by hand calculations according to Eurocodes. There is no particular predefined method to do this, so this report will show how to design a CLT wall by treating CLT as either a homogeneous material or a non-homogeneous/composite material.
Sammanfattning


Foreword

This report is part of my final thesis project in Civil Engineering, BSc, at Umeå University (Högskoleingenjörsprogrammet i byggteknik, 180 hp). The project lasted between January 2014 and March 2014, and was based on a current project managed by Martinsons.

Throughout my whole education, I have been most interested in the structural mechanics aspect of my degree, specifically working with structural timber. I was fortunate enough to be able to work with Martinsons and was granted an insight to working with and designing CLT structures. I would like to thank Håkan Risberg, project supervisor at Martinsons, for giving me the opportunity to work with Martinsons and on his input in choosing a suitable and challenging project for me. My main supervisor at Martinsons was David Rönnqvist, project manager, to whom I would like to thank for his time and help. I thank also Bas Boellaard, structural engineer at Martinsons, for his technical and theoretical assistance; his input was vital to the progression of my project.

I would also like to extend my gratitude to Annika Moström, my supervisor at Umeå University. Her guidance was essential in shaping my project and the report itself.

I feel that now, thanks to having worked in this project for some time, I have gained much experience, knowledge and skills that give me a significant boost in entering the professional world of engineering.

Thomas Brajerski

Umeå, March 2014
# Table of Contents

1. Introduction ......................................................................................................................................... 1  
   1.1. Background .................................................................................................................................. 1  
      1.1.1. Martinsons ............................................................................................................................. 1  
      1.1.2. Brief introduction to Cross-Lamited Timber (CLT) .............................................................. 1  
      1.1.3. Embla 5 project ..................................................................................................................... 3  
   1.2. Aim ............................................................................................................................................... 4  
   1.3. Goals ............................................................................................................................................. 4  
   1.4. Limitations.................................................................................................................................... 4  
   1.5. Eurocodes and equations in this project ....................................................................................... 5  

2. Theory ................................................................................................................................................. 6  
   2.1. Design loads to Eurocodes ........................................................................................................... 6  
      2.1.1. Ultimate limit states (ULS) ................................................................................................... 6  
      2.1.2. Imposed loads ........................................................................................................................ 7  
      2.1.3. Snow loads ............................................................................................................................ 7  
      2.1.4. Wind actions .......................................................................................................................... 8  
      2.1.5. ‘Psi’ factors used in project. (ψ) ........................................................................................ 10  
   2.2. Structural mechanics/analysis of loads ....................................................................................... 11  
      2.2.1. Wind actions ........................................................................................................................ 11  
      2.2.2. Vertical Loads transfer ........................................................................................................ 14  
      2.2.3. Horizontal Loads transfer .................................................................................................... 14  
      2.2.4. Structural analysis of CLT shear wall panels ...................................................................... 16  
   2.3. Stability ...................................................................................................................................... 17  
      2.3.1. How to choose stabilising elements .................................................................................... 17  
      2.3.2. Stability checks.................................................................................................................... 21  
   2.4. Material properties of CLT ......................................................................................................... 24  
      2.4.1. Design value of CLT (EC5-1-1, 2.4.1) ................................................................................ 24  
      2.4.2. Characteristic strength values for CLT ................................................................................ 25  
      2.4.3. Composite theory ................................................................................................................ 26  
      2.4.4. Steiner’s theorem applied to CLT ....................................................................................... 27  
   2.5. Design of CLT wall panels to Eurocode 5 ................................................................................. 29  
      2.5.1. Section properties ................................................................................................................ 29  
      2.5.2. Buckling .............................................................................................................................. 30  
      2.5.3. Combined axial & bending checks to EC5, 6.3.2............................................................... 31  
      2.5.4. Shear check to EC5, 6.1.7 ................................................................................................... 32
1. Introduction

Two aspects of engineering will be focused during this report: the stability of structures and modern timber materials (used for stability).

The ability to stabilise a structure well and have to the foresight to optimise design is the difference between a good engineer and an average one. Stability design is, therefore, seen as one of the most important disciplines of structural mechanics. The engineer must be able to direct the transfer of loads and load actions between structural members in a very specific way; it is as much an art form as it is a science.

There is a growing trend within construction and architecture to use timber in structural design. It is seen to be an aesthetically pleasing, more environmentally friendly alternative to using steel and concrete; although often they are combined with these two materials. Timber products, such as, glued-laminated timber (glulam) and cross-laminated timber (CLT) are increasingly becoming more commonplace as they are strong, light and easily assembled. In this report timber’s environmental aspects will not be focused on, just its structural properties.

1.1. Background

1.1.1. Martinsons

Martinsons is a company based in northern Sweden and is Sweden’s largest producer of glulam and CLT, along with a whole range of other timber products. Approximately half of their produced glulam and CLT is exported overseas for various projects within Europe. Martinsons is unique in the sense that it deals in the whole timber spectrum from forestry to timber production to engineering consultancy. They undertake a wide variety of projects, varying from timber bridges to multi-storey buildings and open long-spanning sports halls.

1.1.2. Brief introduction to Cross-Laminated Timber (CLT)

Cross-laminated timber (CLT for short) is one of the products produced by Martinsons in their factories. CLT panels made up of multiple layers of C24 grade timber panels that are glued together alternating at 90 degrees.

The panels are produced in widths of 1.2m with lengths up to 12m and beyond, although longer lengths can produce greater logistical and transportation difficulties. These 1.2m wide panels can be connected together in the factory to produce prefabricated walls; with openings such as, windows and doors. Prefabricated walls can be delivered directly to building site and allow for easy and rapid assembly. The longer spanning CLT panels are usually used for floors and roof systems.

CLT has great load-bearing capacities that rival steel and concrete and at only a fraction of the weight. Having a light weight has its advantages: complete prefabricated panels can be lifted with relative ease, speed and precision by cranes on site; a lighter self-weight also allows to possibility of building extensions to existing buildings. Horizontal floors panels can span 7m without support.
Figure 1.2 shows some basic standard connections between panels that make up walls. Connecting walls and floors may have different connections depending on their design specifications.

The example on the top shows how two panels can be connected and reinforcement by plywood sheets, which have a greater bearing capacity toward localised stress. This method is how wall panels are connected on the Embla 5 project.

Figure 1.3 shows an assortment of CLT panels that are produced in the Martinsons CLT factory.

70mm-170mm CLT panels are usually used as walls; their larger sizes are used for floors and roofs. Characteristic strength values are shown in section 2.4.2. Characteristic strength values for CLT.

In the Martinsons CLT factory they can are the construct full walls, with openings, to mm precision ready to be delivered on site. The panels are light enough to be lifted into place by cranes.
1.1.3. Embla 5 project

Embla 5 is situated in the centre of Umeå. It is a 6-storey side extension being built adjacent to an existing building and will add office space to the increasing demand. At the time of writing this report the project has completed the design phase and entered the construction phase.

*Figure 1.3* shows a side elevation of the project. The green section shows the original building; the blue section was the original extension, which was a Martinsons project, completed spring 2014; and the red section is the current project.

Like the previous extension, the building is constructed from CLT walls and floors, and glulam beams and columns to support the floors.

The ground and bottom level’s wall are being constructed of a combination of glulam post and beam structure and prefabricated concrete walls. WSP were contracted to design and deliver the prefab-concrete walls, however, it was Martinsons who did the load analysis of the whole structure.

Floors two to six of the extension consist of glulam columns and beams; outer and inner CLT walls and CLT floor/ceiling panels. The floor panels are being supported by the glulam columns and beams and by the CLT walls. CLT walls that have openings (doors or window) are reinforced with plywood to compensate for acting stresses due to lateral loading; each CLT wall will be reinforced, even if it is not a stabilising member, to irradiate any manufacturing mistakes and to simplify on-site erection. Each floor also has a cantilevered CLT balcony.

Floor heights are relatively irregular and are dictated by the original building and its extension.

On level 5 of the building there will be an archive room that will influence the imposed loads.
1.2. Aim

The aim of this project is to gain an understanding of how CLT can be as a stabilising system in multi-storey timber structure. The study will look at the structural aspects of designing CLT elements with respect to stability design; the structural analysis of load transfers throughout the building and additional counter-measures used when stabilising with CLT.

A prerequisite to stability design is the understanding and analysing lateral loads, in this case, wind loads. During this project, a great part would be learning how to calculate acting wind loads and their impact and influence of stability and the general design of CLT elements.

Throughout the project great detail will be paid to design to Eurocodes.

1.3. Goals

Of the four tasks below the first two are considered to be primary tasks that will be supported by calculations. The following tasks are considered as secondary assignments and will be presented in case study form.

- Loads and load actions analysis of the building structure.
  - Vertical loads and actions:
    - Dead loads
    - Imposed loads
    - Snow loads
    - Wind actions (lift)
  - Horizontal actions:
    - Wind actions
- Identification and design of stabilising structural members.
  - Perform stability checks for CLT shear walls.
  - Design of CLT shear wall to Eurocode specifications.
- Exploration into measures taken to combat instability on the Embla project.
  - Connections to prevent sliding.
  - Anchorage against uplift.
- Study how the connections between the 1.2m panels that make the stabilising shear wall.
  - Stress analysis using finite element method
  - Plywood reinforcements.

1.4. Limitations

The main limitation of this project is time; and, as a result, working boundaries and assumptions can be implemented to work more efficiently.

- Only the structural members that will be acting as stabilisers are to be designed and studied. All others’ dimensions can be taken from the existing designs.
- The architect’s initial design should not be altered.
- All designs must be to Eurocode standards.
- When choosing CLT walls, which have openings, for stabilisation use the Martinsons’ template. (figure 2.14)
- Design calculations should be attempted by hand to gain knowledge.
- Additional details such as fittings and insulation do not have to be included in the final design.
1.5. Eurocodes and equations in this project

Equations and codes are that will be used in this project will be come from a number of different sources, such as: structure mechanics books, reference handbooks and the Eurocodes themselves.

At times when using or referencing Eurocodes they will be given in their simplified form to suit their application. If so, they will be accompanied by the abbreviation (EKS 9). This comes from BOVERKET, The Swedish National Board of Housing, Building and Planning, who provide practical interpretations of the standard Eurocodes and apply them to the Swedish system of building.

When using an equation or guideline from Eurocodes, they will be accompanied with their Eurocode reference, including article and equation number. Eurocodes themselves will be written in shorthand mimicking many course books and engineers’ working notes. For example, (SS EN 1991-1-4:2005, 1.1.1) will be written as (EC1-1-4, 1.1.1).

Referencing examples are shown below.

e.g.

\[ \gamma = x + y \]  

(EC1-x-x, xx,x,x EKS 9)
2. Theory

2.1. Design loads to Eurocodes

2.1.1. Ultimate limit states (ULS)

Ultimate limit states deal with direct failure of structures such as collapse and factors that present a safety risk to people.

2.1.1.1. STR

“STR: relating to internal failure or excessive deformation of a structure or structural member”

(EC0, 6.4.1)

The STR ultimate limit state is used for strength checks when designing structural members; it takes into account all variable loads.

The equation below shows the load combination for *unfavourable* strength checks.

\[
Q_d = \gamma_d 1.2G_{k,j,\text{sup}} + \gamma_d 1.5Q_{k,1} + \gamma_d 1.5\psi_0,iQ_{k,i}
\]

(EC0, EKS 9, equ. 6.10b)

Where,

- \(Q_d\): Design load
- \(\gamma_d\): Partial safety factor according to EKS 9
- \(G_k\): Characteristic values of permanent actions
- \(Q_{k,1}\): Characteristic value of *leading* variable actions
- \(Q_{k,i}\): Characteristic values of *accompanying* variable actions
- \(\psi_0\): Combination factor for \(Q_{k,i}\)

For this particular project the variable actions are snow loads, wind actions and various imposed load.

2.1.1.2. EQU

“EQU: relating to the static equilibrium of a structure or any part of it which is considered as a rigid body”

(EC0, 6.4.1)

The EQU ultimate limit state is used for stability checks; it only takes into account the *leading* variable loads, which in this case are wind actions.

The equation below shows the load combination for *favourable* strength checks.

\[
Q_d = \gamma_d 1.1G_{k,j,\text{sup}} + \gamma_d 1.5Q_k
\]

(EC0, EKS 9, equ. 6.10)

Where,

- \(Q_d\): Design load for lift
- \(G_k\): Characteristic values of permanent actions
- \(Q_k\): Characteristic value of variable actions
2.1.2. Imposed loads

Guidelines for imposed loads are given in **EC1-1-1**. Varying factors are taken into account for this such as furniture, movement of people, equipment etc.

**Imposed loads within this project:**  
(EC1-1-1, 6.3.1.2)

- **Category B: office areas** give a load of \( 2.5 \, kN/m^2 \)
- The balconies give a load of \( 3.5 \, kN/m^2 \)
- Level 5 archive room has a imposed load of \( 7.5 \, kN/m^2 \)

When dealing with imposed loads one must also take into consideration reduction factors.

**For floors, beams and roofs:**

\[
\alpha_A = \frac{5}{7} \psi_0 + \frac{A_0}{A} \leq 1.0 \\
\text{(EC1-1-1, equ. 6.1)}
\]

Where,
- \( \alpha_A \) Reduction factor
- \( \psi_0 \) Combination factor given in EC1-1-1 and EKS 9
- \( A_0 = 10.0 \, m^2 \)
- \( A \) Loaded area

**For columns and walls:**

\[
\alpha_n = \frac{2+(n-2)\psi_0}{n} \\
\text{(EC1-1-1, equ. 6.2)}
\]

Where,
- \( \alpha_n \) Reduction factor
- \( \psi_0 \) Combination factor given in EC1-1-1 and EKS 7
- \( n \) Number of storeys (> 2) above of the same category

2.1.3. Snow loads

The snow load is given by the formula below

\[
s = \mu_i C_e C_t s_k \text{ [kN/m}^2\text{]} \\
\text{(EC1-1-3, equ. 5.1)}
\]

Where,
- \( s \) Snow load
- \( \mu_i \) Snow load coefficient
- \( C_e \) Exposure coefficient dependant on the local typography
- \( C_t \) Thermal coefficient
- \( s_k \) Characteristic value for snow on the ground for a particular region
2.1.4. Wind actions

Wind actions are very important to calculate as, in this case, they will be acting as horizontal forces that will need to be stabilised against. However, wind actions do not just act as horizontal load; they can also produce a vertical lift. Timber structures, as opposed to steel and concrete, are far lighter in weight so extra attention is needed to combat uplift; as self-weight of timber is often insufficient to cancel this. When calculating the wind, they can be separated into both internal and external surface pressures.

**Terrain category and basic wind velocity**

There are four terrain categories for calculating wind loads. The Embla 5 project is placed in terrain category III which states:

“Area with regular cover of vegetation or buildings or with isolated obstacles with separation of maximum 20 obstacle heights (such as villages, suburban terrain or permanent forest)”

(EC1-1-4, table 4.1)

Basic wind velocity \( (v_b) \) is given as dependent on geographical location can is given in EKS 9. For Umeå: \( v_b = 22 \text{ m/s} \).

**External pressure:**

\[
w_e = q_p(z_e) \cdot c_{pe} \quad \text{(EC1-1-4, equ. 5.1)}
\]

**Internal pressure:**

\[
w_i = q_p(z_i) \cdot c_{pi} \quad \text{(EC1-1-4, equ. 5.2)}
\]

Where,

- \( w \) Wind load
- \( q_p(z_e) \) Maximum velocity for external pressure for a specific reference height \( (z_e) \)
- \( q_p(z_i) \) Maximum velocity for internal pressure for a specific reference height \( (z_i) \)
- \( c_{pe} \) Pressure coefficient for external pressure
- \( c_{pi} \) Pressure coefficient for internal pressure

In this project, the external pressures and suction (negative pressures) are combined as one coefficient. This gives a slightly larger pressure value and is accepted as a more conservative design load. These individual coefficients can be found within various tables in section 7, EC1-1-4 (“Pressure coefficients for buildings”), and are dependent on building and roof forms as well as wind direction.

Martinsons tend to use the same combined pressure coefficient \( (\mu_{\text{wind}}) \) on many different projects, as long as they meet the same criteria i.e. terrain category and basic wind velocity\((v_b)\), as well as roof form. This pressure coefficient will be used in the calculation section.
Wind surface pressure load (combined coefficients):

\[ w_k = q_p(z) \cdot \mu_{wind} \quad [kN/m^2] \]

Where,

- \( w_k \): Characteristic wind load, surface pressure
- \( q_p(z) \): Maximum velocity pressure for a specific reference height \( z \)
- \( \mu_{wind} \): Combined positive and negative \( c_{pe} \) values ([1] p. 6.3)

Calculating max. velocity pressure \( q_p(z) \):

There are different variations to calculating max. velocity pressure \( q_p(z) \). The simplest method is to take the characteristic values for specific heights from a pre-calculated table (Table C-10a, EKS 9). However, this does not take into account of the terrain factor, wind turbulence nor peak velocity pressure. The more accurate method, which is used in this project, is shown below.

\[ q_p(z) = [1 + 6I_v(z)] \cdot [k_r \cdot \ln \left( \frac{h}{z_0} \right)]^2 \cdot q_b \quad \text{(EKS 9, 4.5(1))} \]

Where,

- \( 1 + 6I_v(z) \): Also referred to as \( C_{dy, gn} \)
- \( [k_r \cdot \ln \left( \frac{h}{z_0} \right)]^2 \): Also referred to as \( C_{exp} \)
- \( q_b \): Basic velocity pressure
- \( I_v(z) = \frac{1}{\ln \left( \frac{h}{z_0} \right)} \): Turbulence intensity factor (EC1-1-4, (4.7))
- \( h \): Height of building or structure
- \( z_0 \): The roughness length, dependent on terrain (EC1-1-4, table 4.1)
- \( k_r = 0.19 \cdot \left( \frac{z_0}{z_{0,II}} \right)^{0.07} \): Terrain factor (EC1-1-4, (4.5))
- \( z \): Height of acting load
- \( z_0 \): The roughness length, dependent on terrain (EC1-1-4, table 4.1)
- \( z_{0,II} \): The roughness length of terrain category II (EC1-1-4, table 4.1)
- \( q_b = \frac{1}{2} \cdot \rho \cdot v_b \)
- \( \rho = 1.25 \text{ kg/m}^3 \): Density of air
- \( v_b \): Basic wind velocity (EKS 9)
Calculating surface pressure \( q_p(z) \) on lower levels:

With accordance to (EC1-1-4, 7.2.2), for vertical walls of rectangular planned buildings.

Up to the height equal to the base:

\[
q_p(z) = q_p(b)
\]

However, Martinsons usually add an extra 0.5 m for added safety.

\[
q_p(z) = q_p(b) + 0.5 \quad (z < b)
\]

Where,

- \( q_p \) : Surface pressure
- \( z \) : Height of acting load
- \( b \) : Length of distributed load

2.1.5. ‘Psi’ factors used in project. (ψ)

‘Psi’ factors are used to reduce non-leading variable loads.

<table>
<thead>
<tr>
<th>Load</th>
<th>( \psi_0 )</th>
<th>( \psi_1 )</th>
<th>( \psi_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imposed load: category B – office areas</td>
<td>0.7</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>Snow load: ( s_k \geq 3 \text{ kN/m}^2 )</td>
<td>0.8</td>
<td>0.6</td>
<td>0.2</td>
</tr>
<tr>
<td>Wind Load</td>
<td>0.3</td>
<td>0.2</td>
<td>-</td>
</tr>
</tbody>
</table>
2.2. Structural mechanics/analysis of loads

The diagram above shows the different dimensions used within the structural analysis calculations. The red wall shows the stabilising CLT element. The breadth of that wall panel is dictated by the manufacturing process i.e. the length in which it is delivered to the building site. The effective length is the actual length minus 0.5 m.

\[ l = l_{\text{actual}} - 0.5 \text{ m} \]

This gives an added factor of safety and takes into account the joining on either side of the wall panel.

2.2.1. Wind actions

2.2.1.1. Calculating point loads from surface pressure:

*Figure 2.3* shows how lateral wind actions can be seen to affect a building or an array of wall panels.

The wind surface pressure is calculated by the equation shown in section 2.1.4. *Wind actions*. The wind point load can then be centralised by multiplying the wind surface pressure by the distributing load length (shown *figure 2.2.*) and the acting height \((z)\); which is the aggregate of half the level height above the point load and half the level height below the point load.

Wind point load:

\[ F_{\text{wind}} = w_k \cdot z \cdot y \ \text{[kN]} \]

Where,

- \( F_{\text{wind}} \) Wind point load [kN] from surface pressure
- \( w_k \) Characteristic wind load [kN/m] shown in section 2.1.4. *Wind actions*
- \( z \) Acting force height i.e. half the level height from below and above of the loaded point.
- \( y \) Length [m] of wind surface pressure distributed load
2.2.1.2. Calculating in plane moments:

The equation below shows how to calculate moment acting against a wall or wall panel.

\[ M = F_{\text{wind}} \cdot h \quad [kNm] \]

\[ M_i = \sum_i (F_{\text{wind}} \cdot z)_i \]

Where,

- \( M \) Moment [kNm] created by wind actions
- \( F_{\text{wind}} \) Wind point load [kN] from surface pressure
- \( h \) Distance [m] from bottom of the element experiencing the moment to the point load

The aggregate total moment is used in the analysis of each CLT wall panel. Figure 2.4. illustrates this. Only horizontal loads above the CLT panel are used to calculate the moments.

To calculate the moment of a particular wall panel, the entire wall’s moment can be proportionately divided among different panels.

\[ M_{\text{panel}} = M \cdot \frac{l}{L} \quad [kNm] \]

Where,

- \( M_{\text{panel}} \) Panel’s moment [kNm]
- \( M \) Wall’s moment [kNm] created by wind actions
- \( l \) Effective length [m] of panel
- \( L \) Total length [m] of wall
2.2.1.3. Calculating cross plane moments

When calculating cross plane moments acting upon a CLT panel, as shown in figure 2.6, it must be assumed that there is sufficient stability resistance against the direction of the wind pressure. With this assumption it is possible to treat the wall as a beam with a uniformly distributed load, as shown in figure 2.7, that has a maximum moment and deflection in the middle. The maximum moment can be calculated by the equation below.

**Cross plane moment**: 

\[ M_{\text{max}} = \frac{q_d \cdot h^2}{8} \text{ [kNm]} \]

Where,

- \( M_{\text{max}} \) Maximum moment [kNm]
- \( q_d \) Design load [kN/m]
- \( w_k \) Characteristic wind load [kN/m] shown in section 2.1.4. Wind actions
- \( l \) Length [m] of panel of which wind surface pressure is action upon
- \( h \) Height [m] of panel

2.2.1.4. Calculating horizontal reactions:

To calculate the lateral forces at each level all the loads that are above the floor are added together. This is a conservative way to view lateral loading as it works with the assumption that there are no reactions working against the loads. This is mostly true for stand-alone structures but if there are structures side by side then those can, theoretically, reduce horizontal loading.

As the horizontal loads act upon the whole length of the wall, the force can be divided by that length to give a distributed load. This distributed load is used in the design of individual CLT wall panels.
**Horizontal reaction:**

\[ \sum F_{\text{wind}} \quad [kN] \]

**Distributed horizontal reaction:**

\[ \frac{\sum F_{\text{wind}}}{L} \quad [kN/m] \]

Where,

- \( H_{Q,\text{wind}} \) - Horizontal reaction load [kN]
- \( H_{Q,\text{wind}} \) - Horizontal reaction load [kN/m]
- \( F_{\text{wind}} \) - Wind load point
- \( L \) - Total length [m] of wall

### 2.2.2. Vertical Loads transfer

*Figure 2.9* shows how forces are transferred vertically. Forces are transferred down uniformly through the CLT walls. The loads from the floors/roof are added at each level and act upon the top of the CLT walls. Roof loads consist of self-weight and snow loads whilst the floor loads consist of self-weight and imposed loads. Reaction forces, at each of the extreme ends of the wall panels, can be taken at every level.

### 2.2.3. Horizontal Loads transfer

Horizontal load tend to be transferred through the stiffest elements. Within this project the forces are transferred either through the CLT shear walls or through CLT floor/roof diaphragms, depending on the location of the stabilising element.

#### 2.2.3.1. Load transfer through shear walls:

*Figure 2.10* illustrates how the horizontal loads are transferred through shear walls (as in this case they are the stiffest elements).

The green arrows represent lateral forces, in this case, they are as a result of horizontal wind loads but they can also be combined with other horizontal forces. (For this project only wind actions are calculated as lateral forces).
The red wall panel is the stabilising element and the blue panel are discrete, load-bearing elements.

The green arrows (wind actions) converge on the shear wall and get transferred directly to and through the stabilising element. That element then has to be designed to withstand these forces which can manifest through shear or sliding force and overturning (uplift). However, it is possible to have more than stabilising panel along a length of a wall so it is permissible for some horizontal loads to be transferred beyond the initial stabilising element.

Varying surface pressures can be observed on the exposed surface as shown in figure 2.3. This diagram only demonstrates the load path.

2.2.3.2. Load transfer through floor diaphragms:

*Figure 2.11* shows how lateral forces will behave on a floor diaphragm. The red panel shows the stabilizing element and the blue walls, column and beams are discrete, load-bearing elements. The green shading represents the load acting on the stabilising element.

The loads acting on the exposed wall surface are distributed equally to the upper and lower floor diaphragms. The floor diaphragms are designed to transfer the whole horizontal load; hence, there is no load transfer through the beams and column. Although, in reality, there would be some transfer.

A characteristic of CLT panels is that they can transport loads in two directions, so it does not matter in which direction they span to transfer loads. As long as the different spanning floors are connected to each other, the floor closest to the stabilizing wall; they will receive direct pressure from each other. It is also vital that the joints between the floor panels offer enough stiffness; especially if the load transfer is in the direction perpendicular to the longitudinal span.

Once the loads reach the stabilising element (or the wall that contains the stabilising element); from both the upper and lower floor diaphragms, they behave as discussed in section 2.2.3.1. *Load transfer through shear walls.*
2.2.4. Structural analysis of CLT shear wall panels

Figure 2.12 shows a free body diagram of how different forces act upon a CLT wall.

Wind loads are covered in section 2.1.4. Wind actions.

Moments resulting from wind loads are covered in section 2.2.1.2. Calculating in plane moments.

Design vertical loading is covered in section 2.2.2. Vertical loads transfer & 2.1.1. Ultimate limit states (ULS).

Sliding force will be discussed and covered in the stability section. See section 2.3.3.3. Sliding resistance.

**NOTE:** different design loads and load combinations are used for different favourable and unfavourable loading scenarios during the design process. Refer back to section 2.1.1. Ultimate limit states (ULS).

### 2.2.4.1. Vertical reactions:

In the calculation of reactions, design values are used to (EC0, 3.3 & EKS 9). Negative results show a lift at the reaction points

\[
R_a = \left( \frac{q_d \cdot l}{2} \right) - \left( \frac{M_{\text{panel}}}{l} \right) \quad [kN]
\]

\[
R_b = \left( \frac{q_d \cdot l}{2} \right) + \left( \frac{M_{\text{panel}}}{l} \right) \quad [kN]
\]

Where,

- \( q_d \) Aggregate distributed design load [kN/m] acting on top of the panel
- \( l \) Effective length [m] of panel

The panel’s moment is calculated to be as shown below:

\[
M_{\text{panel}} = M_d \cdot \frac{l}{L}
\]

- \( M_d \) The design moment [kNm] of a wall
- \( L \) Total length [m] of wall
Substituting for $M_{panel}$ gives:

At point a:

$$R_a = \left( \frac{q_d \cdot l}{2} \right) - \left( \frac{M_d}{L} \right) \quad [kN]$$

At point b:

$$R_b = \left( \frac{q_d \cdot l}{2} \right) + \left( \frac{M_d}{L} \right) \quad [kN]$$

2.3. Stability

When designing CLT walls for stability, it must be ensured that the walls can withstand overturning, shear forces and sliding which arise from lateral loads. According to EC0, 3.3, specific stability checks must be performed for the following:

- Global (whole building) overturning
- Global sliding
- Individual wall panel overturning
- Individual wall panel sliding
- Individual wall panel shear

All design loads must be calculated to ULS (EQU) (SET A) during stability checks.

2.3.1. How to choose stabilising elements

Before checks against stability (listed above and (EC0, 3.3)) can occur, first the engineer must choose which structural element will be utilised for stabilisation.

Choosing how to stabilise a structure is as much an art form as it is a science; there are many different possible solutions, each with their own advantages and disadvantages. Often engineers have to go deep within their calculations to discover different ways to optimise their designs.

*Figure 2.13* shows a very simplified plan layout based on the Embla 5 project and will be used as an example to illustrate the various steps and considerations taken when choosing stabilising elements. The example structure is surrounded by stronger outer CLT shear walls, which have window openings; inner CLT inner; columns and an elevator shaft. The proposed building is also conjoined to an existing, adjacent building as in the case of the Embla 5 project. The plan also shows a balcony that will have a door opening to access it.

At the beginning of a project, an engineer will receive drawings from the architects, along with other specifications, from which they must establish a stability system.
When designing a building stability systems from CLT there are some key issues/criteria to consider:

- Stabilising walls must continue (be connected) on every level. The engineer must check all the architect’s drawings to make sure all potential stabilising elements are aligned. This is particularly important for stabilising inner-walls; it is also important if floor areas are extended on lower levels.

- It is advisable (but not essential) to use walls surrounding elevator shafts. This is because the wall panels are usually stronger and thicker, hence stiffer, than other inner-walls as they have to withstand greater loads due to moving elevators. They also run the whole height of the building. This is the same for stair-wells.

- The way in which the lateral loads should be distributed is vital to assess. The loads should be attempted to be distributed evenly by different stabilising elements; taking into account that winds loads can act from multiple directions.

- The spacing and positioning between stabilising elements is also a factor, as more often than not, the span of a wall may require several of them. If possible, they should be evenly spaced; however, this is rarely the case especially if the building’s design is asymmetrical and non-uniform.

- Openings, such as, windows or doors significantly reduce the stiffness and load-bearing capacities of CLT walls. This is one of the most contributing factors when choosing suitable stabilising wall panels. Martinsons have a template (shown figure 2.14) to show if a wall panel can be used as a stabilising element. If there are no outer walls that match the template criteria, then the engineer would have to discuss with the clients/architects to adjust the initial design of the building to accommodate the need for stabilising outer walls. If a compromise cannot be met to adjust designs, then the engineer will have to seek alternative ways of stabilisation.

![Figure 2.14 Martinsons’ prerequisite template for stabilising CLT panels.](image)

When potential stabilising elements match this criterion then a control (and added reinforcements) must be made in the joints between panel sections during the production
process. This is because they experience great stresses as a result of combined axial compression and lateral actions.

Once choosing stabilising walls that satisfy the openings criteria, it is important to double check that all the walls on each level can be used for stabilisation. It is possible that each individual level has its different opening dimensions.

The walls that do not match the template criteria can still be used as load-bearing element as it is possible to reinforce them or reduce loads with columns and beams.

- Manufacturing lengths of walls panels also dictate the choice of stabilising elements. Lengths can be determined by the engineer for manufacture but they usually should fit within some guidelines. Often an engineer would choose two shorter stabilising panels rather than one oversized one, as they offer more stiffness. The manufacturing lengths are usually between 6-10 m but can go up to 12+ m depending on transportation and erection methods.

- The paths which lateral loads are transferred are important in choosing stabilising element. It is best if loads have a direct route to the stabilising element and will always be transferred through the stiffest structural members. Stiffness checks should be carried out, especially for floor diaphragms.

- Although it is difficult to say if any specific joining will be necessary to combat overturning or panel sliding before a load analysis is completed, the engineer must still make sure that it is possible that such joints and anchors could be placed adequately.

Example: wind blowing from north

![Diagram of stabilising system](image)

*Figure 2.15 Example of stabilising system when wind blowing from north.*

*Figure 2.15* gives an example of a possible combination of stabilising wall panels (red, purple, green and orange). It shows how the wind surface pressure is distributed between the different stabilising wall panels according to load lengths which are spaced out depending on the positioning of the stabilising wall panel. The loads are distributed evenly between the stabilising elements. However, this is only true if the wall (which is having loads applied to) has the same constant stiffness throughout the wall surface. In most cases, this is a safe assumption for calculation.
**Red panel:**
The wall containing the red stabilising panel has a door opening; that particular wall panel would be inadequate for stabilisation. The remainder of the wall has many window openings that satisfy the openings template criteria. The engineer must choose a suitable length of panel to fit within the openings’ criteria and that can withstand the lateral load.

**Purple panel:**
The load here is being transferred through the floor diaphragm and has a larger load to withstand than the red panel as the distributed length is a + b. Again here the engineer must choose a suitable length that can withstand the lateral loads and also fit the stabilisation criteria. Stiffness checks should be done on the floor CLT panels.

**Green panel:**
The stabilising panel here is the inner-wall surrounding the elevator; it is short but fairly stiff as it is thicker than other inner-walls because it has a greater load-bearing capacity. There are no openings to take into account.

**Orange panel:**
The position and length must be decided to withstand the loads, much like the red and purple panels.

**Example: wind blowing from west**

![Diagram](image)

*Figure 2.16 Example of stabilising system when wind blowing from west.*

*Figure 2.16* gives an example of a possible combination of stabilising wall panels (red, purple, green and orange) when the wind is blowing from the side. It shows that the existing neighbouring building is reducing the load on the purple and orange wall panels. For this reduction to be valid checks have to be made. For the safest scenario, no reductions should be implemented.

**Red panel & Green panel:**
The loads are relatively small to be distributed between the two panels; they are transferred through the length of the walls. The panels are adequately spaced from one another at either end of the building side. It is good to have stabilising panels at either end of a building in case of the wind is blowing from each direction. In this case, because of the relatively small loads, it is possible for the panels to act independently withstanding lateral loads from each individual direction.

**Purple panel & Orange panel:**
The load here is being transferred through the floor diaphragm. Stiffness checks should be done on the floor CLT panels. As the panels will be acting together to counteract the lateral loads they can be drawn like the free body diagram below (figure 2.17). The diagram shows three adjacent stabilising wall panels as opposed to two on the plan. This just shows that it is possible to have multiple stabilising elements together to withstand the same load. (The calculation of these moments and reaction is shown in section 2.1.2. Calculating in plane moments.)

![Free body diagram showing how the length of a stabilising element is dependent to determining its moment. Calculation of moments is covered in section 2.2.1.2. Calculating in plane moments.](image1)

2.3.2. Stability checks

As mentioned before, stability checks must be performed on both the whole building and on individual stabilising panels. In this project only individual panel checks are performed, however, the theory is the same.

![Free body diagram showing the loads and reactions of a stabilising element.](image2)

Figure 2.18 shows a free body diagram of a stabilising element. The reaction at point a has a downward direction, as this is the point where uplift is most likely to occur with a clockwise moment. With an anti-clockwise moment acting on the panel the uplift would occur at point b. Sliding and shear forces are always counter-acting lateral loading.

All stability checks should be taken at critical points so only dead loads and wind actions are considered (EQU ultimate limit state).
2.3.2.1. Shear wall overturning check

The theory behind overturning stability is that the overturning moment (i.e. moments created by lateral wind loads) must be countered by the restoring moment (i.e. moments created from dead loads). This is the same for both whole building overturning and individual panel overturning shown by the equation below.

**Overturning check:**

\[
\frac{M_{\text{restoring}}}{M_{\text{overturning}}} > 1.0
\]

Where,

\[
M_{\text{overturning}} = M_d \cdot \frac{l}{L}
\]  
Moment [kNm] for stabilising panel

\[
M_d
\]  
The design moment [kNm] caused by wind actions, as shown in section 2.2.1.2. Calculating in plane moments and combined with ULS EQU (set A) (EC0, EKS 9, equ. 6.10^5)

\[
l
\]  
Effective length [m] of stabilising panel

\[
L
\]  
Total length [m] of wall

And,

\[
M_{\text{restoring}} = \frac{q_{d,\text{total}} \cdot l^2}{2}
\]  
Total design load [kN/m] (self-weight) of the building acting on the panel to ULS EQU (set A) (EC0, EKS 9, equ. 6.10^5)

**Resistance to overturning:**

If,

\[
\frac{M_{\text{restoring}}}{M_{\text{overturning}}} < 1.0
\]

Then the additional holding down resistance required can be calculated by taking the reaction force at point \(a\), as shown in section 2.2.4.1. Vertical reactions.

\[
R_a = \left( \frac{q_d \cdot l}{2} - \frac{M_d}{L} \right) \quad [kN]
\]

The negative reaction force symbolises uplift.

It must be noted that any fixings used to resist from uplift can then not be used to combat sliding according to EC5, 3.3.

**Alternative method:**

It is possible to bypass taking moments for each panel by just taking reactions at point \(a\) of the stabilising wall to ULS EQU (set A) (EC0, EKS 9, equ. 6.10^5). Positive results show that the panels will withstand overturning; negative reactions show uplift, hence inadequate resistance to overturning.
2.3.2.2. Sliding resistance check

Static resistance to sliding (i.e. without mechanical fastenings) is only achieved through friction. Figure 2.20 shows that horizontal forces, $H$, acting on a body are countered by vertical loads multiplied by the coefficient of friction, $\mu V$.

The coefficient of friction is dependent on the materials' properties that are in contact with each other. Wood to wood usually has a coefficient of friction between $\mu_{\text{wood}} = 0.3 - 0.4$. The CLT panel that are produced by Martinsons for this project is taken to be $\mu_{\text{wood}} = 0.4$. This means that the vertical forces reduce horizontal forces by friction between the wall and floor elements.

**Sliding resistance check:**

\[
\frac{\text{Horizontal load}}{\text{Vertical load}} \leq \mu
\]

**Resistance to sliding:**

If,

\[
\frac{\text{Horizontal load}}{\text{Vertical load}} \geq \mu
\]

Then some additional mechanical fastenings may be required. The mechanical resistance required is shown by the equation below.

\[
F_{so} = H - \mu V
\]
2.4. Material properties of CLT

CLT is obviously a timber product and can be seen to have the same properties as normal timber. When calculating using timber properties, it is seen to be homogeneous, meaning that it has constant properties throughout the material. CLT is a composite material made up of timber laminates spanning in different directions; this gives CLT varying strength and section properties than are different to normal timber. At times it can be assumed that CLT is homogeneous to simplify calculations.

2.4.1. Design value of CLT (EC5-1-1, 2.4.1)

The design values for a CLT material is calculated in the same way as other timber products, using the equation below:

\[
f_d = \frac{k_{mod} \cdot f_k}{\gamma_M} \quad [N/mm^2]
\]

Where,

- \( f_d \): Design value of material calculated to (EC5-1-1, equ. 2.14)
- \( f_k \): Characteristic value of a strength property
- \( k_{mod} \): Modification factor dependant on load-duration and moisture content, shown below
- \( \gamma_M = 1.25 \): Partial factor for material property, value given in (EC5-1-1, tab.2.3)

**Modification factor, \( k_{mod} \):**

*Table 2.2 \( k_{mod} \) values for CLT (EC5-1-1, 3.1.3)*

<table>
<thead>
<tr>
<th>Service class</th>
<th>Load-duration class</th>
<th>P</th>
<th>L</th>
<th>M</th>
<th>S</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>

Modification factors, \( k_{mod} \), are derived from load-duration and service classes shown below.

**Load-duration classes:**

*Table 2.3 Load-duration classes (EC5, 2.3.1.2).*

<table>
<thead>
<tr>
<th>Load-duration class</th>
<th>Duration of characteristic load</th>
<th>Load examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent (P)</td>
<td>More than 10 years</td>
<td>Self-weight</td>
</tr>
<tr>
<td>Long-term (L)</td>
<td>6 months to 10 years</td>
<td>Storage</td>
</tr>
<tr>
<td>Medium-term (M)</td>
<td>1 week – six months</td>
<td>Imposed floor loads, snow loads</td>
</tr>
<tr>
<td>Short-term (S)</td>
<td>Less than a week</td>
<td>Wind loads</td>
</tr>
<tr>
<td>Instantaneous (I)</td>
<td>Wind loads, accidental loads</td>
<td></td>
</tr>
</tbody>
</table>

**Service class (climate class):**

This project has the service class 2 which is stated below:

“Service class 2 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.”
2.4.2. Characteristic strength values for CLT

All of the CLT and glulam products produce at the Martinsons factories have been tested by SINTEF to get accurate and official characteristic strengths. SINTEF is a Norwegian based independent research organisation.

Table 2.4 Material characteristic strength values for CLT laminates.

<table>
<thead>
<tr>
<th>Characteristic Properties</th>
<th>Transverse direction [N/sq.mm]</th>
<th>Longitudinal direction [N/sq.mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>14</td>
<td>24</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>8</td>
<td>14</td>
</tr>
<tr>
<td>Tension</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>0,4</td>
<td>0,4</td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>16</td>
<td>21</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>2</td>
<td>2,5</td>
</tr>
<tr>
<td>Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- laminate thickness 19 mm</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>- laminate thickness 31,4 mm</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>- laminate thickness 44 mm</td>
<td>0,7</td>
<td>1</td>
</tr>
<tr>
<td>Rolling shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- laminate thickness 19 mm</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>- laminate thickness 31,4 mm</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>- laminate thickness 44 mm</td>
<td>0,7</td>
<td>1</td>
</tr>
<tr>
<td>Stiffness properties</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic modulus</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- E_0.05</td>
<td>4700</td>
<td>7400</td>
</tr>
<tr>
<td>- E_0,mean</td>
<td>7000</td>
<td>11000</td>
</tr>
<tr>
<td>- E_0.90</td>
<td>230</td>
<td>370</td>
</tr>
<tr>
<td>Shear modulus</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- G_mean</td>
<td>440</td>
<td>690</td>
</tr>
<tr>
<td>- G_90,mean</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Mean density</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- ρ</td>
<td>400</td>
<td>50</td>
</tr>
</tbody>
</table>

Table 2.4 shows just the characteristic strengths and the materials (laminates) that make up the CLT walls. These values can be used along with composite factors to determine effective strength and stiffness properties.
Table 2.3 Specific characteristic strength values CLT panels.

<table>
<thead>
<tr>
<th>Characteristic Properties</th>
<th>Cross-section thickness</th>
<th>95mm</th>
<th>120mm</th>
<th>145mm</th>
<th>170mm</th>
<th>209mm</th>
<th>259mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>( f_{m,0,k} )</td>
<td>13.3</td>
<td>11.5</td>
<td>10.1</td>
<td>9.9</td>
<td>13.1</td>
<td>12.0</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>( f_{m,90,k} )</td>
<td>2.5</td>
<td>3.8</td>
<td>4.8</td>
<td>4.9</td>
<td>2.7</td>
<td>3.4</td>
</tr>
<tr>
<td>Tension</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>( f_{t,0,k} )</td>
<td>5.9</td>
<td>4.7</td>
<td>3.9</td>
<td>4.4</td>
<td>5.4</td>
<td>5.0</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>( f_{t,90,k} )</td>
<td>2.8</td>
<td>3.6</td>
<td>4.2</td>
<td>3.8</td>
<td>3.1</td>
<td>3.3</td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- parallel to grain</td>
<td>( f_{c,0,k} )</td>
<td>12.6</td>
<td>10.0</td>
<td>8.3</td>
<td>9.4</td>
<td>11.5</td>
<td>10.8</td>
</tr>
<tr>
<td>- perpendicular to grain</td>
<td>( f_{c,90,k} )</td>
<td>6.4</td>
<td>8.4</td>
<td>9.7</td>
<td>8.9</td>
<td>7.3</td>
<td>7.8</td>
</tr>
<tr>
<td>Shear</td>
<td>( f_{v,k} )</td>
<td>1.20</td>
<td>1.58</td>
<td>1.57</td>
<td>1.67</td>
<td>1.36</td>
<td>1.46</td>
</tr>
<tr>
<td>Rolling shear</td>
<td>( f_{R,v,k} )</td>
<td>1.00</td>
<td>1.00</td>
<td>0.70</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Stiffness properties</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>( E_{0.05} )</td>
<td>4630</td>
<td>4000</td>
<td>3510</td>
<td>3460</td>
<td>4540</td>
<td>4180</td>
</tr>
<tr>
<td></td>
<td>( E_{0,mean} )</td>
<td>6920</td>
<td>6010</td>
<td>5290</td>
<td>5220</td>
<td>6790</td>
<td>6270</td>
</tr>
<tr>
<td></td>
<td>( E_{0,90} )</td>
<td>880</td>
<td>1330</td>
<td>1690</td>
<td>1730</td>
<td>940</td>
<td>1200</td>
</tr>
<tr>
<td>Shear modulus</td>
<td>( G_{mean} )</td>
<td>590</td>
<td>559</td>
<td>538</td>
<td>551</td>
<td>576</td>
<td>568</td>
</tr>
</tbody>
</table>

Table 2.3 shows effective characteristic strengths of a few selected CLT panels (Martinsons assortment shown in figure 1.3). These strengths are specifically for sections with a width of 1.2m and assuming that it is a homogeneous material.

2.4.3. Composite theory

The composite factors are used to calculate effective strength and stiffness properties. This is because the laminate components in CLT have different strength and stiffness properties depending on their spans. There are four different loading cases which yield different composite, \( k \), factors:

- \( k_1 \) - cross plane loading parallel to span
- \( k_2 \) - cross plane loading perpendicular to span
- \( k_3 \) - in plane loading parallel to span
- \( k_4 \) - in plane loading perpendicular to span

In the case of designing shear walls the \( k_3 \) composite factor is used as shown in figure 2.21.
Composite, \( k \), factor:

\[
k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{\sum b_{\text{long}}}{b_{\text{total}}}
\]

Where,

- \( E_{90} \) Material elastic modulus perpendicular to the grain
- \( E_0 \) Material elastic modulus parallel to the grain
- \( E_0/E_{90} = 30 \) Assumed value
- \( b_{\text{long}} \) Cross-sectional thickness [m] of laminate longitudinally spanned against the load
- \( b_{\text{total}} \) Total cross-sectional thickness of CLT panel

### 2.4.4. Steiner’s theorem applied to CLT

Steiner’s theorem (also known as parallel axis theorem) is a method used to calculate the second moment of area for cross-sections of beams and columns, as shown in the equation below. *Figure 2.22* shows how CLT is calculated as a non-homogeneous composite material. Only laminates that are spanning perpendicular to compression loads are checked against.

*Figure 2.22 Cross-section of a CLT panel showing dimensions used in Steiner’s theorem.*

**Second moment of area (Steiner’s rule):**

\[
I = \sum (l_i + A_i \cdot e_i^2)
\]

Where,

- Second moment of area for a standard rectangle:
  \[
l_i = \frac{y \cdot b^3}{12}
\]

- \( y \) Arbitrary cross-sectional length [m] of a CLT panel shown in *figure 2.22*
- \( b \) Thickness [m] of laminate shown in *figure 2.22*
- \( A_i \) Cross-sectional area [m\(^2\)] of laminate
- \( e_i \) Distance [m] between the panel’s and laminate’s centre of mass
From the second moment of area, it is possible to derive the radius of gyration as shown in the equation below.

**Radius of gyration:**

\[
i = \frac{I_{eff}}{\sqrt{A_{eff}}}
\]

Where,

- \(I_{eff}\): Effective second moment of area
- \(A_{eff}\): Total effective cross-sectional area of laminates as shown in figure 2.22

The radius of gyration is used to calculate buckling factors covered in section 2.5.5. **Buckling.**
2.5. Design of CLT wall panels to Eurocode 5

The method commonly used by structural engineers to design wall panels is to divide them into smaller sections and treat them as columns. In this chapter the theory for doing that will be covered.

2.5.1. Section properties

The calculations of the second moment of area and radius of gyration for CLT panels as composite materials are covered in section 2.4.4. Steiner’s theorem applied to CLT.

Figure 2.23 shows cross-section dimensions of a CLT panel when treated as a homogeneous material; it also shows directions of different axes used.

**Area:**
\[ A = y \cdot b \]

**Second moment of area:**
\[ I_y = \frac{b \cdot y^3}{12} \]
\[ I_z = \frac{y \cdot b^3}{12} \]

**Section modulus:**
\[ W_y = \frac{b \cdot y^2}{6} \]
\[ W_z = \frac{y \cdot b^2}{6} \]

**Radius of gyration:**
\[ i_y = \frac{y}{\sqrt{12}} \]
\[ i_z = \frac{b}{\sqrt{12}} \]
2.5.2. Buckling

When designing against compression it is important to understand the potential to failure resulting from buckling. A structural member will always buckle in its least stiff direction.

Euler’s buckling factors:

![Figure 2.24 Euler's loads](source: www.jfccivilengineering.com)

*Figure 2.24 shows theoretical modes of failure through buckling depending on the joints and connections on the ends of the column.*

**Effective length of compression member:**

\[
 l_{eff} = \beta \cdot h
\]

Where,

- \( h \) Height [m] of shear wall
- \( \beta \) Effective length factor shown in *figure 2.24*

**Slenderness ratios:**

\[
 \lambda_y = \frac{l_{eff}}{i_y}
\]

\[
 \lambda_x = \frac{l_{eff}}{i_z}
\]

\[
 \lambda_{y,rel} = \frac{\lambda_y}{\pi} \sqrt{\frac{F_{c0,k}}{k_{0.05}}} \quad \text{(EC5-1-1, 6.3.2, equ. 6.25)}
\]
\[ \lambda_{x,rel} = \frac{\lambda_x}{\pi} \cdot \sqrt{\frac{f_{c0,k}}{E_{0,05}}} \]  

(EC5-1-1, 6.3.2, equ. 6.26)

Where,

- \( f_{c0,k} \): Characteristic compression strength
- \( E_{0,05} \): Elastic modulus fifth percentile value
- \( l_{eff} \): Effective length [m] of compression member
- \( i \): Effective radius of gyration

Reduction factor, \( k_c \), due to risk of buckling:

- \( k_y = 0.5(1 + \beta_c(\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) \)  
  (EC5-1-1, 6.3.2, equ. 6.27)
- \( k_x = 0.5(1 + \beta_c(\lambda_{rel,x} - 0.3) + \lambda_{rel,x}^2) \)  
  (EC5-1-1, 6.3.2, equ. 6.28)
- \( \beta_c = 0.1 \)  
  (EC5-1-1, 6.3.2, equ 6.29)
- \( k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 + \lambda_{rel,y}^2}} \)  
  Reduction factor. (EC5-1-1, 6.3.2, equ. 6.25)
- \( k_{c,x} = \frac{1}{k_x + \sqrt{k_x^2 + \lambda_{rel,x}^2}} \)  
  Reduction factor. (EC5-1-1, 6.3.2, equ. 6.25)

2.5.3. Combined axial & bending checks to EC5, 6.3.2

According to EC5, stabilising members 'column' must be checked against combined compression and bending forces. Ordinary non-stabilising column subject to compression and bending undergo a different check (EC5, 6.3.1). The section shows the combined axial & bending checks (EC5, 6.3.2) and breaks it down to its components.

Bending Stress:

\[ \sigma_{m,d} = \frac{M_{Ed}}{W} \]

Where,

- \( M_{Ed} \): Design moment
- \( W \): Section modulus

Compression stress:

\[ \sigma_{c,0,d} = \frac{N_{Ed}}{A_{eff}} \]

Where,

- \( N_{Ed} \): Design normal load
- \( A_{eff} \): Effective area.
Combination ratio (EC5 Eq.6.23 & 6.24)

\[
\frac{\sigma_{c,0,d}}{k_{c,y} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \cdot \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

\[
\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

Where,

- \(\sigma_{c,0,d}\) Design compression stress
- \(f_{c,0,d}\) Characteristic design compression strength
- \(\sigma_{m,d}\) Design bending stress
- \(f_{m,d}\) Characteristic design bending strength
- \(k_m = 0.7\) Reduction factor for CLT rectangular cross-section (EC5-1-1, 6.1.6 (2))

### 2.5.4. Shear check to EC5, 6.1.7

Shear checks are a requirement of stability design. This section shows shear checks according to EC5.

**Shear stress:**

\[\tau_d = \frac{1.5 \cdot V_{ld}}{A}\]

Where,

- \(V_{ld}\) Design lateral loading
- \(A\) CLT panel’s cross-sectional area

**Shear stress check:**

\[\tau_d \leq f_{v,d}\]

Where,

- \(\tau_d\) Design shear stress
- \(f_{v,d}\) Characteristic shear strength
3. Method

All calculations were performed by hand using MathCAD application. During this project one stabilising wall would be focused on, running the whole height of the building. MathCAD operates in such a way that variables have to be defined before they are used; this dictated the order of calculation.

At the early stages of a project Martinsons use several different Excel Spreadsheet templates to perform load analysis and stabilisation checks so it was able to compare results at most stages of calculation.

This method chapter will describe how calculations were performed and will reference the hand calculations (found in the end of this report) and to the theory behind the calculations in the theory chapter.

3.1. Choosing the stabilising wall

Factors contributing to selecting a stabilising shear wall are mentioned in section 2.3.1. How to choose stabilising elements.

3.1.1. Inspection of architect’s drawings

From the architect’s initial drawing all loading-bearing elements (walls and columns) were indentified and sub-categorised depending on their load-bearing necessities and exposure to wind actions.

Each wall used then was inspected closer against the architect’s designs to determine the size and position of the stabilising panel.

*Figure 3.1* shows how stabilising walls are distributed across the plan. The top drawing showing the stabilising walls positions and lengths.

*Figure 3.1 Stabilising walls on top of architect’s initial design.*
3.1.2. Choosing wall to design and its dimension

It was decided to focus on the outer wall on the left side of the building (YV1- yellow on figure 3.1).

*Figure 3.2* shows how the length of effective wind load was calculated to be 7.5m by also incorporating the balcony length. The wind load is equally distributed between the exterior wall 1 and the nearest stabilising interior wall.

Once the effective length of wind actions was established it was possible to choose a suitable size for the shear wall panel as shown below, where the length of the shear wall is 6m (*Figure 3.3*). The effective length is 5.5m.

*Figure 3.3* shows the architect’s elevation drawing of exterior wall 1 and the section of the stabilising panel. The bottom floor has no openings and is inside the existing building so it has no exposure to cross plane wind actions. Some of the exterior wall continues into the existing building on the right which gives the total length of the wall 12.5m.

The floor heights were determined to match the same levels of the adjacent building.
3.2. Load analysis of Exterior wall 1 (YV1)

Figure 3.3 shows the heights of each level and reference names for reaction point used for the load analysis.

3.2.1. Snow loads.

Snow loads were calculated according to section 2.1.3. Snow loads.

3.2.2. Wind actions

Wind actions were calculated at each point, where floors and walls connected, starting from the top at point A.

**Pressure loads:**
The wind pressure loads were calculated according to section 2.1.4. Wind Action.

First, the maximum velocity pressure was calculated. Where the terrain category is III and average basic velocity for Umeå is 22m/s.

From the maximum velocity pressure, the wind surface pressure was calculated. The form factor, \( \mu_{\text{wind}} = 0.8 + |−0.5| = 1.2 \), was taken by combining positive and negative \( c_{pe} \) values.

**Acting loads:**
The point loads were calculated according to section 2.2.1.1. Calculating point loads from surface pressure.

From the point loads the aggregate horizontal load distributed along the total length of wall, according to section 2.2.1.4. Calculating horizontal reaction.

**Moments:**
The acting moment were calculated according to section 2.2.1.2. Calculating in plane moments.

3.2.3. Calculation of Ultimate Limit States (ULS) to EC0

**Loads:**
The effective breadth, \( b = 2m \), was taken as half the floor span from the architect’s drawing.

The imposed load values were taken from (EC1-1-1, 6.3.1.2) and the top level being treated as a balcony imposed load. The imposed were also reduced by the reduction factor for walls and columns (EC1-1-1, equ. 6.2).

Self-weights are assumed on the safer side and taken as a standard from Martinsons previous projects. The roof has a 1m high railing that was also calculated.

Self-weight and imposed loads were added together before calculating the different load combinations.
STR load combination:
For the STR load combination self-weights included the floor of the each level, with the exception of the bottom floor which is part of the concrete foundations.

Designs loads were calculated for each of snow, imposed and wind loads as leading variables the corresponding ‘psi’ factor for each case.

EQU load combination:
For the EQU load combination self-weights of the floor were excluded at each level as the design load would be used for stability checks and should not have additional weight to offer resistance.

3.3. Stability checks

All previously calculated wind loads and moment should be multiplied by the coefficient for variable actions: \( \gamma_Q = 1.5 \), this value is the same for both STR and EQU load combinations.

All stability checks used ULS EQU load combinations.

3.3.1. Resistance to overturning check

The resistance to overturning check was performed according to section 2.3.3.2. Shear wall resistance to overturning. Reactions loads were taken at point a \( (R_a, \text{ figure 3.4}) \) at each level to see if each panel had sufficient resistance to overturning. Negative values indicate a need for some anchorage. EQU design loads were used.

To calculate the reaction force at \( R_a \), the design moments had to be calculated and multiplied by the coefficient for variable actions: \( \gamma_Q = 1.5 \).

Then the reactions were calculated according to section 2.2.4.1 Vertical reactions.

3.3.2. Sliding resistance

Sliding resistance was calculated according to section 2.3.3.3. Sliding resistance.

Ratio checks were performed with the coefficient for variable actions: \( \gamma_Q = 1.5 \) used for the horizontal wind load. For the storeys that did not have enough sliding resistance, the resistance force was calculated.
3.4. Calculation and determination of design loads

All design loads were calculated for both medium-term and short-term loading durations and STR ULS. For medium-term loading, imposed loads as leading variable were used as they were greater than snow loads. Wind loads as leading variable are considered as short-term loading.

3.4.1. Design bending moments

In plane:
The design bending moments were calculated according to section 2.2.1.2. Calculating in plane moments and multiplied by the coefficient for variable actions, $\gamma_0$ for both medium-term and short-term loading; and medium-term also being reduced by the $\psi_0$ factor for wind.

Cross plane:
Wind surface pressure acting in the cross plane direction was calculated according to section 2.2.1.3. Calculating cross plane moments and multiplied by the coefficient for variable actions, $\gamma_0$ for both medium-term and short-term loading; and medium-term also being reduced by the $\psi_0$ factor for wind.

The assumption was made that wind blowing into the walls would have the same surface pressure as the side winds that were previously calculated.

The bottom floor wall has no exposure to wind thus had no surface pressure.

3.4.2. Vertical design compression load

Reactions at point b were taken ($R_b$, figure 3.4) as the compression load would be greatest at this point. Calculations were performed according to section 2.2.4.1. Vertical reactions.

3.5. Design of Exterior wall (YV1) to EC5

The most common method of designing a shear wall by hand calculations is to split it into sections, usually 1m to make calculations simpler, and treat a section as a column. As the greatest reaction force is at point b (figure 3.4), that particular 1m section will be designed as a ‘column’ to withstand maximum compression and combined bending.

In plane (Y-Y axis, figure 2.23) bending stress is completely disregarded as there is no chance of buckling in that direction because shear wall is extremely stiff along its span.

Floors 1, 2 & 3 were designed as they were exposed to greatest compression stresses. Figures 3.5, 3.6 & 3.7 show the dimensions of those walls. The heights vary from the heights used in the load analysis as depth of floor diaphragms was considered. The bottom floor has no wind exposure so the Z-Z axis (figure 2.23) bending moment was disregarded.
When designing CLT, it is possible to treat the panels as either homogeneous or composites, each having different applicable characteristic stiffness and strength properties. In this project both methods of calculation were explored.

3.5.1 Homogeneous properties

Section properties:
When treating the CLT panels as homogeneous, the section properties were calculated according to 2.5.4. Section properties for the specific dimensions of CLT panel being checked.

Characteristic strength values:
The characteristic strength values used for designing the exterior wall were taken from table 2.5 for the dimensions of CLT panel being checked. These values are specifically for 1.2m long section but it can be assumed for 1m sections.

3.5.2 Non-homogeneous properties

Section properties:
When treating the CLT panels as non-homogeneous, effective section properties must be determined by assuming that only the longitudinal laminates will withstand compression and bending. The radius of gyration, \( i_z \), can be determined from the second moment of inertia, \( I_z \), according to section 2.4.4. Steiner’s rule applied to CLT.

The composite theory was used to obtain the effective area, which was needed to determine compression stress. The \( k_3 \) factor was used according to 2.4.3. Composite theory.

Characteristic strength values:
The characteristic strength values used for non-homogeneous CLT were the material characteristic strengths table 2.4.

3.5.3 Combined axial & bending checks

The design checks were performed in accordance with sections 2.5.6. Combined axial & bending checks to EC5, 6.3.2 and 2.5.5. Bucking.

Buckling:
Slenderness ratios and k-factors were determined for both cases, homogeneous and non-homogeneous, using the relevant section and characteristic strength properties.

Duration loads:
Tests were performed for both medium-term; using imposed loads as the design loads, and short-term; using wind loads as the design load. Each having a different modification factor, \( k_{mod} \).
**Design moments:**
In plane moments were dismissed, whilst cross plane moments were calculated according to section 2.2.1.3. *Calculating cross plane moments*. Cross plane moments were only present on levels 2 & 3.

**Design compression stress:**
The design loads from reactions taken at point b (*figure 3.4*) were divided by effective area determined by the section properties.

**Design strengths:**
Design strengths were determined according to 2.4.1. *Design value of CLT (EC5-1-1, 2.4.1)*, using the corresponding characteristic strengths for homogeneous and non-homogeneous assumptions.

### 3.5.4. Shear check

Shear checks were performed for both medium-term and short-term loadings according to section 2.5.7. *Shear check to EC5, 6.1.7*.

**Design load:**
The horizontal loads were calculated according to 2.2.1.4 *Calculating horizontal reactions*. 
4. Results

The complete results can be viewed in the MathCAD calculations in the appendix. In this chapter the results are summarised.

4.1. Stability Design of Exterior wall

Resistance to overturning and uplift reactions:
- **Roof**: sufficient resistance to overturning
- **Level 6**: sufficient resistance to overturning
- **Level 5**: sufficient resistance to overturning
- **Level 4**: sufficient resistance to overturning
- **Level 3**: sufficient resistance to overturning
- **Level 2**: sufficient resistance to overturning, however, resistance is minimal and potential anchorage should be taken into consideration.
- **Level 1**: insufficient resistance to overturning
  Uplift reaction force: **-12.5kN**

Resistance to sliding:
- **Roof**: inadequate resistance to sliding
  Resistance required: **1.6 kN/m**
- **Level 6**: inadequate resistance to sliding
  Resistance required: **0.9 kN/m**
- **Level 5**: borderline resistance to sliding, some resistance required
- **Level 4**: adequate resistance to sliding, some resistance required
- **Level 3**: adequate resistance to sliding, some resistance required
- **Level 3**: adequate resistance to sliding, some resistance required

4.2. Design of Exterior wall to EC5

4.2.1. Level 1

Checks for CLT panel b=145mm.

4.2.1.1. Treating CLT as homogeneous

**Combined axial & bending checks:**

Medium-term: ULS STR with imposed loads as leading variable

\[
\frac{\sigma_{c,0,d}}{k_{c,x} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.88 \quad OK!
\]
Short-term: ULS STR with wind loads as leading variable
\[
\frac{\sigma_{c.o.d}}{k_{c.z} \cdot f_{c.o.d}} + k_m \frac{\sigma_{m.y,d}}{f_{m,y,d}} + \frac{\sigma_{m.z,d}}{f_{m,z,d}} = 0.94 \quad OK!
\]

Shear checks:
\[
\frac{\tau_d}{f_{v,d}} = 0.52 \quad OK!
\]

4.2.1.2. Treating CLT as non-homogeneous

Combined axial & bending checks:

Medium-term: ULS STR with imposed loads as leading variable
\[
\frac{\sigma_{c.o.d}}{k_{c.z} \cdot f_{c.o.d}} + k_m \frac{\sigma_{m.y,d}}{f_{m,y,d}} + \frac{\sigma_{m.z,d}}{f_{m,z,d}} = 0.68 \quad OK!
\]

Short-term: ULS STR with wind loads as leading variable
\[
\frac{\sigma_{c.o.d}}{k_{c.z} \cdot f_{c.o.d}} + k_m \frac{\sigma_{m.y,d}}{f_{m,y,d}} + \frac{\sigma_{m.z,d}}{f_{m,z,d}} = 0.72 \quad OK!
\]

Shear checks:
\[
\frac{\tau_d}{f_{v,d}} = 0.49 \quad OK!
\]

4.2.2. Level 2

Checks for CLT panel b=120mm.

4.2.2.1. Treating CLT as homogeneous

Combined axial & bending checks:

Medium-term: ULS STR with imposed loads as leading variable
\[
\frac{\sigma_{c.o.d}}{k_{c.z} \cdot f_{c.o.d}} + k_m \frac{\sigma_{m.y,d}}{f_{m,y,d}} + \frac{\sigma_{m.z,d}}{f_{m,z,d}} = 0.62 \quad OK!
\]

Short-term: ULS STR with wind loads as leading variable
\[
\frac{\sigma_{c.o.d}}{k_{c.z} \cdot f_{c.o.d}} + k_m \frac{\sigma_{m.y,d}}{f_{m,y,d}} + \frac{\sigma_{m.z,d}}{f_{m,z,d}} = 0.67 \quad OK!
\]
4.2.2.2. Treating CLT as non-homogeneous

**Combined axial & bending checks:**

**Medium-term:** ULS STR with imposed loads as leading variable

$$\frac{\sigma_{c,0,d}}{k_{cz} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.50 \text{ OK!}$$

**Short-term:** ULS STR with wind loads as leading variable

$$\frac{\sigma_{c,0,d}}{k_{cz} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.53 \text{ OK!}$$

**Shear checks:**

$$\frac{\tau_d}{f_{v,d}} = 0.46 \text{ OK!}$$

4.2.3. Level 3

Checks for CLT panel b=120mm.

4.2.3.1. Treating CLT as homogeneous

**Combined axial & bending checks:**

**Medium-term:** ULS STR with imposed loads as leading variable

$$\frac{\sigma_{c,0,d}}{k_{cz} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.78 \text{ OK!}$$

**Short-term:** ULS STR with wind loads as leading variable

$$\frac{\sigma_{c,0,d}}{k_{cz} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.84 \text{ OK!}$$
Shear checks:

\[ \frac{\tau_d}{f_{v,d}} = 0.49 \text{ OK!} \]

4.2.3.2. Treating CLT as non-homogeneous

**Combined axial & bending checks:**

**Medium-term:** ULS STR with imposed loads as leading variable

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.60 \text{ OK!} \]

**Short-term:** ULS STR with wind loads as leading variable

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.64 \text{ OK!} \]

Shear checks:

\[ \frac{\tau_d}{f_{v,d}} = 0.40 \text{ OK!} \]
5. Discussion

When comparing the design result from the calculations in this report to the actual design dimensions used in the Embla 5 project, they pass the same systems checks.

For the load analysis and initial designs of CLT panels Martinsons use an Excel spreadsheet template. Performing calculations by hand (i.e. using MathCAD) is definitely more time consuming and requires more visual space to represent the same data. Where it is possible to use two different spreadsheets, the calculations in this project amounted to 35 pages. However, MathCAD has been a useful tool to clearly validate and verify the calculations in this project; as it is easy to miss-enter data into spreadsheets and possible to skip certain steps.

Many discussion points have arisen during the course of this project. The main points are summarised in this chapter.

5.1. Discussion topics from calculation phase

5.1.1. Load analysis and determination of design loads

From a structural mechanics perspective, the method used to perform the load analysis was fairly definitive; there were not many factors that could have altered the results. The end results (i.e. design loads obtained) from the calculations vary only slightly from that of Martinsons and would not influence the final design of the exterior walls.

Variations can be explained by Martinsons application of Eurocodes according to EKS 9.

5.1.2. Stability design and stability checks

Like the load analysis, there is a definitive method to performing stability checks. The checks performed on this project did not give any surprise results.

It was more unclear to how the lengths and positioning of stabilising elements affected the design of stabilising walls.

Theoretically, by decreasing the length of a stabilising wall, the moment exerted on the panel will also decrease. This is an advantage when designing against compression loads as the reaction forces are smaller. At the same time, it can be a disadvantage as towards stability as overturning resistance is decreased. There is definitely potential to optimise lengths of stabilising elements.

5.1.3. Design of shear wall

Treating CLT panels as columns:
The method adopted, by many structural engineers, to design CLT shear walls is to treat them as columns and perform checks for combined axial and compression. This brings up many discussion points concerning design loads, moment and buckling.

The section lengths in which the CLT panels are checked against are usually 1m (or designed per meter), but if this arbitrary was made to be much shorter or longer, then the capacities of the column would significantly vary.
CLT as homogeneous or composite material:
Treating CLT as a homogeneous material is a contentious subject. It certainly simplifies calculations but does take into account internal stresses such as rolling shear for example.

The material characteristic values for specific sizes of panels were calculated by SINTEF, but were only for 1.2m panels that are produced in the Martinsons CLT factory. When checking CLT walls as columns, having a section length not 1.2m meant that the actual characteristic values would vary. This was not taken into account during the calculation stage.

Just from the results, it is noticeable that calculating CLT as different types of material gives differing capacities. Treating CLT as homogeneous shows that it has a reduced capacity to withstand combined axial & bending loads. It can be argued that this should be the preferred method to perform checks as it is in a more critical state, however, adequate theory should be used to back this claim.

Buckling:
Calculating buckling in CLT was very difficult, as long spanning panels do not behave in the same way as columns. In plane moments were completely disregarded from the calculations as it was assumed that the risk of buckling in that direction was insignificant.

Calculation of slenderness ratios and buckling factors varied significantly, for CLT as homogenous and composite materials, as section properties differ greatly. Steiner’s theorem was used to calculate the section properties of CLT, but working with the assumption that only the longitudinal laminates would take compression. This may not be the case in reality.

5.2. Discussion topics based on theory and experience of working on the Embla 5 project

During the course of this project work, much knowledge and experience was acquired that had not been directly used to the calculation section. Here are some additional discussion points that are worth mentioning for the Embla 5 project.

5.2.1. Measures taken against uplift

When individual stabilising panels do not satisfy the overturning resistance check, there is an uplift force created. There are different methods used to counter uplift on the Embla 5 project.

It is possible to reduce to uplift forces by transferring the loads to adjacent non-stabilising wall panels or columns. For the case of the exterior wall 1 (the wall that was designed in this project), the entire uplift force was reduced by a 0.7m extension to the nearest column.

For walls that still had uplift forces remaining, even after some of the uplift was reduced, an anchorage system was set in place. Figures 5.1 & 5.2 show steel anchors that were used on the Embla 5 project; the anchors consist of a steel rod and two steel plates held together by a pair of nuts, a groove is cut out from the CLT panel to accommodate the anchors.
5.2.2. Measures taken against sliding

Usually when stabilising panels do not satisfy sliding checks, as in the case of the top two floors of Exterior wall 1; extra connections should be provided to counter sliding. However, on the Embla 5 project the risk of sliding was dismissed because the horizontal sliding forces were deemed to be insignificant. Considering the entire lengths of walls and existing connections with the addition of imposed loading, were enough to withstand sliding.

5.2.3. Connections

The theory and design shown in this project is sufficient enough to be able to establish initial design dimensions for load-bearing elements; however, it is vital to do additional checks around connections, joints and openings.

The connections between the manufactured CLT panels were made by gluing and nailing plywood sheets, in the factory. The plywood sheets would also act as additional reinforcements to counter high stress values. Stress analyses against lateral loading around openings were needed to be able to design the plywood connections.

The stress analyses were made using RFEM software, which using the finite element method (FEM) to calculate stresses on composite materials. Figures 5.3 shows the analysis made for level 3 of External wall 1. The red and yellow sections show extremely high stresses. From these results it was possible to match suitable plywood reinforcements by comparing resulting stress values from the analysis to a pre-calculated design strength spreadsheet for plywood.
6. Conclusion

Ultimately the purpose of this project was to show, by worked example, the theory and practice behind the design of CLT stabilising system in multi-storey structures. In that respect the project and report is successful.

The theory shown in this report along with the corresponding calculation example (and also corresponding Eurocodes) would show the reader accurately:

- How to analyse wind actions subjected upon building surfaces to Eurocode specifications;
- Calculation of ultimate limit states design loads and use them accordingly, both for vertical loading and horizontal loading, taking into account different leading variables and load durations;
- How to perform stability checks, for individual stabilising elements, against resistance to overturning and resistance to sliding; and how to calculate required resistance required to counter uplift or sliding;
- A brief introduction to using composite theory and its applications;
- Understanding CLT as a structural material and being able to consider it as a homogeneous material or a composite (non-homogeneous) material focusing on:
  - The differences between characteristic material strengths
  - The different section properties which influence slenderness ratios and buckling coefficients.
- The theory and method behind being able to design CLT wall panels according to Eurocode 5.
7. Future work

During the course of this project, there were many aspects that could have been studied much deeper. Stabilisation in engineering is a very broad subject and can be studied further. CLT as a material opens many avenues for further research both from a material science perspective and from a structural mechanics view.

Here are some points and topic that arose during this project that could be studied further:

- Wind actions on buildings – a better understanding of how wind forces behave on buildings of different varying form.

- Choosing stabilising element – how lengths of stabilising members effect overall stability and also if having multiple stabilising walls allows the engineer to optimise design.

- Openings of stabilising elements – Martinsons have a template (figure 2.14) showing the prerequisite for choosing stabilising CLT walls with openings. It would be interesting to be able to optimise the template for varying height. In addition to that, connections reinforcements could play a role in the calculations.

- Buckling – when designing CLT wall panels as columns the same buckling rules do not necessarily apply. Yet, there is no distinct simple method to calculate buckling of CLT panel.

- CLT material properties – a greater study of treating CLT panels as either homogeneous or composite materials. This would incorporate aspects of composite theory.
References

Bibliography


Eurocodes


Analysis and Design of Exterior Wall 1 (YV 1) - Calculations

Calculations – Table of contents

Building elevation dimensions ................................................................. 1
Snow loads ............................................................................................... 1
Wind actions .......................................................................................... 1
Calculation of ultimate limit states ......................................................... 6
Stability check – resistance to overturning ............................................. 10
Stability check – resistance to sliding .................................................... 11
Calculation of design loads ................................................................. 13
Composite theory .................................................................................. 17
Design of wall – Level 1 ................................................................. 18
Design of wall – Level 2 ................................................................. 24
Design of wall – Level 3 ................................................................. 30
Analysis and Design of exterior wall (YV1)

Wall dimensions

\[ L := 12.5 \, m \] Total length of wall
\[ l := 5.5 \, m \] Effective length of stabilising panel
\[ l_w := 7.5 \, m \] Length of wind distributed load

Snow load

\[ \mu_i := 0.8 \] Snow load coefficient
\[ C_e := 1.2 \] Exposure coefficient
\[ C_t := 1.0 \] Thermal coefficient
\[ s_k := 3 \, kN \cdot m^{-2} \] Characteristic snow load
\[ s_d := \mu_i \cdot C_e \cdot C_t \cdot s_k \] Snow load
\[ s_d := 2.88 \, kN \cdot m^{-2} \]

Wind actions

\[ \mu_{\text{wind}} := 1.15 \] Pressure coefficient
\[ h := 21.8 \, m \] Height of building
\[ z_0 := 0.3 \, m \] Roughness length
\[ z_{0,II} := 0.05 \, m \] Roughness length, Cat.II
\[ k_r := 0.19 \cdot \left( \frac{z_0}{z_{0,II}} \right)^{0.07} = 0.2154 \] Terrain factor
\[ \rho := 1.25 \, kg \cdot m^{-3} \] Density of air
\[ v_b := 22 \, m \cdot s^{-1} \] Basic wind velocity
\[ q_b := 0.5 \cdot \rho \cdot v_b^2 = 0.3 \, kN \cdot m^{-2} \] Basic wind pressure

Wind actions @ A

\[ z_A := 21.8 \, m \] Height from ground at which load is acting
\[ z_{\text{act},A} := 0.5 \, m \] Vertical length of acting load
\[ q_{p,A} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot \left[ k_r \cdot \ln \left( \frac{z_A}{z_0} \right) \right]^2 \cdot q_b = 0.62 \, kN \cdot m^{-2} \] Max. velocity pressure
\[ w_{k,A} := q_{p,A} \cdot \mu_{\text{wind}} = 0.71 \, kN \cdot m^{-2} \] Wind surface pressure load
\[ F_{\text{wind},A} := w_{k,A} \cdot z_{\text{act},A} \cdot l_w = 2.7 \, kN \] Point load @ A
\[ H_A := \frac{F_{\text{wind},A}}{L} = 0.21 \, kN \cdot m^{-1} \] Agreggate horizontal load distributed along the total length of wall
Wind actions @ B

\[ z_B = 20.8 \, \text{m} \]  
Height from ground at which load is acting

\[ z_{act.B} = 2.2 \, \text{m} \]  
Vertical length of acting load

\[
q_{p,B} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot \left[ k_r \cdot \ln \left( \frac{z_B}{z_0} \right) \right]^2 \cdot q_b = 0.61 \, kN \cdot m^{-2}
\]  
Max. velocity pressure

\[
w_{k,B} := q_{p,B} \cdot \mu_{wind} = 0.7 \, kN \cdot m^{-2}
\]  
Wind surface pressure load

\[
F_{wind.B} := w_{k,B} \cdot z_{act.B} \cdot l_w = 11.5 \, kN
\]  
Point load @ A

\[
H_B := \frac{F_{wind.B}}{L} + H_A = 1.13 \, kN \cdot m^{-1}
\]  
Aggregate horizontal load distributed along the total length of wall

Wind moment:

\[
M_{wind,B} := F_{wind.A} \cdot 1 \, m
\]

\[ M_{wind,B} = 2.67 \, kN \cdot m \]

Wind actions @ C

\[ z_C = 17.4 \, \text{m} \]  
Height from ground at which load is acting

\[ z_{act.C} = 3.4 \, \text{m} \]  
Vertical length of acting load

\[
q_{p,C} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot \left[ k_r \cdot \ln \left( \frac{z_C}{z_0} \right) \right]^2 \cdot q_b = 0.56 \, kN \cdot m^{-2}
\]  
Max. velocity pressure

\[
w_{k,C} := q_{p,C} \cdot \mu_{wind} = 0.64 \, kN \cdot m^{-2}
\]  
Wind surface pressure load

\[
F_{wind.C} := w_{k,C} \cdot z_{act.C} \cdot l_w = 16.3 \, kN
\]  
Point load @ A

\[
H_C := \frac{F_{wind.C}}{L} + H_B = 2.43 \, kN \cdot m^{-1}
\]  
Aggregate horizontal load distributed along the total length of wall

Wind moment

\[
M_{wind,C} := F_{wind.A} \cdot 4.4 \, m + F_{wind.B} \cdot 3.4 \, m
\]

\[ M_{wind,C} = 50.78 \, kN \cdot m \]
Wind actions @ D

- \( z_D := 14.0 \, \text{m} \)  
- \( z_{\text{act},D} := 3.4 \, \text{m} \)

\[ q_{p,D} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot k_r \cdot \ln \left( \frac{z_D}{z_0} \right)^2 \cdot q_b = 0.5 \, \text{KN} \cdot \text{m}^{-2} \]  

Max. velocity pressure

\[ w_{k,D} := q_{p,D} \cdot \mu_{\text{wind}} = 0.57 \, \text{KN} \cdot \text{m}^{-2} \]  

Wind surface pressure load

\[ F_{\text{wind},D} := w_{k,D} \cdot z_{\text{act},D} \cdot l_w = 14.6 \, \text{kN} \]  

Point load @ A

\[ H_D := \frac{F_{\text{wind},D}}{L} + H_C = 3.6 \, \text{KN} \cdot \text{m}^{-1} \]  

Agregate horizontal load distributed along the total length of wall

Wind moment

\[ M_{\text{wind},D} := F_{\text{wind},A} \cdot 7.8 \, \text{m} + F_{\text{wind},B} \cdot 6.8 \, \text{m} + F_{\text{wind},C} \cdot 3.4 \, \text{m} \]

\[ M_{\text{wind},D} = 154 \, \text{KN} \cdot \text{m} \]

Wind actions @ E

- \( z_E := 10.6 \, \text{m} \)  
- \( z_{\text{act},E} := 3.5 \, \text{m} \)

\[ q_{p,E} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot k_r \cdot \ln \left( \frac{z_E}{z_0} \right)^2 \cdot q_b = 0.43 \, \text{KN} \cdot \text{m}^{-2} \]  

Max. velocity pressure

\[ w_{k,E} := q_{p,E} \cdot \mu_{\text{wind}} = 0.49 \, \text{KN} \cdot \text{m}^{-2} \]  

Wind surface pressure load

\[ F_{\text{wind},E} := w_{k,E} \cdot z_{\text{act},E} \cdot l_w = 12.9 \, \text{kN} \]  

Point load @ A

\[ H_E := \frac{F_{\text{wind},E}}{L} + H_C = 4.64 \, \text{KN} \cdot \text{m}^{-1} \]  

Agregate horizontal load distributed along the total length of wall

Wind moment

\[ M_{\text{wind},E} := F_{\text{wind},A} \cdot 11.2 \, \text{m} + F_{\text{wind},B} \cdot 10.2 \, \text{m} + F_{\text{wind},C} \cdot 6.8 \, \text{m} + F_{\text{wind},D} \cdot 3.4 \, \text{m} \]

\[ M_{\text{wind},E} = 307 \, \text{KN} \cdot \text{m} \]
Wind actions @ F

*Actual z = 7m but 8m is used*

\[ z_F := 8 \text{ m} \quad \text{Height from ground at which load is acting} \]
\[ z_{act,F} := 3.25 \text{ m} \quad \text{Vertical length of acting load} \]

\[ q_{p,F} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \left[ k_r \cdot \ln \left( \frac{z_F}{z_0} \right) \right]^2 \cdot q_b = 0.36 \text{ kN} \cdot \text{m}^{-2} \quad \text{Max. velocity pressure} \]

\[ w_{k,F} := q_{p,F} \cdot \mu_{wind} = 0.42 \text{ kN} \cdot \text{m}^{-2} \quad \text{Wind surface pressure load} \]
\[ F_{wind,F} := w_{k,F} \cdot z_{act,F} \cdot l_w = 10.2 \text{ kN} \quad \text{Point load @ A} \]
\[ H_F := \frac{F_{wind,F}}{L} + H_E = 5.45 \text{ kN} \cdot \text{m}^{-1} \quad \text{Agregate horizontal load distributed along the total length of wall} \]

Wind moment
\[ M_{wind,F} := F_{wind,A} \cdot 14.8 \text{ m} + F_{wind,B} \cdot 13.8 \text{ m} + F_{wind,C} \cdot 10.4 \text{ m} + F_{wind,D} \cdot 7 \text{ m} + F_{wind,E} \cdot 3.6 \text{ m} \]
\[ M_{wind,F} = 516 \text{ kN} \cdot \text{m} \]

Wind actions @ G

*Actual z = 7m but 8m is used*

\[ z_G := 8 \text{ m} \quad \text{Height from ground at which load is acting} \]
\[ z_{act,G} := 3.5 \text{ m} \quad \text{Vertical length of acting load} \]

\[ q_{p,G} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \left[ k_r \cdot \ln \left( \frac{z_G}{z_0} \right) \right]^2 \cdot q_b = 0.36 \text{ kN} \cdot \text{m}^{-2} \quad \text{Max. velocity pressure} \]

\[ w_{k,G} := q_{p,G} \cdot \mu_{wind} = 0.42 \text{ kN} \cdot \text{m}^{-2} \quad \text{Wind surface pressure load} \]
\[ F_{wind,G} := w_{k,G} \cdot z_{act,G} \cdot l_w = 11 \text{ kN} \quad \text{Point load @ A} \]
\[ H_G := \frac{F_{wind,G}}{L} + H_F = 6.33 \text{ kN} \cdot \text{m}^{-1} \quad \text{Agregate horizontal load distributed along the total length of wall} \]

Wind moment
\[ M_{wind,G} := F_{wind,A} \cdot 17.7 \text{ m} + F_{wind,B} \cdot 16.7 \text{ m} + F_{wind,C} \cdot 13.3 \text{ m} + F_{wind,D} \cdot 9.9 \text{ m} + F_{wind,E} \cdot 6.5 \text{ m} + F_{wind,F} \cdot 2.9 \text{ m} \]
\[ M_{wind,G} = 713 \text{ kN} \cdot \text{m} \]
Wind actions @ H

Actual $z = 7m$ but $8m$ is used

$z_H := 8 \text{ m}$  \hspace{1cm} Height from ground at which load is acting

$z_{act,H} := 2.05 \text{ m}$  \hspace{1cm} Vertical length of acting load

$$q_{p,H} := \left[ 1 + \frac{6}{\ln \left( \frac{h}{z_0} \right)} \right] \cdot \left[ k_v \cdot \ln \left( \frac{z_H}{z_0} \right) \right]^2 \cdot q_b = 0.36 \text{ kN} \cdot \text{m}^{-2}$$  \hspace{1cm} Max. velocity pressure

$$w_{k,H} := q_{p,H} \cdot \mu_{wind} = 0.42 \text{ kN} \cdot \text{m}^{-2}$$  \hspace{1cm} Wind surface pressure load (distributed)

$$F_{wind,H} := w_{k,H} \cdot z_{act,H} \cdot l_w = 6.4 \text{ kN}$$  \hspace{1cm} Point load @ A

$$H_H := \frac{F_{wind,H}}{L} + H_G = 6.84 \text{ kN} \cdot \text{m}^{-1}$$  \hspace{1cm} Aggregate horizontal load distributed along the total length of wall

Wind moment

$$M_{wind,H} := F_{wind,A} \cdot 21.8 \text{ m} + F_{wind,B} \cdot 20.8 \text{ m} + F_{wind,C} \cdot 17.4 \text{ m} + F_{wind,D} \cdot 14 \text{ m} + F_{wind,E} \cdot 10.6 \text{ m} + F_{wind,F} \cdot 7 \text{ m} + F_{wind,G} \cdot 4.1 \text{ m}$$

$$M_{wind,H} = 1038 \text{ kN} \cdot \text{m}$$
Calculation of ultimate limit states (ULS), EC0

\[ \gamma_d = \text{Factor of safety} \]
\[ n = 6 \quad \text{Number of storeys} \]
\[ \psi_{0,\text{snow}} = 0.8 \quad \text{’Psi’ factor for snow loads} \]
\[ \psi_{0,\text{wind}} = 0.3 \quad \text{’Psi’ factor for wind loads} \]
\[ \psi_{0,\text{live}} = 0.7 \quad \text{’Psi’ factor for imposed loads} \]
\[ \alpha_n = \frac{2 + (n-2) \cdot \psi_{0,\text{live}}}{n} = 0.8 \quad \text{Reduction factor for imposed loads} \]

\textbf{Loads}

\[ b_{\text{area}} = 2 \text{ m} \quad \text{Breath of distributed horizontal load area} \]
\[ Q_{\text{snow}} = s_d \cdot b_{\text{area}} = 5.76 \text{ kN} \cdot \text{m}^{-1} \quad \text{Snow load (distributed)} \]
\[ Q_{\text{wind}} = 0 \cdot \text{ kN} \cdot \text{m}^{-2} \cdot b_{\text{area}} \quad \text{Vertical downward wind load} \]
\[ G_{\text{wall}} = 0.9 \text{ kN} \cdot \text{m}^{-2} \quad \text{Self-weight of CLT wall} \]
\[ G_{\text{floor}} = 1.5 \text{ kN} \cdot \text{m}^{-2} \cdot b_{\text{area}} \quad \text{Self-weight of CLT floor} \]
\[ G_{\text{railing}} = 0.3 \text{ kN} \cdot \text{m}^{-2} \cdot 1 \text{ m} \quad \text{Self-weight of roof railing} \]
\[ Q_{\text{balcony}} = \alpha_n \cdot 3.5 \text{ kN} \cdot \text{m}^{-2} \cdot b_{\text{area}} \quad \text{Imposed load of balcony} \]
\[ Q_{\text{office}} = \alpha_n \cdot 2.5 \text{ kN} \cdot \text{m}^{-2} \cdot b_{\text{area}} \quad \text{Imposed load of office space} \]

\textbf{Calculation of live and deads loads at each storey}

\textbf{Roof}

\[ G_{k,\text{roof}} := G_{\text{railing}} + G_{\text{floor}} = 3.3 \text{ kN} \cdot \text{m}^{-1} \quad \text{Self-weight} \]
\[ Q_{k,\text{roof}} := Q_{\text{balcony}} = 5.6 \text{ kN} \cdot \text{m}^{-1} \quad \text{Imposed load} \]

\textbf{Level 6}

\[ G_{k,6} := G_{\text{railing}} + 2 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 3.4 \text{ m} = 9.4 \text{ kN} \cdot \text{m}^{-1} \quad \text{Self-weight} \]
\[ Q_{k,6} := Q_{\text{balcony}} + Q_{\text{office}} = 9.6 \text{ kN} \cdot \text{m}^{-1} \quad \text{Imposed load} \]

\textbf{Level 5}

\[ G_{k,5} := G_{\text{railing}} + 3 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 6.8 \text{ m} = 15.4 \text{ kN} \cdot \text{m}^{-1} \quad \text{Self-weight} \]
\[ Q_{k,5} := Q_{\text{balcony}} + 2 \cdot Q_{\text{office}} = 13.6 \text{ kN} \cdot \text{m}^{-1} \quad \text{Imposed load} \]

\textbf{Level 4}

\[ G_{k,4} := G_{\text{railing}} + 4 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 10.2 \text{ m} = 21.5 \text{ kN} \cdot \text{m}^{-1} \quad \text{Self-weight} \]
\[ Q_{k,4} := Q_{\text{balcony}} + 3 \cdot Q_{\text{office}} = 17.6 \text{ kN} \cdot \text{m}^{-1} \quad \text{Imposed load} \]

\textbf{Level 3}

\[ G_{k,3} := G_{\text{railing}} + 5 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 13.8 \text{ m} = 27.7 \text{ kN} \cdot \text{m}^{-1} \quad \text{Self-weight} \]
\[ Q_{k,3} := Q_{\text{balcony}} + 4 \cdot Q_{\text{office}} = 21.6 \text{ kN} \cdot \text{m}^{-1} \quad \text{Imposed load} \]
Level 2

\[ G_{k.2} := G_{\text{railing}} + 6 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 16.7 \ m = 33.3 \ kN \cdot m^{-1} \]  
Self-weight

\[ Q_{k.2} := Q_{\text{balcony}} + 5 \cdot Q_{\text{office}} = 25.6 \ kN \cdot m^{-1} \]  
Imposed load

Level 1

\[ G_{k.1} := G_{\text{railing}} + 6 \cdot G_{\text{floor}} + G_{\text{wall}} \cdot 20.8 \ m = 37 \ kN \cdot m^{-1} \]  
Self-weight

\[ Q_{k.1} := Q_{k.2} = 25.6 \ kN \cdot m^{-1} \]  
Imposed load

**ULS (STR) with Snow load as leading variable**

**Roof**

\[ Q_{d.\text{roof.snow}} := \gamma_d \cdot 1.2 \cdot G_{k.\text{roof}} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.\text{roof}} = 18.5 \ kN \cdot m^{-1} \]

Level 6

\[ Q_{d.6.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.6} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.6} = 30 \ kN \cdot m^{-1} \]

Level 5

\[ Q_{d.5.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.5} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.5} = 41.4 \ kN \cdot m^{-1} \]

Level 4

\[ Q_{d.4.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.4} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.4} = 52.9 \ kN \cdot m^{-1} \]

Level 3

\[ Q_{d.3.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.3} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.3} = 64.6 \ kN \cdot m^{-1} \]

Level 2

\[ Q_{d.2.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.2} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.2} = 75.5 \ kN \cdot m^{-1} \]

Level 1

\[ Q_{d.1.\text{snow}} := \gamma_d \cdot 1.2 \cdot G_{k.1} + \gamma_d \cdot 1.5 \cdot Q_{\text{snow}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.1} = 79.9 \ kN \cdot m^{-1} \]

**ULS (STR) with Imposed load as leading variable**

**Roof**

\[ Q_{d.\text{roof.imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.\text{roof}} + \gamma_d \cdot 1.5 \cdot Q_{\text{roof}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 19.3 \ kN \cdot m^{-1} \]

Level 6

\[ Q_{d.6.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.6} + \gamma_d \cdot 1.5 \cdot Q_{k.6} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 32.5 \ kN \cdot m^{-1} \]

Level 5

\[ Q_{d.5.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.5} + \gamma_d \cdot 1.5 \cdot Q_{k.5} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 45.8 \ kN \cdot m^{-1} \]
Level 4
\[ Q_{d.4.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.4} + \gamma_d \cdot 1.5 \cdot Q_{k.4} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 59.1 \, kN \cdot m^{-1} \]

Level 3
\[ Q_{d.3.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.3} + \gamma_d \cdot 1.5 \cdot Q_{k.3} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 72.6 \, kN \cdot m^{-1} \]

Level 2
\[ Q_{d.2.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.2} + \gamma_d \cdot 1.5 \cdot Q_{k.2} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 85.3 \, kN \cdot m^{-1} \]

Level 1
\[ Q_{d.1.\text{imposed}} := \gamma_d \cdot 1.2 \cdot G_{k.1} + \gamma_d \cdot 1.5 \cdot Q_{k.1} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 89.7 \, kN \cdot m^{-1} \]

**ULS (STR) with Wind actions as leading variable**

**Roof**

\[ Q_{d.\text{roof.wind}} := \gamma_d \cdot 1.2 \cdot G_{k.\text{roof}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.\text{roof}} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 16.8 \, kN \cdot m^{-1} \]

Level 6
\[ Q_{d.6.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.6} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.6} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 28.2 \, kN \cdot m^{-1} \]

Level 5
\[ Q_{d.5.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.5} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.5} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 39.7 \, kN \cdot m^{-1} \]

Level 4
\[ Q_{d.4.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.4} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.4} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 51.2 \, kN \cdot m^{-1} \]

Level 3
\[ Q_{d.3.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.3} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.3} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 62.9 \, kN \cdot m^{-1} \]

Level 2
\[ Q_{d.2.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.2} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.2} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 73.8 \, kN \cdot m^{-1} \]

Level 1
\[ Q_{d.1.\text{wind}} := \gamma_d \cdot 1.2 \cdot G_{k.1} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{live}} \cdot Q_{k.1} + \gamma_d \cdot 1.5 \cdot \psi_{0,\text{snow}} \cdot Q_{\text{snow}} = 78.2 \, kN \cdot m^{-1} \]

**Conclusion:** for medium term loads ULS with imposed loads as leading variable should be used as is greater than snow loads as leading variable.
ULS (EQU) with **Wind actions** as only variable

*All loads are to be taken over the floor for stability checks*

**Roof**

\[ Q_{EQU, \text{roof}} := \gamma_d \cdot 1.1 \cdot (G_{k, \text{roof}} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 0.33 \text{ kN} \cdot \text{m}^{-1} \]

**Level 6**

\[ Q_{EQU,6} := \gamma_d \cdot 1.1 \cdot (G_{k,6} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 7 \text{ kN} \cdot \text{m}^{-1} \]

**Level 5**

\[ Q_{EQU,5} := \gamma_d \cdot 1.1 \cdot (G_{k,5} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 13.7 \text{ kN} \cdot \text{m}^{-1} \]

**Level 4**

\[ Q_{EQU,4} := \gamma_d \cdot 1.1 \cdot (G_{k,4} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 20.3 \text{ kN} \cdot \text{m}^{-1} \]

**Level 3**

\[ Q_{EQU,3} := \gamma_d \cdot 1.1 \cdot (G_{k,3} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 27.2 \text{ kN} \cdot \text{m}^{-1} \]

**Level 2**

\[ Q_{EQU,2} := \gamma_d \cdot 1.1 \cdot (G_{k,2} - G_{floor}) + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 33.4 \text{ kN} \cdot \text{m}^{-1} \]

**Level 1**

\[ Q_{EQU,1} := \gamma_d \cdot 1.1 \cdot G_{k,1} + \gamma_d \cdot 1.5 \cdot Q_{\text{wind}} = 40.7 \text{ kN} \cdot \text{m}^{-1} \]
Stability Checks

Resistance to overturning

*Check for resistance to overturning by taking reactions at A. negative values give show uplift.*

\[ \gamma_Q = 1.5 \quad \text{Variable action coefficient for USL (EQU) & (STR)} \]

**Roof**

\[ M_{\text{roof}} := \gamma_Q \cdot M_{\text{wind,B}} \cdot \frac{l}{L} = 1.8 \, \text{kN} \cdot \text{m} \]

\[ R_{a,\text{roof}} := \frac{Q_{\text{EQU,roof}} \cdot l}{2} - \frac{M_{\text{roof}}}{l} = 0.6 \, \text{kN} \quad \text{Reaction @ a} \]

**Level 6**

\[ M_6 := \gamma_Q \cdot M_{\text{wind,C}} \cdot \frac{l}{L} = 33.5 \, \text{kN} \cdot \text{m} \]

\[ R_{a,6} := \frac{Q_{\text{EQU,6}} \cdot l}{2} - \frac{M_6}{l} = 13.1 \, \text{kN} \quad \text{Reaction @ a} \]

**Level 5**

\[ M_5 := \gamma_Q \cdot M_{\text{wind,D}} \cdot \frac{l}{L} = 101.8 \, \text{kN} \cdot \text{m} \]

\[ R_{a,5} := \frac{Q_{\text{EQU,5}} \cdot l}{2} - \frac{M_5}{l} = 19.1 \, \text{kN} \quad \text{Reaction @ a} \]

**Level 4**

\[ M_4 := \gamma_Q \cdot M_{\text{wind,E}} \cdot \frac{l}{L} = 202.8 \, \text{kN} \cdot \text{m} \]

\[ R_{a,4} := \frac{Q_{\text{EQU,4}} \cdot l}{2} - \frac{M_4}{l} = 19 \, \text{kN} \quad \text{Reaction @ a} \]

**Level 3**

\[ M_3 := \gamma_Q \cdot M_{\text{wind,F}} \cdot \frac{l}{L} = 340.5 \, \text{kN} \cdot \text{m} \]

\[ R_{a,3} := \frac{Q_{\text{EQU,3}} \cdot l}{2} - \frac{M_3}{l} = 12.9 \, \text{kN} \quad \text{Reaction @ a} \]
Level 2

\[ M_2 := \gamma_Q \cdot M_{\text{wind.G}} \cdot \frac{l}{L} = \frac{470.9}{l} \text{ kN} \cdot \text{m} \]
Design moment

\[ R_{a.2} := \frac{Q_{\text{EQU.2}} \cdot l}{2} - \frac{M_2}{l} = 6.1 \text{ kN} \]
Reaction @ a

Level 1

\[ M_1 := \gamma_Q \cdot M_{\text{wind.H}} \cdot \frac{l}{L} = \frac{684.9}{l} \text{ kN} \cdot \text{m} \]
Design moment

\[ R_{a.1} := \frac{Q_{\text{EQU.1}} \cdot l}{2} - \frac{M_1}{l} = -12.5 \text{ kN} \]
Reaction @ a

**Conclusion:** There is definite uplift on level 1 and would, therefore, have to be anchored. Level 2 and the roof have a very small resistance to overturning so should be closer examined and potentially anchored. The rest of the storeys show a sufficient resistance to overturning.

**Resistance to sliding**

The ratio between the horizontal should be lower than the coefficient of friction (0.4 for wood against wood).

Level 6

\[ \frac{\gamma_Q \cdot H_B}{Q_{\text{EQU.roof}}} = 515\% \quad \text{Not OK!} \quad \text{Friction ratio} \]

\[ F_{s0.6} := (\gamma_Q \cdot H_B - 0.4 \cdot Q_{\text{EQU.roof}}) = 1.6 \text{ kN} \cdot \text{m}^{-1} \]
Slide force

Level 5

\[ \frac{\gamma_Q \cdot H_C}{Q_{\text{EQU.6}}} = 52\% \quad \text{Not OK!} \quad \text{Friction ratio} \]

\[ F_{s0.5} := (\gamma_Q \cdot H_C - 0.4 \cdot Q_{\text{EQU.6}}) = 0.9 \text{ kN} \cdot \text{m}^{-1} \]
Slide force

Level 4

\[ \frac{\gamma_Q \cdot H_D}{Q_{\text{EQU.5}}} = 40\% \quad \text{Not OK!} \quad \text{Friction ratio} \]

\[ F_{s0.4} := (\gamma_Q \cdot H_D - 0.4 \cdot Q_{\text{EQU.5}}) = -0.06 \text{ kN} \cdot \text{m}^{-1} \]
Slide force
Conclusion: Levels 5 & 6 show a lot of sliding tendencies; fastenings should be provided to combat sliding. Level 4 is on the border of sliding; it should be closer examined and some precautionary measures against sliding should be taken.
Calculation and determination of design Loads

Bending Moments

Medium term - Imposed loads as leading variable

**Roof**

\[ M_{y,MT.roof} := \gamma_Q \cdot \psi_{0.wind} \cdot M_{\text{wind},B} \cdot \frac{l}{L} = 0.53 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.roof} := \gamma_Q \cdot \psi_{0.wind} \cdot 0.5 \cdot (w_{k,A} + w_{k,B}) \]

Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.roof} = 0.32 \text{ kN} \cdot \text{m}^{-2} \]

**Level 6**

\[ M_{y,MT.6} := \gamma_Q \cdot \psi_{0.wind} \cdot M_{\text{wind},C} \cdot \frac{l}{L} = 10.1 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.6} := \gamma_Q \cdot \psi_{0.wind} \cdot 0.5 \cdot (w_{k,B} + w_{k,C}) \]

Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.6} = 0.3 \text{ kN} \cdot \text{m}^{-2} \]

**Level 5**

\[ M_{y,MT.5} := \gamma_Q \cdot \psi_{0.wind} \cdot M_{\text{wind},D} \cdot \frac{l}{L} = 30.5 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.5} := \gamma_Q \cdot \psi_{0.wind} \cdot 0.5 \cdot (w_{k,C} + w_{k,D}) \]

Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.5} = 0.27 \text{ kN} \cdot \text{m}^{-2} \]

**Level 4**

\[ M_{y,MT.4} := \gamma_Q \cdot \psi_{0.wind} \cdot M_{\text{wind},E} \cdot \frac{l}{L} = 60.9 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.4} := \gamma_Q \cdot \psi_{0.wind} \cdot 0.5 \cdot (w_{k,D} + w_{k,E}) \]

Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.4} = 0.24 \text{ kN} \cdot \text{m}^{-2} \]
Level 3

\[ M_{y,MT.3} = \gamma_Q \psi_{0,wind} \cdot M_{\text{wind,F}} \cdot \frac{l}{L} = 102.2 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.3} = \gamma_Q \psi_{0,wind} \cdot 0.5 \cdot (w_{k,E} + w_{k,F}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.3} = 0.2 \text{ kN} \cdot \text{m}^{-2} \]

Level 2

\[ M_{y,MT.2} = \gamma_Q \psi_{0,wind} \cdot M_{\text{wind,G}} \cdot \frac{l}{L} = 141.3 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,MT.2} = \gamma_Q \psi_{0,wind} \cdot 0.5 \cdot (w_{k,F} + w_{k,G}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,MT.2} = 0.19 \text{ kN} \cdot \text{m}^{-2} \]

Level 1

\[ M_{y,MT.1} = \gamma_Q \psi_{0,wind} \cdot M_{\text{wind,H}} \cdot \frac{l}{L} = 205.5 \text{ kN} \cdot \text{m} \]

Design moment y-axis

The wall on Level 1 is not exposed to wind actions on the z-axis

Short term - Wind loads as leading variable

Roof

\[ M_{y,ST.roof} = \gamma_Q \cdot M_{\text{wind,B}} \cdot \frac{l}{L} = 1.76 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,ST.roof} = \gamma_Q \cdot 0.5 \cdot (w_{k,A} + w_{k,B}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,ST.roof} = 1.06 \text{ kN} \cdot \text{m}^{-2} \]

Level 6

\[ M_{y,ST.6} = \gamma_Q \cdot M_{\text{wind,C}} \cdot \frac{l}{L} = 33.5 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,ST.6} = \gamma_Q \cdot 0.5 \cdot (w_{k,B} + w_{k,C}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,ST.6} = 1 \text{ kN} \cdot \text{m}^{-2} \]

Level 5

\[ M_{y,ST.5} = \gamma_Q \cdot M_{\text{wind,D}} \cdot \frac{l}{L} = 101.8 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,ST.5} = \gamma_Q \cdot 0.5 \cdot (w_{k,C} + w_{k,D}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,ST.5} = 0.91 \text{ kN} \cdot \text{m}^{-2} \]

Level 4

\[ M_{y,ST.4} = \gamma_Q \cdot M_{\text{wind,E}} \cdot \frac{l}{L} = 202.8 \text{ kN} \cdot \text{m} \]

Design moment y-axis

\[ w_{z,ST.4} = \gamma_Q \cdot 0.5 \cdot (w_{k,D} + w_{k,E}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z,ST.4} = 0.8 \text{ kN} \cdot \text{m}^{-2} \]
Level 3
\[ M_{y.ST.3} := \gamma_Q \cdot M_{\text{wind.F}} \cdot \frac{L}{L} = 340.5 \text{ kN} \cdot \text{m} \]
Design moment y-axis

\[ w_{z.ST.3} := \gamma_Q \cdot 0.5 \cdot (w_{k.E} + w_{k.F}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z.ST.3} = 0.68 \text{ kN} \cdot \text{m}^{-2} \]

Level 2
\[ M_{y.ST.2} := \gamma_Q \cdot M_{\text{wind.G}} \cdot \frac{L}{L} = 470.9 \text{ kN} \cdot \text{m} \]
Design moment y-axis

\[ w_{z.ST.2} := \gamma_Q \cdot 0.5 \cdot (w_{k.F} + w_{k.G}) \]
Design surface pressure in the z-axis contributing to bending

\[ w_{z.ST.2} = 0.63 \text{ kN} \cdot \text{m}^{-2} \]

Level 1
\[ M_{y.ST.1} := \gamma_Q \cdot M_{\text{wind.H}} \cdot \frac{L}{L} = 684.9 \text{ kN} \cdot \text{m} \]
Design moment y-axis

The wall on Level 1 is not exposed to wind actions on the z-axis

Compression loads

Medium term - Imposed loads as leading variable
Reactions at point b given the maximum compression load

Roof
\[ R_{b.MT.roof} := \frac{Q_{d.roof.imposed} \cdot l}{2} + \frac{M_{y.MT.roof}}{l} = 53.1 \text{ kN} \]
Design compression load

Level 6
\[ R_{b.MT.6} := \frac{Q_{d.6.imposed} \cdot l}{2} + \frac{M_{y.MT.6}}{l} = 91.3 \text{ kN} \]
Design compression load

Level 5
\[ R_{b.MT.5} := \frac{Q_{d.5.imposed} \cdot l}{2} + \frac{M_{y.MT.5}}{l} = 131.5 \text{ kN} \]
Design compression load

Level 4
\[ R_{b.MT.4} := \frac{Q_{d.4.imposed} \cdot l}{2} + \frac{M_{y.MT.4}}{l} = 173.6 \text{ kN} \]
Design compression load

Level 3
\[ R_{b.MT.3} := \frac{Q_{d.3.imposed} \cdot l}{2} + \frac{M_{y.MT.3}}{l} = 218.2 \text{ kN} \]
Design compression load

Level 2
\[ R_{b.MT.2} := \frac{Q_{d.2.imposed} \cdot l}{2} + \frac{M_{y.MT.2}}{l} = 260.3 \text{ kN} \]
Design compression load

Level 1
\[ R_{b.MT.1} := \frac{Q_{d.1.imposed} \cdot l}{2} + \frac{M_{y.MT.1}}{l} = 284.1 \text{ kN} \]
Design compression load
Short term - Wind loads as leading variable

Reactions at point b given the maximum compression load

**Roof**

\[ R_{b,ST.\text{roof}} := \frac{Q_{d,\text{roof\_wind}} \cdot l}{2} + \frac{M_{y,ST.\text{roof}}}{l} = 46.4 \text{ kN} \]

Design compression load

**Level 6**

\[ R_{b,ST.6} := \frac{Q_{d,6\_wind} \cdot l}{2} + \frac{M_{y,ST.6}}{l} = 83.7 \text{ kN} \]

Design compression load

**Level 5**

\[ R_{b,ST.5} := \frac{Q_{d,5\_wind} \cdot l}{2} + \frac{M_{y,ST.5}}{l} = 127.7 \text{ kN} \]

Design compression load

**Level 4**

\[ R_{b,ST.4} := \frac{Q_{d,4\_wind} \cdot l}{2} + \frac{M_{y,ST.4}}{l} = 177.6 \text{ kN} \]

Design compression load

**Level 3**

\[ R_{b,ST.3} := \frac{Q_{d,3\_wind} \cdot l}{2} + \frac{M_{y,ST.3}}{l} = 234.8 \text{ kN} \]

Design compression load

**Level 2**

\[ R_{b,ST.2} := \frac{Q_{d,2\_wind} \cdot l}{2} + \frac{M_{y,ST.2}}{l} = 288.5 \text{ kN} \]

Design compression load

**Level 1**

\[ R_{b,ST.1} := \frac{Q_{d,1\_wind} \cdot l}{2} + \frac{M_{y,ST.1}}{l} = 339.6 \text{ kN} \]

Design compression load
Composite theory design to k3

CLT panel - 120 mm

\[ b_{120} := 120 \text{ mm} \]

\[ k_{3.120} := 1 - \left( 1 - \frac{1}{30} \right) \cdot \frac{63}{120} = 0.49 \]

\[ b_{\text{eff.120}} := k_{3.120} \cdot b_{120} = 59 \text{ mm} \]

Effective thickness of CLT panel

CLT panel - 145 mm

\[ b_{145} := 145 \text{ mm} \]

\[ k_{3.145} := 1 - \left( 1 - \frac{1}{30} \right) \cdot \frac{88}{145} = 0.41 \]

\[ b_{\text{eff.145}} := k_{3.145} \cdot b_{145} = 60 \text{ mm} \]

Effective thickness of CLT panel

CLT panel - 170 mm

\[ b_{170} := 170 \text{ mm} \]

\[ k_{3.170} := 1 - \left( 1 - \frac{1}{30} \right) \cdot \frac{94.5}{170} = 0.46 \]

\[ b_{\text{eff.170}} := k_{3.170} \cdot b_{170} = 79 \text{ mm} \]

Effective thickness of CLT panel

Characteristic strength values

CLT panel - Material characteristic strength

Bending \[ f_{m,k} := 24 \cdot \frac{N}{\text{mm}^2} \]

Compression \[ f_{c,0,k} := 21 \cdot \frac{N}{\text{mm}^2} \]

Shear \[ f_{v,k} := 4 \cdot \frac{N}{\text{mm}^2} \]

Elastic modulus \[ E_{0.05} := 7400 \cdot \frac{N}{\text{mm}^2} \]

CLT panel - 120 mm

Bending \[ f_{120.m.y,k} := 11.1 \cdot \frac{N}{\text{mm}^2} \]

\[ f_{120.m.z,k} := 3.8 \cdot \frac{N}{\text{mm}^2} \]

Shear \[ f_{120.v.k} := 1.58 \cdot \frac{N}{\text{mm}^2} \]

Elastic modulus \[ E_{120.0} := 4000 \cdot \frac{N}{\text{mm}^2} \]

CLT panel - 145 mm

Bending \[ f_{145.m.y,k} := 10.1 \cdot \frac{N}{\text{mm}^2} \]

\[ f_{145.m.z,k} := 4.8 \cdot \frac{N}{\text{mm}^2} \]

Shear \[ f_{145.v.k} := 1.57 \cdot \frac{N}{\text{mm}^2} \]

Elastic modulus \[ E_{145.0} := 3510 \cdot \frac{N}{\text{mm}^2} \]

CLT panel - 170 mm

Bending \[ f_{170.m.y,k} := 10.1 \cdot \frac{N}{\text{mm}^2} \]

\[ f_{170.m.z,k} := 4.8 \cdot \frac{N}{\text{mm}^2} \]

Shear \[ f_{170.v.k} := 1.57 \cdot \frac{N}{\text{mm}^2} \]

Elastic modulus \[ E_{170.0} := 3510 \cdot \frac{N}{\text{mm}^2} \]

Created with PTC Mathcad Express. See www.mathcad.com for more information.
**Design of YV1 - level 1**

**Wall properties**

- Height of wall $h := 4 \text{ m}$
- Length of wall panel $y_{total} := 6 \text{ m}$
- Effective length of wall $l = 5.5 \text{ m}$
- Total length of wall $L = 12.5 \text{ m}$
- Section length $y := 1 \text{ m}$

---

**Treating CLT as a homogenous material**

Let $b = 145\text{ mm}$

**Section properties**

- Effective cross-sectional area of section $A_{eff} := b \cdot y = 0.145 \text{ m}^2$
- Effective radius of gyration $i_z := \frac{b}{\sqrt{12}} = 42 \text{ mm}$
- Effective section modulus $W_z := \frac{b^2 \cdot y}{6} = \left(3.5 \cdot 10^6\right) \text{ mm}^3$

---

**Combined Axial & Bending checks to EC5-1-1, 6.3.2**

**Slenderness ratios**

- Effect length of 'column' $l_{eff} := h = 4 \text{ m}$
- Slenderness ratio $\lambda_z := \frac{l_{eff}}{i_z} = 95.56$
- Slenderness ratio $\lambda_{rel.z} := \frac{\lambda_z}{\pi} \sqrt{\frac{f_{145,c.0,k}}{E_{145.0}}} = 1.48$

**k - coefficients**

- Factor for straightness limits $\beta_c := 0.1$
- Re-distribution of stresses factor $k_m := 0.7$

\[
k_z := 0.5 \left(1 + \beta_c \cdot (\lambda_{rel.z} - 0.3) + \lambda_{rel.z}^2\right)\]

\[
k_{c.z} := \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel.z}^2}} = 0.418\]
Medium term - imposed load as leading variable

$k_{mod} := 0.8$  
$\gamma_m := 1.25$

**Design compression strength:**

$$f_{c,0,d} := \frac{k_{mod} \cdot f_{145,c.0,k}}{\gamma_m} = 5.31 \ N \cdot mm^{-2}$$

**Applied compression stress:**

$$N_{Ed} := R_{b,MT,1} = 284.13 \ kN$$

$$\sigma_{c,0,d} := \frac{N_{Ed}}{A_{eff}} = 1.96 \ N \cdot mm^{-2}$$

**Design bending strength:**

$$f_{m.y,d} := \frac{k_{mod} \cdot f_{145,m.y,k}}{\gamma_m} = 6.46 \ N \cdot mm^{-2}$$

$$f_{m.z,d} := \frac{k_{mod} \cdot f_{145,m.z,k}}{\gamma_m} = 3.07 \ N \cdot mm^{-2}$$

**Applied bending stress:**

$$\sigma_{m.y,d} := 0 \ N \cdot mm^{-2}$$

$$M_{Ed,z} := 0 \ kN \cdot m$$  
No acting wind pressure in z-axis

$$\sigma_{m.z,d} := \frac{M_{Ed,z}}{W_z} = 0 \ N \cdot mm^{-2}$$

**Combination ratio:** buckling along Z-Z axis

$$\frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m.y,d}}{f_{m.y,d}} + \frac{\sigma_{m.z,d}}{f_{m.z,d}} = 0.88$$  
Ok!

Short term - wind load as leading variable

$k_{mod} := 0.9$  
$\gamma_m := 1.25$

**Design compression strength:**

$$f_{c,0,d} := \frac{k_{mod} \cdot f_{145,c.0,k}}{\gamma_m} = 5.98 \ N \cdot mm^{-2}$$
Applied compression stress:

\[ N_{Ed} := R_{b,ST.1} = 339.62 \, kN \]

\[ \sigma_{c,0,d} := \frac{N_{Ed}}{A_{eff}} = 2.34 \, N \cdot mm^{-2} \]

Design bending strength:

\[ f_{m,y,d} := k_{mod} \cdot f_{145,m,y,k} = 7.27 \, N \cdot mm^{-2} \]

\[ f_{m,z,d} := k_{mod} \cdot f_{145,m,z,k} = 3.46 \, N \cdot mm^{-2} \]

Applied bending stress:

\[ \sigma_{m,y,d} := 0 \cdot N \cdot mm^{-2} \]

\[ M_{Ed,z} = 0 \, kN \cdot m \quad \text{No acting wind pressure in } Z-Z \text{ axis} \]

\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 0 \, N \cdot mm^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.94 \quad \text{Ok!} \]

**Shear checks to EC5-1-1, 6.1.7**

Applied shear stress

\[ F := \gamma_Q \cdot H_T \cdot l = 56.4 \, kN \]

\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 0.58 \, N \cdot mm^{-2} \]

Design shear strength

\[ f_{v,d} := k_{mod} \cdot f_{145,v,k} = 1.13 \, N \cdot mm^{-2} \]

\[ \frac{\tau_d}{f_{v,d}} = 0.52 \quad \text{Ok!} \]
Treating the CLT as a non-homogenous material

Let \( b=145 \text{ mm} \)

Section properties

\[
\begin{align*}
b_{\text{long}} & := 19 \text{ mm} \\
b_{\text{trans}} & := 44 \text{ mm}
\end{align*}
\]

\[A_{\text{eff}} := b_{\text{eff.145}} \cdot y = 0.06 \text{ m}^2\] Effective cross-sectional area of section acting as a column

\[
I_z := 3 \cdot \left( \frac{y \cdot b_{\text{long}}^3}{12} \right) + 2 \cdot y \cdot b_{\text{long}} \cdot \left( b_{\text{long}} + b_{\text{trans}} \right)^2
\]

\[I_z = (1.53 \cdot 10^8) \text{ mm}^4\] Second moment of inertia

\[
i_z := \sqrt{\frac{I_z}{y \cdot 3 \cdot b_{\text{long}}}} = 52 \text{ mm}\] Effective radius of gyration

\[
W_z := \frac{b_{\text{eff.145}}^2 \cdot y}{6} = (5.987 \cdot 10^5) \text{ mm}^3\] Effective section modulus

Combined Axial & Bending checks to EC5-1-1, 6.3.2

Slenderness ratios

\[l_{\text{eff}} := h = 4 \text{ m}\] Effect length of 'column'

\[\lambda_z := \frac{l_{\text{eff}}}{i_z} = 77.32\]

\[
\lambda_{\text{rel.z}} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = 1.31
\]

\(k\) - coefficients

\[\beta_c := 0.1\] Factor for straightness limits

\[k_m := 0.7\] Re-distribution of stresses factor

\[
k_z := 0.5 \left( 1 + \beta_c \cdot (\lambda_{\text{rel.z}} - 0.3) + \lambda_{\text{rel.z}}^2 \right)
\]

\[
k_{c,z} := \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel.z}}^2}} = 0.518
\]
Medium term - imposed load as leading variable

\[ k_{\text{mod}} := 0.8 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

**Design compression strength:**

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{c,0,k}}{\gamma_m} = 13.44 \text{ N} \cdot \text{mm}^{-2} \]

**Applied compression stress:**

\[ N_{\text{Ed}} := R_{b,MT.1} = 284.13 \text{ kN} \]
\[ \sigma_{c,0,d} := \frac{N_{\text{Ed}}}{A_{\text{eff}}} = 4.74 \text{ N} \cdot \text{mm}^{-2} \]

**Design bending strength:**

\[ f_{m,y,d} := \frac{k_{\text{mod}} \cdot f_{m,k}}{\gamma_m} = 15.36 \text{ N} \cdot \text{mm}^{-2} \]
\[ f_{m,z,d} := \frac{k_{\text{mod}} \cdot f_{m,k}}{\gamma_m} = 15.36 \text{ N} \cdot \text{mm}^{-2} \]

**Applied bending stress:**

\[ \sigma_{m,y,d} := 0 \cdot \text{N} \cdot \text{mm}^{-2} \]
\[ M_{\text{Ed,z}} := 0 \text{ kN} \cdot \text{m} \quad \text{No acting wind pressure in Z-Z axis} \]
\[ \sigma_{m,z,d} := \frac{M_{\text{Ed,z}}}{W_z} = 0 \text{ N} \cdot \text{mm}^{-2} \]

**Combination ratio: buckling along Z-Z axis**

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.68 \quad \text{Ok!} \]

Short term - wind load as leading variable

\[ k_{\text{mod}} := 0.9 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

**Design compression strength:**

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{c,0,k}}{\gamma_m} = 15.12 \text{ N} \cdot \text{mm}^{-2} \]
Applied compression stress:

\[ N_{Ed} := R_{b,ST.1} = 339.62 \, kN \]

\[ \sigma_{c,0.d} := \frac{N_{Ed}}{A_{eff}} = 5.67 \, N \cdot mm^{-2} \]

Design bending strength:

\[ f_{m,y,d} := \frac{k_{mod} \cdot f_{m,k}}{\gamma_m} = 17.28 \, N \cdot mm^{-2} \]

\[ f_{m,z,d} := \frac{k_{mod} \cdot f_{m,k}}{\gamma_m} = 17.28 \, N \cdot mm^{-2} \]

Applied bending stress:

\[ \sigma_{m,y,d} := 0 \cdot N \cdot mm^{-2} \]

\[ M_{Ed,z} := 0 \, kN \cdot m \]

No acting wind pressure in Z-Z axis

\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 0 \, N \cdot mm^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0.d}}{k_{c,z} \cdot f_{c,0.d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.72 \]  
OK!

Shear checks to EC5-1-1, 6.1.7

Applied shear stress

\[ F := \gamma_Q \cdot H_l \cdot l = 56.4 \, kN \]

\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 1.41 \, N \cdot mm^{-2} \]

Design shear strength

\[ f_{v,d} := \frac{k_{mod} \cdot f_{v,k}}{\gamma_m} = 2.88 \, N \cdot mm^{-2} \]

\[ \frac{\tau_d}{f_{v,d}} = 0.49 \]  
OK!
Design of YV1 - level 2

Wall properties

Height of wall \( h := 2.69 \ m \)
Length of wall panel \( y_{total} := 6 \ m \)
Effective length of wall \( l = 5.5 \ m \)
Total length of wall \( L = 12.5 \ m \)
Section length \( y := 1 \ m \)

Treating CLT as a homogenous material

Let \( b = 120\ mm \)

**Section properties**

\[
A_{\text{eff}} := b \cdot y = 0.12 \ m^2
\]

Effective cross-sectional area of section

\[
i_z := \frac{b}{\sqrt{12}} = 35 \ mm
\]

Effective radius of gyration

\[
W_z := \frac{b^2 \cdot y}{6} = (2.4 \cdot 10^6) \ mm^3
\]

Effective section modulus

**Combined Axial & Bending checks to EC5-1-1, 6.3.2**

Slenderness ratios

\[
l_{\text{eff}} := h = 2.69 \ m
\]

Effect length of 'column'

\[
\lambda_z := \frac{l_{\text{eff}}}{i_z} = 77.65
\]

\[
\lambda_{rel,z} := \frac{\lambda_z}{\pi} \sqrt{\frac{f_{120,c,0,k}}{E_{120,0}}} = 1.24
\]

**k - coefficients**

\[
\beta_c := 0.1 \quad \text{Factor for straightness limits}
\]

\[
k_m := 0.7 \quad \text{Re-distribution of stresses factor}
\]

\[
k_z := 0.5 \left( 1 + \beta_c \cdot (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2 \right)
\]

\[
k_{c,z} := \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} = 0.573
\]
Medium term - imposed load as leading variable

\[ k_{\text{mod}} := 0.8 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

Design compression strength:

\[ f_{c,0.d} := \frac{k_{\text{mod}} \cdot f_{120.c.0.k}}{\gamma_m} = 6.4 \, \text{N} \cdot \text{mm}^{-2} \]  

Applied compression stress:

\[ N_{Ed} := R_{b,MT.2} = 260.28 \, \text{kN} \]
\[ \sigma_{c,0.d} := \frac{N_{Ed}}{A_{\text{eff}}} = 2.17 \, \text{N} \cdot \text{mm}^{-2} \]

Design bending strength:

\[ f_{m.y.d} := \frac{k_{\text{mod}} \cdot f_{120.m.y.k}}{\gamma_m} = 7.1 \, \text{N} \cdot \text{mm}^{-2} \]
\[ f_{m.z.d} := \frac{k_{\text{mod}} \cdot f_{120.m.z.k}}{\gamma_m} = 2.43 \, \text{N} \cdot \text{mm}^{-2} \]

Applied bending stress:

\[ \sigma_{m.y.d} := 0 \cdot \text{N} \cdot \text{mm}^{-2} \]
\[ M_{Ed.z} := \frac{w_{z,MT.2} \cdot y \cdot h^2}{8} = 0.17 \, \text{kN} \cdot \text{m} \]
\[ \sigma_{m.z.d} := \frac{M_{Ed.z}}{W_z} = 0.07 \, \text{N} \cdot \text{mm}^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \frac{\sigma_{m.y,d}}{f_{m.y,d}} + \frac{\sigma_{m.z,d}}{f_{m.z,d}} = 0.62 \quad \text{Ok!} \]

Short term - wind load as leading variable

\[ k_{\text{mod}} := 0.9 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

Design compression strength:

\[ f_{c,0.d} := \frac{k_{\text{mod}} \cdot f_{120.c.0.k}}{\gamma_m} = 7.2 \, \text{N} \cdot \text{mm}^{-2} \]
Applied compression stress:

\[ N_{Ed} := R_{b,ST.2} = 288.53 \text{ kN} \]
\[ \sigma_{c.0,d} := \frac{N_{Ed}}{A_{eff}} = 2.4 \text{ N/mm}^{-2} \]

Design bending strength:

\[ f_{m.y.d} := \frac{k_{mod} \cdot f_{120.m.y.k}}{\gamma_m} = 7.99 \text{ N/mm}^{-2} \]
\[ f_{m.z.d} := \frac{k_{mod} \cdot f_{120.m.z.k}}{\gamma_m} = 2.74 \text{ N/mm}^{-2} \]

Applied bending stress:

\[ \sigma_{m.y.d} := 0 \text{ N/mm}^{-2} \]
\[ M_{Ed.z} := \frac{w_{z,ST.2} \cdot y \cdot h^2}{8} = 0.57 \text{ kN.m} \]
\[ \sigma_{m.z.d} := \frac{M_{Ed.z}}{W_z} = 0.24 \text{ N/mm}^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c.0,d}}{k_{c.z} \cdot f_{c.0,d}} + k_m \cdot \frac{\sigma_{m.y.d}}{f_{m.y.d}} + \frac{\sigma_{m.z.d}}{f_{m.z.d}} = 0.67 \quad \text{Ok!} \]

Shear checks to EC5-1-1, 6.1.7

Applied shear stress

\[ F := \gamma_Q \cdot H_G \cdot l = 52.2 \text{ kN} \]
\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 0.65 \text{ N/mm}^{-2} \]

Design shear strength

\[ f_{v.d} := \frac{k_{mod} \cdot f_{120.v.k}}{\gamma_m} = 1.14 \text{ N/mm}^{-2} \]
\[ \frac{\tau_d}{f_{v,d}} = 0.57 \quad \text{Ok!} \]
Let \( b = 120 \, \text{mm} \)

Section properties
\[
\begin{align*}
\textcolor{blue}{b_{\text{long}} := 19 \, \text{mm}} \\
\textcolor{blue}{b_{\text{trans}} := 31.5 \, \text{mm}} \\
\end{align*}
\]

\[
A_{\text{eff}} := b_{\text{eff,120}} \cdot y = 0.059 \, \text{m}^2
\]

Effective cross-sectional area of section acting as a column

\[
I_z := 3 \cdot \left( \frac{y \cdot b_{\text{long}}^3}{12} \right) + 2 \cdot y \cdot b_{\text{long}} \cdot (b_{\text{long}} + b_{\text{trans}})^2
\]

\[
= (9.86 \cdot 10^7) \, \text{mm}^4
\]

Second moment of inertia

\[
i_z := \sqrt{\frac{I_z}{y \cdot 3 \cdot b_{\text{long}}}} = 42 \, \text{mm}
\]

Effective radius of gyration

\[
W_z := \frac{b_{\text{eff,120}}^2 \cdot y}{6} = (5.821 \cdot 10^5) \, \text{mm}^3
\]

Effective section modulus

Combined Axial & Bending checks to EC5-1-1, 6.3.2
Slenderness ratios
\[
l_{\text{eff}} := h = 2.69 \, \text{m}
\]

Effect length of 'column'

\[
\lambda_z := l_{\text{eff}} = 64.67
\]

\[
\lambda_{\text{rel,z}} := \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = 1.1
\]

\[
k - \text{coefficients}
\]

\[
\beta_c := 0.1 \quad \text{Factor for straightness limits}
\]

\[
k_m := 0.7 \quad \text{Re-distribution of stresses factor}
\]

\[
k_z := 0.5 \left( 1 + \beta_c \cdot (\lambda_{\text{rel,z}} - 0.3) + \lambda_{\text{rel,z}}^2 \right)
\]

\[
k_{c,z} := \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel,z}}^2}} = 0.687
\]
Medium term - imposed load as leading variable

\[ k_{\text{mod}} := 0.8 \]  
\[ \gamma_m := 1.25 \]

Modification factor
Partial factor for material property

Design compression strength:

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{c,0,k}}{\gamma_m} = 13.44 \, N \cdot mm^{-2} \]

Applied compression stress:

\[ N_{Ed} := R_{b,MT.2} = 260.28 \, kN \]
\[ \sigma_{c,0,d} := \frac{N_{Ed}}{A_{\text{eff}}} = 4.4 \, N \cdot mm^{-2} \]

Design bending strength:

\[ f_{m,y,d} := \frac{k_{\text{mod}} \cdot f_{m,k}}{\gamma_m} = 15.36 \, N \cdot mm^{-2} \]
\[ f_{m,z,d} := \frac{k_{\text{mod}} \cdot f_{m,k}}{\gamma_m} = 15.36 \, N \cdot mm^{-2} \]

Applied bending stress:

\[ \sigma_{m,y,d} := 0 \cdot N \cdot mm^{-2} \]
\[ M_{Ed,z} := \frac{w_{z,MT.2} \cdot y \cdot h^2}{8} = 0.17 \, kN \cdot m \]
\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 0.29 \, N \cdot mm^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.5 \quad \text{Ok!} \]

Short term - wind load as leading variable

\[ k_{\text{mod}} := 0.9 \]  
\[ \gamma_m := 1.25 \]

Modification factor
Partial factor for material property

Design compression strength:

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{c,0,k}}{\gamma_m} = 15.12 \, N \cdot mm^{-2} \]
Applied compression stress:

\[ N_{Ed} := R_{b,ST.2} = 288.53 \text{ kN} \]

\[ \sigma_{c,0.d} := \frac{N_{Ed}}{A_{eff}} = 4.88 \text{ N \cdot mm}^{-2} \]

Design bending strength:

\[ f_{m.y.d} := \frac{k_{mod} \cdot f_{m.k}}{\gamma_m} = 17.28 \text{ N \cdot mm}^{-2} \]

\[ f_{m.z,d} := \frac{k_{mod} \cdot f_{m.k}}{\gamma_m} = 17.28 \text{ N \cdot mm}^{-2} \]

Applied bending stress:

\[ \sigma_{m.y,d} := 0 \cdot \text{N \cdot mm}^{-2} \]

\[ M_{Ed.z} := \frac{w_{z,ST.2} \cdot y \cdot h^2}{8} = 0.57 \text{ kN \cdot m} \]

\[ \sigma_{m.z,d} := \frac{M_{Ed.z}}{W_z} = 0.97 \text{ N \cdot mm}^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0.d}}{k_{c.z} \cdot f_{c,0.d}} + k_{m} \cdot \frac{\sigma_{m.y,d}}{f_{m.y,d}} + \frac{\sigma_{m.z,d}}{f_{m.z,d}} = 0.53 \quad \text{Ok!} \]

Shear checks to EC5-1-1, 6.1.7

Applied shear stress

\[ F := \gamma_Q \cdot H_G \cdot l = 52.2 \text{ kN} \]

\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 1.32 \text{ N \cdot mm}^{-2} \]

Design shear strength

\[ f_{v,d} := \frac{k_{mod} \cdot f_{v,k}}{\gamma_m} = 2.88 \text{ N \cdot mm}^{-2} \]

\[ \frac{\tau_d}{f_{v,d}} = 0.46 \quad \text{Ok!} \]
Design of YV1 - level 3

Wall properties

- Height of wall: \( h = 3.35 \text{ m} \)
- Length of wall panel: \( y_{\text{total}} = 6 \text{ m} \)
- Effective length of wall: \( l = 5.5 \text{ m} \)
- Total length of wall: \( L = 12.5 \text{ m} \)
- Section length: \( y = 1 \text{ m} \)

**Treating CLT as a homogenous material**

Let \( b = 120 \text{ mm} \)

**Section properties**

- Effective cross-sectional area of section: \( A_{\text{eff}} = b \cdot y = 0.12 \text{ m}^2 \)
- Effective radius of gyration: \( i_z = \frac{b}{\sqrt{12}} = 35 \text{ mm} \)
- Effective section modulus: \( W_z = \frac{b^2 \cdot y}{6} = \left(2.4 \cdot 10^6\right) \text{ mm}^3 \)

**Combined Axial & Bending checks to EC5-1-1, 6.3.2**

**Slenderness ratios**

- Effective length of 'column': \( l_{\text{eff}} = h = 3.35 \text{ m} \)
- Slenderness ratio: \( \lambda_z = \frac{l_{\text{eff}}}{i_z} = 96.71 \)

- Effective slenderness ratio: \( \lambda_{\text{rel},z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{120.0}}{E_{120.0}}} = 1.54 \)

**k - coefficients**

- Factor for straightness limits: \( \beta_c = 0.1 \)
- Re-distribution of stresses factor: \( k_m = 0.7 \)

- Combined axial & bending factor: \( k_z = 0.5 \left(1 + \beta_c \cdot (\lambda_{\text{rel},z} - 0.3) + \lambda_{\text{rel},z}^2\right) \)

- Combined axial & bending factor: \( k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel},z}^2}} = 0.389 \)
**Medium term - imposed load as leading variable**

\[ \begin{align*}
\text{k}_{\text{mod}} & := 0.8 \quad \text{Modification factor} \\
\gamma_m & := 1.25 \quad \text{Partial factor for material property}
\end{align*} \]

**Design compression strength:**

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{120,c,0,k}}{\gamma_m} = 6.4 \, N \cdot mm^{-2} \]

**Applied compression stress:**

\[ N_{Ed} := R_{b,MT.3} = 218.16 \, kN \]

\[ \sigma_{c,0,d} := \frac{N_{Ed}}{A_{eff}} = 1.82 \, N \cdot mm^{-2} \]

**Design bending strength:**

\[ f_{m,y,d} := \frac{k_{\text{mod}} \cdot f_{120,m,y,k}}{\gamma_m} = 7.1 \, N \cdot mm^{-2} \]

\[ f_{m,z,d} := \frac{k_{\text{mod}} \cdot f_{120,m,z,k}}{\gamma_m} = 2.43 \, N \cdot mm^{-2} \]

**Applied bending stress:**

\[ \sigma_{m,y,d} := 0 \cdot N \cdot mm^{-2} \]

\[ M_{Ed,z} := \frac{w_{z,MT.3} \cdot y \cdot h^2}{8} = 0.29 \, kN \cdot m \]

\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 0.12 \, N \cdot mm^{-2} \]

**Combination ratio: buckling along Z-Z axis**

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + \frac{k_m \cdot \sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.78 \quad \text{Ok!} \]

**Short term - wind load as leading variable**

\[ \begin{align*}
\text{k}_{\text{mod}} & := 0.9 \quad \text{Modification factor} \\
\gamma_m & := 1.25 \quad \text{Partial factor for material property}
\end{align*} \]

**Design compression strength:**

\[ f_{c,0,d} := \frac{k_{\text{mod}} \cdot f_{120,c,0,k}}{\gamma_m} = 7.2 \, N \cdot mm^{-2} \]
Applied compression stress:

\[ N_{Ed} := R_{b,ST,3} = 234.77 \text{ kN} \]

\[ \sigma_{c,0,d} := \frac{N_{Ed}}{A_{eff}} = 1.96 \text{ N} \cdot \text{mm}^{-2} \]

Design bending strength:

\[ f_{m.y,d} := \frac{k_{mod} \cdot f_{120.m.y.k}}{\gamma_m} = 7.99 \text{ N} \cdot \text{mm}^{-2} \]

\[ f_{m.z,d} := \frac{k_{mod} \cdot f_{120.m.z.k}}{\gamma_m} = 2.74 \text{ N} \cdot \text{mm}^{-2} \]

Applied bending stress:

\[ \sigma_{m.y,d} := 0 \text{ N} \cdot \text{mm}^{-2} \]

\[ M_{Ed,z} := \frac{w_{z,ST,3} \cdot y \cdot h^2}{8} = 0.96 \text{ kN} \cdot \text{m} \]

\[ \sigma_{m.z,d} := \frac{M_{Ed,z}}{W_z} = 0.4 \text{ N} \cdot \text{mm}^{-2} \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m.y,d}}{f_{m.y,d}} + \frac{\sigma_{m.z,d}}{f_{m.z,d}} = 0.84 \quad \text{Ok!} \]

Shear checks to EC5-1-1, 6.1.7

Applied shear stress

\[ F := \gamma_Q \cdot H_F \cdot l = 45 \text{ kN} \]

\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 0.56 \text{ N} \cdot \text{mm}^{-2} \]

Design shear strength

\[ f_{v,d} := \frac{k_{mod} \cdot f_{120.v.k}}{\gamma_m} = 1.14 \text{ N} \cdot \text{mm}^{-2} \]

\[ \frac{\tau_d}{f_{v,d}} = 0.49 \quad \text{Ok!} \]
**Treating the CLT as a non-homogenous material**

Let \( b = 120 \text{mm} \)

Section properties

\[
\begin{align*}
 b_{\text{long}} &:= 19 \text{ mm} \\
 b_{\text{trans}} &:= 31.5 \text{ mm}
\end{align*}
\]

\[
A_{\text{eff}} := b_{\text{eff.120}} \cdot y = 0.059 \text{ m}^2
\]

Effective cross-sectional area of section acting as a column

\[
I_z := 3 \cdot \left( \frac{y \cdot b_{\text{long}}^3}{12} \right) + 2 \cdot y \cdot b_{\text{long}} \cdot (b_{\text{long}} + b_{\text{trans}})^2
\]

\[
I_z = \left( 9.86 \cdot 10^7 \right) \text{ mm}^4
\]

Second moment of inertia

\[
i_z := \sqrt{\frac{I_z}{y \cdot 3 \cdot b_{\text{long}}}} = 42 \text{ mm}
\]

Effective radius of gyration

\[
W_z := \frac{b_{\text{eff.120}}^2 \cdot y}{6} = \left( 5.821 \cdot 10^5 \right) \text{ mm}^3
\]

Effective section modulus

Combined Axial & Bending checks to EC5-1-1, 6.3.2

Slenderness ratios

\[
l_{\text{eff}} := h = 3.35 \text{ m}
\]

Effect length of 'column'

\[
\lambda_z := \frac{l_{\text{eff}}}{i_z} = 80.54
\]

\[
\lambda_{\text{rel.z}} := \frac{\lambda_z}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} = 1.37
\]

\[
k - \text{coefficients}
\]

\[
\beta_c := 0.1 \quad \text{Factor for straightness limits}
\]

\[
k_m := 0.7 \quad \text{Re-distribution of stresses factor}
\]

\[
k_z := 0.5 \left( 1 + \beta_c \cdot (\lambda_{\text{rel.z}} - 0.3) + \lambda_{\text{rel.z}}^2 \right)
\]

\[
k_{c,z} := \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{\text{rel.z}}^2}} = 0.483
\]
Medium term - imposed load as leading variable

\[ k_{\text{mod}} := 0.8 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

**Design compression strength:**

\[ f_{c.0,d} := \frac{k_{\text{mod}} \cdot f_{c.0,k}}{\gamma_m} = 13.44 \ \text{N} \cdot \text{mm}^{-2} \]

**Applied compression stress:**

\[ N_{Ed} := R_{b,MT.3} = 218.16 \ \text{kN} \]
\[ \sigma_{c.0,d} := \frac{N_{Ed}}{A_{\text{eff}}} = 3.69 \ \text{N} \cdot \text{mm}^{-2} \]

**Design bending strength:**

\[ f_{m.y,d} := \frac{k_{\text{mod}} \cdot f_{m.k}}{\gamma_m} = 15.36 \ \text{N} \cdot \text{mm}^{-2} \]
\[ f_{m.z,d} := \frac{k_{\text{mod}} \cdot f_{m.k}}{\gamma_m} = 15.36 \ \text{N} \cdot \text{mm}^{-2} \]

**Applied bending stress:**

\[ \sigma_{m.y,d} := 0 \cdot \text{N} \cdot \text{mm}^{-2} \]
\[ M_{Ed,z} := \frac{w_{z,MT.3} \cdot y \cdot h^2}{8} = 0.29 \ \text{kN} \cdot \text{m} \]
\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 0.49 \ \text{N} \cdot \text{mm}^{-2} \]

**Combination ratio:** buckling along Z-Z axis

\[ \frac{\sigma_{c.0,d}}{k_{c.z} \cdot f_{c.0,d}} + k_m \cdot \frac{\sigma_{m.y,d}}{f_{m.y,d}} + \frac{\sigma_{m.z,d}}{f_{m.z,d}} = 0.6 \quad \text{Ok!} \]

Short term - wind load as leading variable

\[ k_{\text{mod}} := 0.9 \quad \text{Modification factor} \]
\[ \gamma_m := 1.25 \quad \text{Partial factor for material property} \]

**Design compression strength:**

\[ f_{c.0,d} := \frac{k_{\text{mod}} \cdot f_{c.0,k}}{\gamma_m} = 15.12 \ \text{N} \cdot \text{mm}^{-2} \]
Applied compression stress:

\[ N_{Ed} := R_{b,ST,3} = 234.77 \text{ kN} \]

\[ \sigma_{c,0,d} := \frac{N_{Ed}}{A_{eff}} = 3.97 \text{ kN/mm}^2 \]

Design bending strength:

\[ f_{m,y,d} := \frac{k_{mod} \cdot f_{m,k}}{\gamma_m} = 17.28 \text{ kN/mm}^2 \]

\[ f_{m,z,d} := \frac{k_{mod} \cdot f_{m,k}}{\gamma_m} = 17.28 \text{ kN/mm}^2 \]

Applied bending stress:

\[ \sigma_{m,y,d} := 0 \text{ kN/mm}^2 \]

\[ M_{Ed,z} := \frac{w_{z,ST,3} \cdot y \cdot h^2}{8} = 0.96 \text{ kN/m} \]

\[ \sigma_{m,z,d} := \frac{M_{Ed,z}}{W_z} = 1.64 \text{ kN/mm}^2 \]

Combination ratio: buckling along Z-Z axis

\[ \frac{\sigma_{c,0,d}}{k_{c,z} \cdot f_{c,0,d}} + k_m \cdot \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} = 0.64 \quad \text{Ok!} \]

Shear checks to EC5-1-1, 6.1.7

Applied shear stress

\[ F := \gamma_Q \cdot H \cdot l = 45 \text{ kN} \]

\[ \tau_d := \frac{1.5 \cdot F}{A_{eff}} = 1.14 \text{ kN/mm}^2 \]

Design shear strength

\[ f_{v,d} := \frac{k_{mod} \cdot f_{v,k}}{\gamma_m} = 2.88 \text{ kN/mm}^2 \]

\[ \frac{\tau_d}{f_{v,d}} = 0.4 \quad \text{Ok!} \]

Page 35 of 35