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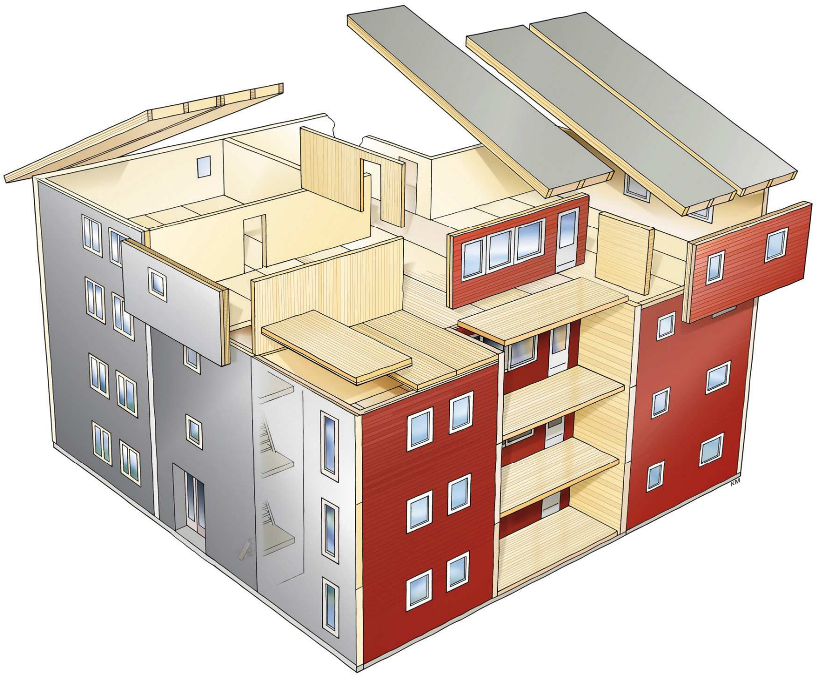

Wind-induced vibrations of a multi-storey residential building in cross-laminated timber in the serviceability limit state

A case study at Södra Älvstranden, Göteborg

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

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Cover:
Assembly of CLT elements in a multi-storey residential building produced by Martinsons, illustrated by Kjell Magnusson.

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ABSTRACT

The municipality of Gothenburg is working towards connecting the city centre with the riverfront, creating new residential, office and recreational environments along the quay. Along with new development at the south side of the river an organization has formed in order to promote sustainable development and construction with environmentally friendly building materials. Multi-storey building construction in timber is becoming more common and many new composite materials are competing on the market. Cross-laminated timber (CLT) provides higher stiffness compared to traditional onsite timber framing construction and consist of planar elements with cross-wise layers of timber lamellas. CLT elements can be used for all structural members in a building system, shear transmission and vertical load-bearing.

As for all light-weight structures dynamic aspects are of concern in serviceability limit state. This thesis will check the dynamic response of an eight storey high residential timber building in CLT located at Skeppsbron, a district in Gothenburg close to Göta Älv, with regard to wind-induced vibrations. Discomfort due to wind-induced low-frequency vibration may be of concern and comfort criteria regulations differ in threshold limits and evaluation methodology between standards. Through analysis of a fictitious building conducted in the finite element program "FEM design 3D structure" natural frequencies were obtained for which acceleration was calculated in order to check vibration criteria given in regulations. Regulations checked were those provided in Eurocode, ISO, Canadian, Japanese and British standards.

Results from static and dynamic analysis showed that the building fulfils requirements of lateral deflection according to German codes and comfort criteria according to Canadian and ISO standard. According to Japanese and British regulations, residents will perceive motion. Acceleration obtained from calculations shows that there is a 20% probability of perception according to Japanese regulations and a low probability of adverse comment according to British.

Key words: wind-induced vibrations, CLT, cross-laminated timber, multi-storey timber building, comfort criteria, Skeppsbron

Vindinducerade vibrationer av ett flervåningsbostadshus i korslaminerat trä i bruksstadiet
En studie vid Södra Älvstranden, Göteborg
Examensarbete inom Structural Engineering and Building Performance Design
MATILDA KRYH
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SAMMANFATTNING

Göteborgs stadskärna ligger nära Göta Älv men upplevs som distanserad från vattnet, genom att skapa nya ytor för boende, kontorslokaler och allmänna platser längs med kajen kan vattnet integreras som en naturlig del i stadsplaneringen. I samband med planeringen ligger fokus på hållbar utveckling och byggande med miljövänliga byggnadsmaterial. Flervåningshus i trä börjar bli allt vanligare och många nya kompositmaterial konkurrerar på marknaden. Korslaminerat trä (CLT) är ett material med högre styvhet jämfört med platsbyggda skivregelsystem i trä och består av element uppbyggda av träplankor med lager i olika riktningar. CLT kan fungera som bärande väggar samt ta upp skjuvkrafter och kan användas både som vägg och golv element.

Detta examensarbete behandlar den dynamiska responsen av ett fiktivt åttavåningsbostadshus i CLT, beläget vid Skeppsbron, med hänsyn till vindinducerade vibrationer. Den dynamiska responsen för strukturer byggda i lättviktmaterial är viktigt att undersöka för att se hur byggnaden fungerar i bruksstadiet och om komfortkraven för boende uppfylls. Vibrationer som uppkommer av vind är lågfrekventa och kan ge upphov till obehag. Byggnadsregler och standarder för komfortkrav skiljer sig åt vid kritiska värden, begränsningar och utvärderingsmetoder.

Genom att modellera en fiktiv byggnad i finita element programmet ”FEM design 3D structure” så erhölls egenfrekvenser för byggnaden som användes för att beräkna accelerationer som sedan jämfördes med olika byggnadsregler. De byggnadsregler och standarder som jämfördes var Eurocode, ISO, kanadensisk, japansk och brittisk standard.

Resultat från statisk och dynamisk analys indikerar att byggnaden uppfyller kraven för horisontella förskjutningar enligt tysk kod och komfortkrav enligt ISO och kanadensisk standard. Enligt japansk byggnadsnorm uppgår sannolikheten för vibrationskänning till 20% och sannolikheten för klagomål är låg enligt brittisk.

Nyckelord: vindinducerade vibrationer, korslaminerat trä, CLT, flervåningshus i trä, komfortkrav, Skeppsbron

Contents

Preface

This thesis is the final part of the Master program Structural engineering and Building performance design and has been carried out at COWI AB, Göteborg from June to December 2012 in cooperation with the Division of Structural Engineering at Chalmers University of Technology, Sweden. Professor Robert Kliger was the examiner for the thesis.

The modelling of the structure was made in “FEM design 3D structure”, a finite element software developed by Strusoft and the modelling was eased by the support from co-workers at COWI and the support at Strusoft.

We would like to thank our supervisors Thomas Hallgren, Magnus Nilber and Robert Kliger who have motivated and encouraged us during the process and contributed with knowledge and feedback during our work. We would especially like to thank Thomas Hallgren at COWI for all the help concerning dynamic analysis, experience in the area and the helpfulness during the whole project.

A special thanks to our opponent Anna Teike for great feedback and cooperation throughout the project.

Finally, we would like to thank the co-workers at COWI for their helpful attitude and encouragement during the whole process

Göteborg, december 2012

Matilda Kryh

Mia Nilsson

Notations

Roman upper case letters

A	Peak acceleration
A_i	Area of the individual layers
B	Height of the composite panel
B_d	Background response factor
$\boldsymbol{C}$	System damping matrix
E_0	Modulus of elasticity parallel to grain
E_{90}	Modulus of elasticity perpendicular to grain
$E_{c.k}$	Effective characteristic modulus of elasticity in compression
E_{mean}	Mean modulus of elasticity
$E_{m.k}$	Effective characteristic modulus of elasticity in bending
E_{eff}	Effective modulus of elasticity
E_i	Modulus of elasticity of the individual layers
$E_{t.k}$	Effective characteristic modulus of elasticity in tension
$(EI)_{eff}$	Effective bending stiffness
$(EI)_{lam}$	Effective bending stiffness of the cross-section
$(ES)(z)$	First moment of axial stiffness at level z of cross-section
$(GA)_{lam}$	Effective shear modulus of the cross-section
G_k	Shear modulus
G_{mean}	Mean shear modulus
$G_{v.k}$	Characteristic shear modulus
$G_{r.k}$	Characteristic rolling shear modulus
$G_{R.mean}$	Mean rolling shear modulus
I	Moment of inertia
I_i	Moment of inertia of the individual layers
I_v	Turbulence intensity factor
I_{vh}	Turbulence intensity factor at reference height
$\boldsymbol{K}$	System springer stiffness matrix
L	Span length
$\boldsymbol{M}$	System mass matrix
R_D	Resonant response factor
R_v	Root mean square of the resonance factor
X	Acceleration

$X_{EC.5}$	Acceleration according to Eurocode based of the first natural frequency with a 5 year wind
$X_{EC.skepp}$	Acceleration according to Eurocode based of the first natural frequency with mean wind at Skeppsbron
$X_{ISO.1}$	Acceleration according to ISO based of the first natural frequency with a 1 year wind
$X_{ISO.skepp}$	Acceleration according to ISO based of the first natural frequency with mean wind at Skeppsbron
T	Total period of the day or night which vibration may occur
T_a	Return period in years
$VDV_{b/d,day/night}$	Vibration dose value
VDV_{ISO}	Vibration dose value using acceleration calculated according to ISO
VDV_{EC}	Vibration dose value using acceleration calculated according to Eurocode
$VDV_{day.skepp}$	Vibration dose value during day for mean wind at Skeppsbron
$VDV_{night.skepp}$	Vibration dose value during night for mean wind at Skeppsbron
W_b	Frequency weighting modulus for vertical vibration
W_d	Frequency weighting modulus for horizontal vibration

Roman lower case letters

a_i	Distance from local and global centre of gravity
a_m	Total thickness of a CLT panel
a_{m-2}	Thickness of a CLT panel minus the outer two lamellas
a_{m-4}	Thickness of a CLT panel minus the outer four lamellas
$a(t)$	Frequency-weighted acceleration in m/s^2
b	Width of panel
b_c	Thickness of the cross-section/composites
b_f	Width of building face
c_f	Force coefficient for wind load
c_{exp}	Exposure factor
d	Total thickness of panel
g	Acceleration of gravity
g_{DB}	Peak factor for background component
h_{tot}	Total height/thickness of panel

f_0	First natural frequency in a structural direction of a building and in torsion
f_n	Natural frequency
k	Structural stiffness
k_i	Composition factor
k_p	Peak acceleration factor
k_{topog}	Peak topography exposure factor
$k_{tr.change}$	Peak terrain roughness change exposure factor
$k_{tr.z}$	Peak terrain roughness and height exposure factor
m	Structural mass
m_e	Mass per unit length
n	Number of layer
n_1	First mode natural frequency
n_{EC}	First natural frequency according to Eurocode
$p(t)$	Variable load with regard to time
q_{pv}	Wind pressure
t	Thickness of panel
t_{layer}	Thickness of one cross-layer
t_p	Time duration peak wind
$u(t)$	Displacement of a system with regard to time
$\dot{u}(t)$	Velocity of a system with regard to time
$\ddot{u}(t)$	Acceleration of a system with regard to time
v_b	Reference wind speed
v_m	Mean wind speed
v_{ref}	Reference wind speed
v_{site}	Wind speed at site
v_{Ta}	Wind speed for a return period T_a
x_1	Maximum structural displacement with structural mass acting as a load at the end of a cantilever in the vibration direction
x_D	Displacement under static wind loading

Greek letters

β	Shear correction factor
γ	Reduction factor for imaginary fasters
σ_v	Standard deviation of turbulence

σ_x	Standard deviation of acceleration
τ_0	Nominal shear stress in plane
τ_v	Shear stress in cross section
τ_T	Shear stress due to torsional moment in gluing interface
υ	Up-crossing frequency
ϕ_1	Fundamental flexural mode
φ	Variable for load action dependent of storey occupancy
$\psi_{E.i}$	Combination coefficient for a variable action *i*, to be used when determining the effects of the design seismic action
$\psi_{2.i}$	Combination coefficient for a quasi-permanent value of variable action *i*
ω_n	Un-damped circular natural frequency

Abbreviations

AIJ	Architectural Institute of Japan
CFD	Computational Fluid Dynamics
CLT	Cross-Laminated Timber
DIN	Deutsche Institut für Normung
EC	Eurocode
FEM	Finite Element Model
ISO	International Organization of Standardization
NBCC	National Building Code of Canada
RMS	Root Mean Square
SDOF	Single Degree Of Freedom
SHMI	Swedish Metrological and Hydrological Institute
SLS	Serviceability Limit State
ULS	Ultimate Limit State
VDV	Vibration Dose Value

1 Introduction

The city of Gothenburg is facing infrastructural and urban development changes and in accordance with these there are several on-going projects run by the municipal of Gothenburg to reunite the city centre with the riverside. The Urban Planning Department has proposed that a new "meeting area" for the citizens. This area should be built on the south side of Göta Älv, creating a connection between the two sides of the river. In the proposal, the area should consist of four building blocks with a mixture of residential buildings, offices and commercial enterprise. Södra Älvstranden Utveckling AB, a municipality-owned corporate group, is responsible for the development of the area and focuses on sustainable building in Gothenburg; therefore a multi-storey timber building could be a good alternative for Skeppsbron, district at the south side of Göta Älv.

Since the regulations for construction of multi-storey timber buildings in Sweden changed in 1994 many wood processing companies has developed building systems for multi-storey buildings in timber. The highest building with a structural system constructed in timber in Sweden today is Limnologen, situated in Växjö, consisting of four eight stories high buildings with a planar building system in cross-laminated timber (CLT). Building systems in CLT are relatively stiff in comparison with other more traditional timber construction techniques, as timber framing. Although CLT elements provide stiffer structures, horizontal stability and dynamic aspects are critical in design as for all light-weight structures.

The performance of timber structures in serviceability limit state often focus on their acoustic performance, vertical deflection and vibrations induced by occupants. Due to the light-weight construction other aspect that needs to be taken into account are vibrations induced by external sources causing discomfort for occupants. In high-rise buildings, wind-induced low-frequency vibrations can cause discomfort for occupants and regulations of these types of vibration were developed although none is internationally accepted.

The wind conditions at Skeppsbron are not extreme however they will act throughout the entire year and this paper will investigate wind-induced horizontal vibration and lateral displacements causing discomfort for occupants.

1.1 Aim

The aim of the thesis was to investigate wind-induced horizontal vibrations and lateral displacements of a multi-storey timber building constructed in a coastal area with relatively harsh wind-conditions, regarding serviceability limit state. Horizontal low-frequency vibrations can cause discomfort for residents in high-rise buildings and this thesis investigates if vibrations in a light-weight timber structure situated in the quay area of Gothenburg would be of the magnitude causing discomfort for residents. An evaluation of vibration was made based on the regulations found in Eurocode, ISO-, British- Canadian- and Japanese standards.

1.2 Method

The project was divided into three main parts; literature study, modelling of building and comparison and evaluation of results with regard to regulations. Different building systems for existing and planned multi-storey buildings in cross-laminated timber were studied as well as material properties and design methods for CLT elements. The influence of wind on high-rise and low-rise buildings and different design regulations against horizontal vibrations with regard to residents comfort was considered. The wind conditions at the site Skeppsbron were investigated.

The development plan for Skeppsbron was decisive when determining the dimensions of the fictitious building for which a model in ”FEM design 3D structure” was made. Global checks in ultimate limit state was performed in ”FEM design 3D structure” in order to confirm that the building has satisfying load bearing capacity under self-weight, wind and residential loading. Lateral displacement and natural frequency of the building was obtained from the FEM model, verified with hand calculations and then used in further calculations in order to check vibrations and accelerations with regulations.

1.3 Limitations

The building system studied in this report was only analysed with regard to wind-induced vibrations and lateral deformations in the serviceability limit state. The timber building studied is fictitious with a structural system in cross-laminated timber based on an already developed building system. Loads taken into account were those induced by wind.

Due to the complexity and time consuming aspects, no measurements of an existing building or surveys in order to evaluate the regulations of comfort criteria was made. The checks performed was therefore made under the assumption that the regulations are proven to be reasonable and a good indicator of human response and thresholds to vibrations in buildings.

2 Skeppsbron

The Urban Planning Department of Gothenburg has recently published a paper in accordance with the programme, *Program för Södra Älvstranden,* approved by the municipal assembly in 2007. The programmes' purpose is to give suggestions on how to change sites along the south parts of Göta Älv and create new public areas for citizens, reuniting the central parts of Gothenburg with the riverfront. The intention of the area is to have a mixture between residential buildings, hotels, offices and a public areas; making the quay at Skeppsbron a vibrant environment for both tourists and citizens of Gothenburg (Söderberg & Lööf, 2012).

As a result of the infrastructural changes due to the construction of Västlänken, Skeppsbron and Packhuskajen is proposed to be the new connection for public transport between the northern parts of the river and the central parts of Gothenburg, see Figure 2.1 (Söderberg & Lööf, 2012).

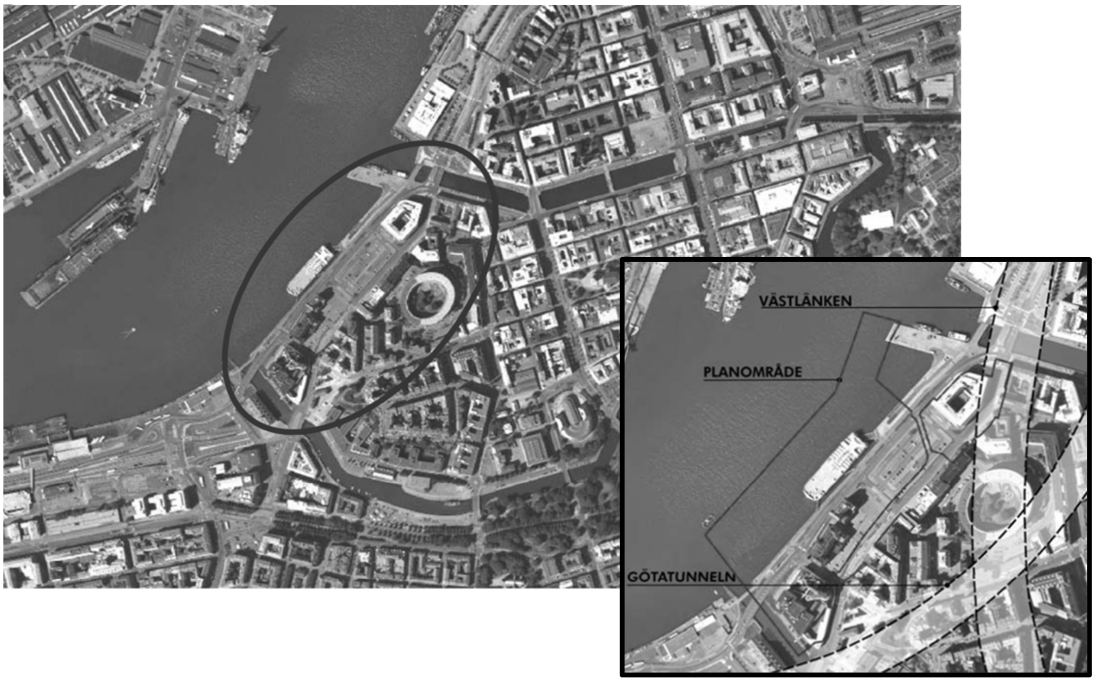


Figure 2.1 Map of Södra Älvstranden at Skeppsbron (Söderberg & Lööf, 2012)

2.1 Development plan

Within the area of Skeppsbron, four building blocks consisting of 40 000 square meters of offices, market and hotels and 400 new apartments should be built according to the development plan, see Figure 2.2. The existing buildings in the area are 7-11 stories high and the building height for new construction is regulated by the total allowed building height in Gothenburg (Söderberg & Lööf, 2012).

Älvstranden Utveckling AB, which is a municipality-owned corporate group responsible for the development at Skeppsbron, is part of a collaborative project organisation called Green Gothenburg. The main focus and idea behind the organisation is to promote environmental and sustainable development in the city of Gothenburg (Älvstranden Utveckling AB, 2012). Construction of new buildings in the area should be done using environmental and sustainable materials as well as have energy efficient building technique solutions as has been done on the north side of the riverfront.

In a comparison made by Canadian wood council in 2004 it is stated that *"Relative to the wood design, the steel and concrete designs embodies: 26% and 57% more energy, emit 34% and 81% more greenhouse gases, release 24% and 47% more pollutants into the air, discharge 400% and 350% more water pollution, produce 8% and 23% more solid waste, and use 11% and 81% more resources (from a weighted resource use perspective)"* (Canadian wood council, 2009). Timber has great potential and may be the construction material for a sustainable future. Since timber construction is on the upcoming and may be classified as environmental friendly in comparison with traditional construction in concrete, timber buildings may be well suited in the area. Using timber in construction at Skeppsbron may act as a statement for further timber construction and sustainability propagation in Gothenburg.

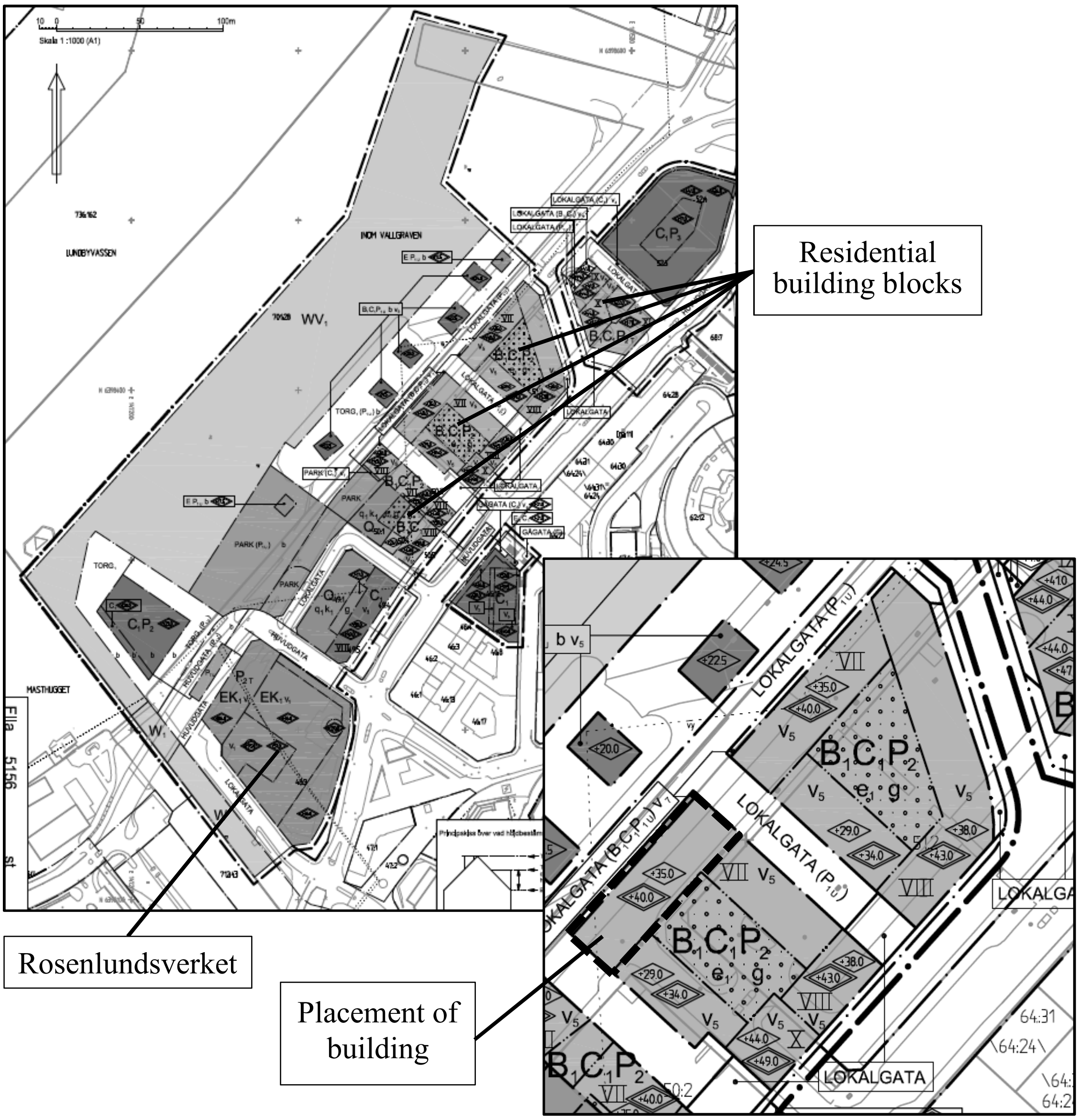


Figure 2.2 Plan sketch for Skeppsbron (Söderberg & Lööf, 2012)

In the plan sketch for Skeppsbron, Figure 2.2, the placement and layout of the proposed building blocks are given. On the south-west side the existing district heating plant, Rosenlundsverket is situated, whereas no residential buildings are to be built in the nearby area due to safety and acoustic reasons. Four building blocks suitable for residential occupancy are located in the north-east side of Skeppsbron. The number of storeys is regulated to seven, which may be increased by one if made as a furnished attic. The total building height should not be greater than 40 meters above the city's zero-plane, which for the city of Gothenburg is 10.07 meters below the mean sea level. Roughly the buildings height should therefore not be greater 25 meters assuming the base of the building starting at 5 meters above sea level.

The location of the fictitious building being analysed in this thesis can be seen in Figure 2.2. The decision was based on where the most critical wind conditions for the area can be found and for which still suits the layout according to the development plan. The building was therefore placed as near the quay as possible and made rectangular with the long side facing the river, making it as slender as possible under wind-loading from west. Dimension was set to 40x16 meters, which is within the dimension according to the plan sketch of 45x25 meters.

2.2 Wind conditions at Skeppsbron

In 2009 the Swedish metrological and hydrological institute, SMHI, investigated the area of Skeppsbron with regard to wind and river-current conditions in order to prevent problems with floating ice and waste at the dock (Gyllenram et al., 2009). In total, six areas were identified and investigated; 1, the outdoor pool, 2, the main walking area, 3, the terminal for buses, trams and boats, 4, the inner yards, 5, the side streets in between the buildings and 6, the quay, see Figure 2.3. The spot for which the building is placed and for which the wind conditions will be investigated is marked in Figure 2.3.

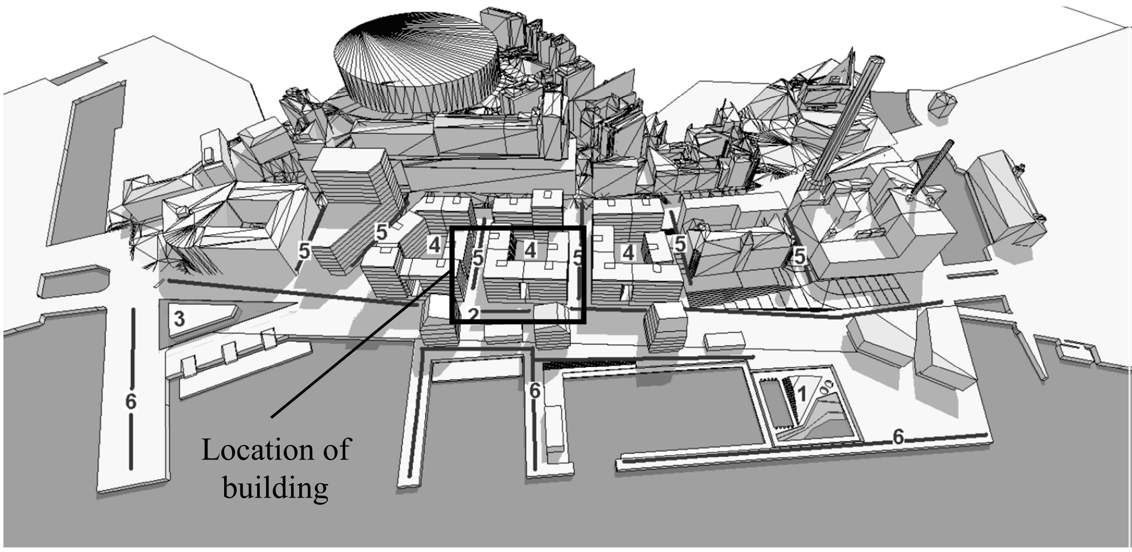


Figure 2.3 *The 6 main spots for wind investigation; 1 the outdoor pool, 2 the main walking area, 3 terminal for buses, trams and boats, 4 inner yards of buildings, 5 side streets in between the buildings, 6 the quays (Gyllenram et al., 2009)*

The investigation was based on measured data from the weather station in Säve, situated northwest of Södra Älvstranden and climate conditions for Skeppsbron was assumed to be the same. Statistical data of wind direction and velocity was measured every third hour between 1961 until 1999 at a height of ten meters and has been compiled to show the main wind direction and yearly mean wind speed in the area.

As can be seen in Figure 2.4 most frequent wind direction is west and winds from south and southwest follow in frequency. The wind rose visualises the frequency of wind direction and is given as a percentile of the total wind over a one-year period, illustrated by the rings. West orientated wind represents 12% of the total wind during a year. The intensity of the wind speed is given by the colours and is distributed dependent of their frequency in occurrence. The main west orientated wind speed lies between 4.5-6.5 m/s and can on occasion get as strong as over 16.5 m/s during a one year period.

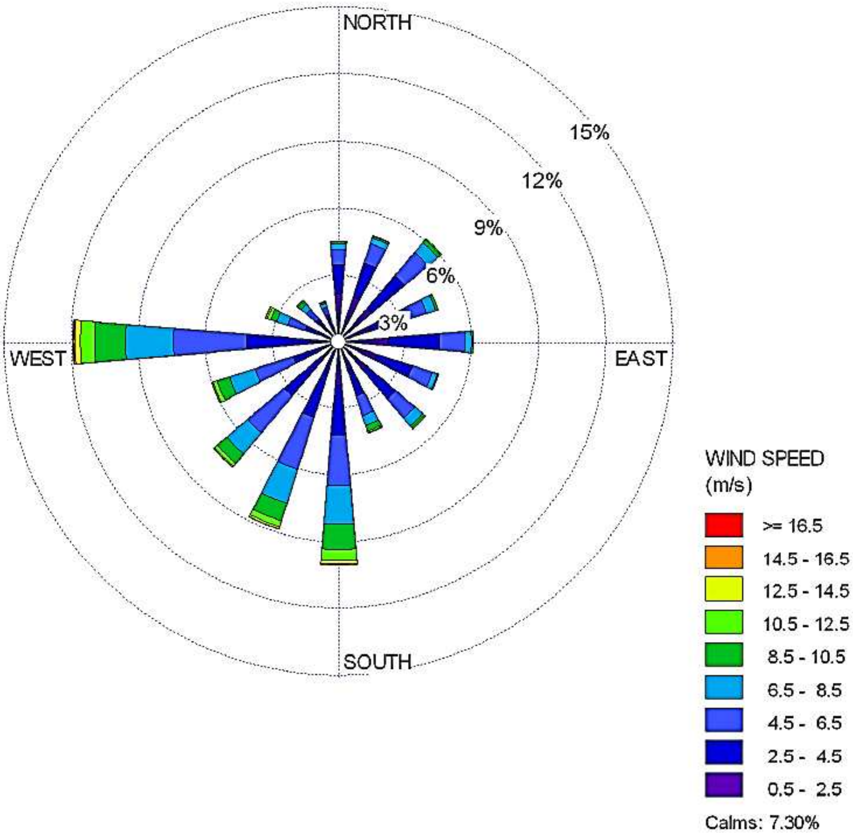


Figure 2.4 Wind rose at Säve weather station, visualising the percentile distribution of the wind direction and wind velocity on a height of ten meters (Gyllenram et al., 2009)

The calculation technique that has been used for the wind flow is a CFD-technique (Computational Fluid Dynamics) where the equations for speed, pressure and turbulence are solved in a large number of points in the model. The method can be seen as a numerical wind tunnel test where a calculation mesh is made for the buildings according to the development plan. Results are given in a three-dimensional wind vector with direction and size, wind pressure and the energy that is created by the winds turbulence. The incoming wind was assumed to have a logarithmical vertical profile and was set to a wind velocity of 3 m/s on a height of ten meters.

With the measured data and the CFD-model an analysis was made for the 6 different areas presented in Figure 2.3. To evaluate the obtained values, the data are compared with a reference value of 5 m/s on an open field. As can be seen in Figure 2.5, the quays and the spots closest to the riverfront are the most critical whilst the inner yards and the leeward side of the buildings are the most protected.

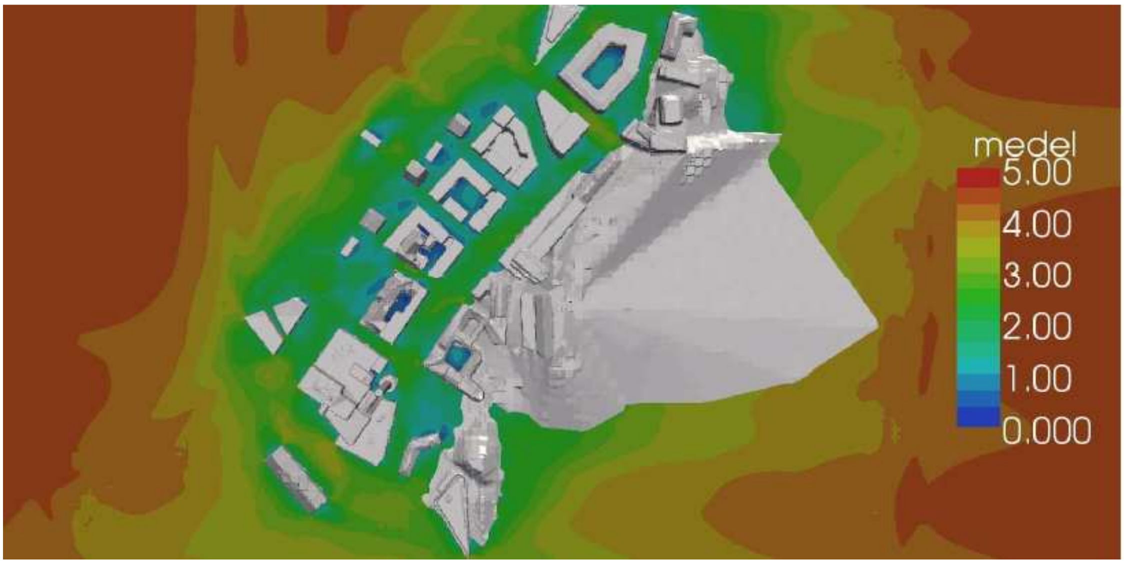


Figure 2.5 *Wind speeds at Skeppsbron with a reference wind of 5 m/s (Gyllenram et al., 2009)*

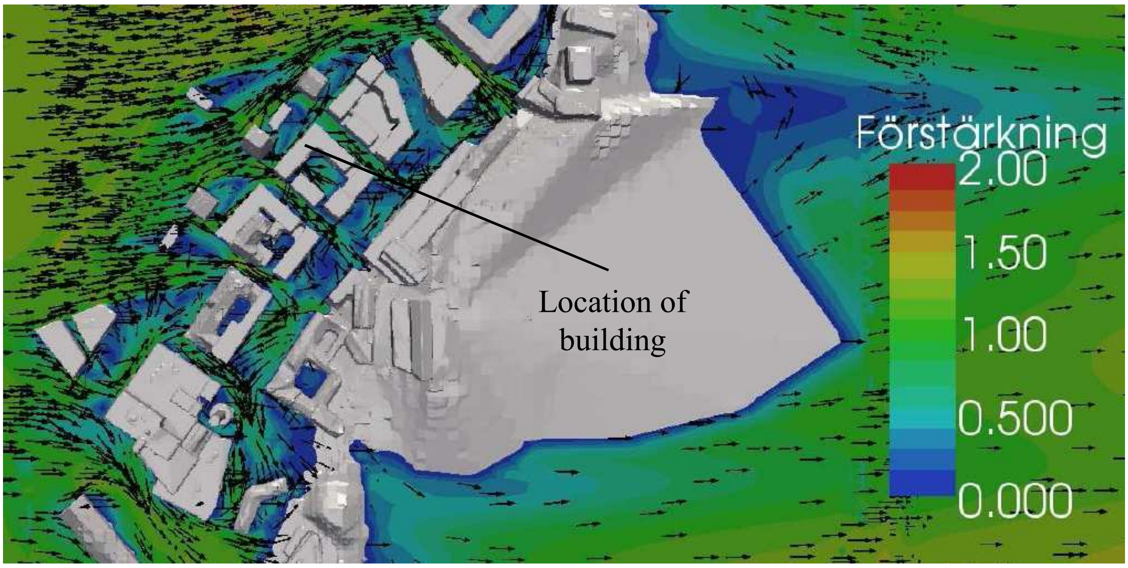


Figure 2.6 *Amplification of a reference wind of 5 m/s and wind direction from west (Gyllenram et al., 2009)*

Generally obstacles such as buildings lower the wind speed but locally a positive acceleration could occur. The amplification in different parts of Skeppsbron is given in Figure 2.6 where the scale represents the amplification, 1.00 represents the reference value.

3 Wind Loading on Buildings

Wind actions on a structure creates occurrences where the flow of the wind interact with the surrounding structures and environment which gives rise to whirls of varying sizes and different rotation patterns. This behaviour creates the gusty and turbulent character of the wind and an example of whirls created around a building is visualized in Figure 3.1. Tall buildings with a slender shape can respond dynamically to wind-loads and can lead to failure if a coupled torsional and flexural mode of oscillation is developed (Mendis et al., 2007).

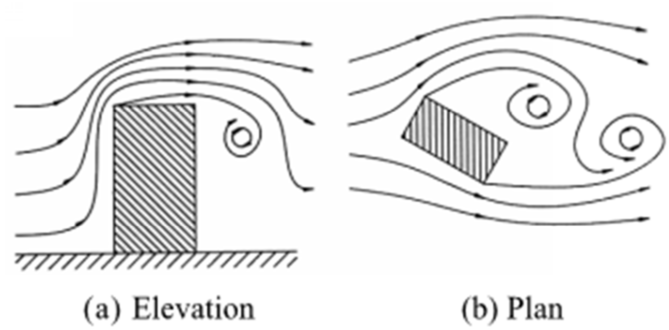


Figure 3.1 Generation of whirls (Mendis et al., 2007)

The wind is considered as a lateral dynamic force where the pressure on the building is divided into a mean part and a fluctuating part according to Eurocode 1 (EC 1). The mean part is calculated by pressure and load coefficients and the fluctuating part is taken into account by including the intensity of turbulence at site, size reduction factors and dynamic amplification.

Wind pressure on external and internal surfaces should be calculated according to EC1. The pressure directed towards the surface is taken as positive and suction directed away from the surface as negative. The roof and walls are subdivided into different zones with specific pressure coefficient in order to calculate the wind-loads on the structure. The internal pressure is dependent on the area and location of the openings in the structure. The surrounding terrain can be categorized according to their associated roughness length and varies from open terrain to close surrounding buildings and obstacles. In EC 1 there are 5 different terrain categories which are visualized in Figure 3.2.

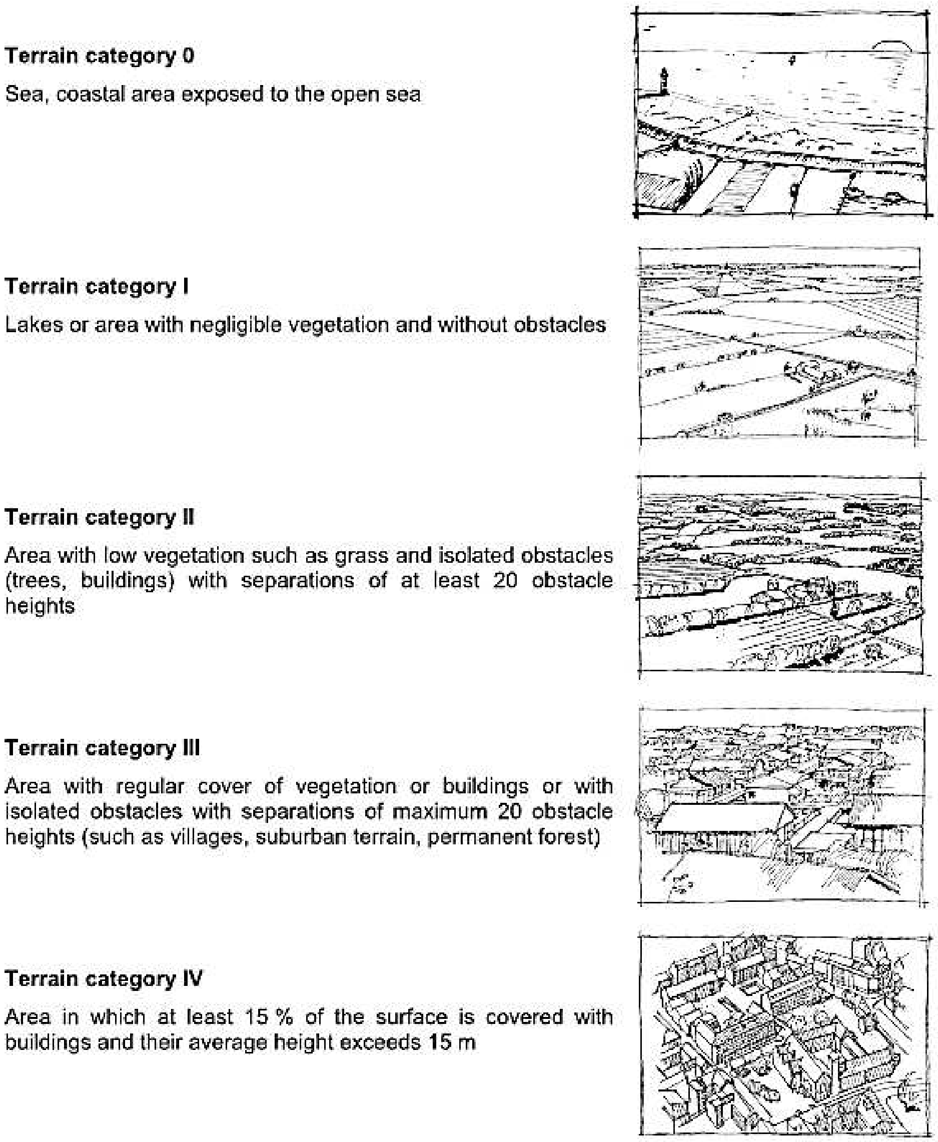


Figure 3.2 Terrain categories 0-IV (Eurocode 1, 2005)

The wind pressure on a structure varies due to fluctuating wind and needs to be considered in design. The pressure differs depending on the shape and geometry of the structure and the effect of the upwind. Wind loading acting on a structure gives a building response that can be subdivided into three parts; along-wind, cross-wind and torsion, see Figure 3.3. The along-wind response of a building can be estimated by taking the mean wind component and the fluctuating component consisting of whirls and gusts into account, if the influence of surrounding buildings and terrain is not significant. Cross-wind response and torsion is mainly a problem when designing tall and flexible structures (Mendis et al., 2007).

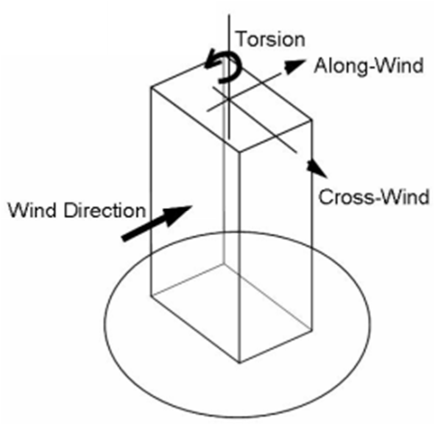


Figure 3.3 Response of wind-loading on building (Mendis et al., 2007)

If the conditions regarding wind and surrounding environment are difficult, the aerodynamic shape of the building is complex or if the building is very flexible a wind-tunnel test can be conducted. Testing in a wind-tunnel gives a more accurate evaluation of the effects on the structure and is common in design of tall buildings. A model of the specific building and the surrounding buildings are placed in a wind-tunnel and rotated to find the behaviour of the structure due to wind from all directions (Mendis et al., 2007). Another way to estimate the behaviour of the structure at a concept design stage is by using CFD techniques as was mentioned in Chapter 2.

3.1 Measuring of wind speed

As stated before, wind pressure is assumed to consist of one mean and one fluctuating part where the mean part could be seen as a mean value of measured wind speeds. This is adopted in codes when calculating acceleration where the mean values of wind speed are used as reference wind speeds and turbulence is used to represent fluctuation of wind. Wind speeds are measured for a long period of time and mean values can be presented in different ways depending on what should be evaluated or investigated. The terms 3 second, 10 minute and hourly averaging wind speeds are often used and represent the amount of peak values included in a mean value of wind measurement. The variation in wind speed is quite large and statistics are used in order to get representative values. For example a 10 minute averaging wind with a 10 year return period represents a mean value of minor mean values during a 10 minute measurement time, with a 10% probability of exceedance in one year. The peak values are smoothened out the longer averaging time is used. The 3 second averaging wind is often used when affects from gust-winds or cyclonic winds should be investigated and for evaluation of more normal wind conditions a 10 minute or hourly averaging wind are used instead.

Figure 3.5 represents wind measurements over a 10 minute period and the different wind averaging mean values are presented by lines in the figure. The thin line represents the true mean value of the wind speed where peaks are equalized and the mean wind speed averaging at 10 minutes becomes 8.6 m/s. The thicker lines represent the mean value of a 1 minute averaging wind, where the mean of varying wind speeds are calculated each 1 minute interval. The sum of the 1 minute mean values gives a higher value and is not equal to the true mean wind speed where fewer wind peaks are included (Harper, Kepert & Ginger, 2008).

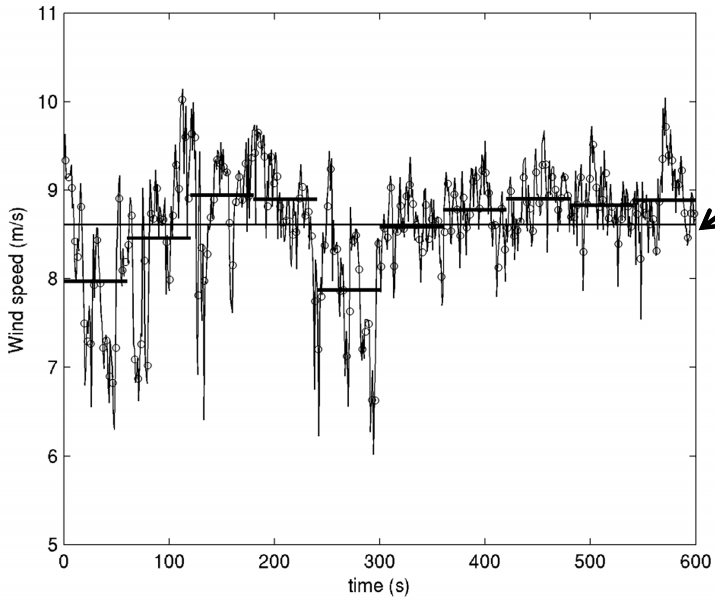


Figure 3.5 Mean wind speeds for a 10 minute time period measured at North West Cape, Western Australia (Harper, Kepert & Ginger, 2008).

3.2 Wind-load on building at Skeppsbron

The eight-story structure that is investigated in this thesis is situated at Skeppsbron which is a coastal area with few obstacles hence terrain category 0 is chosen according to Figure 3.2. The height of the structure is 24 meters which leads to that the shape profile of velocity pressure are subdivided into two parts as can be seen in Figure 3.6.

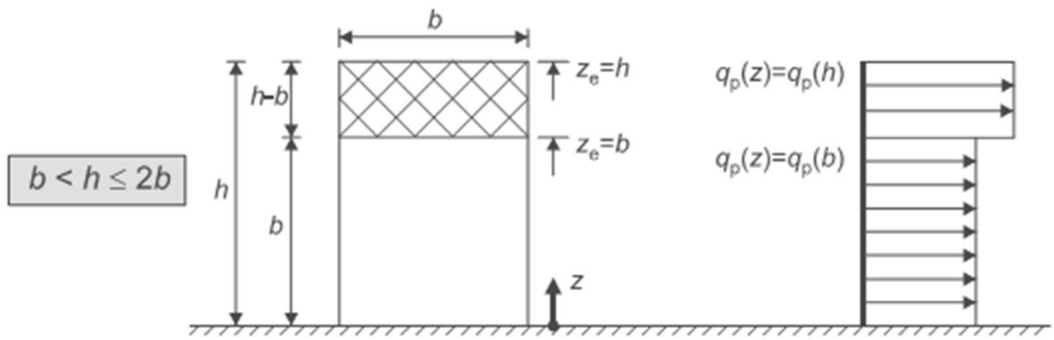


Figure 3.6 Reference heights, dependent on h and b and corresponding velocity pressure profile (Eurocode 1, 2005)

A flat roof is used and the wind-calculations are presented in Appendix A where the resistance against tilting is calculated. In the national annex of EC 1 there are reference wind-velocities presented for different parts of Sweden which are based on measured statistical metrological data. This represents the mean peak wind speed with ten minute duration and a return period of 50 years. For Gothenburg the reference wind speed value is 25 m/s.

4 Wind-Induced Vibrations

Timber buildings have lower stiffness and mass density compared to structural systems made in concrete and steel which means that the structural system of timber needs to be investigated thoroughly regarding dynamic behaviour and horizontal sway caused by wind loads.

Building occupants may sense low-frequency motion in three ways; by balance organs, visual cues and audio cues. This chapter covers the effect of the structural system due to lateral forces, the human response to wind-induced vibrations and also how the design codes and standards for serviceability state takes wind-induced vibrations into account.

4.1 Dynamic response due to wind forces

Loads that are considered dynamic are for example earthquake, wind, wave, explosion and collision forces, in order to find the real behaviour of a structure a dynamic analysis has to be done. The dynamic behaviour of a structural system is mainly dependent on four parameters, stiffness, mass, damping, load intensity and load distribution as a function of time. To find the dynamic response the structural system can be simplified as a mass-spring-dashpot system with one single degree of freedom, see Figure 4.1, which is described with the second-order differential Equation 4.1.

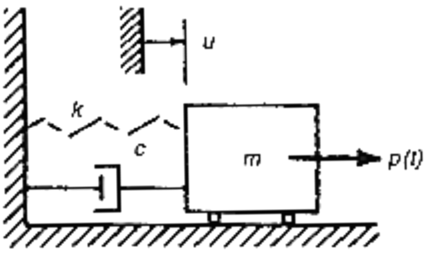


Figure 4.1 Mass-springer-dashpot systems with one degree of freedom (Craig & Kurdila, 2006)

$$\mathbf{M}\ddot{u}(\mathrm{t}) + \mathbf{C}\dot{u}(\mathrm{t}) + \mathbf{K}u(\mathrm{t}) = p(\mathrm{t}) \qquad (4.1)$$

Where:

$\boldsymbol{M}$	is the systems mass matrix
$\boldsymbol{C}$	is the systems damping matrix
$\boldsymbol{K}$	is the springer stiffness matrix
$p(t)$	is the variable load with regard to time
$u(t)$	is the displacement of the system with regard to time
$\dot{u}(t)$	is the velocity of the system with regard to time
$\ddot{u}(t)$	is the acceleration of the system with regard to time

A parameter that is essential in dynamic response calculations is the natural frequency which is determined by Equations 4.2 and 4.3.

$$f_n = \omega_n / 2\pi \tag{4.2}$$

$$\omega_n = \sqrt{k/m} \tag{4.3}$$

Where:

f_n	is the natural frequency
ω_n	is the un-damped circular natural frequency
k	is the structural stiffness
m	is the structural mass

The mass and stiffness of a structure can be obtained from a finite element analysis, where the structure is subdivided into a finite number of elements and the geometric and material properties are assembled into stiffness and mass matrixes. The main difficulty is to determine the damping matrix since many different aspects needs to be taken into account and is most often determined in an experimental way or approximated by damping ratios.

If the dynamic loads acting on the building result in a frequency identical to the natural frequency of the system resonance will occur, this may lead to uncontrolled vibrations and can theoretically in the worst case eventually cause collapse. If the system is un-damped the system, could continue to sway for infinity, although that scenario is rather unlikely to occur. Large excitations on a structure could lead to fatigue considering long-term effects.

From the four previously mentioned parameters which effect dynamic behaviour there are three different ways to reduce the sway of the structural system. The load intensity cannot be changed since it is an exterior force but the building design can be made in a more or less aerodynamically way and this effect how the load distribution is transferred to the ground.

By increasing the structural stiffness the natural frequency will increase. Although increased stiffness may lead to increased area of structural system and less architectonic freedom. Increased mass of the structure will decrease the natural frequency. To obtain difference in frequency the mass has to increase significantly which can be hard to obtain without effecting the primary layout and design.

4.2 Human response to wind-induced vibrations in buildings

Human response to wind-induced vibrations in buildings can be difficult to measure, since it dependent of both psychological and physiological aspects and the sensibility towards motion is individual. Even though one can measure thresholds for nausea and motion sickness by simulated vibrations and surveys it is problematic to find methods that give satisfying data when trying to find design criteria for comfort levels of occupants' living in buildings. It is the complex collaboration between the occupants' motion perception, the motion acceleration and frequency that give guidance to what criteria's that are needed in design (Melbourne, 1998).

The expression *"occupant comfort"* is quite vague and is highly a matter of interpretation; it can involve everything from actual noticeable vibrations affecting the everyday living activities to less noticeable vibrations that indirectly affect the occupants' wellbeing, and in the long-term effecting the overall attitude towards light-weight timber structures. It is not only the acceleration and frequency that affect how people react to vibrations; duration of vibrations and knowledge of the vibration source can affect the human tolerance threshold.

Simulated human response tests where frequencies of horizontal sinusoidal motion between 0.1-1.2 Hz for a constant acceleration measured both body and head acceleration and head displacement. The results show peak values at low frequencies, below 0.5 Hz due to inactivity of the para-spinal muscles which act as dampers for body acceleration (Burton et al., 2006).

Field studies have been made in the matter of trying to find human tolerance thresholds for discomfort associated with wind-induced vibrations and evaluate design criteria in serviceability limit state. Surveys with questions about comfort and disturbance focusing on slender high-rise buildings have been done in the past; one is of more interest where three airport control towers in Australia were studied (Denoon et al., 2000). Occupants in the three control towers were asked to press buttons at the start; end and during their shift declaring on a level from 1-5 the experience and discomfort of motion. Acceleration and frequency meters where placed to combine the results of with different wind speeds.

Results from the investigation show that although both control towers in Brisbane and Sydney airport were within the limits given in ISO 6897; concerning guidelines of evaluation of the response of occupants in fixed structures to low-frequency horizontal motion; the occupants in the Sydney tower experienced more disturbance. The metrological differences between the towers are that winds in the Sydney area tend to last during a longer period of time whilst the Brisbane area experiences gust-winds, giving reason to believe that human comfort threshold against wind-induced vibrations decrease with increased wind duration (Kwok, Hitchcock & Burton, 2009).

4.3 Design codes for occupant comfort in serviceability limit state

The most common way to assess wind-induced vibrations in buildings is by evaluating acceleration and frequency, since this is the easiest and most comparable way of describing vibrations based on human perception to motion. Two different ways of presenting acceleration is either by peak acceleration or normalized root-mean-square (RMS) values. The peak acceleration at different frequencies can be an indicator for human motion threshold, but when evaluating annoyance or disturbance from a vibration of a continuous nature RMS is a better indicator (Setareh, 2010). The peak acceleration is good when looking at vibrations induced by gust-winds, which will be of high magnitude and have relatively short duration, although they are noticed by occupants the tolerance may be higher towards them in contrast to vibrations of lower magnitude with longer duration.

The first full-scale study to evaluate human perception of wind-induced vibrations in tall buildings was made in the early 1970s by Hansen et al. where two buildings were analysed and it was based on two surveys and a tentative criterion was established. By combining occupant perception with the expected number of times per year a person would be willing to tolerate these motions and incorporate the number of complaints that a building owner would have to receive before taking action. The resulting criterion was set to a maximum RMS acceleration of 5 milli-g (0.05 m/s^2) with a return period of six years (Kwok, Hitchcock & Burton, 2009).

Based on this study further research was initiated in the area of perception criteria in wind codes/standards and led to new results by different researchers. Davenport utilized peak accelerations as opposed to RMS values, which became the accepted standard in North America. Irwin developed a concept of frequency-dependent vibrations based on sinusoidal motions, which was later adopted in ISO 6897, which is used in the majority of the world (Bashor, Kijewski-Correa & Kareem, 2005).

4.3.1 Regulations according to Eurocode

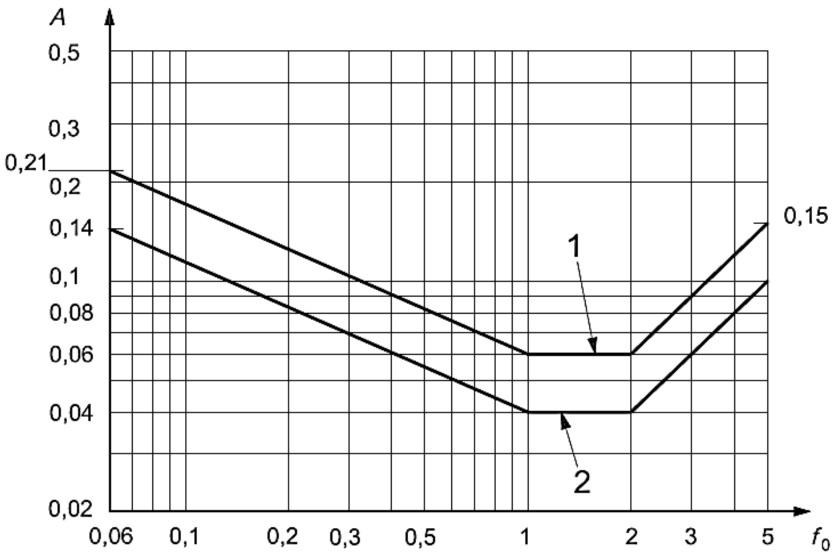
When designing a residential timber building for serviceability state with regard to vibrations according to Eurocode 5, general requirements state that it shall be ensured that actions on the structure does not cause unacceptable vibrations with regard to discomfort for occupants. When looking at the effects of wind-induced vibrations and horizontal acceleration, Eurocode 5 does not provide guidance or regulations other than mentioned above. A regulation for wind-loads in serviceability state only concerns limits of horizontal displacement. In a document written by Boverket, the Swedish national board of housing, building and planning, comments and applications of Eurocodes are presented. Concerning comfort criteria the regulations follows ISO 6897 where the wind velocity is calculated as a root mean square value with a return period of five years and presents criteria in the frequency range 0.063-1 Hz (Boverket, 2011).

4.3.2 Regulations according to ISO

For residential buildings regulations, ISO 10137 state that the accelerations should be kept within limits of "*daily living conditions with respect to human response to relatively ordinary motions of buildings, and horizontal accelerations of building with a one year-return period*" (International Organization for Standardization, 2008). In Chapter 7.2 the human threshold for vibrations is discussed and the complexity of the subject is clearly stated, acceptable vibrations are given in Annex C and D of the standard and may be adjusted to fit the circumstances. Annex C gives examples of comfort criteria for vibrations in general within the range 1-80 Hz and for vibrations below 1 Hz the designer is advised to use ISO 6897.

Guidance for human response to wind-induced horizontal motions in buildings is given in Annex D of ISO 10137 which is based on criteria from ISO 6897 and ISO 2631. Horizontal accelerations for winds with a return period of one year are applied; other return periods can be adopted if adjusted with multiplying factors. The criteria are based on peak accelerations for the first natural frequency in the principal structural direction and in torsion (International Organization for Standardization, 2008). In Figure 4.2 the critical values for residents lay between 1-2 Hz for accelerations 0.04 m/s^2, where the threshold is “independent” of a change in frequency.

The designer should use regulations in ISO 6897 for low-frequency horizontal motion, below 1 Hz. The criteria are based on root mean square values for accelerations with a return period of five years (International Organization for Standardization, 1984). Comfort regulations for occupants with average sensitivity to horizontal motion in an area where the environment is assumed to be relatively stable is given by curve 1 in Figure 4.3. Curve 1 represents residential buildings and curve 2 offices.



A peak acceleration, m/s²

f_0 first natural frequency in a structural direction of a building and in torsion, Hz

1 offices

2 residences

Figure 4.2 Evaluation curves for wind-induced vibrations in building in a horizontal (x, y) direction for a one-year return period in ISO 10137 (International Organization for Standardization, 2008).

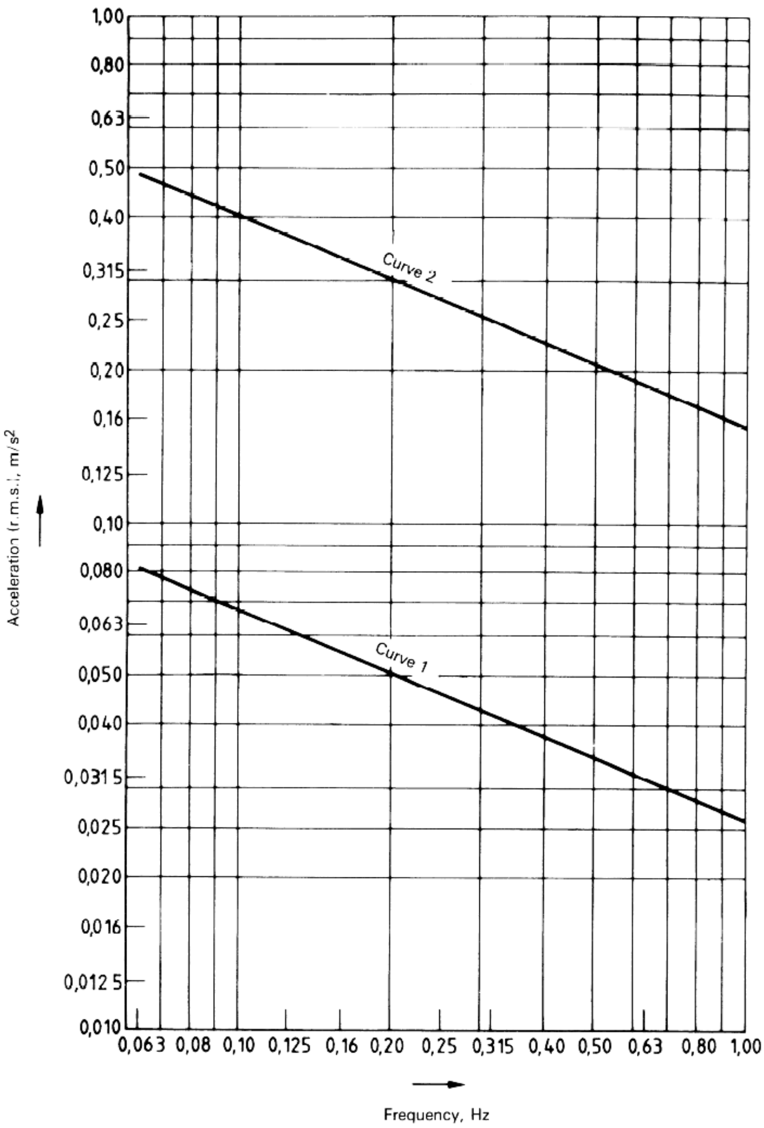


Figure 4.3 Suggested satisfactory magnitude of horizontal motion in buildings used for general purposes in ISO 6897 (International Organization for Standardization, 1984).

4.3.3 Regulations in Canada according to NBCC

The National Building Code of Canada, NBCC 2005 has no limiting criteria for wind-induced vibrations due to human perception. In the commentary of the code, the designer is advised to check vibration limits in serviceability limit state and implies that a design check should be made if one of the conditions below is fulfilled:

1. the building height is greater than four times its minimum effective width
2. the building height is greater than 120 meters
3. the building is light-weight
4. the building has low frequencies
5. the building has low damping properties

Since timber buildings are light-weight, wind-induced vibrations should always be checked in the serviceability limit state.

A review, given in the commentary of NBCC 2005, of design criteria according to North American and ISO standard it is stated that it should only be used as a guideline in design. The North American standard criterion is based on the peak acceleration with a ten year return period of an hourly averaging wind speed (Pozos-Estrada, Hong & Galsworthy, 2010). The criterion is independent of a variance in frequency and in range between 1.5%-3.5% of acceleration due to gravity, i.e. 0.14 m/s^2-0.25 m/s^2 (Hu, 2012). The lower value usually implies for residential buildings and it is stated in the code that the performance of buildings designed according to these regulations has been satisfactory.

4.3.4 Regulations in Japan according to AIJ

The building codes in Japan regarding wind-induced vibrations are given by the AIJ, Architectural Institute of Japan and the most recent building code regarding habitability evaluation is given by AIJ-GBV-2004. Peak acceleration for a 10 minute averaging wind speed with a one year return period should be used in evaluation (Pozos-Estrada, Hong & Galsworthy, 2010). The AIJ-GBV-2004 provides five different curves: H-10, H-30, H-50, H-70 and H-90 where the number of each curve indicate the perception probability of motion as a percentage for natural frequency in the range 0.1-5 Hz, see Figure 4.4. For example 50 % of the occupants can perceive vibration specified by the H-50 curve (Tamura et al., 2004).

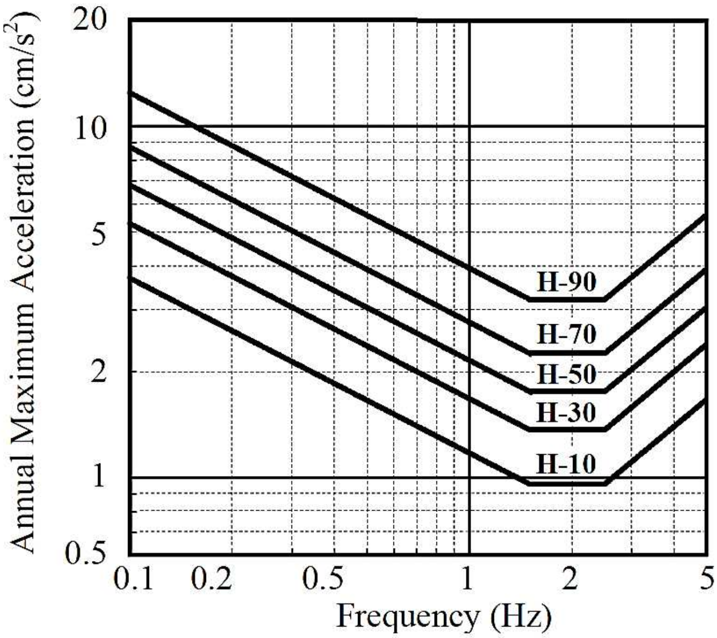


Figure 4.4 Probabilistic perception thresholds given in AIJ-GBV-2004 (Tamura et al., 2004).

No maximum allowed perception level for residential buildings are given in AIJ-GBV-2004 which can make it difficult to judge and select one of the curves as a design criterion; the basic concept of the code is to provide a guideline and it is up to the building owner to decide what probability perception level is acceptable (Tamura et al., 2004). Wind-induced responses on 286 buildings, constructed in different materials, were computed and compared with the perception levels given in the code. This was made as an attempt to give a better understanding of the guidelines for designers and building owners. The results from the investigation can be seen in Figure 4.5, where the different buildings are marked.

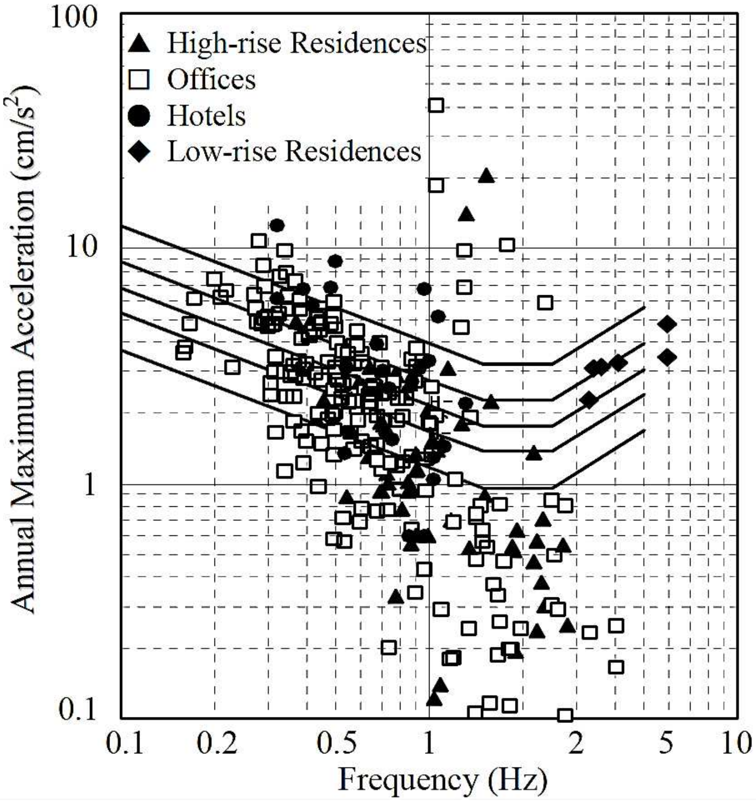


Figure 4.5 Existing buildings plotted in the perception thresholds given in AIJ-GBV-200 (Tamura et al., 2004)

4.3.5 Regulation according to British standard

Criterion in the British standard, BS6472-1:2008, differs from other regulations mentioned. Like the ISO-standards it initially defines the vibration in terms of acceleration, the difference is that it includes the durability and at what time a day the vibration occurs as a factor for human tolerance against vibration[1]. The magnitude of the vibration is presented as frequency-weighted acceleration which is used to calculate a vibration dose value (VDV). The acceleration is multiplied with a frequency modulus depending on the vibration direction, as given in Figures 4.6 and 4.7. The VDV is compared with acceptable values which are based on the probability of complaints from occupants; see Table 4.1 (British Standard Institution, 2008).

The idea behind VDV is that a wind with a short duration might not cause disturbance whilst a longer lasting wind is more likely to enhance occupant discomfort. Eurocode 1 is not easily adoptable when calculating wind-acceleration since the acceleration term in the expression for VDV is dependent on wind duration.

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Table 4.1 Vibration dose values, $VDV_{b/d,day/night}$ for residential buildings (British Standard Institution, 2008).

Vibration dose value range which might result in various Probabilities of adverse comment within residential buildings

Place and time	**Low probability of adverse comment** $m/s^{1.75}$	**Adverse comment possible** $m/s^{1.75}$	**Adverse comment probable** $m/s^{1.75}$
Residential buildings 16 h day	0.2 to 0.4	0.4 to 0.8	0.8 to 1.6
Residential buildings 8 h night	0.1 to 0.2	0.2 to 0.4	0.4 to 0.8

NOTE For offices and workshops, multiplying factors of 2 and 4 respectively should be applied to the above vibration dose value ranges for a 16 h day.

$$VDV_{b/d,day/night} = \left(\int_o^T a^4(t)dt\right)^{0.25} \quad (4.4)$$

Where:

$VDV_{b/d,day/night}$ is the vibration dose value in $m/s^{1.75}$

$a(t)$ is the frequency-weighted acceleration in m/s^2, using W_b or W_d as appropriate

T is the total period of the day or night during in s which vibration may occur

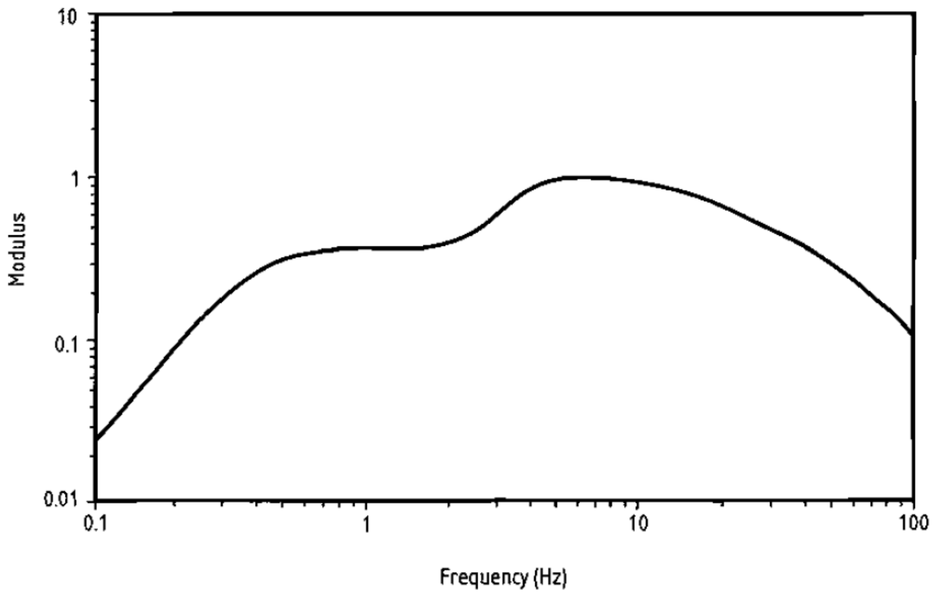


Figure 4.6 Frequency weighting curve (W_b) appropriate for vertical vibration (British Standard Institution, 2008)

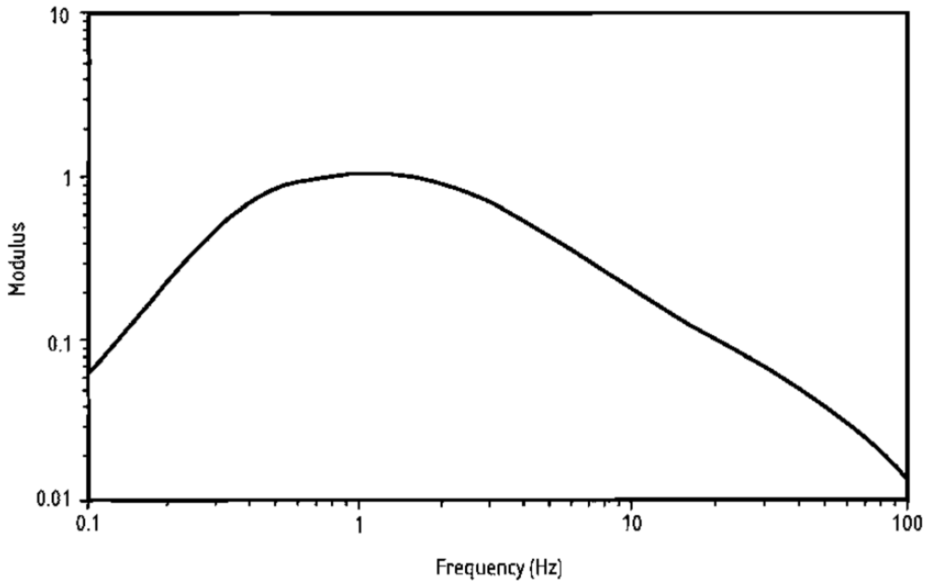


Figure 4.7 *Frequency weighting curve (W_d) appropriate for horizontal vibration (British Standard Institution, 2008)*

4.3.6 Comparison of standards and regulations

A comparison of Canadian, ISO 10137 and Japanese standards and regulations has been made by Pozos-Estrada, Hong & Galsworthy, 2010 and Figure 4.8 presents their relationship. Since the criteria compared evaluates wind speeds with different averaging times and return periods the values were scaled in order to be compared. The scaling was done such that the criteria comply with an hourly averaging wind with a 10-year return period, according to regulations given in the commentary of NBCC (Pozos-Estrada, Hong & Galsworthy, 2010).

It can be seen in Figure 4.8 that the limit of perception for residential buildings according to ISO 10137 corresponds to the H90 perception level for the AIJ code in the low-frequency range. The acceleration level is constant according to NBCC and is subdivided into different limits of perception for offices and residential buildings whilst AIJ and ISO are frequency dependent as previously stated.

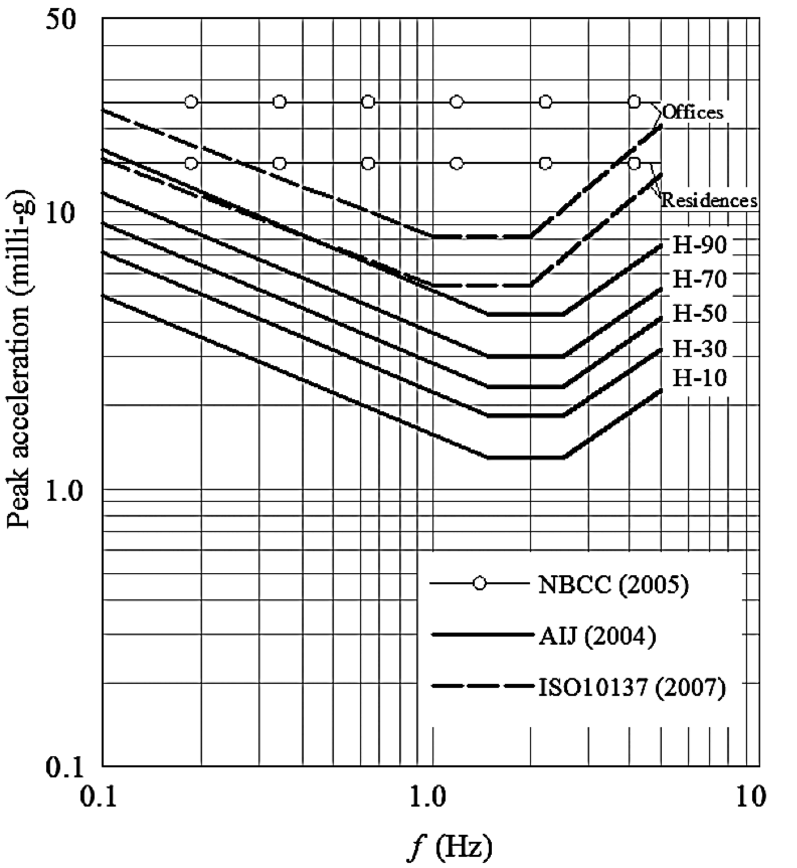


Figure 4.8 Comparison of limits of perception (Pozos-Estrada, Hong & Galsworthy, 2010).

5 Residential Buildings in Timber

Since 1994 when the regulations in Sweden changed and allowed construction of timber buildings with more than two storeys, the development of building systems from different producers with high prefabrication ratio has increased. Today construction of multi-storey timber buildings is becoming more common internationally, although building regulations differ between countries and delay the development and markets acceptance pace. The acceptance of new technologies and innovations takes time and there are various reasons why; the involvement of many different actors and companies in the building industry, the long life-time of a building which makes it challenging to evaluate the new innovations and a general resistance towards change in the construction industry (Goverse et al., 2001).

In a research conducted by Roos, Woxholm and McCluskey in 2010 structural engineers and architects attitude towards timber-framed houses was investigated. Reluctance to use timber was based on the uncertainty regarding fire safety and the materials durability, stability and sound transmission. It could be noticed that they wanted to use timber to a larger extent but due to lack of experience and cooperation with producers and the perception of timber-framed houses as complicated and difficult to work with often resulted in usage of more conventional materials such as concrete or steel (Roos, Woxholm & McCluskey, 2010).

This chapter presents information about buildings systems in CLT and more detailed on how the elements and connections are designed and the load-transfer between elements.

5.1 Building system in CLT

There are a wide range of methods of constructing multi-storey buildings in timber and the technology varies between countries. The most conventional structural system for multi-storey timber buildings is a beam and post system although it is becoming more common to use prefabricated planar elements. The different types of elements can be subdivided into three main categories; light frame, solid wood or composite timber elements. The focus of this report will be on a loadbearing structure made of solid wood elements, cross-laminated timber panels. CLT is an engineered wood product with high stiffness properties relative other engineered wood products. The building studied consists entirely of CLT elements for loadbearing walls, shear walls, flooring and stabilizing elevator shafts. Some of the advantages with CLT are strength and shear properties and it allows faster erection time due to the high prefabrication rate.

The highest existing multi-storey timber building in Sweden is Limnologen which is situated in Växjö and consists of eight storeys; see Figure 5.1 (b). The structural system is made of cross-laminated timber developed by Martinsons Byggsystem. Solid composite timber was used in both walls and joist floors and traditional stud construction in some of the apartment dividing walls.

Internationally the market for cross-laminated timber is prospering and in London a 9-storey residential building called Stadthaus was constructed in 2008. The cross-laminated solid timber panels form a cellular structure as can be seen in Figure 5.1 (a) where the loadbearing walls including all stairs and lift cores and slabs are made of cross-laminated timber. The first storey is made of concrete and the CLT-panels were prefabricated and were craned into position at the arrival of the building site which reduced the construction time; the entire structure was assembled in nine weeks.

(a) (b)

Figure 5.1 (a) Assembly of Stadthaus in London (Techniker, 2010) (b) Limnologen in Växjö during construction (Midroc, 2008)

5.1.1 Elements

The walls are constructed to transfer vertical loads as well as shear forces. The horizontal forces acting on the façade are subdivided into tributary areas and then distributed to the floor diaphragm. To distribute loads to the ground the shear walls take up the shear and racking force. The shear capacity is dependent on the material behaviour in both floor and walls and the connections between walls and floor (Crocetti, 2011). In comparison to more traditional sheeted timber framed shear walls the stiffness is rather high with regard to in-plane deformations and the shear stiffness is mainly dependent on connections.

In warmer countries the wall elements made of CLT can be applied without extra insulation, but in Nordic countries the colder climate demands alternative solutions. The external walls are completed with insulating material, moist- and convection-barriers and façade material to obtain an adequate climate shell. CLT-producers have different solutions but a general solution made by KLH is presented in Figure 5.4.

Floor elements can either be made as plane solid wood panels, cassettes stiffened with under-laying I-beams or solid wood panels completed with a concrete cover (Massivträhandboken, 2006). The design of floor elements is from a structural point of view often governed by serviceability requirements regarding vibrations and deflection. By using CLT plane elements the influence of moisture related motions are small and shear stiffness is high. The deflection should not exceed L/300 and the frequency should be over 8 Hz in order to avoid human discomfort according to EC 5. To reduce the acoustic transmission either a non-structural hanging ceiling or isolating flooring can be added as presented in Figure 5.5.

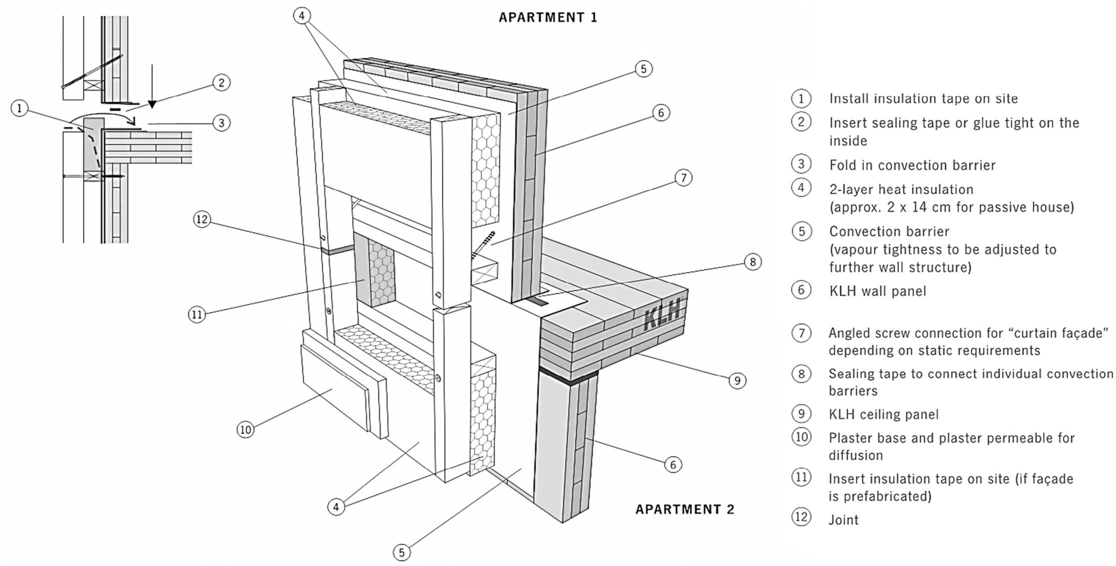


Figure 5.4 Assembly of external prefabricated wall element (KLH, 2011)

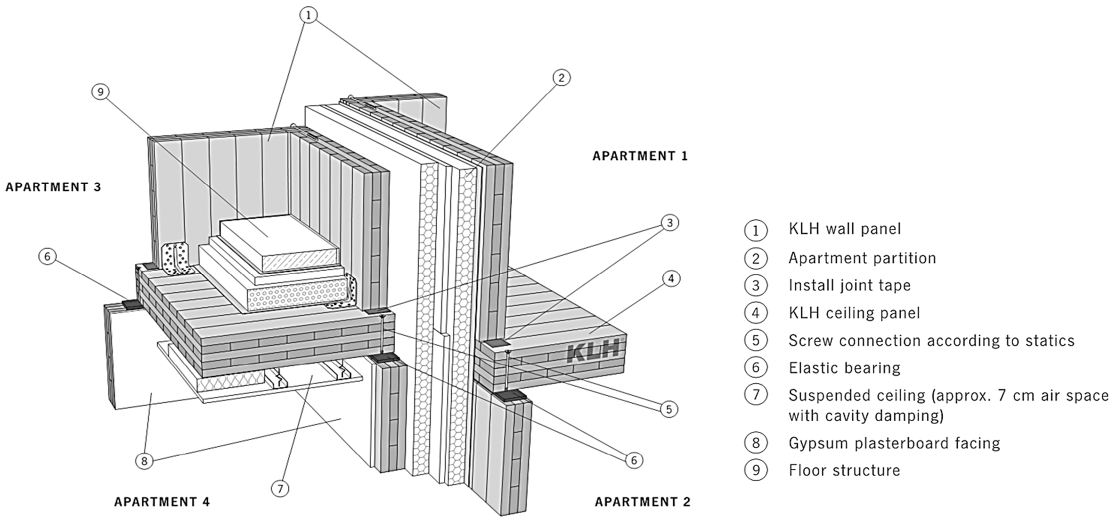


Figure 5.5 Partition between apartments (KLH, 2011)

5.1.2 Connections

Connections in a building system influences the structural behaviour and need to be done in a manner so that the connection is air and dust-tight, which is accomplished by using rubber profiles, sheeting and sealing bands to seal the connections. The size of elements is restricted by limitations in the production process or by the transport, and therefore the panels need to be connected at site (Augustin, 2008).

The seams between two joist floor panels are loaded both in longitudinal and perpendicular direction. It can be executed either by a half-lapped connection merging the layers together either by gluing or with screws or by a double surface spline, see Figure 5.6. The connection with a screwed half-lap joint can carry normal and transversal loads but is not able to transfer moment. The joint with double-sided surface spline consist of a spline with an engineered wood product, the connection can transfer moment (Augustin, 2008).

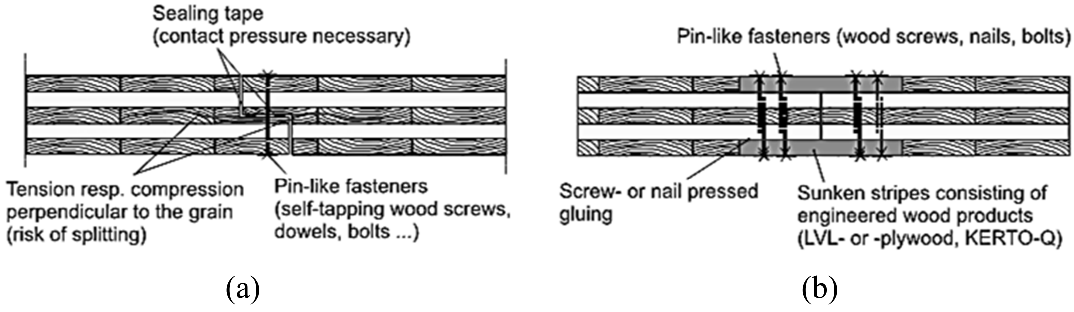


Figure 5.6 Connection of parallel panels (a) screwed half-lapped (b) double-sided surface spline (Augustin, 2008)

Connections between wall and floor can be made with steel angle brackets, joints with self-tapping wood screws and glued-in rods, see Figure 5.7. This connection can transfer the horizontal loads from the walls down to the floor and can resist lifting forces from the wall element (Augustin, 2008).

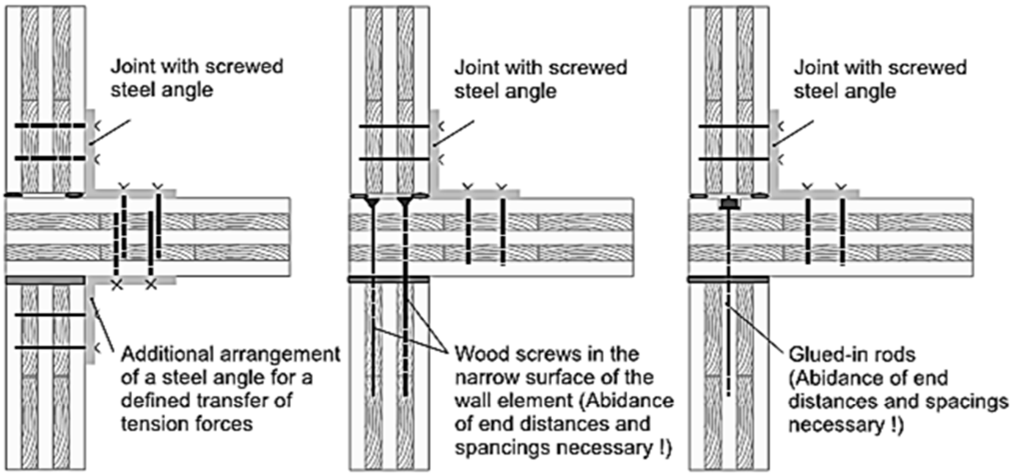


Figure 5.7 Connection between walls and floor with steel angle brackets (Augustin, 2008)

Connections between walls and foundation can be jointed by steel plates or steel angle brackets and screws. In order to preserve the wood and to adjust the height edge veneers or wooden profiles made of hardwood or engineered wood products can be used, as can be seen in Figure 5.8.

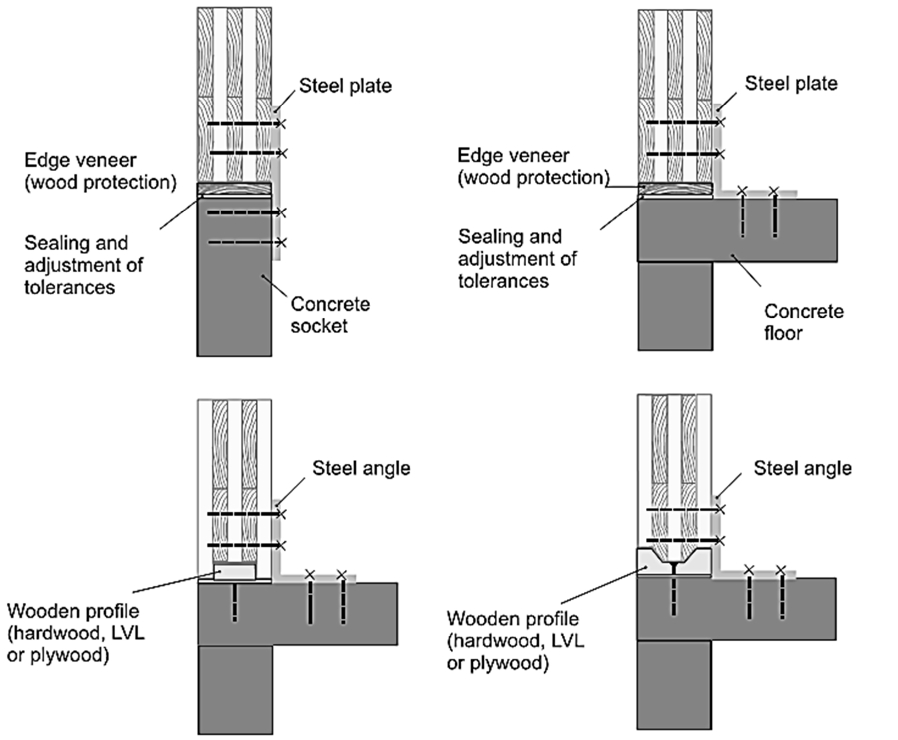


Figure 5.8 Connection between wall and foundation (Augustin, 2008)

CLT panels are considered very rigid in comparison to the connections and a majority of the flexural deflections originate from the connections since the shear and flexural deflection in a panel is assumed to be very small (Gavric, Ceccotti & Fragiacomo, 2011).

5.1.3 Load transfer

The lateral forces cause horizontal, twisting and torsional deformations. Wind forces acting on the side of the building will be picked up of the surface members and transferred to secondary frame elements (Crocetti, 2011).

External wall elements needs not only be designed for vertical load but also for transferring and resisting horizontal loads. In the case of solid timber panels each wall consists of a cross-laminated timber component which transfer load by shear action and are considered ridged. It is not only the vertical elements in a structure that needs to withstand lateral forces; the horizontal elements are also influenced.

The general model used for a one-storey timber building loaded by lateral forces can be seen in Figure 5.9. The lateral load, in this case wind, is transferred by the building face to the shear walls and then transferred by shear action to the foundation. This model can be used for multi-storey buildings, with the adjustment that the shear forces at the bottom rail are transferred to the underlying floor instead of directly to the foundation (Vessby, 2008). The floor diaphragms are in this model seen as elastic; in simplified plastic models they are seen as infinitely stiff. Horizontal elements can be seen as bracing members and create higher structural stiffness and may be designed to transfers lateral load to side vertical elements.

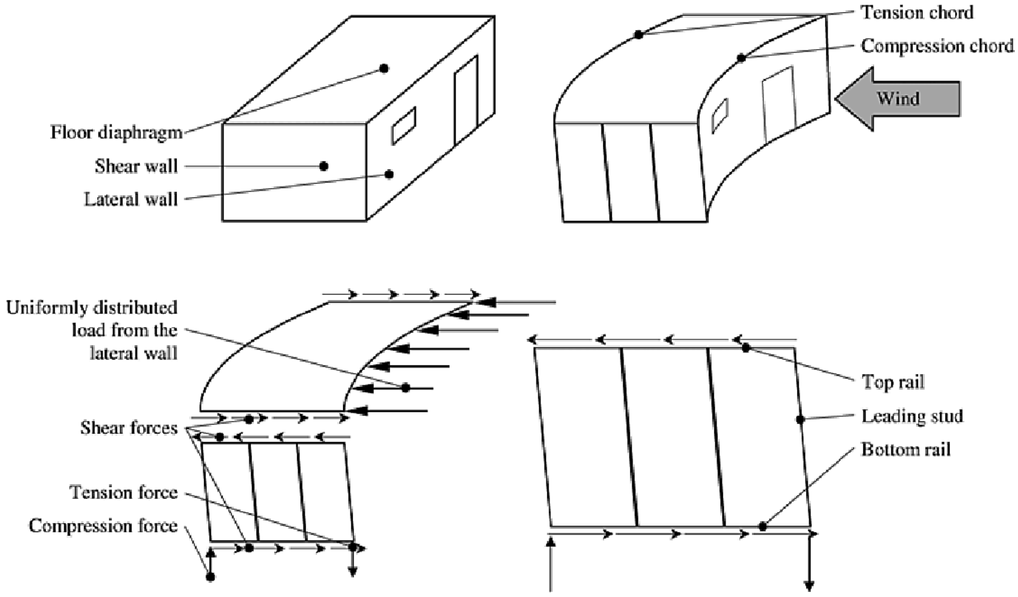


Figure 5.9 Model for load transfer in a one storey high timber building (Vessby, 2008)

6 Cross-laminated Timber

The first initiative to develop a building material of solid timber, cross-laminated in several layers was introduced in Zurich and Lausanne, Switzerland in the early 1990s. In 1996 Austria undertook an industry-academia research effort that resulted in the development of modern CLT (Crespell & Gagnon, 2010).

Cross-laminated timber panels are made up by cross wise layering of timber planks where the grain direction of the individual layers are placed orthogonal one another, see Figure 6.1. The planks are most commonly made with C16-C24 classified timber and are kiln dried to a moisture content of 12% ± 2%, preventing dimensional variations and surface cracking.

The panels are lengthwise merged by finger-jointing to obtain desired length; the sizes of the panels vary depending on the manufacturer and transportation regulations. Interior or exterior polyurethane adhesives are normally used to glue the layers of planks together and are pressed together in the vertical and horizontal direction by hydraulic presses or in some cases vacuum or compressed air presses depending on the thickness of the layers (Crespell & Gagnon, 2010). The number of layers vary from three and up with odd numbers and the outer most layers are made such that the grain direction runs parallel to the main loading direction, further on referred to as main grain direction.

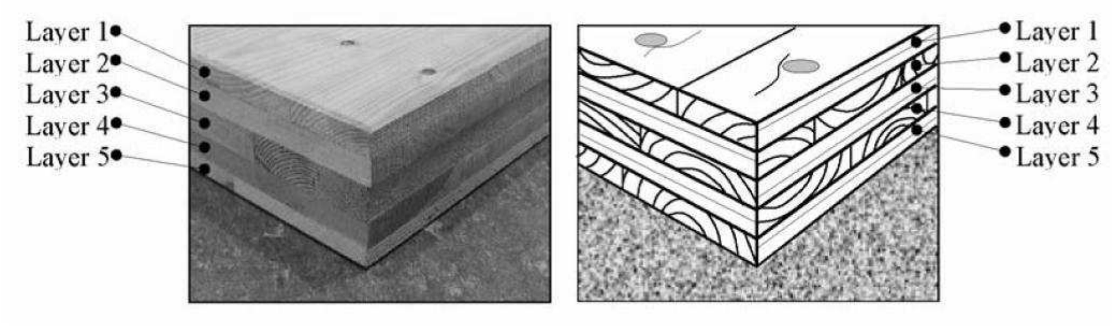


Figure 6.1 A five-layered cross-laminated timber panel (Vessby et al., 2010)

CLT producers have their own standards when it comes to thickness of the layers and effective values for material properties of the composite panels. Since it is a relatively new product there is no standardized method for determining strength and stiffness properties of CLT panels. Mechanical properties are provided by each manufacturer and the approval process is divided into several steps where the ETA (European technical approval) allows manufacturers to place CE- marking on the CLT-product.

Due to the anisotropy of CLT panels, their stiffness properties are different depending on grain direction, setup of the layers and if the panel is subjected to loading perpendicular to plane or loading in plane. The governing stresses causing deformation are also dependent on main grain direction and loading conditions. The properties of the CLT panels can be seen as a combination of plywood, glulam and solid timber, where not only the layers properties in different directions are taken into account but the lamination effect as well.

6.1 Strength and stiffness tests

In a test conducted at the Linnaeus University in Växjö CLT specimens were tested. The specimen consisted of Norway spruce (*Picea abies*) with five layers, a total thickness of 96 mm and main grain direction perpendicular to load. Four different specimens were tested and the load was applied according to the test setup given in Figure 6.2.

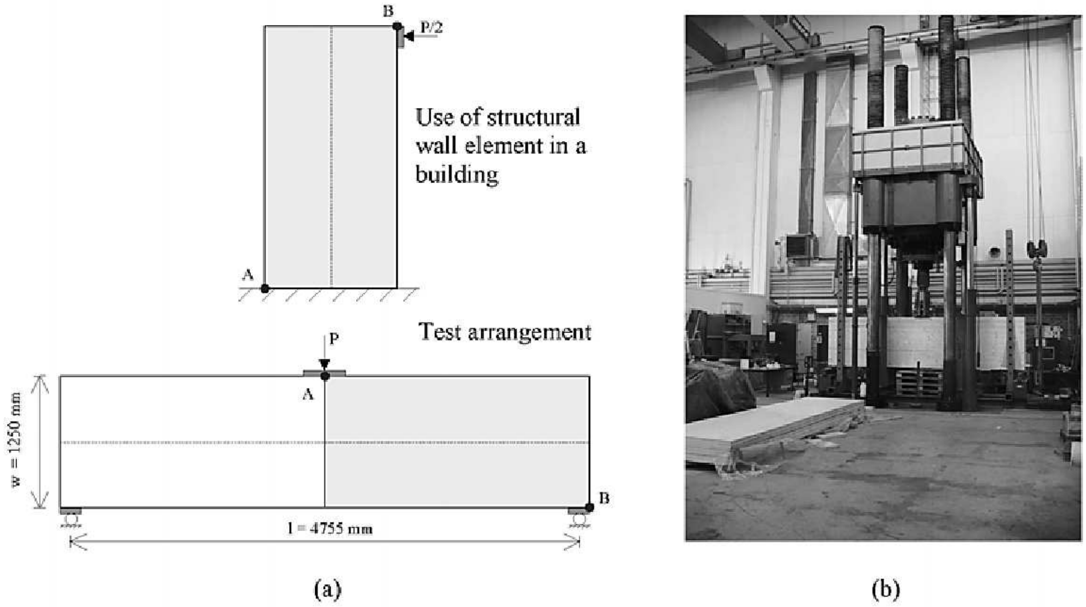


Figure 6.2 (a) schematic plan of testing setup, (b) photograph of the testing machine (The dotted line indicate a longitudinal joint such as for a beam element with joints) (Vessby et al., 2010)

To calculate the mid-deflection material properties of the panels were assumed. The longitudinal stiffness was set to 12 000 MPa and the transversal to 400 MPa, which used in order to calculate an effective modulus of elasticity of E= 7360 MPa. The shear modulus of the CLT panel was assumed to be 750 MPa, and calculated value of mid-deflection due to a concentrated load of 200 kN became 7.1mm. In the experimental tests of the specimens average mid-deflections were 6.1mm. The comparison between test results and calculated value shows that the assumed longitudinal E-modulus was higher than 12000 MPa (Vessby et al., 2010), CLT elements are in fact stiffer in reality than estimated by calculations.

6.2 Influence of shear

In panels loaded perpendicular to the plane, shear stresses develop in the cross-section between layers and are strained by rolling shear, see Figure 6.3. Due to the low rolling shear stiffness of timber the global shear deformation is mainly caused by deformations in the local cross-layers and the deformation will increase with increasing thickness of the rolling shear layer. Rolling shear modulus for timber is dependent on the density of timber and the orientation of the annular rings. It may therefore be hard to distinguish representative values of shear modulus and advanced techniques have been developed determined from measured frequencies of bending members. Rolling shear modulus varies between 40 and 80 MPa and it has been verified with test results that values may be taken as $G_{R,mean} \approx G_{mean}/10$ where the shear moduli is $G_{mean} \approx E_{parallel}/16$ (Fellmoser & Blass, 2004).

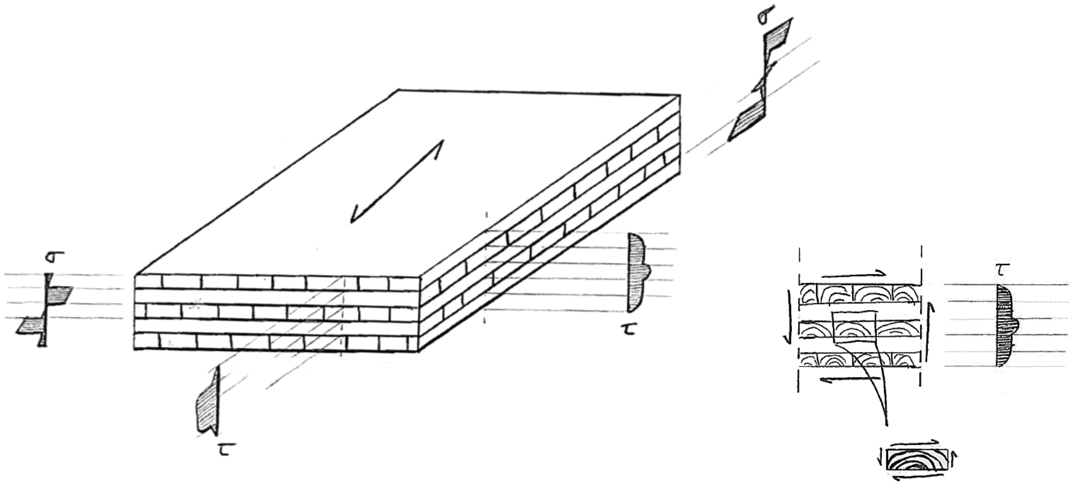


Figure 6.3 Resulting bending, shear and rolling shear stresses of a panel loaded perpendicular to plane, parallel or perpendicular to main grain direction.

Studies have showed that for an element with high span to depth ratio the shear does not influence deformation significantly, with increasing span length the shear influence decreases and bending will dominate in deformation. The shear deformation may therefore be neglected under following circumstances, where L is span length and d is total thickness of the panel (Blass & Fellmoser, 2004):

- $L/d \geq 30$ when loading perpendicular to plane and parallel to main grain direction (direction of the outer most layers in the panel)
- $L/d \geq 20$ when loading perpendicular to the plane and perpendicular to main grain direction (direction of the outer most layers in the panel)

For panels loaded in plane, shear stresses develop in the panels' cross-section, due to internal vertical and external horizontal loads on the building. Deformation is caused by slippage between layers due to loss of strength in adhesives. A difference is made between edge bonded element and non-edge bonded elements, for edge bonded panels the assumption is made that no slip between layers occur and for non-edge bonded elements the shear flow will be at the face of layers with perpendicular grain direction, see Figure 6.4 (Schickhofer & Theil, 2011). Shear stresses may also occur due to torsional moment in the glued interface between layers. A schematic picture of how the stresses act in a CLT panel is shown in Figure 6.4.

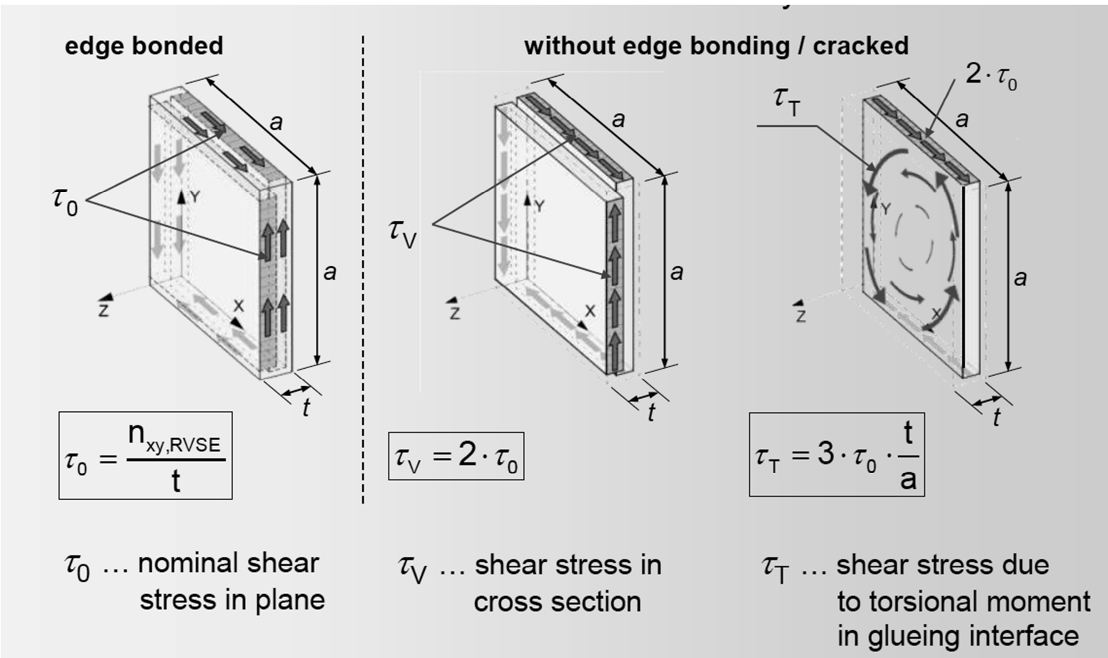


Figure 6.4 Shear stresses in edge- and non-edge-bonded wall panels (Schickhofer & Theil, 2011).

6.3 Design methods for stiffness properties

There is currently no accepted design method for determining strength and stiffness properties of a CLT panel, therefore a combination of different methods are used. CLT elements have different properties in different directions; this is taken into account by the composite theory. Shear deformation in bending is disregarded when using the composite theory and can therefore only be used for elements with high span to depth ratios; where shear influence may be neglected. When determining the influence of shear deformation, the theory of mechanically jointed beams may be used (Blass & Fellmoser, 2004).

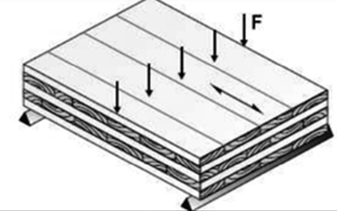
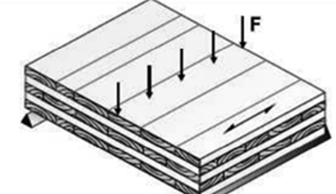
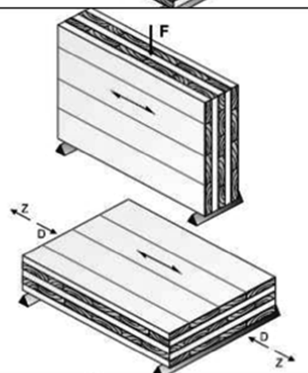
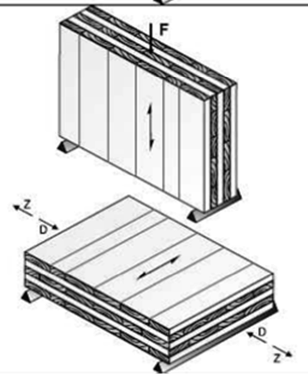
For roof and floor elements, composite theory and theory for jointed beam are to be used. For wall elements, panels mainly loaded in plane, theory for jointed beam and wall elements seen as beams and lintels a simplified or composition factor method may be used in design (Gagnon, 2011).

6.3.1 Composite theory

According to the composite theory, the individual layer's properties should be taken into account and the individual layer's properties depend on the orientation of the grain and the layer thicknesses.

One way of calculating a cross-section bending stiffness may be by using the method used for plywood, where the total bending stiffness of a cross section is taken as the sum of the contribution from the individual layers. Layers with grain direction parallel to load are assumed to have no contribution of bending stiffness since the modulus of elasticity perpendicular grain is so much smaller than for parallel grain. This method will give an effective modulus of elasticity for the composite material determined by the number of layers in each load direction, layer thicknesses and loading in plane or perpendicular to plane. Studies have shown contradictions when calculating stiffness properties using the "plywood method" and testing of cross-layered specimens, this might have to do with the fact that layers running perpendicular grain directions are neglected (Blass & Fellmoser, 2004).

In German design codes composition factors are used to determine the panel's effective properties and it is governed by the number of layers, layer thicknesses and load direction, see Figure 6.5. This method is based on the same composite theory used for plywood but takes the transverse layers into account as well and the stiffness perpendicular grain is taken as $E_{90} = E_0/30$ (Blass & Fellmoser, 2004).

Load configuration	k_i
	$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_2 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \ldots \pm a_1^3}{a_m^3}$
	$k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$
	$k_4 = \frac{E_{90}}{E_0} + \left(1 - \frac{E_{90}}{E_0}\right) \cdot \frac{a_{m-2} - a_{m-4} + \ldots \pm a_1}{a_m}$

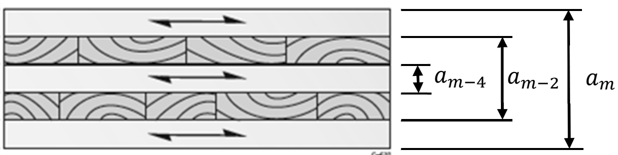


Figure 6.5 Composition factors k_i for solid wood panels with cross layers (Gagnon, 2011).

The modulus of elasticity parallel grain is multiplied with the composite factors and the effective stiffness values describe the stress distribution in the composite panel according to Equations 6.1 and 6.2. These factors may be used to correlate stiffness properties as well as strength properties.

$$E_{eff} = E_0 \cdot k_i \tag{6.1}$$

$$(EI)_{eff} = E_{eff} \cdot \frac{b \cdot h_{tot}^3}{12} \tag{6.2}$$

Where:

E_0	is the modulus of elasticity parallel grain
k_i	is the composition factor
b	is the width of the panel
h_{tot}	is the total height/thickness of the panel

When using material properties for C24, the lamination effect is disregarded and characteristic values for glulam GL28c may give a more accurate value according to Blass and Fellmoser at the University of Karlsruhe (Blass & Fellmoser, 2004).

6.3.2 Theory for jointed beam (Gamma method)

The theory for jointed beam is based on Annex B in EC 5, which is a design tool for elements jointed with mechanical fasteners. According to Eurocode 5, this method is only valid for simply-supported beam with a sinusoidal load distribution, which in the case of CLT complies for small span to depth ratios (Blass & Fellmoser, 2004).

To be able to account for the rolling shear deformation, imaginary fasteners are used, the longitudinal cross layers are assumed to be beams jointed with fasteners having stiffness equal to that of the material's rolling shear.

When calculating the bending stiffness using this method, layers in transverse direction are neglected in the same manner as for plywood (Gagnon, 2011). The contribution from the imaginary fasteners is given by the reduction factor, γ, in the bending stiffness equation according to Equation 6.3. The reduction factor varies between 0 and 1 and is taken as 1 for ridged connections and 0 for no interaction between members, see Figure 6.6. The factor can be seen as a connection efficiency factor of the imaginary connections and are usually in the range 0.85-0.99.

$$(EI)_{eff} = \sum_{i=1}^{n} (E_i I_i + \gamma E_i A_i a_i^2) \tag{6.3}$$

Where:

n	is the number of layers
γ	is the reduction factor for imaginary fasters
E_i	is the modulus of elasticity of the individual layers
I_i	is the moment of inertia of the individual layers
A_i	is area of the individual layers
a_i	is the distance between local and global centre of gravity

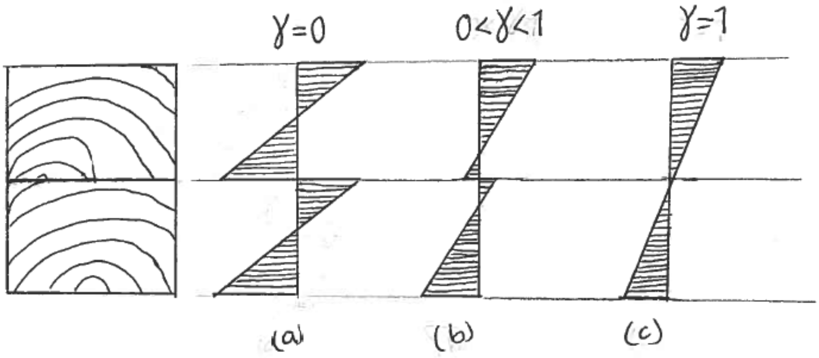


Figure 6.6 Variation of connection efficiency factors between no interaction to full interaction for mechanically jointed members (a) no interaction, (b) partial interaction (c) complete interaction (Gagnon, 2011).

6.3.3 Shear correction factor

The shear correction factor was introduced by Igor Popov in 1990 and represents the modification of the shear deflection depending on the shape of the cross-section. (Popov, 1990) The shear correction factor relates to the equivalent shear angle and stress to the actual ones that vary over the cross-section, by differentiating the normal stress, the nominal shear stress along the whole width of the cross-section at the z-level. The shear correction factor can then be determined by equating energy of the equivalent shear angle/stress model with the energy of the corresponding real situation as can be seen in Equation 6.4, given by Tlustochowicz, 2011.

$$\beta = \frac{(GA)_{lam}}{(EI)_{lam}^2} \int_0^B \frac{[(ES)(z)]^2}{G_{lam} b_c(z)}\, dz \tag{6.4}$$

Where:

β	is the shear correction factor
$(GA)_{lam}$	is the effective shear modulus of the cross-section
$(EI)_{lam}$	is the effective bending stiffness of the cross-section
$(ES)(z)$	is the first moment of axial stiffness at level z of cross-section
b_c	is the thickness of the cross-section/composites
B	is the height of the composite panel

6.4 Comparison of design methods

As has been mentioned before there are a number different design methods and assumptions when determining stiffness properties of CLT panels and in order to see how they vary a comparison between them is given in this chapter. Bending stiffness of a 1.2 x 3m panel is considered under loading perpendicular to plane and in plane where the main grain direction of the panel is set according to Figure 6.7. The panels are 95 mm thick with five layers and material properties for C24 timber and GL28c glulam are set in accordance with EC, see Table 6.1.

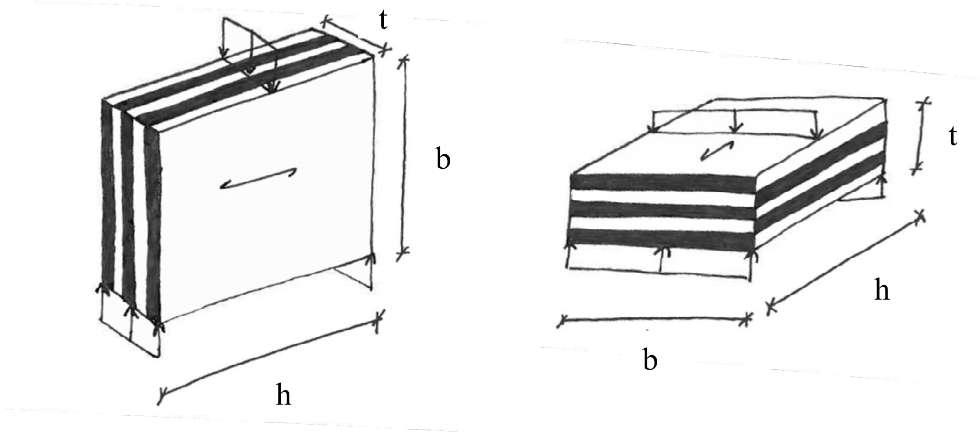


Figure 6.7 CLT panel, loaded perpendicular to plane and in plane

The variations in stiffness properties for the CLT panels calculated with the different theories are given in Tables 6.2 and 6.3. The stiffness calculations according to plywood and composition factors method give quite similar results, this shows that the influence of layer having grain direction parallel loading direction is small. For gamma method a slightly lower stiffness value is obtained.

Table 6.1 Material property data according to Eurocode 5

Material	E_0 [MPa]	E_{90} [MPa]
C24	11000	370
GL28c	12000	370

Table 6.2 Bending stiffness when loading perpendicular to plane

Method	I [m^4]	EI_{C24} [MNm^2]	EI_{glulam} [MNm^2]
Plywood	$\frac{3 \cdot t_{layer} b^3}{12} = 0.0082$	$E_0 I = 90.29$	$E_0 I = 103.42$
Composition factors	$\frac{tb^3}{12} = 0.014$	$k_3 E_0 I = 92.31$	$k_3 E_0 I = 105.45$

Table 6.3 Bending stiffness when loading in plane

Method	I [m^4]	EI_{C24} [MNm^2]	EI_{glulam} [MNm^2]
Plywood	$\frac{3 \cdot b t_{layer}{}^3}{12} + 2 b t_{layer} (2 t_{layer})^2 = 0.000039$	$E_0 I = 0.747$	$E_0 I = 0.856$
Composition factors	$\frac{bt^3}{12} = 0.214$	$k_1 E_0 I = 0.754$	$k_1 E_0 I = 0.862$
Gamma	$\frac{3 \cdot b t_{layer}{}^3}{12} + 2 \cdot \gamma b t_{layer} (2 t_{layer})^2 = 0.000036$	$E_0 I = 0.689$	$E_0 I = 0.789$

7 Description of Model

The computer program “FEM design 3D structure” was used for analysis of the chosen timber building. “FEM design 3D structure” is developed by Structural Design Software in Europe AB (Strusoft) and is an advanced modelling software program for FE-analysis and design of load-bearing structures in concrete, steel and timber according to Eurocode including national Annexes. In order to obtain lateral deflection and natural frequency that occurs due to wind forces a static and a dynamic analysis was performed.

7.1 Dimensions

The design of the structure was done with focus to create a structure as slender as possible in order to get significant influence from wind loading. The structure had to fulfil requirements stated in the plan sketch of Skeppsbron, described in Chapter 2. The height of each floor was set to 3 meters which gives a total building height of 24 meters; the length 40 meters and the width 16 meters; see Figure 7.1. The internal wall distribution was made symmetrical and can be seen as an assembling of two 10x16 m buildings, where an elevator shaft was placed in the centre of each. The walls around the elevator shaft were modelled as CLT elements and were included in the model. All internal walls are load-bearing and were placed in order to fulfil maximum span lengths of floors in accordance with Martinsons CLT handbook (Martinsons, 2006). The ground plate was made 0.5 m thick with C40/50 concrete.

The loadbearing structure was made of CLT panels; CLT 95 was used for vertical elements and CLT 145 for horizontal elements. The stiff direction of elements was; for wall panels set by default in the vertical direction and for floor panels it is defined by the designer. The stiff direction of floors was set parallel global x-direction, i.e. parallel wind loading direction to obtain maximum stiffness. Thicknesses, widths and span lengths were set in accordance with proposed design values in Martinsons CLT handbook (Martinsons, 2006) whereas all floor spans were set to 4m. To reduce the number of degrees of freedom in the model the assumption was made that jointing of panels during manufacturing can be seen as satisfactory and elements consisting of many panels may be seen as one continuous panel. The panels were therefore modelled as 2m wide, walls 3m high and floor spans 4m.

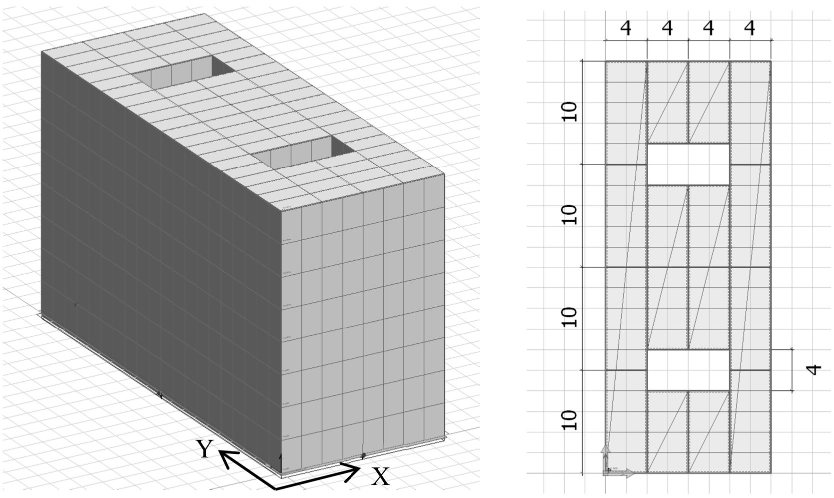


Figure 7.1 FEM design 3D structure model

Reinforcement design in the ground plate was done using a function in ”FEM design 3D structure” called surface reinforcement auto-design. The program calculates the most suitable top and bottom reinforcement distribution and dimension with regard to crack width, shrinkage and load action for the chosen concrete strength class and minimum concrete cover. Required dimensions and spacing for bottom and top reinforcement can be seen in Figure 7.2 and the amount of surface reinforcement can be seen in Figure 7.3.

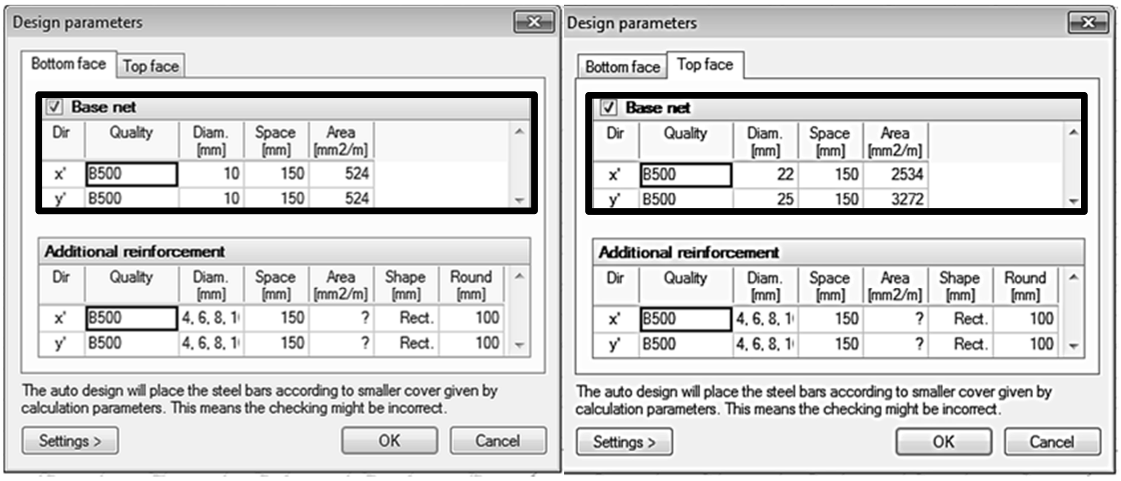


Figure 7.2 Required bottom and top reinforcement

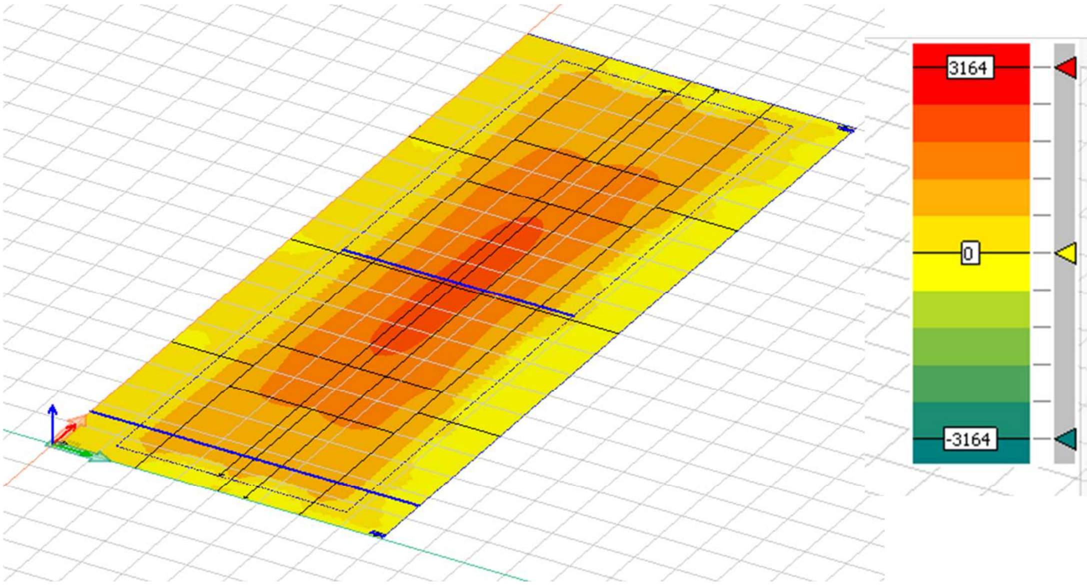


Figure 7.3 Amount of reinforcement given as area reinforcement per meter slab, [mm²/m]

The ground plate was set as fixed with semi-ridged group line supports along all sides, motions associated with ground condition was therefore neglected. The connections rigidity is given by springer stiffness in the model see Figure 7.4. For rigid connection motion and rotation is allowed in all direction and springer stiffness values are pre-set in x-, y- and z-direction as 10 GN/m/m for motion and 174.5 MNm/m/° for rotation. Connections between panels and between elements borders were set as hinged. For hinged connections, motion in all directions is allowed whilst rotation is not. Motion was pre-set as 10 GN/m/m in x-, y- and z-direction.

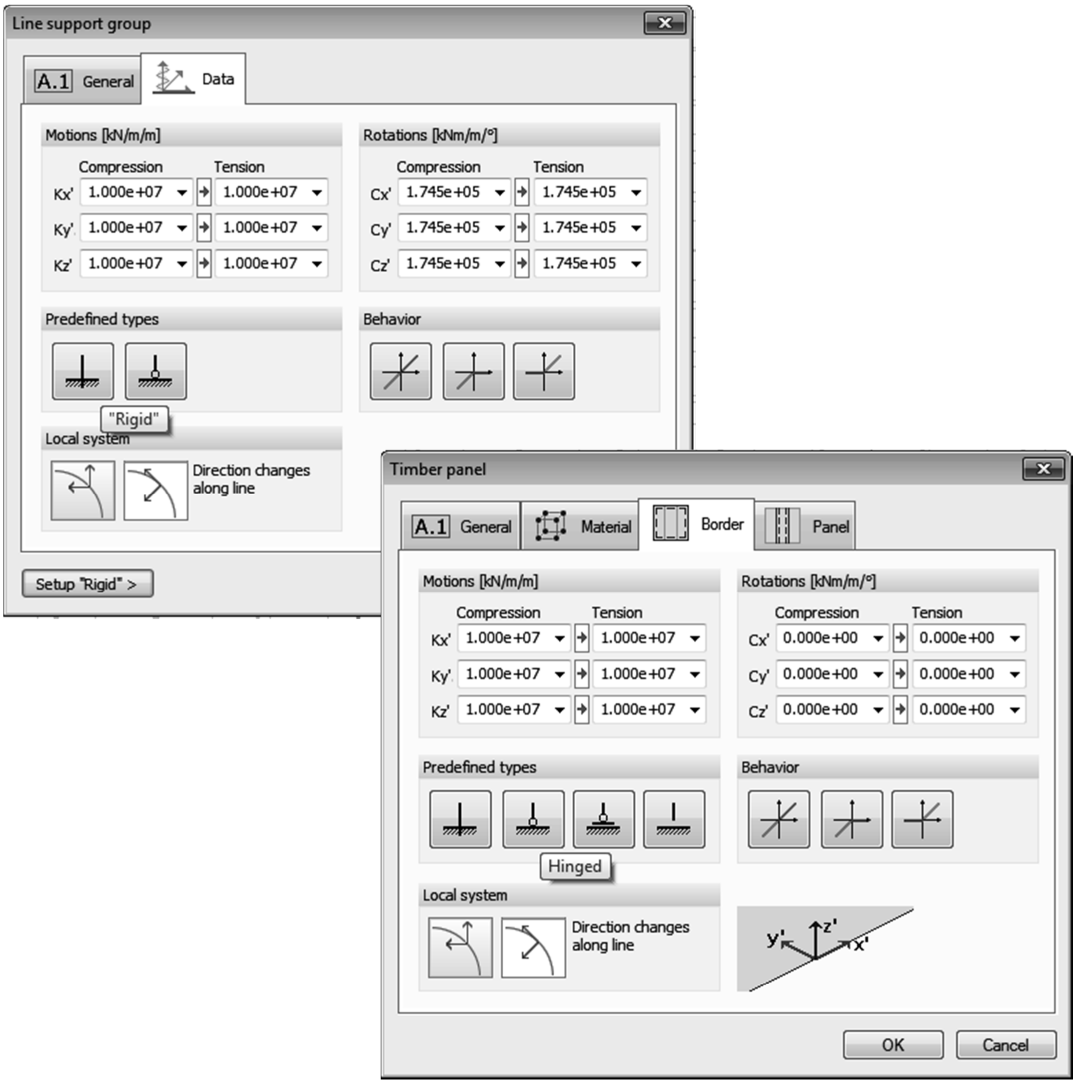


Figure 7.4 Connection rigidity set up in FEM design 3D structure for a rigid line support group and hinged timber panel borders.

7.2 Loads

The vertical loads acting on the structure consisted of self-weight, which was set as permanent and a 2kN/m² uniformly distributed imposed load from residents with medium-term load duration. The self-weight of the structure was generated by FEM-design, distributed over the structure and the total weight of building became 1386 ton including all structural elements. The wind load on the building's long-side/face was applied as a uniformly distributed load with short-term load duration. Horizontal displacements of the model under loading are presented in Chapter 8.

As was established from the CFD model for Skeppsbron in Chapter 2.2, wind will hit the building at the southwest corner whereas the wind load was modelled acting in the positive x- and y-direction for the two walls nearest the quay. According to the CFD model there will be some amplification hitting the southwest corner of the building, although close to 1 and will therefore be neglected in further analysis.

In ”FEM design 3D structure” there are two ways of modelling wind load, one where a uniformly-distributed load is placed at each floor and the other where it is placed as a cover acting on the faces of the building as pressure or suction. Two analyses were made and when using the first method the wind load acts as line loads on each floor, in positive x- and y-direction. Values for wind load for each storey are based on a reference wind speed, set as 25 m/s in accordance with the national annex of Eurocode 1:1-4, general actions- wind actions. The tributary area was then automatically calculated by the program and the wind was distributed to each floor as uniformly distributed line loads, see Figure 7.5.

Another way of modelling is by assigning covers for wind load and the load will then be recalculated by the program as surface loads corresponding to pressure and suction acting on the building faces, se Figure 7.6. ”FEM design 3D structure” generates 4 different load combinations for the different loading directions and the worst case is then chosen by the designer and used in the final load combination in ULS and SLS. Load case 1 for X and Y direction was in this case worst and therefore used in further analysis.

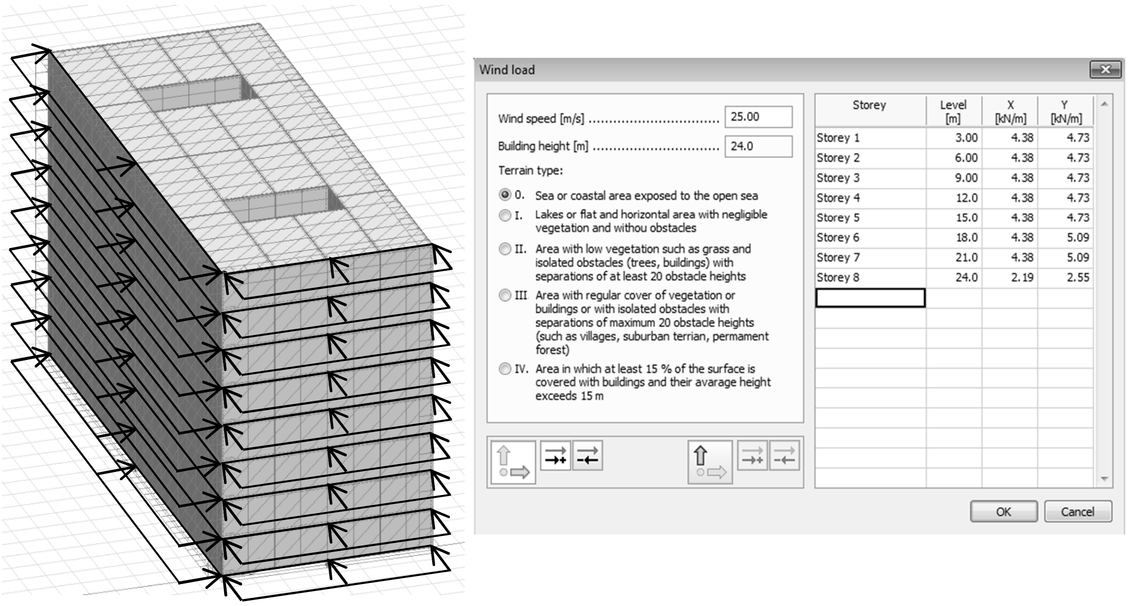


Figure 7.5 Wind load acting in positive x- and y-direction as line loads on each floor

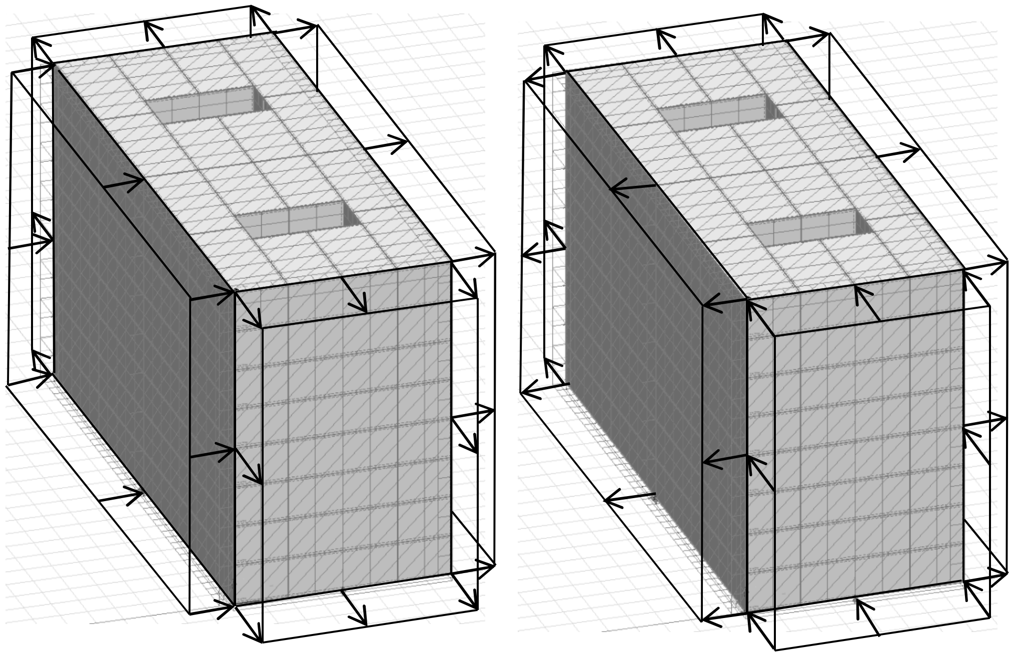


Figure 7.6 Wind load acting as surface loads on building faces in positive x- and y-direction

7.3 Mesh

Meshing in ”FEM design 3D structure” is generated automatically, where the program calculates the most suitable element sizes in order to get good results without making the element sizes larger than needed. In connections between two elements the program takes into account the needed element sizes for each element and generates the best suitable transition. The same is done for mesh under loads, where finer mesh is suitable. The average size of elements in the model became 0.46m.

7.4 CLT in *FEM design 3D structure*

The program has a pre-setting for timber design, where the designer is free to choose between predefined materials or define new materials by changing its properties. The predefined CLT timber panels may function as wall or floor elements and takes into account the orthotropic properties of timber. The designer can either choose to use the predefined material CLT or make a shell element with the same properties as what would be expected of CLT.

According to Fredrik Lagerström[2], the program treats the panels as a solid homogeneous material with the orthotropic properties of CLT. Although neither the number of layers or layer thicknesses can be given as input data the mean elasticity values of the pre-set materials vary for different material thicknesses. The different properties of elasticity and strength have been given by the company Martinsons (Martinsons, 2006) and are results from testing of panels. The characteristic values of modulus of elasticity in ”FEM design 3D structure” may therefore be seen as effective stiffness properties taking into account the different layers and thicknesses.

[2] Fredrik Lagerström, Technical Sales and Support at Strusoft Design Software in Europe AB.

In calculations the design effective values are used and are conducted according to EC 5.

To verify if the material parameters in the pre-setting influence of shear and bending stiffness in calculation of deflection, one single CLT panel was modelled with a horizontal concentrated patch load of 10kN at the top left corner. The panel was assumed to be fully fixed at the support and was seen as a cantilever beam deflecting due to shear and bending, as can be seen in Figure 7.7.

The width of the panel was set to 1.2 meter and the height 3 and 10 meters respectively. The influence of shear and bending can therefore be compared since shear is expected to dominate deflection of the shorter element and decrease with increased height.

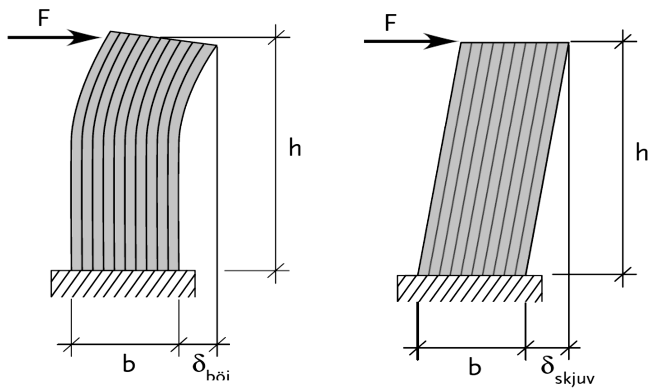


Figure 7.7 Deflection due to bending and shear respectively (Martinsons, 2006).

When using the pre-set CLT panels, the model became quite large with a large number of degrees of freedom. In order to reduce the size of the model and calculation time two different ways of modelling CLT panels was compared, one with the predefined CLT 95 and one with a shell element with assigned properties of CLT 95. This was done in order to see if they would behave the same with regard to bending and shear.

When converting the input data for material properties of CLT panels to shell element, elasticity modulus in compression and tension was neglected and only the mean modulus of elasticity is used. The characteristic values for material properties are given in Table 7.1.

Table 7.1 Characteristic material properties for CLT 95 panels and shell elements.

	CLT 95	
Grain direction	0°	90°
$E_{m.k}$ [N/mm^2]	7400	2200
$E_{c.k}$ [N/mm^2]	5700	3900
$E_{t.k}$ [N/mm^2]	5700	3900
$G_{v.k}$ [N/mm^2]	110	-
$G_{r.k}$ [N/mm^2]	110	-

	Shell element	
Grain direction	0°	90°
E_{mean} [N/mm^2]	7400	2200
G_{mean} [N/mm^2]	110	-

When comparing the deflection of the CLT panel with a shell element the results differ. For a 3 m high wall element, the deflection became 13% greater for the shell element than for the CLT element and for a 10m high wall the deflection was 37% higher, see Table 7.2 for comparison. This indicates that the two elements are treated in different ways in calculation and since the CLT panel show higher stiffness it is assumed to show a more realistic behaviour compared with the shell element. CLT elements were therefore used for the model and in further analysis.

Table 7.2 Comparison of deflection for a shell element and a CLT panel

Height	Shell element	CLT panel
3m:	7.17mm	6.34mm
10m:	115.78mm	84.80mm

In order to further understand how the ”FEM design 3D structure” evaluates the predefined CLT material; the input data values for characteristic elasticity modulus were changed to see how it effects the deflection. The results indicate no difference in deflection when changing the values for mean modulus of elasticity perpendicular and parallel grain. When changing the values of elasticity in compression perpendicular and parallel grain a change in response is noticed, revealing that these parameters are being used in stiffness calculations. These values are therefore used in hand calculations when verifying the model.

8 Static Analysis

The static analysis was conducted to investigate the lateral deflections due to wind-loads and load-bearing capacity of the chosen structure. The building system presented in Chapter 7 was analysed through preliminary static design and the maximal lateral deflection at the top of the structure was compared with results from the finite element model.

The load-bearing system is sufficient against vertical and lateral loading in ULS. The combination of loading actions was set according to Eurocode 0 and the loads considered was self-weight, residential and wind load in positive x- and y-direction as described in Chapter 7. The check was performed in ”FEM design 3D structure” where all structural elements were evaluated and the mean utilization ratio became about 20% for line and surface wind loading. Although it is a low value the dimensions of elements were kept since it complies with design handbook given by the CLT producer Martinsons.

8.1 Static deflection

Depending on how the structural system is designed the building reacts to wind loads differently. The structure could be assumed to act as a cantilever beam with uniformly distributed effective bending stiffness and mass. Observations on structural systems made of different building materials shows that relatively stiff structures such as concrete perceive a larger influence of bending deformation whilst weaker systems is more influenced by shear deflection as can be seen in Figure 8.1. ”FEM design 3D structure” does not provide checks in SLS but is available for ULS; therefore checks in SLS are made by hand-calculations.

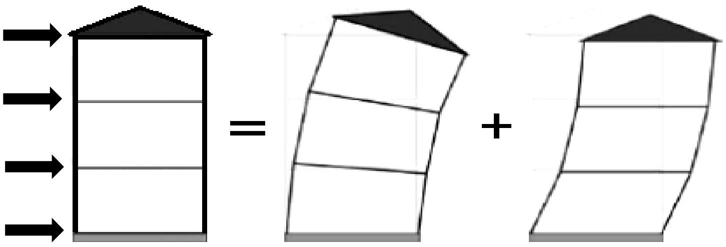


Figure 8.1 Behaviour of structure due to wind-load, b) typical for very stiff system such as concrete, c) typical for weaker system such as wood-based shear walls (Källsner, 2008).

In order to estimate the expected deflection and to compare and verify the obtained ”FEM design 3D structure” results a simplified hand calculation, given in Appendix B. The deflection was calculated for the structure with the same material properties as used in ”FEM design 3D structure” , i.e. mean effective value of modulus of elasticity and shear, given in Table 7.1. The lateral deflection from hand-calculations produced a value of 8.9mm. Deflections obtained from ”FEM design 3D structure” is given in Figures 8.2 and 8.3 and mean values at top for wind loading in positive x-direction are ≈9.2mm for line wind model and ≈12.4mm for surface wind model. The hand-calculations are very simplified where the walls are seen as continuous panels without hinged connection, the wind only acts in the x-direction on the along-side as a lineload and the behaviour is simplified to a cantilever beam. The deflection of the across-side wall is relatively small for both line and surface load.

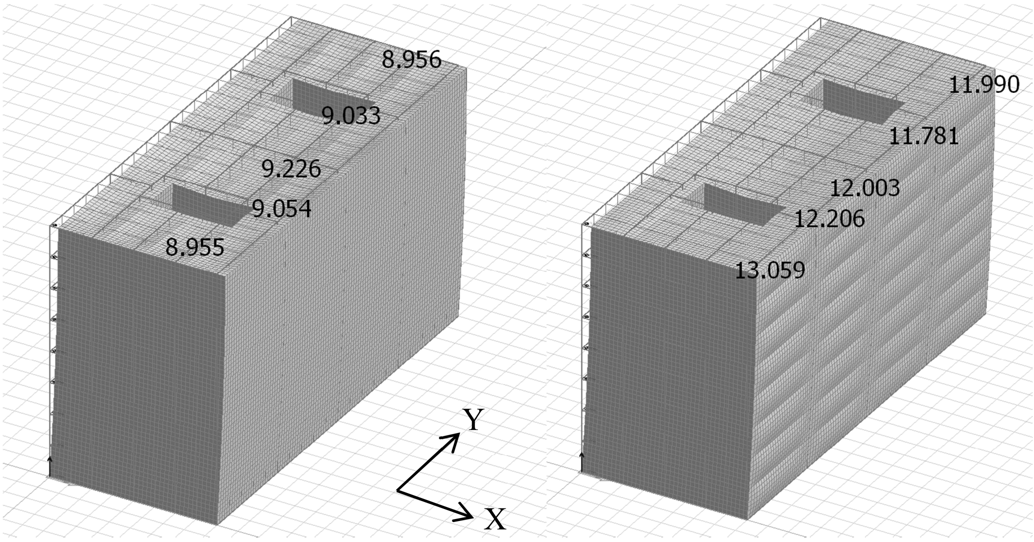


Figure 8.2 Lateral deflections in SLS for x- -direction, in mm, for line (left) and surface (right) wind loading.

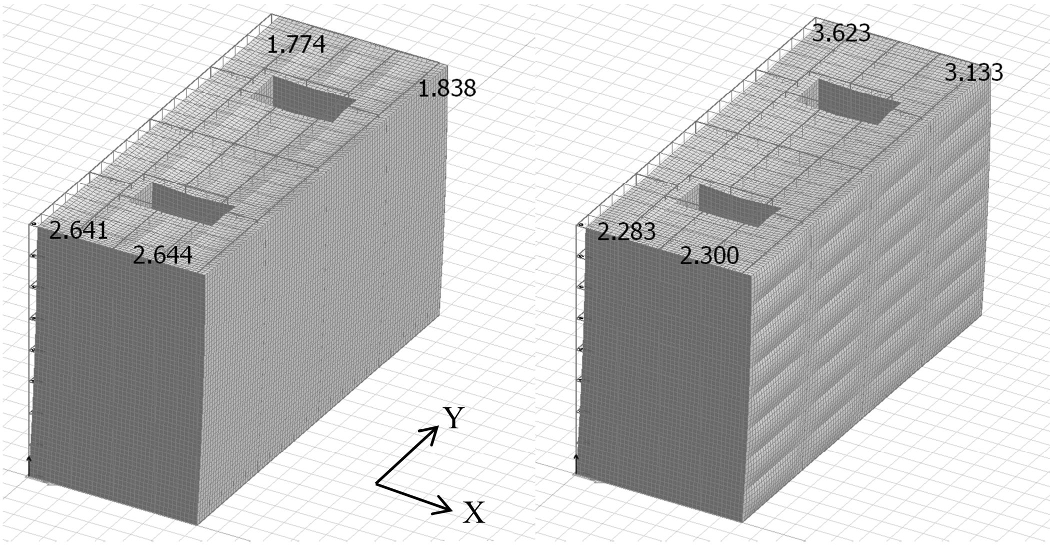


Figure 8.3 Lateral deflections in SLS for y-direction, in mm, for line (left) and surface (right) wind loading.

The judgement regarding maximum allowed horizontal displacements is left to the designer since the current code for design, EC5, does not provide any limitations. The existing research material is not sufficient enough to formulate requirements (Källsner, 2008). For multi-storey structures large deflections may cause cracking, permanent damage and brittle failure. In previous Eurocodes from 1989 the maximum allowed displacement was given as L/300 where L is the height of the structure (Källsner, 2008). This regulation was removed since it was no longer up to date for the increasing number of multi-storey structures and for the increasing quality demands. At the moment the strictest regulation is L/500, according to German design code (DIN, 1994) and lateral deflections should not exceed 48mm. The results from both hand-calculations and model fulfil the requirements.

8.2 Horizontal stability

The stability due to tilting was checked for both favourable and unfavourable load-cases, given in Appendix A. For a favourable load combination the structure fulfilled the demands although for the unfavourable load-combination the limit was exceeded. The moment resistance of the structure was not enough and there is a need for further anchorage in order to fulfil stability conditions. Geotechnical investigations are given in the development plan for Skeppsbron and states that piling is needed to avoid further settlements in the area (Söderberg & Lööf, 2012). The geotechnical conditions at Skeppsbron consist mainly of clay, sensitive to settlements with an underlying layer of friction-soil down to bedrock. The depth of the soil-layer varies between 50 to 80 meters (Söderberg & Lööf, 2012). Since the piles counteracts the moment induced by wind and unintended inclination the structure is safe against tilting as long as the connections between slab, piles and walls are sufficient.

9 Dynamic Analysis

A dynamic analysis was performed and the acceleration will be calculated based on the natural frequency obtained from "FEM design 3D structure" and further on compared with different comfort criteria mentioned in Chapter 4.3. Other dynamic loads that could have been considered are earthquakes or vibrations induced by traffic and industries close to the building. Since Sweden is situated in an area with low tectonic activity earthquakes are infrequent in this area. Skeppsbron is an area with surrounding roads, tunnels, ferries and planned tram lines but due to complex measurements and uncertain conditions at the site this thesis does not evaluate the influence of these kinds of vibrations. In this chapter the dynamic analysis is described more in detail and the obtained accelerations are compared to the different codes and regulations.

9.1 Frequency

In order to order the natural frequency of the building at different mode shapes, the "FEM design 3D structure" built-in function for mass conversion and mass distribution was used. This function is mainly used as a first step for further dynamic analysis when determining the seismic effects induced by earthquakes. It is made in accordance with EC 8 stating that the load cases acting in the direction of gravity should be converted into mass and distributed along the structure. Self-weight should be converted by a factor 1 whilst variable actions should be reduced by factor $\psi_{E.i}$. This reduction factor is a combination coefficient taking into account the likelihood of the variable loads not being present all over the structure at the event of an earthquake according to Equation 9.1. The reduction factor becomes quite small and therefore also the influence of residential loading for mass conversion.

$$\psi_{E.i} = \varphi \cdot \psi_{2.i} \tag{9.1}$$

Where:

$\varphi =$ is the variable for load action dependent on storey occupancy

$\psi_{2.i}=$ is the combination coefficient for a quasi-permanent value of variable action

Worst case scenario for a structure at ULS under seismic loading is during an earthquake and an empty building, where only the self-weight is present resulting in maximum acceleration and sway which will cause greatest damage to the structure. The natural frequency was evaluated for both dead-weight and variable residential loading is present, since worst case is when wind load acts on the structure during a longer period of time and with resident present.

The first natural frequency is dependent on the mass, stiffness, load intensity and damping of the structure. "FEM design 3D structure" does not have any built in structural damping other than the material itself and damping obtained by self-weight and vertical loads. Damping is taken into account when acceleration calculations are conducted by empirical values, chosen values is given in Appendix E. The first natural frequency obtained from "FEM design 3D structure" produces a value of 1.687Hz with both self-weight and reduced value for residential loading.

According to EC 1 part 1-4, general actions on structures – wind actions, the first natural frequency of a cantilever with a height below 50m should be calculated according to an SDOF model. The SDOF model is an easy way of calculating the dynamic response of a complex structure since the building can be estimated as a cantilever beam. The maximum displacement of the building is determined by placing the total self-weight as a static load in the direction of the vibration. Bending and shear stiffness is approximated along the structure. The first natural frequency is given by Equation 9.2. Calculation of fundamental frequency according to Eurocode, a single freedom (SDOF) model, is given in Appendix C.

$$n_{EC} = \frac{1}{2\pi} \cdot \sqrt{\frac{g}{x_1}} = 1.633\ Hz \tag{9.2}$$

Where:

g is the acceleration of gravity

x_1 is the maximum structural displacement with total mass acting as a load in the vibration direction

9.2 Acceleration according to Eurocode

In the national annex for Sweden of Eurocode 1:1-4 it is stated that when comfort criteria is evaluated a wind speed with a 5-year return period should be used. The wind speed was recalculated according to national annex and has the value of 21.4m/s. The formula used to recalculate the wind speed with different return periods is given by Equation 9.3.

$$v_{Ta} = 0.75 v_b \sqrt{1 - 0.2\ln\left(-\ln(1 - \frac{1}{T_a})\right)} \tag{9.3}$$

Where:

T_a is the return period in years
v_b is the reference wind speed

Wind turbulence is taken into account by the turbulence intensity factor I_v, which depends on the standard deviation of turbulence and the mean wind velocity.

$$I_v = \sigma_v / v_m \tag{9.4}$$

Where:

σ_v is the standard deviation of turbulence
v_m is the mean wind speed

The total damping of the structure is determined by the summation of structural damping and aerodynamic damping. Since no value for structural damping of timber structures is given in the code it is approximated as a slightly lower value than for steel structures which is given in table F.2 in EC1:1-4, aerodynamic damping is dependent of force coefficient, wind speed, first natural frequency, density of air and total mass of structure per unit area of building face. The resonance factor of the structure is calculated according to a background factor submitted in the national annex and different parameters given in Eurocode.

Finally the standard deviation of the acceleration and a peak factor is calculated whereas a maximum acceleration is obtained.

$$\sigma_x = \frac{3I_v R_v q_{pv} b_f c_f \phi_1}{m_e} \tag{9.5}$$

Where:

I_v	is the turbulence intensity factor
R_v	is the root mean square of the resonance factor
q_{pv}	is the wind pressure
b_f	is the width of the building face
c_f	is the force coefficient for wind load
ϕ_1	is the fundamental flexural mode
m_e	is the mass per unit length

$$k_p = \sqrt{2\ln(v \cdot t_p)} + \frac{0.6}{\sqrt{2\ln(v \cdot t_p)}} \tag{9.6}$$

Where:

v	is the up-crossing frequency
t_p	is the time duration of peak wind

$$X = k_p \cdot \sigma_x \tag{9.7}$$

Where:

k_p	is the peak acceleration factor
σ_x	is the standard deviation of acceleration

Maximum accelerations for a 10 minute wind with a 5-year return period and a mean wind at Skeppsbron with a 1-year return period are presented below.

$$\mathrm{X_{EC.5}} = 0.022\ {}^{m}/_{s^2}$$

$$\mathrm{X_{EC.skepp}} = 0.000056\ {}^{m}/_{s^2}$$

9.3 Acceleration according to ISO standard

In ISO 4354:2009, Wind actions on structures; the acceleration is calculated in a similar way as presented in the national annex to Eurocode but takes different parameters and wind return period into account. The ISO-standard evaluates the first natural frequency in main structural direction and torsion, the first natural frequency obtained from "FEM design 3D structure" for the building analysed was 1.687Hz and the torsional mode occurs at the fifth natural frequency at 5.917Hz.

The reference speed is the specified values for wind speed at the geographical area in which the structure is located. It refers to a standard exposure concerning height, roughness and topography, averaging time of 3 seconds and probability of exceedance in one year. In the code it is stated that the hourly averaging wind speed is meaningful for synoptic storms, whereas 10 minute and 3 second wind should be used for more harsh wind conditions like thunderstorms and tropical cyclones.

The reference wind for Gothenburg is taken as stated in Eurocode, a 10 minute wind with a return period of 50 years. According to Thomas Hallgren[3] the conversion from reference 10 minute wind to an hourly wind can be done by reducing 25m/s^2 by 0.95 giving a value of 23.75m/s^2. This value was used in calculation of the peak acceleration according to ISO standard and was recalculated for a wind speed with return period of 1 year as stated in Equation 9.3.

The peak terrain roughness is based on an hourly mean averaging wind which in turn influence of the turbulence intensity. The exposure factor c_{exp} is calculated according to Equation 9.8.

$$c_{exp} = k_{tr.z} k_{tr.change} k_{topog} \tag{9.8}$$

Where:

$k_{tr.z}$ is the peak terrain roughness and height exposure factor
$k_{tr.change}$ is the peak terrain roughness change exposure factor
k_{topog} is the peak topography exposure factor

The reference hourly wind speed is converted to fit the topography conditions at site, see Equation 9.9.

$$v_{site} = v_{ref}\, c_{exp} \tag{9.9}$$

Where:

v_{ref} is the reference wind speed

c_{exp} is the exposure factor

The damping of the structure should be reduced to 75% of the values given in table E.3 when evaluating horizontal habitability comfort. There is no given value of damping for timber structures so it was approximated to 1.2% which is a slightly lower value than the given value for steel structures.

Acceleration of a building exposed to wind is given by Equation 9.10 below.

$$X = (2\pi n_1)^2 \frac{2 g_{DB} I_{vh} R_D}{1 + 2 g_{DB} I_{vh} \sqrt{B_D{}^2 + R_D{}^2}} x_D \tag{9.10}$$

Where:

n_1 is the first mode natural frequency
g_{DB} is the peak factor for background component
I_{vh} is the turbulence intensity factor for wind speed at reference height
R_D is the resonant response factor
B_d is the background response factor
x_D is the displacement under static wind loading

[3] Thomas Hallgren, Structural Engineer at COWI AB, Byggteknik Hus.

Accelerations for an hourly averaging wind with a 1-year return period for the first and fifth mode shape frequency and mean-wind velocity at Skeppsbron are presented below.

$$X_{ISO.1} = 0.013\,m/s^2$$

$$X_{ISO.1.torsion} = 0.055\,m/s^2$$

$$X_{ISO.skepp} = 0.0070\,m/s^2$$

$$X_{ISO.skepp.torsion} = 0.024\,m/s^2$$

9.4 Comfort criteria

The obtained natural frequency, from ”FEM design 3D structure” for which acceleration was calculated, was used in order to check the buildings behaviour according to comfort criteria described in Chapter 4.3. The different codes use different values for wind return periods and frequency in evaluation and the assumptions made and results are described in the following chapters. Accelerations calculated according to Eurocode and ISO differ a bit and the choice was made to follow the individual codes requirements for wind speeds for acceleration calculations based on comfort evaluation. ISO evaluates an hourly averaging wind with a return period of 1 year, Eurocode with a 10 min averaging wind and return period of 5years. Wind conditions at Skeppsbron represent a yearly mean wind speed which will be compared with the different criteria presented in following chapters.

9.4.1 Eurocode

In the Swedish national annex of Eurocode it is stated that a mean 10 minute wind with a 5-year return period should be used when evaluating comfort criteria. Accelerations should be calculated for the first natural frequency, which corresponds to bending in the principal structural direction of the building studied. Accelerations obtained from Eurocode calculations were evaluated by comfort criteria according to ISO, Japanese, British and Canadian standard. Since Eurocode doesn't have any comfort criteria due to wind-induced vibrations, accelerations was checked according to the above mentioned codes even though the wind duration and return periods used in Eurocode might conflict with the ones used in the codes. Results are presented in the individual codes chapters that follow.

9.4.2 ISO 10137

The comfort criteria presented in ISO 10137 is based on peak acceleration for a wind with a 1-year return period. It is stated in the code that for synoptic winds other than cyclonic storms and thunderstorms an averaging time of 1 hour should be used in evaluation; other averaging time periods give extreme design values. The frequency of the first mode in principle structural direction and in torsion should be checked, which corresponds to the 1st and 5th mode shape, see Appendix D. ISO standard 6897 is not relevant since it only complies for natural frequencies below 1Hz.

The acceleration calculated for an hourly averaging wind with a 1-year return period for the first natural frequency of 1.687Hz was acceptable according to regulations and has a lower value than what is presented in Figure 9.1. For the mode in torsion the frequency becomes higher than for which criteria is given and is therefore assumed to be satisfactory. Acceleration obtained using wind data from Skeppsbron calculated according to Eurocode and ISO fulfil requirements and lay under values given in regulations. Accelerations for a 10 minute averaging wind with a 5-year return period for the first natural frequency calculated according to Eurocode fulfil requirements for residential building. Accelerations that lay within range of regulations according to ISO 10137 are presented in Figure 9.1.

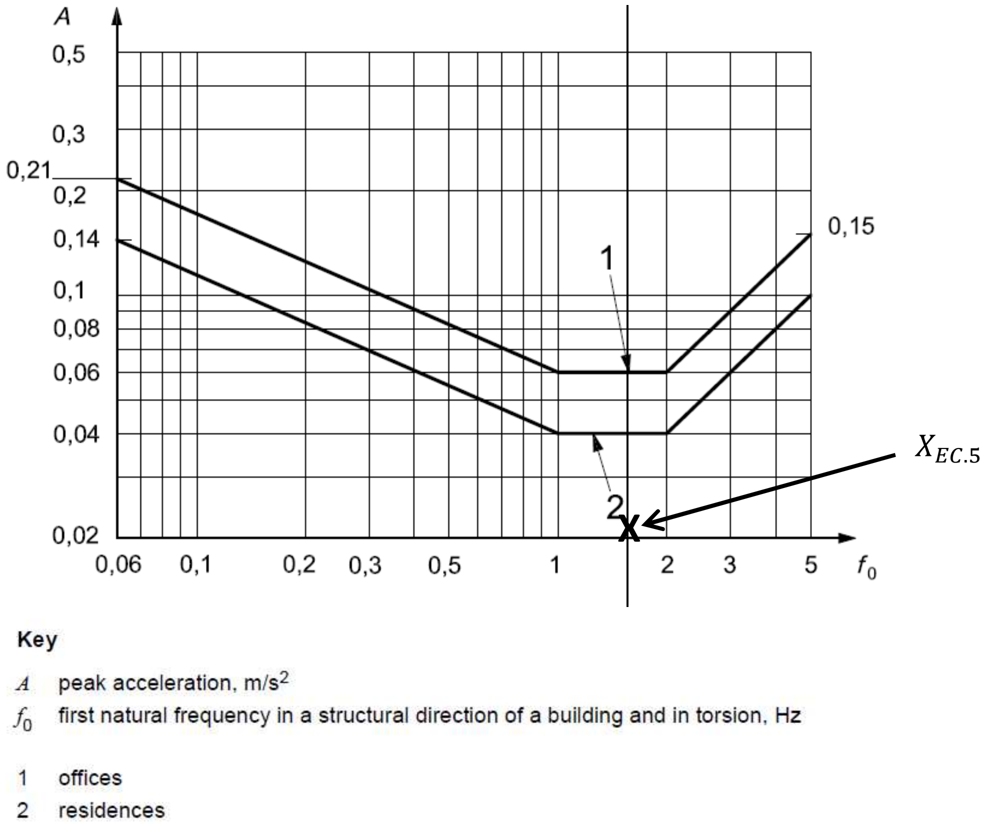


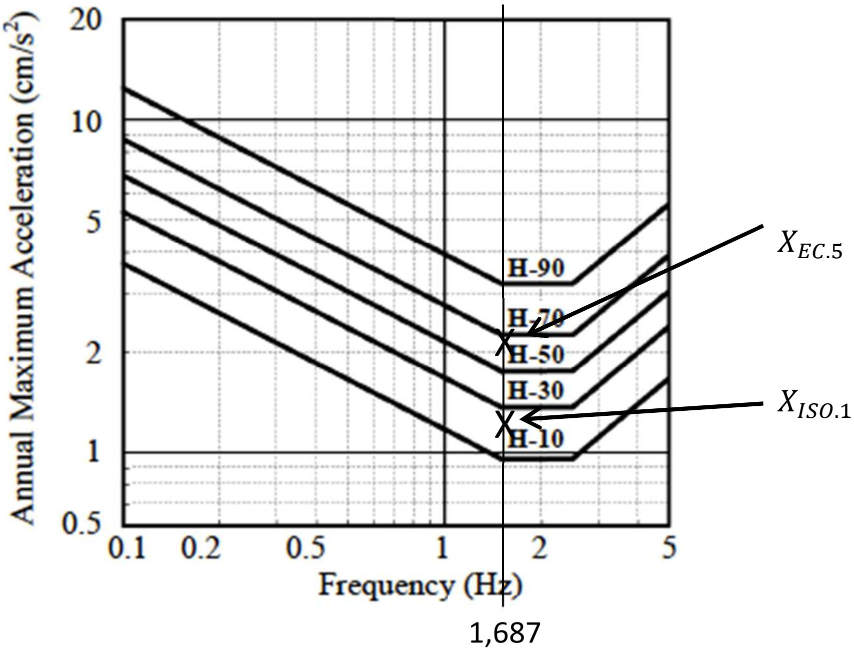
Figure 9.1 Comfort criteria according to ISO 10137 with results from dynamic analysis calculations (International Organization for Standardization, 2008)

9.4.3 NBCC

The Canadian regulation is based on mean hourly peak acceleration with a 10-year return period. The criterion is independent of variation in frequency and in the range 1.5%-3.5% of acceleration due to gravity which corresponds to approximately 0.14m/s^2-0.25m/s^2. The acceleration due to a 10-year return period calculated according to ISO became 0.023m/s^2 and is below the limit for residential buildings, which is represented by the lower value in the acceleration range. Values for a 5-year wind and wind at Skeppsbron calculated according to Eurocode will also fulfil these requirements.

9.4.4 AIJ

The Japanese code is based on the probability of residents that will perceive the vibration expressed in percentage and not a specific value of disturbance or discomfort threshold. Peak acceleration for a 10 minute averaging wind speed with a one year return period should be used in evaluation according to AIJ and acceleration was calculated according to ISO. As can be seen in Figure 9.2 a wind with a 5-year and a 1-year return period is perceptible by approximately 65% and 20% respectively of the residents and mean annular wind at Skeppsbron lay under the scale.



9.2 Probabilistic perception thresholds according to AIJ-GBV-2004 with results from dynamic analysis calculations (Architectural institute of Japan, 2004)

9.4.5 British standard

The British standard is based on vibration dose values that indicate the human tolerance against vibrations and the probability of adverse complaints from residents. There is no recommended wind speed, duration or return period to use in the code, so in order to get a comparison the vibration dose values were investigated for accelerations obtained by calculations according to Eurocode, ISO and Skeppsbron. The code is not compatible with Eurocode and according to Thomas Reynolds[4] wind calculated according to Eurocode can be adopted by assuming the mean wind duration is 10 min.

The code takes into account the duration of the wind during either day or night, where residents are assumed to be more disturbed by vibration during night. The vibration dose value can be calculated in two ways dependent of the wind data available; either the duration of the wind is known or the number of times the vibration will occur. The assumption was made that the peak wind of ISO has an hourly duration; mean wind

[4] Thomas Reynolds, Department of Architecture & Civil Engineering, University of Bath, Bath, UK Email: T.P.S.Reynolds@bath.ac.uk

according to Eurocode 10 minutes and mean wind at Skeppsbron during the entire day/night. Calculations are given in Appendix F and results presented in Table 9.1. The mean wind at Skeppsbron acting on the structure will not give rise to comments according to the regulations. For a 5-year and 1-year return period wind will give rise to low probability of adverse comment during night only.

$$VDV_{\mathrm{ISO}} = 0.10\ {}^{m}/_{s^{1.75}}$$

$$VDV_{\mathrm{EC}} = 0.11\ {}^{m}/_{s^{1.75}}$$

$$VDV_{day.skepp} = 0.11\ {}^{m}/_{s^{1.75}}$$

$$VDV_{night.skepp} = 0.091\ {}^{m}/_{s^{1.75}}$$

Table 9.1 Vibration dose values, $VDV_{b/d\,,day/night}$ for residential buildings (British Standard Institution, 2008).

Vibration dose value range which might result in various Probabilities of adverse comment within residential buildings

Place and time	**Low probability of adverse comment** m/s$^{1.75}$	**Adverse comment possible** m/s$^{1.75}$	**Adverse comment probable** m/s$^{1.75}$
Residential buildings 16 h day	0.2 to 0.4	0.4 to 0.8	0.8 to 1.6
Residential buildings 8 h night	0.1 to 0.2	0.2 to 0.4	0.4 to 0.8

NOTE For offices and workshops, multiplying factors of 2 and 4 respectively should be applied to the above vibration dose value ranges for a 16 h day.

VDV_{EC} VDV_{ISO}

10 Discussion

The main focus of this thesis has been to investigate problems connected to residents' comfort concerning wind-induced horizontal vibrations in a multi-storey building constructed with CLT and throughout the project uncertainties with determining material properties and how design calculations should be computed has been revealed. Since CLT is rather new on the market material properties used are often those given by the individual producers conducted by tests, which in turn leads to a restricted data access and limited results. Methods regarding stiffness calculations of the material differ and the offered data from producers' does not support any guidance in design calculations due to the producers' confidentiality. More research needs to be done in order to obtain a standardized method and values for material properties.

”FEM design 3D structure” has a pre-set material function for CLT for which the material data was given by Martinsons. The validation of the model was therefore done with the same material data, the effective mean modulus of shear and elasticity compiled from tests. Fredrik Lagerström at Strusoft provided the information concerning material properties in ”FEM design 3D structure”; he also mentioned that the function for CLT in ”FEM design 3D structure” is under development and that the initiative behind a CLT function was made by Martinsons. Approximations for the material properties were made in the program, where the influence of layers was not included. Since the material properties is given by Martinsons, there is not an alternative way to design with CLT in ”FEM design 3D structure”, in other finite element software the designer could have more insight in calculations and influence of material properties.

In order to run an analysis, the panels had to be redesigned compared to the initial design in order to reduce the number of panels, connections and to simplify the mesh. Openings were not made since it was not possible to define load distribution over openings and load bearing structure in the wind-cover model, which might give different results in terms of natural frequency and deflection. The alternative to “FEM design 3D structure” is to model the structure in a more advanced computer program where the CLT-material can be modelled in layers. Problem is that it would be a heavy model due to the size of the structure and numbers of degrees of freedom and due to the impact of slip/racking of panels and layers of laminates.

In comparison with lateral deflection the methodology assumed to be used by “FEM design 3D structure” was adopted in hand-calculations and the material was seen as solid timber panels with orthotropic material behaviour. Wind-loading was applied as line-loads acting on each floor and the effective shear and bending stiffness was approximated for the structure and floors were set as infinite stiff. The values obtained from hand calculation correlates with the obtained value for the model where wind was applied as a line load in “FEM design 3D structure” as would be expected. In the model wind was applied in both x and y direction whilst calculations only considered along-wind response, which affects the deflection. Wind-loading in x-direction will on the other hand have larger influence and will give larger deflection since wind is applied in the weak structural direction.

On the other hand no connections were taken into account in hand-calculations and all connections between walls were assumed to be rigid. A parameter that could affect the results substantially is the connections between panels and also connections to the slab. In “FEM design 3D structure” all connections between elements was modelled as hinged and the structure was fully fixed to the slab, but in reality the connections could behave differently, more rigid in some connections and less rigid in some parts which makes the connections a possible source of error.

The human perception of vibrations depends on many variables and it could be hard to decide an exact limit value of what is considered "habitable". The ISO-standard limit for residential buildings is connected to the perception probability limit given by the Architectural institute of Japan limit for perception probability percentage of 90% by residents, based on results from surveys for different buildings. There is no stated wind return period that should be used in evaluation presented in the Japanese code and it is up to the designer to judge what values are seen as tolerable and within the limits of “residential comfort”. Even though one can get threshold values of perception, disturbance from vibrations are harder to evaluate. Even though 70% may experience motion it may not be equivalent with how many who experience it as disturbing. The only code that evaluates actual disturbance probability is the British, based on duration of the vibrations during day or night. The background for the limits of the vibration dose values is rather unclear and evaluation and comparison with other limitations are therefore hard to do. Assumptions were made for duration of vibrations since the code is not directly adaptable with wind speeds used as stated in Eurocode, which makes the results uncertain and whether they are lower or higher than in reality is also hard to judge. Even though a building may be within the limits some residents may still consider noticeable vibrations as a disturbing feature whilst others may not experience any motion at all and the psychological aspect is almost impossible to predict and use as a design criteria. How the designer evaluates and decides what is considered acceptable when it comes to the percentage of occupants who experience disturbance is a fine balance between efficient design and the designer reputation in terms of complaints.

The codes are based on wind data with different return periods and mean peak wind speed durations. The mean peak wind speed for a 10 minute averaging wind is lowered by almost half when changing the return period from 5 to 1 years but does not change substantially when converting to an hourly averaging wind for wind speed with the same return period. Wind speed at Skeppsbron is the mean wind speed during an entire year from data collected during a long period of time and may not be used in comparison with regulations. Shorter mean wind speed time periods are represent wind gusts whilst annual mean wind speed represent the true mean wind speed. The codes on the other hand evaluate mean peak values and the annual mean wind speed at Skeppsbron does not give representative accelerations when comparing with comfort criteria.

In calculations for accelerations approximations had to be made in order to get adequate results. Calculations according to national annex of Eurocode had a straightforward method whilst in ISO some factors, such as frictional coefficient had to be approximated. The peak factor for background component and according to ISO an appropriate value is 3.4 for a fluctuating wind speed. It is also stated that it could be equated to the average peak factor which is dependent on the averaging time of wind speed where the value 3.4 is closer to a 3 second wind and the value for an hourly wind was equal to zero. This seemed strange since no acceleration can be obtained using the value for hourly averaging wind in calculation and therefore the value was set to 0.28 which corresponds to a 10 minute averaging wind.

11 Conclusions

Requirements stated in ISO-10137 and Canadian standard are fulfilled with values well below the limit for residential buildings. In ISO 10137 it is stated that the torsional mode should be investigated as well, that occurs at a frequency of 5.9Hz which is outside the range of the scale in Figure 9.1 and torsion is therefore assumed to have a low probability of occurrence. According to the Architectural institute of Japan the probability of perception percentage from residents is 65% based on accelerations according to Eurocode and 20% according to ISO, whether it is a perception of disturbance or perception of motion is unclear. The code may be seen as a guideline for design rather than a regulation; we believe a 65% and 20% probability of perception is acceptable in a 5 respectively 1 year period when discussing comfort. Compared to the other regulations, the probability of perception percentage curves given by the Japanese standard are quite low as can be seen in Figure 11.1 which also supports the statement above.

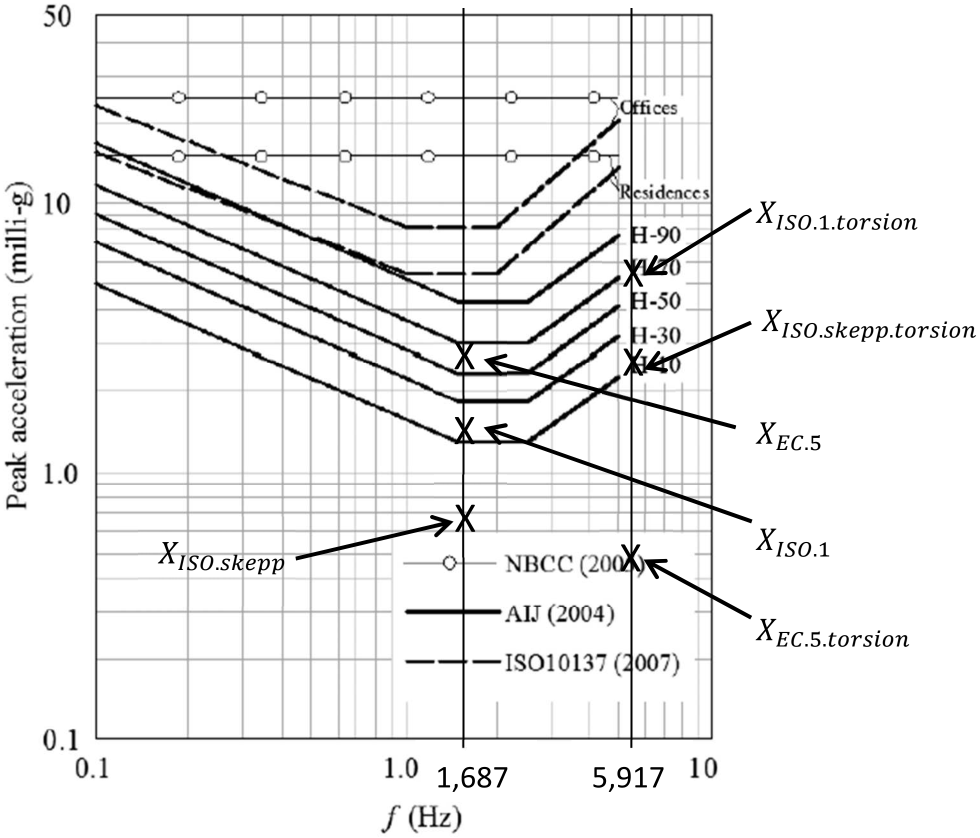


Figure 11.1 Comfort criteria of accelerations calculated according to ISO and Eurocode for 5, 1 year return period and wind speed at Skeppsbron (Pozos-Estrada, Hong & Galsworthy, 2010)

The British standard is altered from the ISO-standards and when comparing the results calculated according to EC and ISO-standard there is a low risk of adverse comment from residents due to a 5-year and 1-year return period wind which is assumed to be acceptable on the same basis as for the Japanese regulations stated above.

To investigate the specific conditions at Skeppsbron an analysis of the acceleration due to the mean wind obtained from SHMI, Swedish Metrological and Hydrological Institute, measurements was used to see how the perception would be for residents for a smother wind acting throughout the year. Calculations according to ISO and Eurocode are based on mean peak wind speed and therefore the results may not be as representative as the British standard where the duration of wind was taken into account. The acceleration calculated according to Eurocode fell out of the range of Figure 11.1, the acceleration calculated in ISO is in the lower part of the figure and the vibration dose value gave results below low probability of adverse comment.

The lateral deflection of the building was investigated through a static analysis with wind-loads from both x and y direction with two different load applications, line loads which are used in Eurocode calculations and by a wind-cover which was applied in the finite element analysis. The mean lateral deflection in the x-direction became 9.25mm and 12.2mm respectively which are satisfactory according to German regulations, in this case 48mm.

The building is therefore seen to fulfil the requirements according to regulations against lateral deflection and wind-induced vibrations presented in this thesis.

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Appendix A - Horizontal stability

Geometry and input

$h_{storey} := 3m$

$n_{floor} := 8$

$h_{building} := h_{storey} \cdot n_{floor} = 24\,m$

$l_{building} := 44m$

$b_{building} := 16m$

Selfweights of components

$G_k := 5 \frac{kN}{m^3}$ selfweight CLT

$G_{k.roof} := 0.8 \frac{kN}{m^2}$

$G_{k.exwall} := 0.85 \frac{kN}{m^2}$

$G_{k.floor} := 0.9 \frac{kN}{m^2}$

$G_{k.divwall} := 1.05 \frac{kN}{m^2}$

$G_{k.F} := 5.55 \frac{kN}{m^2}$ selfweight foundation

flat roof $\mu := 0.8$ $S_k := 1.5 \frac{kN}{m^2}$

$Q_{snow} := S_k \cdot \mu = 1.2 \cdot \frac{kN}{m^2}$

$q_{k.imposed} := 2 \frac{kN}{m^2}$ $\alpha_A := 0.6$

$q_{k.wind} := 1 \frac{kN}{m^2}$

$\gamma_m := 1.25$

Geometry and input

Loads

Loads

$g_{roof} := \dfrac{l_{building} \cdot b_{building} \cdot G_{k.roof}}{2} = 281.6 \cdot kN$ Selfweight of the roof

$q_{snow} := \dfrac{S_k \cdot l_{building} \cdot b_{building}}{2} = 528 \cdot kN$

$$g_{floor.edge} := G_{k.exwall} \cdot \left(l_{building} + \frac{b_{building}}{4} \right) h_{storey} \ldots = 280.8 \cdot kN$$
$$+ G_{k.floor} \cdot l_{building} \cdot \frac{b_{building}}{4}$$

$$q_{floor.edge} := q_{k.imposed} \cdot l_{building} \cdot \frac{b_{building}}{4} = 352 \cdot kN$$

$$g_{floor.mid} := G_{k.exwall} \cdot \left(\frac{b_{building}}{2} + l_{building} \right) \cdot h_{storey} \ldots = 449.4 \cdot kN$$
$$+ G_{k.floor} \cdot l_{building} \cdot \frac{b_{building}}{2}$$

$$q_{floor.mid} := q_{k.imposed} \cdot l_{building} \cdot \frac{b_{building}}{2} = 704 \cdot kN$$

Selfweight as unfavorable

$V_{d8.edge} := 1.10 \cdot g_{roof} + 1.50 \cdot q_{snow} = 1.102 \times 10^3 \cdot kN$

$V_{d8.mid} := 1.10 \cdot g_{roof} + 1.50 \cdot q_{snow} = 1.102 \times 10^3 \cdot kN$

$V_{d7.edge} := V_{d8.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 1.939 \times 10^3 \cdot kN$

$V_{d7.mid} := V_{d8.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 2.652 \times 10^3 \cdot kN$

$V_{d6.edge} := V_{d7.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 2.776 \times 10^3 \cdot kN$

$V_{d6.mid} := V_{d7.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 4.202 \times 10^3 \cdot kN$

$V_{d5.edge} := V_{d6.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 3.612 \times 10^3 \cdot kN$

$V_{d5.mid} := V_{d6.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 5.753 \times 10^3 \cdot kN$

$V_{d4.edge} := V_{d5.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 4.449 \times 10^{3} \cdot kN$

$V_{d4.mid} := V_{d5.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 7.303 \times 10^{3} \cdot kN$

$V_{d3.edge} := V_{d4.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 5.286 \times 10^{3} \cdot kN$

$V_{d3.mid} := V_{d4.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 8.853 \times 10^{3} \cdot kN$

$V_{d2.edge} := V_{d3.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 6.123 \times 10^{3} \cdot kN$

$V_{d2.mid} := V_{d3.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 1.04 \times 10^{4} \cdot kN$

$V_{d1.edge} := V_{d2.edge} + 1.10 \cdot g_{floor.edge} + 1.50 \cdot q_{floor.edge} = 6.96 \times 10^{3} \cdot kN$

$V_{d1.mid} := V_{d2.mid} + 1.10 g_{floor.mid} + 1.50 \cdot q_{floor.mid} = 1.195 \times 10^{4} \cdot kN$

<u>Selfweight as favorable</u>

$V_{d8.edge.f} := 0.9 g_{roof} = 253.44 \cdot kN$

$V_{d8.mid.f} := 0.9 g_{roof} = 253.44 \cdot kN$

$V_{d7.edge.f} := V_{d8.edge.f} + 0.9 \cdot g_{floor.edge} = 506.16 \cdot kN$

$V_{d7.mid.f} := V_{d8.mid.f} + 0.9 g_{floor.mid} = 657.9 \cdot kN$

$V_{d6.edge.f} := V_{d7.edge.f} + 0.9 \cdot g_{floor.edge} = 758.88 \cdot kN$

$V_{d6.mid.f} := V_{d7.mid.f} + 0.90 g_{floor.mid} = 1.062 \times 10^{3} \cdot kN$

$V_{d5.edge.f} := V_{d6.edge.f} + 0.90 \cdot g_{floor.edge} = 1.012 \times 10^{3} \cdot kN$

$V_{d5.mid.f} := V_{d6.mid.f} + 0.90 g_{floor.mid} = 1.467 \times 10^{3} \cdot kN$

$V_{d4.edge.f} := V_{d5.edge.f} + 0.9 \cdot g_{floor.edge} = 1.264 \times 10^{3} \cdot kN$

$V_{d4.mid.f} := V_{d5.mid.f} + 0.90 g_{floor.mid} = 1.871 \times 10^{3} \cdot kN$

$V_{d3.edge.f} := V_{d4.edge.f} + 0.90 \cdot g_{floor.edge} = 1.517 \times 10^{3} \cdot kN$

$V_{d3.mid.f} := V_{d4.mid.f} + 0.90 g_{floor.mid} = 2.276 \times 10^{3} \cdot kN$

$V_{d2.edge.f} := V_{d3.edge.f} + 0.9 \cdot g_{floor.edge} = 1.77 \times 10^{3} \cdot kN$

$V_{d2.mid.f} := V_{d3.mid.f} + 0.90 g_{floor.mid} = 2.68 \times 10^{3} \cdot kN$

$V_{d1.edge.f} := V_{d2.edge.f} + 0.90 \cdot g_{floor.edge} = 2.022 \times 10^{3} \cdot kN$

$$V_{d1.mid.f} := V_{d2.mid.f} + 0.90g_{floor.mid} = 3.085 \times 10^{3} \cdot kN$$

Loads

Unintended inclination

Unintended inclination

$$\alpha_0 := 0.003$$

$$\alpha_d := 0.012$$

$n := 3$ *Number of supporting walls*

$$\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 9.928 \times 10^{-3}$$

Horizontal loads - unfavourable

$$F_{h8} := \left(V_{d8.edge} \cdot 2 + V_{d8.mid}\right) \cdot \alpha_{md} = 32.815 \cdot kN$$

$$F_{h7} := \left(V_{d7.edge} \cdot 2 + V_{d7.mid}\right) \cdot \alpha_{md} = 64.825 \cdot kN$$

$$F_{h6} := \left(V_{d6.edge} \cdot 2 + V_{d6.mid}\right) \cdot \alpha_{md} = 96.835 \cdot kN$$

$$F_{h5} := \left(V_{d5.edge} \cdot 2 + V_{d5.mid}\right) \cdot \alpha_{md} = 128.844 \cdot kN$$

$$F_{h4} := \left(V_{d4.edge} \cdot 2 + V_{d4.mid}\right) \cdot \alpha_{md} = 160.854 \cdot kN$$

$$F_{h3} := \left(V_{d3.edge} \cdot 2 + V_{d3.mid}\right) \cdot \alpha_{md} = 192.863 \cdot kN$$

$$F_{h2} := \left(V_{d2.edge} \cdot 2 + V_{d2.mid}\right) \cdot \alpha_{md} = 224.873 \cdot kN$$

$$F_{h1} := \left(V_{d1.edge} \cdot 2 + V_{d1.mid}\right) \cdot \alpha_{md} = 256.882 \cdot kN$$

Horizontal loads - favourable

$$F_{h8.f} := \left(V_{d8.edge.f} \cdot 2 + V_{d8.mid.f}\right) \cdot \alpha_{md} = 7.549 \cdot kN$$

$$F_{h7.f} := \left(V_{d7.edge.f} \cdot 2 + V_{d7.mid.f}\right) \cdot \alpha_{md} = 16.582 \cdot kN$$

$$F_{h6.f} := \left(V_{d6.edge.f} \cdot 2 + V_{d6.mid.f}\right) \cdot \alpha_{md} = 25.616 \cdot kN$$

$$F_{h5.f} := \left(V_{d5.edge.f} \cdot 2 + V_{d5.mid.f}\right) \cdot \alpha_{md} = 34.65 \cdot kN$$

$$F_{h4.f} := \left(V_{d4.edge.f} \cdot 2 + V_{d4.mid.f}\right) \cdot \alpha_{md} = 43.683 \cdot kN$$

$$F_{h3.f} := \left(V_{d3.edge.f} \cdot 2 + V_{d3.mid.f}\right) \cdot \alpha_{md} = 52.717 \cdot kN$$

$$F_{h2.f} := \left(V_{d2.edge.f} \cdot 2 + V_{d2.mid.f}\right) \cdot \alpha_{md} = 61.751 \cdot kN$$

$$F_{h1.f} := \left(V_{d1.edge.f} \cdot 2 + V_{d1.mid.f}\right) \cdot \alpha_{md} = 70.784 \cdot kN$$

Unintended inclination

Tilting

Tilting

The building is safe against tilting if the the vertical resultant caused by all horizontal loads are within the border of the core.

$t_{wall} := 0.09m$ $\quad h_s := 3m$

$t_{floor} := 0.145m$

$b_{panel} := 1.2 \cdot m$

$h_{roof} := 0m$

Wind calculations, Terrain category 0

$$v_b := 25 \frac{m}{s}$$

$c_{e1} := 3.28$ $\quad (z_e = 16)$ $\quad$ exposure factor

$c_{e2} := 3.5$ $\quad (z_e = 24)$

$$\rho := 1.25 \frac{kg}{m^3}$$

$$q_b := 0.5 \cdot \rho \cdot v_b^2 = 0.391 \cdot \frac{kN}{m^2}$$

$q_{p1} := c_{e1} \cdot q_b = 1.281 \cdot \frac{kN}{m^2}$ $\quad$ peak velocity pressure

$q_{p2} := c_{e2} \cdot q_b = 1.367 \cdot \frac{kN}{m^2}$ $\quad$ peak velocity pressure above 16 meter

Pressure coefficents, external walls

$$\frac{h_{building}}{b_{building}} = 1.5$$

Across side wall

$c_{pe.A} := -1.2$

$c_{pe.B} := -0.8$

Along longside wall

$c_{pe.D} := 0.8$

$$c_{pe.E} := -0.7 - 0.3 \cdot \frac{5 - 1.5}{5 - 1} \cdot -0.5 = -0.569$$

flat roof, sharp eaves

$c_{pe.F} := -1.8$

$c_{pe.G} := -1.2$

$c_{pe.H} := -0.7$

$c_{pe.I} := 0.2$

Pressure coefficents, internal wall

$c_{pi} := -0.3$ Area of openings unknown

Characteristic wind load

$$w_{w.A1} := \left(c_{pe.A} - c_{pi}\right) \cdot q_{p1} = -1.153 \cdot \frac{kN}{m^2}$$

$$w_{w.A2} := \left(c_{pe.A} - c_{pi}\right) \cdot q_{p2} = -1.23 \cdot \frac{kN}{m^2}$$

$$w_{w.B1} := \left(c_{pe.B} - c_{pi}\right) \cdot q_{p1} = -0.641 \cdot \frac{kN}{m^2}$$

$$w_{w.B2} := \left(c_{pe.B} - c_{pi}\right) \cdot q_{p2} = -0.684 \cdot \frac{kN}{m^2}$$

$$w_{w.D1} := \left(c_{pe.D} - c_{pi}\right) \cdot q_{p1} = 1.409 \cdot \frac{kN}{m^2}$$

$$w_{w.D2} := (c_{pe.D} - c_{pi}) \cdot q_{p2} = 1.504 \cdot \frac{kN}{m^2}$$

$$w_{w.E1} := (c_{pe.E} - c_{pi}) \cdot q_{p1} = -0.344 \cdot \frac{kN}{m^2}$$

$$w_{w.E2} := (c_{pe.E} - c_{pi}) \cdot q_{p2} = -0.367 \cdot \frac{kN}{m^2}$$

$$w_{w.F} := (c_{pe.F} - c_{pi}) \cdot q_{p2} = -2.051 \cdot \frac{kN}{m^2}$$

$$w_{w.G} := (c_{pe.G} - c_{pi}) \cdot q_{p2} = -1.23 \cdot \frac{kN}{m^2}$$

$$w_{w.H} := (c_{pe.H} - c_{pi}) \cdot q_{p2} = -0.547 \cdot \frac{kN}{m^2}$$

$$w_{w.I} := (c_{pe.I} - c_{pi}) \cdot q_{p2} = 0.684 \cdot \frac{kN}{m^2}$$

$$e_{wind} := \min\left[l_{building}, 2 \cdot (h_{building})\right] = 44\,m$$

$$Q_{wind.F} := w_{w.F} \cdot \left(\frac{e_{wind}}{10} \cdot \frac{e_{wind}}{4} \cdot 2\right) = -198.516 \cdot kN$$

$$Q_{wind.G} := w_{w.G} \cdot \left(\frac{e_{wind}}{10} \cdot \frac{e_{wind}}{2}\right) = -119.109 \cdot kN$$

$$Q_{wind.H} := w_{w.H} \cdot \left(\frac{e_{wind}}{2} - \frac{e_{wind}}{10}\right) \cdot b_{building} = -154 \cdot kN$$

$$Q_{wind.I} := w_{w.I} \cdot \left(\frac{e_{wind}}{2}\right) \cdot b_{building} = 240.625 \cdot kN$$

$$Q_{wind.ver} := (Q_{wind.F} + Q_{wind.G} + Q_{wind.H} + Q_{wind.I}) = -231 \cdot kN$$

$$Q_{wind.side} := (w_{w.D1} + w_{w.E1}) \cdot l_{building} \cdot 16m + (w_{w.D2} + w_{w.E2}) \cdot l_{building} \cdot 8m = 1.15 \cdot MN$$

$$Q_{wind.acrosside} := \left(w_{w.A1} \cdot \frac{b_{building}}{5} + w_{w.B1} \cdot \frac{4b_{building}}{5}\right) \cdot 16m \ldots = -393.24 \cdot kN$$
$$+ \left(w_{w.A2} \cdot \frac{b_{building}}{5} + w_{w.B2} \cdot \frac{4b_{building}}{5}\right) \cdot 16m$$

$$\gamma_Q := 1.5$$

$$e_{tilt} := \frac{b_{building}}{6} = 2.667\,m$$

$$M_{Sd} := \gamma_Q \cdot \begin{pmatrix} F_{h8} \cdot 8 \cdot h_{storey} + F_{h7} \cdot 7 \cdot h_{storey} + F_{h6} \cdot 6 \cdot h_{storey} + F_{h5} \cdot 5 \cdot h_{storey} \ldots \\ + F_{h4} \cdot 4 \cdot h_{storey} + F_{h3} \cdot 3 \cdot h_{storey} + F_{h2} \cdot 2 \cdot h_{storey} + F_{h1} \cdot h_{storey} \end{pmatrix} \ldots = 3.904 \times 10^4 \cdot kN \cdot m$$
$$+ -\gamma_Q \cdot Q_{wind.ver} \cdot e_{tilt} + \gamma_Q \cdot Q_{wind.side} \cdot h_{storey} \cdot 4$$

$$M_{Sd.f} := \gamma_Q \cdot \begin{pmatrix} F_{h8.f} \cdot 8 \cdot h_{storey} + F_{h7.f} \cdot 7 \cdot h_{storey} + F_{h6.f} \cdot 6 \cdot h_{storey} \ldots \\ + F_{h5.f} \cdot 5 \cdot h_{storey} + F_{h4.f} \cdot 4 \cdot h_{storey} + F_{h3.f} \cdot 3 \cdot h_{storey} \ldots \\ + F_{h2.f} \cdot 2 \cdot h_{storey} + F_{h1.f} \cdot h_{storey} \end{pmatrix} \ldots = 2.626 \times 10^4 \cdot kN \cdot m$$
$$+ -\gamma_Q \cdot Q_{wind.ver} \cdot e_{tilt} + \gamma_Q \cdot Q_{wind.side} \cdot h_{storey} \cdot 4$$

$$H_{Sd} := \gamma_Q \cdot \left(F_{h8} + F_{h7} + F_{h6} + F_{h5} + F_{h4} + F_{h3} + F_{h2} + F_{h1}\right) = 1.738 \cdot MN$$

$$H_{Sd.f} := \gamma_Q \cdot \left(F_{h8.f} + F_{h7.f} + F_{h6.f} + F_{h5.f} + F_{h4.f} + F_{h3.f} + F_{h2.f} + F_{h1.f}\right) = 0.47 \cdot MN$$

<u>Resisting moment against tilting</u>

Horizontal distribution of wind load on stabilizing walls

All walls are of same thickness and material, rotational centre is in the middle of the building.
The horizontal distribution of wind load is affected mainly by the effective length of the walls.

Number of floors

$b_{window} := 1m$ $\qquad$ $b_{door} := 1.5m$

$A_{window} := 1.5m^2$ $\qquad$ $A_{door} := 3m^2$

$n_{window} := 6$

$wall_1 := 16m$

$wall_2 := 4m$

$wall_3 := 8m$

$l_{ef.1} := wall_1 = 16\,m$ $\quad n_1 := 2$ $\quad$ wall no.1

$l_{ef.2} := wall_1 = 16\,m$ $\quad n_2 := 1$ $\quad$ wall no. 2

$l_{ef.3} := wall_2 = 4\,m$ $\quad n_3 := 4$ $\quad$ walls no. 4-6

$l_{ef.4} := wall_3 = 8\,m$ $\quad n_4 := 4$ $\quad$ walls no. 8-11

$$l_{ef.tot} := l_{ef.1} \cdot n_1 + l_{ef.2} \cdot n_2 + l_{ef.3} \cdot n_3 + l_{ef.4} \cdot n_3 = 96\,m$$

$$\begin{aligned} G_{selfw} := {} & G_{k.roof} \cdot b_{building} \cdot l_{building} \cdots \\ & + G_{k.floor} \cdot 8 \cdot l_{building} \cdot b_{building} \cdots \\ & + G_{k.exwall} \cdot \left(wall_1 \cdot n_1 + l_{building} \cdot 2\right) \cdot n_{floor} \cdot h_{storey} \cdots \\ & + G_{k.divwall} \cdot n_{floor} \cdot \left(wall_1 \cdot n_2 + wall_2 \cdot n_3 + wall_3 \cdot n_4 + 2 \cdot l_{building}\right) \cdot h_{storey} \cdots \\ & + G_k \cdot l_{building} \cdot b_{building} \cdot 0.3\,m \end{aligned} \qquad = 12.966 \cdot MN$$

$\gamma_{fav} := 0.9$

$\gamma_{unfav} := 1.1$

$G_{dim.f} := \gamma_{fav} \cdot G_{selfw} = 11.67 \cdot MN$

$G_{dim} := \gamma_{unfav} \cdot G_{selfw} = 14.263 \cdot MN$

$M_{Rd.f} := G_{dim.f} \cdot e_{tilt} = 3.112 \times 10^4 \cdot kN \cdot m$

$M_{Rd} := G_{dim} \cdot e_{tilt} = 3.803 \times 10^4 \cdot kN \cdot m$

$\dfrac{M_{Sd}}{M_{Rd}} = 102.634 \cdot \%$ NOT OK!!

The moment of resistance is not enough against tilting, there would be a need for tying the foundation to the groundwork

$\dfrac{M_{Sd.f}}{M_{Rd.f}} = 84.38 \cdot \%$ OK!!

Resisting friction force against sliding

$\phi_{friction} := 35 \cdot deg$ Friction of soil for sand

$$\tau := \frac{G_{dim} - 1.5 \cdot Q_{wind.ver}}{l_{building} \cdot b_{building}} \cdot \tan\left(\phi_{friction}\right) = 14.531 \cdot \frac{kN}{m^2}$$

$$\tau_f := \frac{G_{dim.f} - 1.5 \cdot Q_{wind.ver}}{l_{building} \cdot b_{building}} \cdot \tan\left(\phi_{friction}\right) = 11.952 \cdot \frac{kN}{m^2}$$

$$H_{Rd} := \tau \cdot l_{building} \cdot b_{building} = 10.23 \cdot MN$$

$$H_{Rdf} := \tau_f \cdot l_{building} \cdot b_{building} = 8.414 \cdot MN$$

$$\frac{H_{Sd}}{H_{Rd}} = 16.992 \cdot \%$$

The structure is safe against sliding

$$\frac{H_{Sd.f}}{H_{Rdf}} = 5.586 \cdot \%$$

Tilting

Appendix B- Lateral deflection

Input

Geometry

$h := 24m$

$b := 40m$

$d := 16m$

$h_s := 3m$

$n_{floor} := 8$

$t_{floor} := 145mm$

$t_{wall} := 95mm$

$\rho_{CLT} := 550 \frac{kg}{m^3}$

$A_{floor} := b \cdot d = 640\, m^2$

$A_{wall} := 2 \cdot d \cdot h_s + 2 \cdot b \cdot h_s + 2 \cdot \left(6 \cdot 8 \cdot m \cdot h_s + 2 \cdot 4 \cdot m \cdot h_s\right) = 672\, m^2$

$A_{CLT} := \left(A_{floor} + A_{wall}\right) \cdot n_{floor} = 1.05 \times 10^4\, m^2$

$V_{floor} := A_{floor} \cdot t_{floor} = 92.8 \cdot m^3$

$V_{wall} := A_{wall} \cdot t_{wall} = 63.84 \cdot m^3$

$V_{CLT} := n_{floor} \cdot \left(V_{wall} + V_{floor}\right) = 1.253 \times 10^3 \cdot m^3$

Mass

$m_{tot} := V_{CLT} \cdot \rho_{CLT} = 759.731 \cdot ton$

$m_{FEM} := 800ton$

Self-weight

$g_{FEM} := \frac{\left(m_{FEM} \cdot g\right)}{h} = 296.548 \cdot \frac{kN}{m}$

$g_{self} := \frac{m_{tot} \cdot g}{h} = 281.621 \cdot \frac{kN}{m}$

Input

Bending and shear stiffness

Bending stiffness

The mean bending stiffness is approximated to be uniformly distributed over the height of the cantilever structure.

$\gamma_M := 1.3$

$k_{def} := 0.6$

Distances from rotational to centre of wall

$l_{wall} := 4m$ $\quad x_{z1} := 4m$ $\quad z_{wall.1} := 6m$

$b_{wall} := 4m$ $\quad x_{z2} := 8m$ $\quad z_{wall.2} := 2m$

$b_{wall.3} := 8m$ $\quad x_{z.3} := 4m$

$x_{z.shaft} := 2m$

Material properties from FEM-design

$E_{0.mean.FEM} := 5700MPa$

$E_{90.mean.FEM} := 3900MPa$

$$E_{d.0.mean.FEM} := \frac{E_{0.mean.FEM}}{(1 + k_{def})} = 3.563 \times 10^9 \, Pa$$

$$E_{d.90.mean.FEM} := \frac{E_{90.mean.FEM}}{(1 + k_{def})} = 2.438 \times 10^9 \, Pa$$

$$G_{mean.FEM} := \frac{110}{(1 + k_{def})} MPa$$

Dimensions

$$t_{layer} := \frac{t_{wall}}{5} = 0.019\,m$$

$A_{cross} := t_{wall} \cdot d = 1.52\,m^2$ $\qquad A_{along} := t_{wall} \cdot b = 3.8\,m^2$

$A_{shaftwall} := t_{wall} \cdot (2 \cdot x_{z2} + 2 \cdot x_{z1}) = 2.28\,m^2$ $\qquad A_4 := (b - x_{z.shaft} \cdot 4) \cdot t_{wall} = 3.04\,m^2$

Second moment of inertia

$$I_{wall.1} := \frac{t_{wall} \cdot d^3}{12} = 32.427 \cdot m^4$$

Gable wall

$$I_{wall.2} := \frac{b \cdot t_{wall}^{\;3}}{12} + b \cdot t_{wall} \cdot x_{z2}^{\;2} = 243.203\,m^4$$

Along side wall

$$I_{shaft} := 2 \cdot \frac{t_{wall} \cdot l_{wall}^{3}}{12} + 2 \cdot t_{wall} \cdot l_{wall} \cdot z_{wall.1}^{2} + 2 \cdot \frac{t_{wall} \cdot b_{wall.3}^{3}}{12} \cdots + 2 \cdot \frac{b_{wall} \cdot t_{wall}^{3}}{12} + 2 \cdot t_{wall} \cdot b_{wall} \cdot z_{wall.2}^{2} = 39.521\,m^{4}$$

Across wall with elevatorshaft included

$$I_{wall.3} := \frac{b \cdot t_{wall}^{3}}{12} + b \cdot t_{wall} \cdot x_{z.3}^{2} = 60.803\,m^{4}$$

Inner wall along-side

$$I_{wall.4} := \frac{\left(b - x_{z.shaft} \cdot 4\right) \cdot t_{wall}^{3}}{12} = 2.286 \times 10^{-3}\,m^{4}$$

Mid inner wall

$$I_{tot} := 3 \cdot I_{wall.1} + 2 \cdot I_{shaft} + 2 \cdot I_{wall.2} + I_{wall.3} \cdot 2 + I_{wall.4} = 784.335\,m^{4}$$

Elastic and shear stiffness with material properties from FEM-design

$$EI_{tot} := E_{d.0.mean.FEM} \cdot \left(3 \cdot I_{wall.1} + 2 \cdot I_{shaft}\right) \cdots + E_{d.0.mean.FEM} \cdot \left(2 I_{wall.2} + 2 \cdot I_{wall.3} + I_{wall.4}\right) = 2.794 \times 10^{3} \cdot GN \cdot m^{2}$$

$$GA_{tot} := 3 \cdot A_{cross} \cdot G_{mean.FEM} + 2 \cdot A_{shaftwall} \cdot G_{mean.FEM} \cdots + 2 \cdot A_{along} \cdot G_{mean.FEM} + \left(2 \cdot A_{along} + A_{4}\right) \cdot G_{mean.FEM} = 1.881\,m^{2} \cdot GPa$$

Bending and shear stiffness

Windload

Wind calculations, Terrain category 0

$$v_{b} := 19.64 \frac{m}{s}$$

$$h_{w} := 8m$$

$c_{e} := 3.5$ ($z_{e} = H$) exposure factor

$$\rho := 1.25 \frac{kg}{m^{3}}$$

$$q_{b} := 0.5 \cdot \rho \cdot v_{b}^{2} = 0.241 \cdot \frac{kN}{m^{2}}$$

$$q_p := c_e \cdot q_b = 0.844 \cdot \frac{kN}{m^2}$$ peak velocity pressure

Alongside wall

$$c_{pe.D} := 0.8$$

$$c_{pe.E} := -0.7 - 0.3 \cdot \frac{5 - 1.5}{5 - 1} \cdot -0.5 = -0.569$$

Pressure coefficents, internal wall

$$c_{pi} := -0.3$$

Characteristic wind load

$$q_{w.D} := \left(c_{pe.D} - c_{pi}\right) \cdot q_p = 0.928 \cdot \frac{kN}{m^2}$$

$$q_{w.E} := \left(c_{pe.E} - c_{pi}\right) \cdot q_p = -0.227 \cdot \frac{kN}{m^2}$$

Along-side wall D, uniformly distributed wind load acting on each floor

$$q_7 := \frac{1}{2} \cdot h_s \cdot q_{w.D} = 1.392 \cdot \frac{kN}{m}$$

$$q_6 := h_s \cdot q_{w.D} = 2.784 \cdot \frac{kN}{m}$$

$$q_5 := q_6 = 2.784 \cdot \frac{kN}{m}$$

$$q_4 := q_6$$

$$q_3 := q_6$$

$$q_2 := q_6$$

$$q_1 := q_6$$

$$q_0 := q_6$$

$$q_{tot} := 7q_6 + q_7 = 20.884 \cdot \frac{kN}{m}$$ total wind load on bottom floor

Windload

Deflection whole structure, Material properties from FEM-design

$$z_{floor} := 0, 3m .. 21m = \begin{array}{|r|}\hline 0 \\ \hline 3 \\ \hline 6 \\ \hline 9 \\ \hline 12 \\ \hline 15 \\ \hline 18 \\ \hline 21 \\ \hline \end{array}\ m$$

$$q_{floor} := \begin{pmatrix} q_{tot} \\ 7 \cdot q_0 \\ 6 \cdot q_0 \\ 5q_0 \\ 4 \cdot q_0 \\ 3 \cdot q_0 \\ 2q_0 \\ q_0 \end{pmatrix}$$

$$\delta_{shear.floor} := \begin{bmatrix} q_{floor_0} \cdot \left[\dfrac{\left(h - z_{floor_0}\right)^2}{GA_{tot}}\right] \\ q_{floor_1} \cdot \left[\dfrac{\left(h - z_{floor_1}\right)^2}{GA_{tot}}\right] \\ q_{floor_2} \cdot \left[\dfrac{\left(h - z_{floor_2}\right)^2}{GA_{tot}}\right] \\ q_{floor_3} \cdot \left[\dfrac{\left(h - z_{floor_3}\right)^2}{GA_{tot}}\right] \\ q_{floor_4} \cdot \left[\dfrac{\left(h - z_{floor_4}\right)^2}{GA_{tot}}\right] \\ q_{floor_5} \cdot \left[\dfrac{\left(h - z_{floor_5}\right)^2}{GA_{tot}}\right] \\ q_{floor_6} \cdot \left[\dfrac{\left(h - z_{floor_6}\right)^2}{GA_{tot}}\right] \\ q_{floor_7} \cdot \left[\dfrac{\left(h - z_{floor_7}\right)^2}{GA_{tot}}\right] \end{bmatrix} = \begin{pmatrix} 6.395 \\ 4.57 \\ 2.878 \\ 1.665 \\ 0.853 \\ 0.36 \\ 0.107 \\ 0.013 \end{pmatrix} \cdot mm$$

$$\delta_{bend.floor} := \begin{bmatrix} q_{floor_0} \cdot \left[\frac{\left(h - z_{floor_0}\right)^4}{EI_{tot}} \right] \\ q_{floor_1} \cdot \left[\frac{\left(h - z_{floor_1}\right)^4}{EI_{tot}} \right] \\ q_{floor_2} \cdot \left[\frac{\left(h - z_{floor_2}\right)^4}{EI_{tot}} \right] \\ q_{floor_3} \cdot \left[\frac{\left(h - z_{floor_3}\right)^4}{EI_{tot}} \right] \\ q_{floor_4} \cdot \left[\frac{\left(h - z_{floor_4}\right)^4}{EI_{tot}} \right] \\ q_{floor_5} \cdot \left[\frac{\left(h - z_{floor_5}\right)^4}{EI_{tot}} \right] \\ q_{floor_6} \cdot \left[\frac{\left(h - z_{floor_6}\right)^4}{EI_{tot}} \right] \\ q_{floor_7} \cdot \left[\frac{\left(h - z_{floor_7}\right)^4}{EI_{tot}} \right] \end{bmatrix} = \begin{pmatrix} 2.48 \\ 1.357 \\ 0.628 \\ 0.252 \\ 0.083 \\ 0.02 \\ 2.583 \times 10^{-3} \\ 8.072 \times 10^{-5} \end{pmatrix} \cdot mm$$

$$\delta_{tot} := \left(\delta_{bend.floor_0} + \delta_{shear.floor_0}\right) = 8.875 \cdot mm$$ total deflection at top

$$\delta_{FEM.lineload} := 9.2528mm$$

$$\delta_{FEM.cover} := 12.2076mm$$

Deflections obtained from FEM-design

$$u_{tot.FEM.lineload} := 1 - \frac{\delta_{FEM.lineload}}{\delta_{tot}} = -4.261 \cdot \%$$

$$u_{tot.FEMcover} := 1 - \frac{\delta_{FEM.cover}}{\delta_{tot}} = -37.556 \cdot \%$$

Deflection whole structure, Material properties from FEM-design

Appendix C - Fundamental frequency

Input

$h := 24m$
$b := 40m$
$d := 16m$

$h_s := 3m$
$n_{floor} := 8$

$t_{floor} := 145mm$
$t_{wall} := 95mm$
$t_{btg} := 500mm$

Mass

$$m_{FEM} := 1386000 \cdot kg$$

Self-weight

$$g_{FEM} := \frac{m_{FEM} \cdot g}{h} = 566.334 \cdot \frac{kN}{m}$$

Input

Bending and shear stiffness

The mean bending stiffness is approximated to be uniformly distributed over the height of the cantilever obtained from stiffness calculations in Appendix B, Lateral deflection.

$$EI_{tot} := 3628GN \cdot m^2$$

$$GA_{tot} := 1.881m^2 \cdot GPa$$

Bending and shear stiffness

EC

Maximum displacement the building seen as a cantilever with mass applied in direction of vibration.

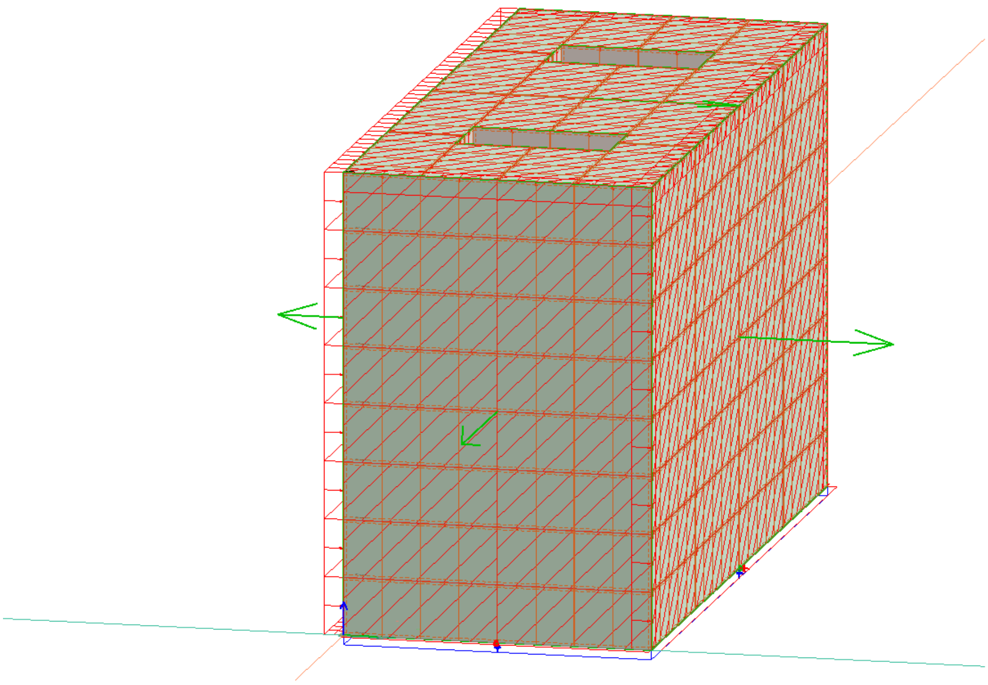
$$x_1 := \frac{g_{FEM} \cdot h^4}{8 \cdot EI_{tot}} + \frac{g_{FEM} \cdot h^2}{2 \cdot GA_{tot}} = 93.185 \cdot mm$$

$$n_{EC} := \frac{1}{2\pi} \cdot \sqrt{\frac{g}{x_1}} = 1.633 \cdot Hz$$

EC

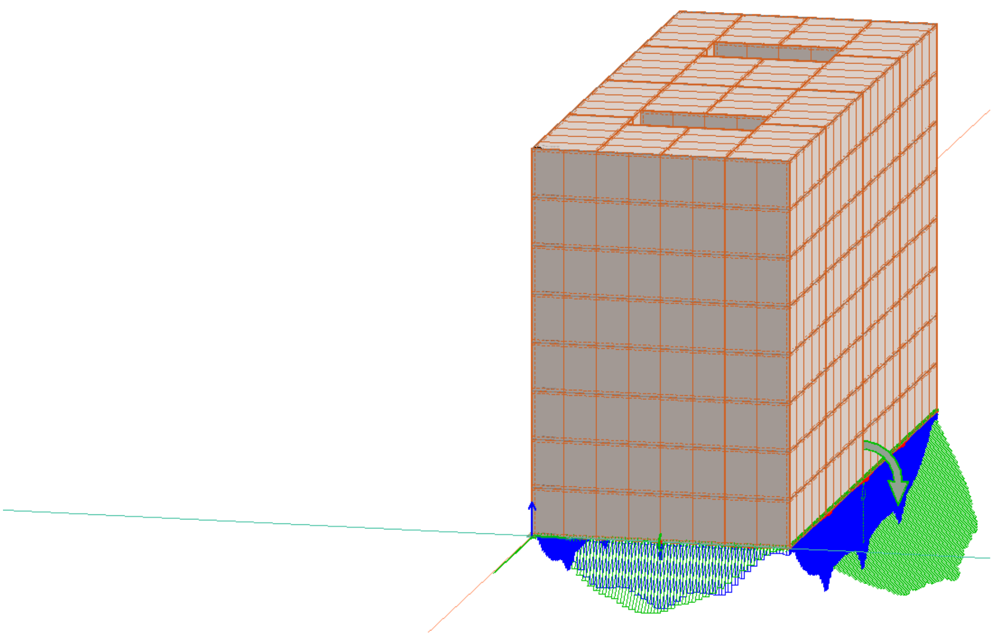
Appendix D- Pictures of model

Eurocode (NA: Swedish)

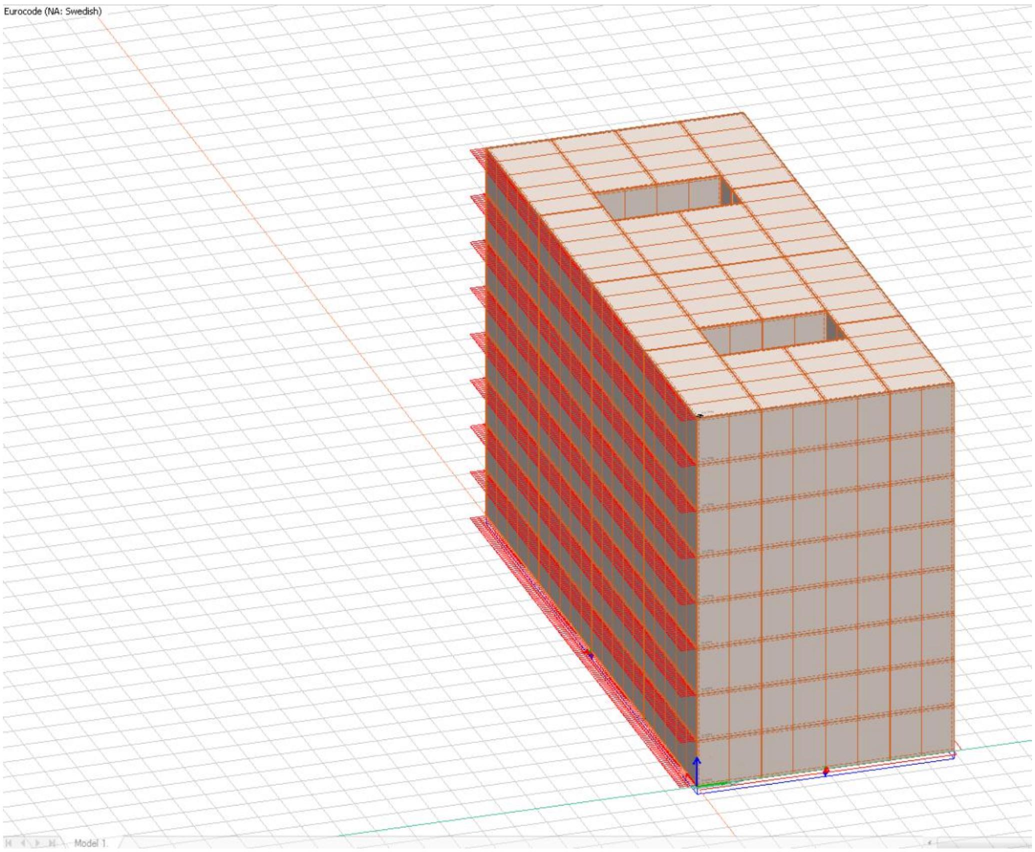


Load-distribution along-side wall

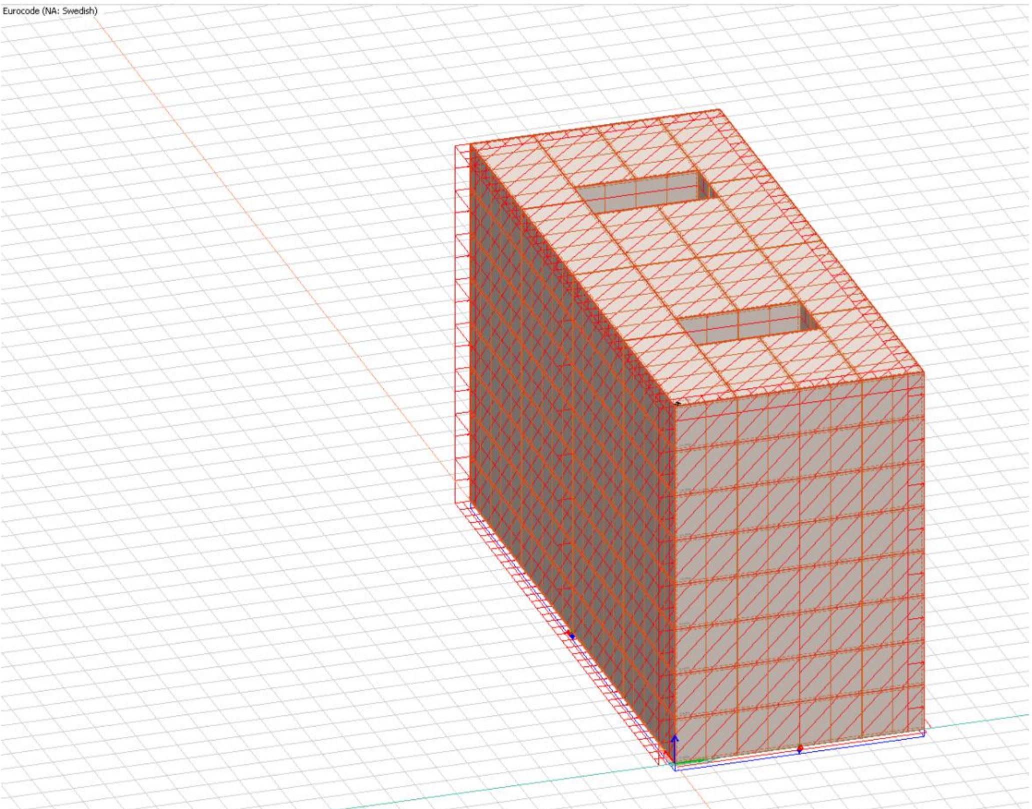
Eurocode (NA: Swedish) code: 1st order theory - Load combinations - ULS - Reactions - [kN, kNm, kN/m, kNm/m, kN/m2]



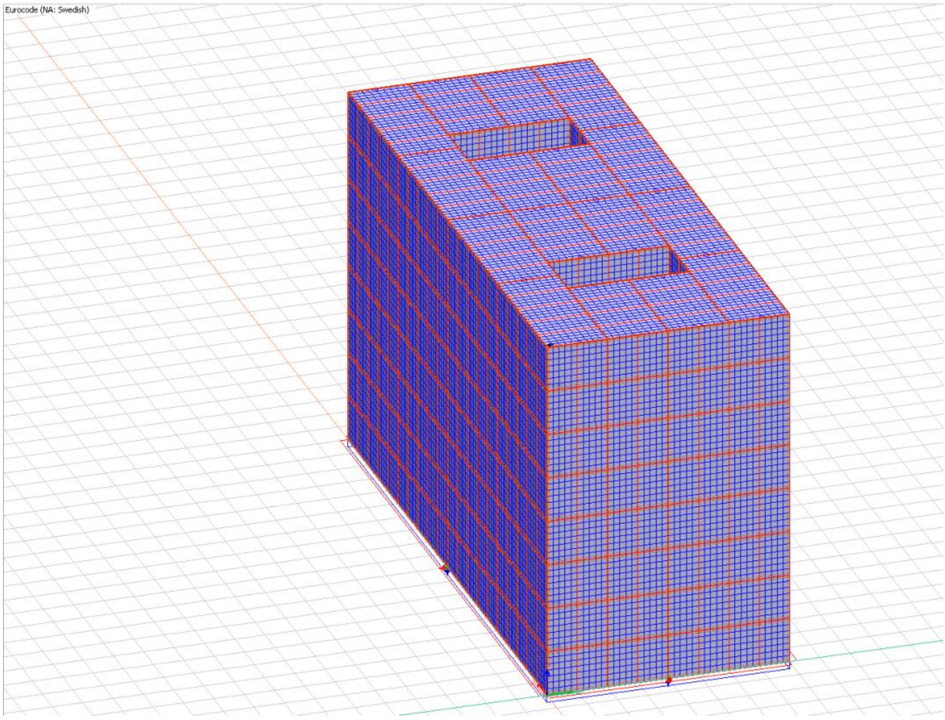
Reaction forces



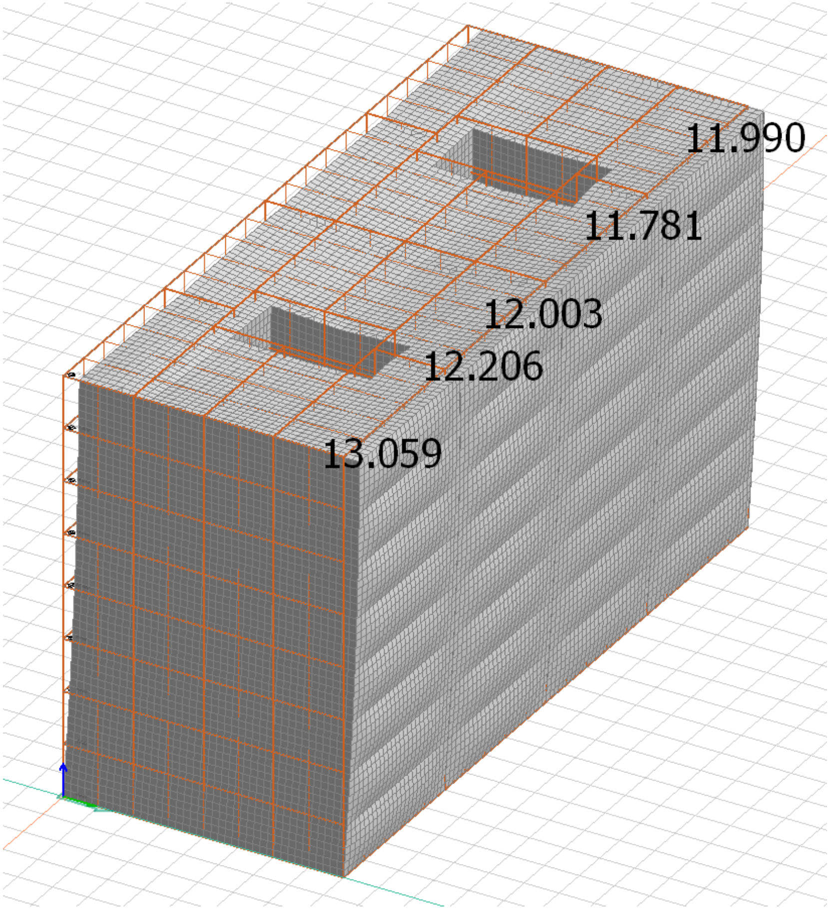
Wind load, line load



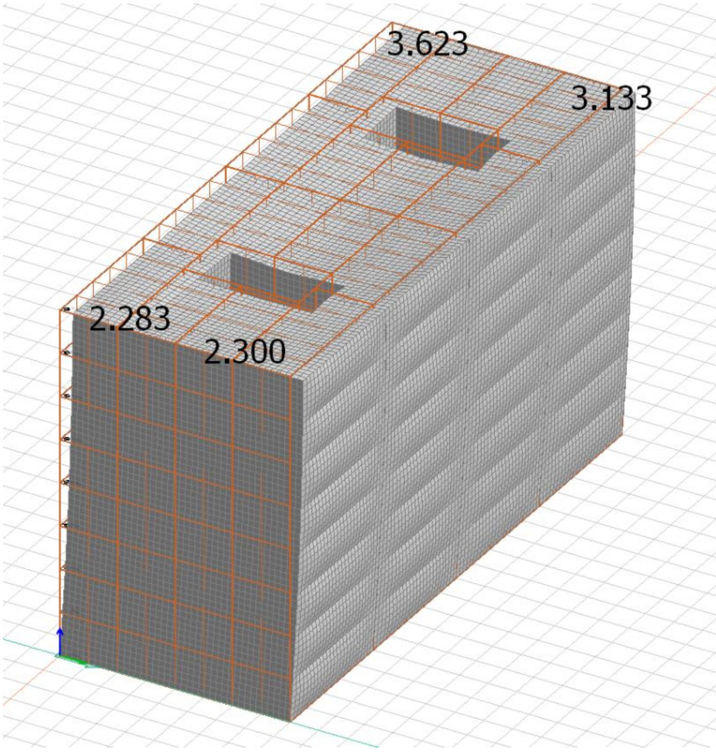
Wind load, surface cover load



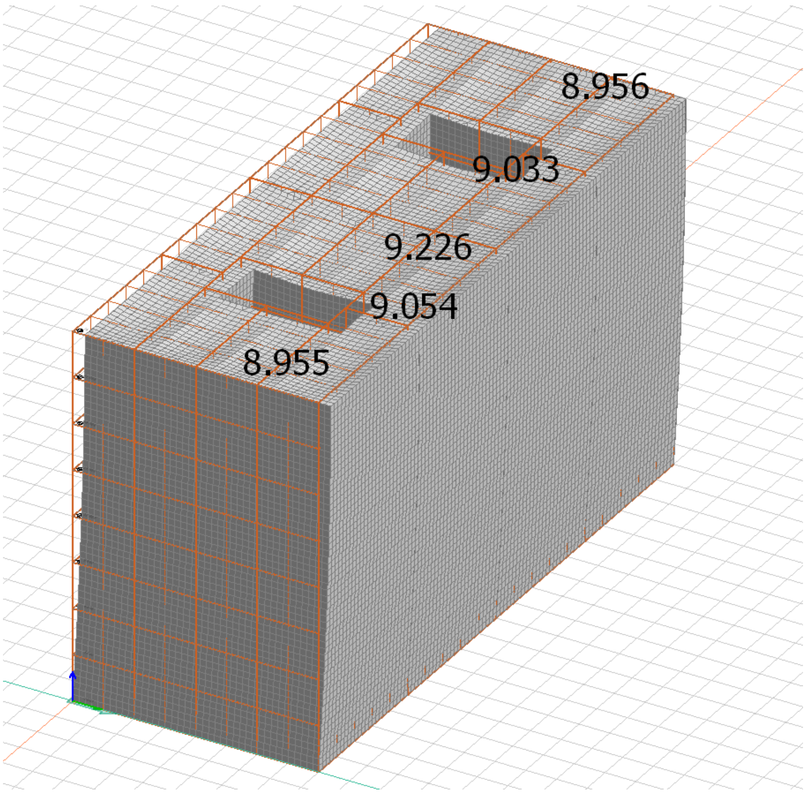
Mesh



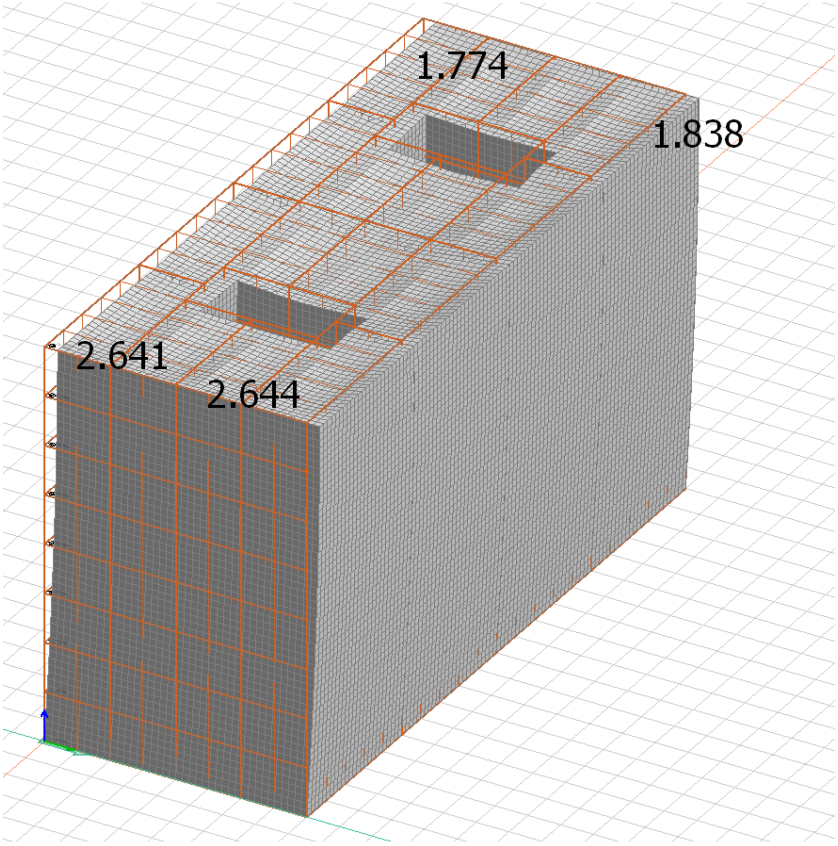
Deflection SLS x-direction, Cover



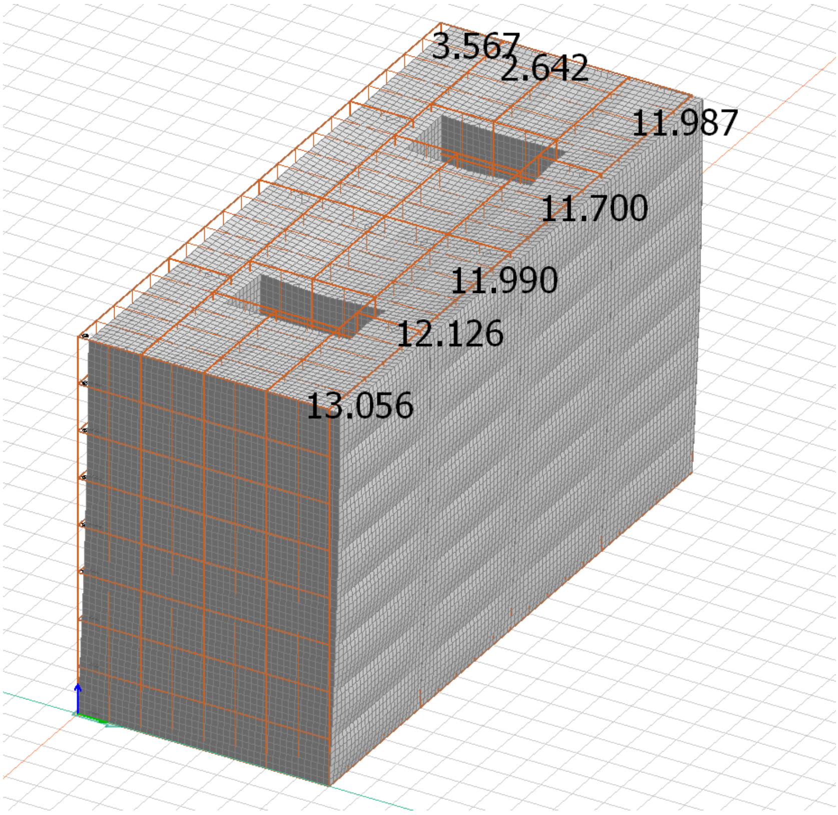
Deflection SLS y-direction, Cover



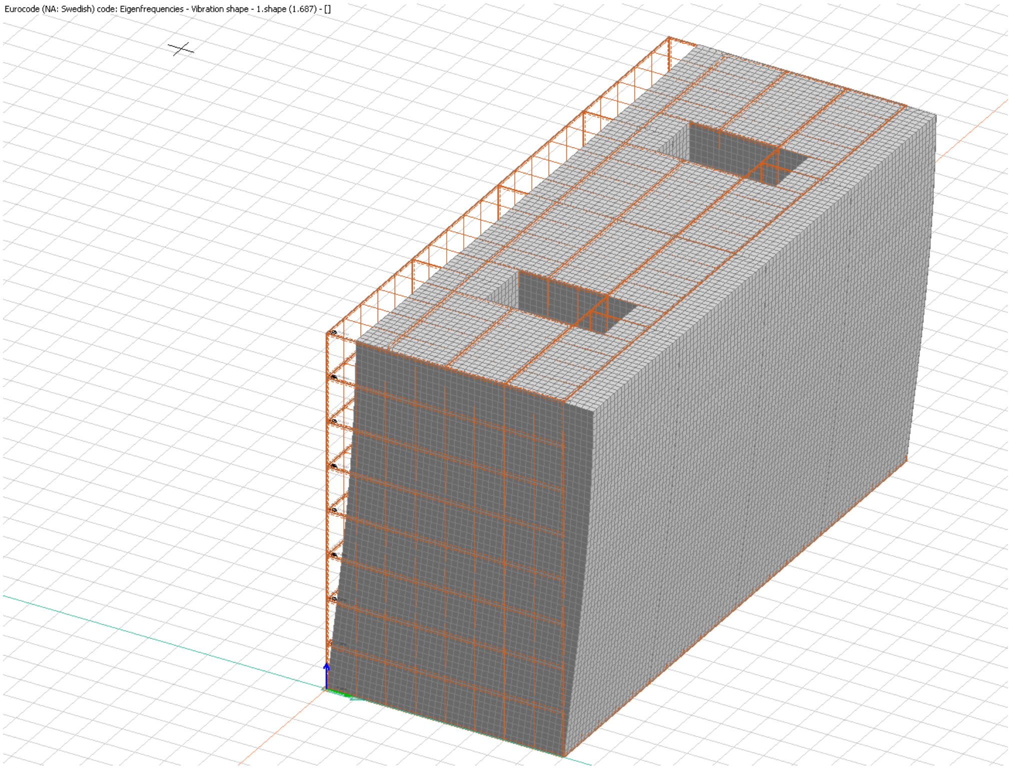
Deflection SLS x-direction, line-load



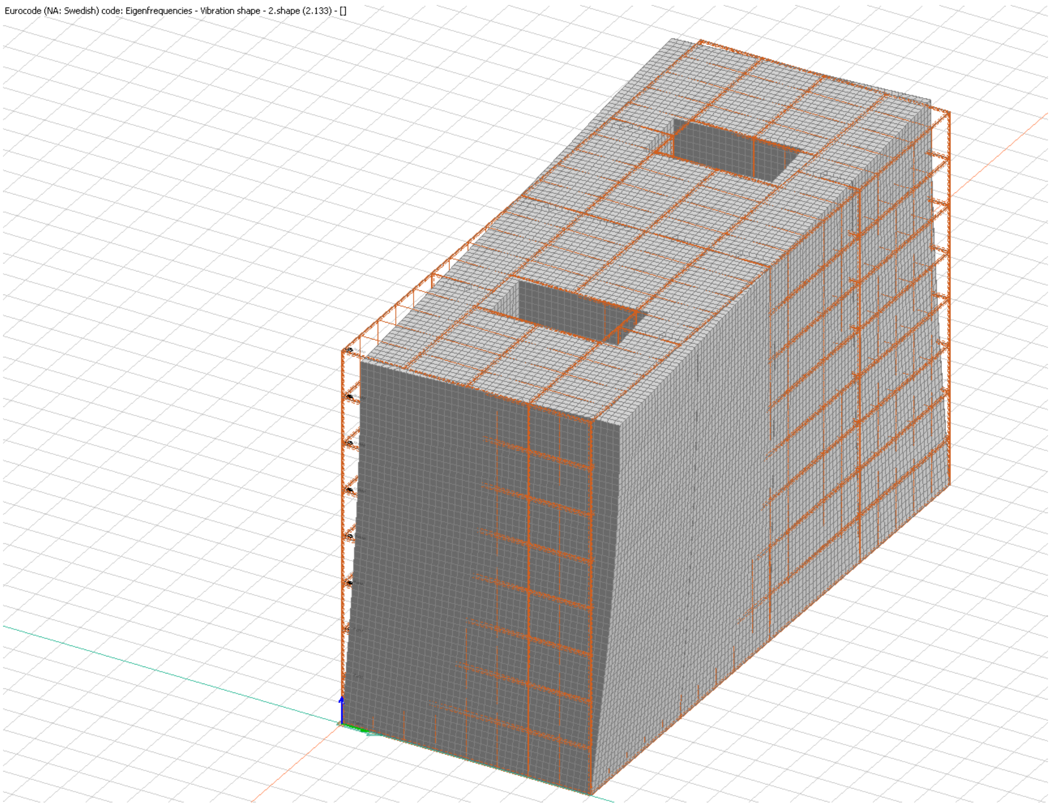
Deflection SLS y-direction, Cover



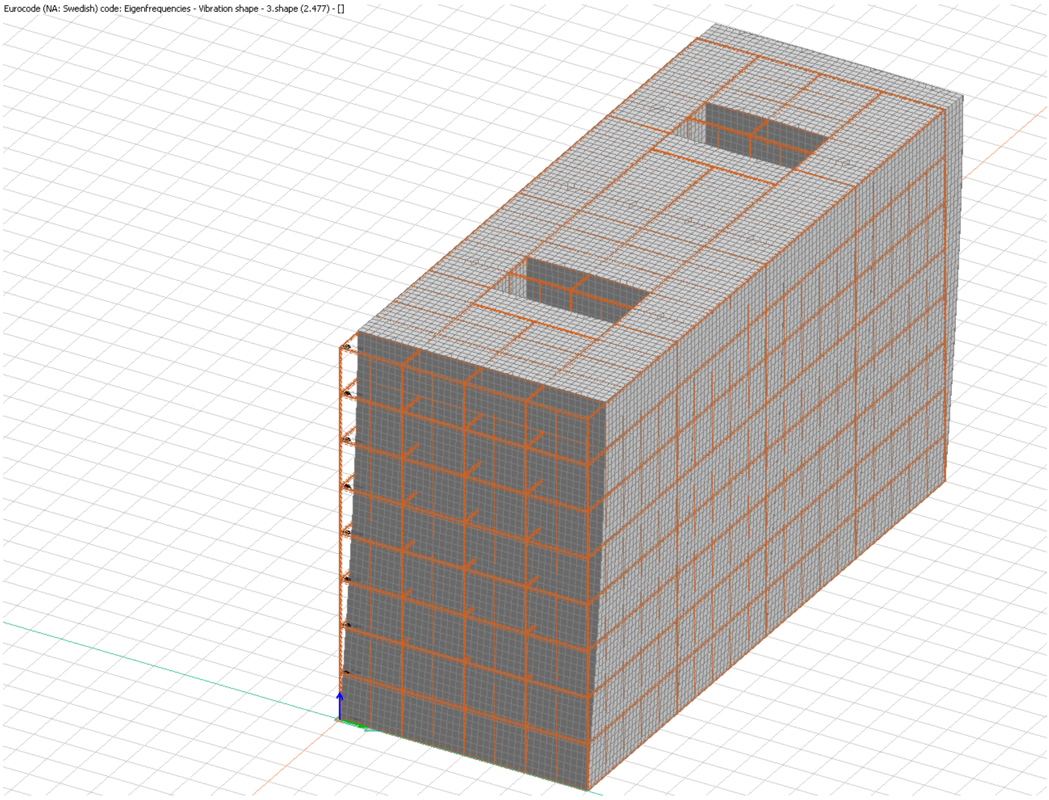
Deflection ULS in x and y direction



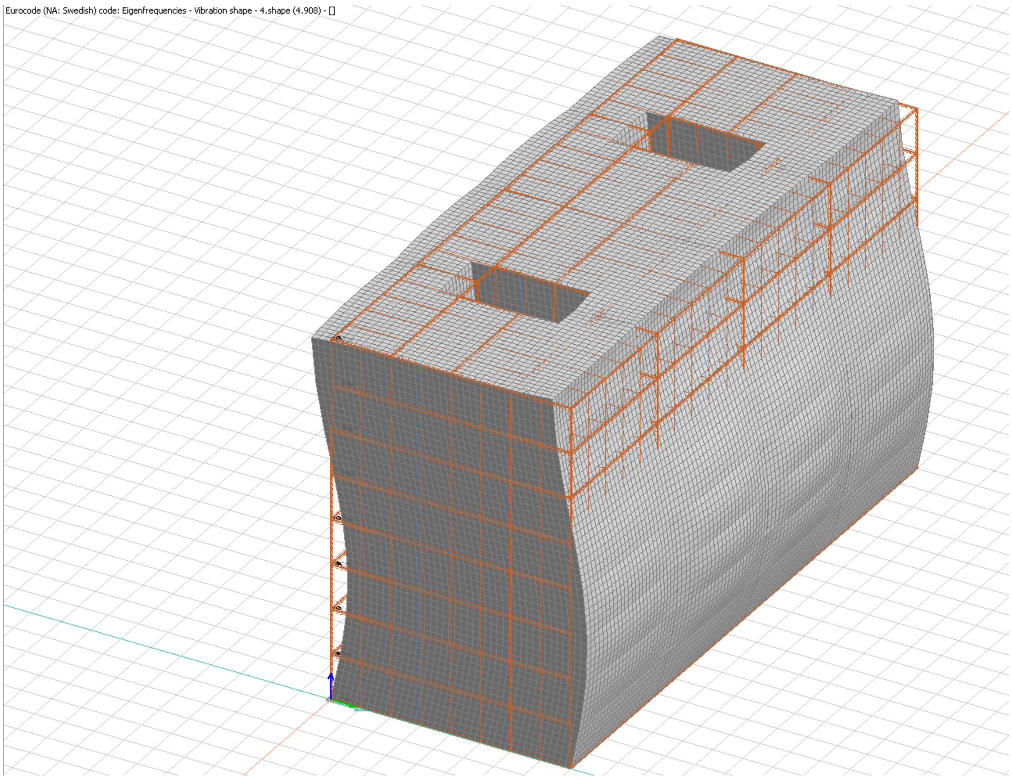
First natural frequency, 1.687 Hz



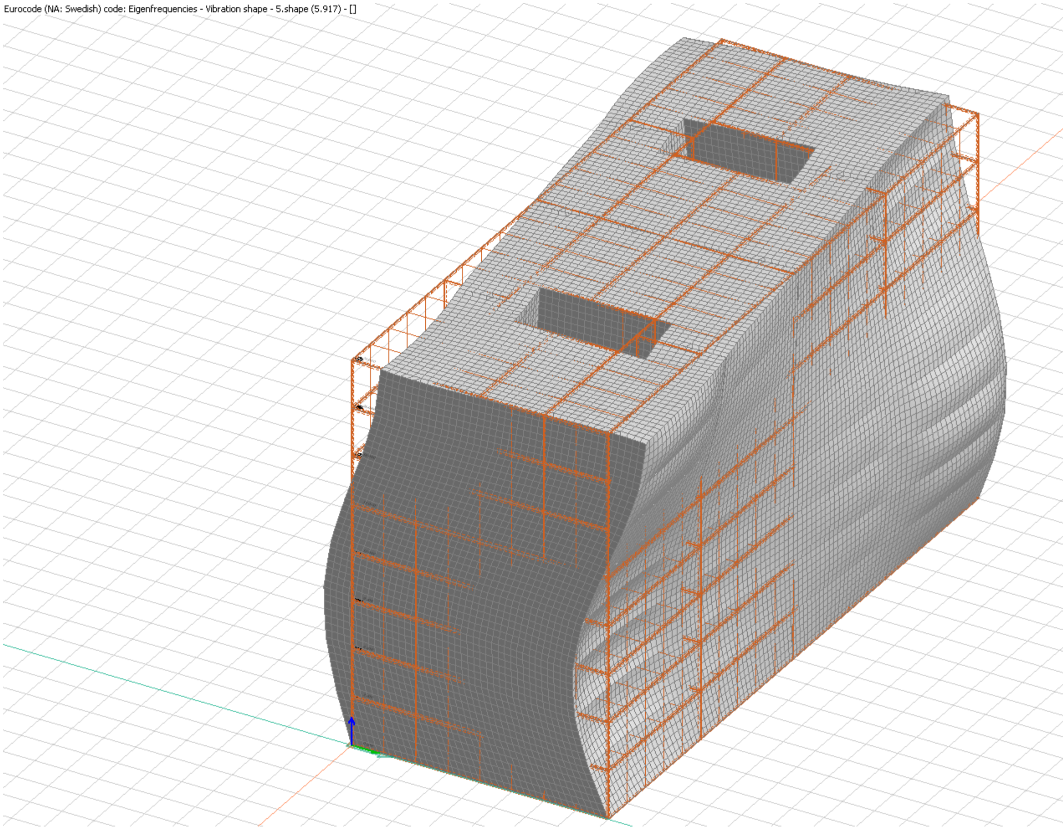
Second frequency, 2.133 Hz



Third frequency, 2.477 Hz



Fourth frequency, 4.908 Hz



Fifth frequency, 5.917 Hz

Appendix E- Dynamic analysis according to EC and national annex and ISO

Input

Building dimensions

$h := 24\text{m}$
$b := 40\text{m}$
$d := 16\text{m}$

$\rho_{air} := 1.25 \frac{\text{kg}}{\text{m}^3}$ density of air

$\frac{h}{d} = 1.5$

Wind speed

Reference wind speed, for a 50 year return period in Gothenbourg

$v_b := 25 \frac{\text{m}}{\text{s}}$

wind speed for a 5 year wind, which should be used in comfort critera investigation according to national annex of EC1-4.

$T_a := 5$ years

$$v_{Ta} := 0.75 \cdot v_b \cdot \sqrt{\left[1 - 0.2 \cdot \ln\left[-\ln\left[1 - \left(\frac{1}{T_a}\right)\right]\right]\right]} = 21.378 \frac{\text{m}}{\text{s}}$$

$v_{m.skepp} := 5 \frac{\text{m}}{\text{s}}$ mean wind speed, 1 year return period

$v_{ref.50} := 40 \frac{\text{m}}{\text{s}}$ reference 3 s wind with a return period of 50 years over a flat open country terrain

$T_{a.ISO} := 1.000001$ years 1 year return period

$$v_{ref.1.600} := 0.75 \cdot v_b \cdot \sqrt{\left[1 - 0.2 \cdot \ln\left[-\ln\left[1 - \left(\frac{1}{T_{a.ISO}}\right)\right]\right]\right]} = 12.92 \frac{\text{m}}{\text{s}}$$ reference 10 min wind for a return period of 1 year

$v_{ref.1.3600} := v_{ref.1.600} \cdot 0.95 = 12.274 \frac{\text{m}}{\text{s}}$ reference 1 hour wind for a return period of 1 year

Wind pressure

$q_b := 0.5 \cdot \rho_{air} \cdot v_b^2 = 390.625\ \text{Pa}$

$q_{b.Ta} := 0.5 \cdot \rho_{air} \cdot v_{Ta}^2 = 285.642\ \text{Pa}$

$c_{pe.10} := 0.8$

$q_p := c_{pe.10} \cdot q_b = 312.5\ \text{Pa}$

$q_{p.Ta} := c_{pe.10} \cdot q_{b.Ta} = 228.514\,\text{Pa}$

<u>Data from FEMdesign model</u>

$n_1 := 1.687\text{Hz}$ — first natural frequency, from FEM analysis

$m_{tot} := 1418 \cdot \text{ton} = 1.286 \times 10^6\,\text{kg}$ — total mass of building

Mass at top of building, where largest ampitude of mode occurs

$\mu_e := \frac{m_{tot}}{b \cdot d} = 2.01 \times 10^3 \frac{\text{kg}}{\text{m}^2}$ — mass per unit area

$m_e := \frac{m_{tot}}{\frac{h}{3}} = 1.608 \times 10^5 \frac{\text{kg}}{\text{m}}$ — mass per unit length, of 1/3 of building height figure F.1

Height above ground

$z := h$

$z_s := 0.6 \cdot h = 14.4\,\text{m}$ — reference height according to figure 6.1 EC-1-4

Roughness lenths, dependent of terrain category

$z_0 := 0.003\text{m}$ — terrain category 0, sea or costal areas

$z_{0.2} := 0.05\text{m}$

$z_{min} := 1\text{m}$ $\quad z_{max} := 200\text{m}$

Input

Mean wind velocity

Mean wind speed, variation with height above terrain. at z=h

$k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.2}}\right)^{0.07} = 0.156$ — terrain factor

$c_r := k_r \cdot \ln\left(\frac{z}{z_0}\right) = 1.402$ — roughness factor

$c_o := 1$ — orography factor, change in terrain may increase wind speed. Hills, cliffs etc. are assumed to have no effect here.

mean wind velocity

$v_{m.Ta} := c_r \cdot c_o \cdot v_{Ta} = 29.979 \frac{\text{m}}{\text{s}}$ $\quad T_a = 5$ years

$$v_{skepp} := \frac{v_{m.skepp}}{c_r \cdot c_o} = 3.566 \frac{m}{s}$$

$$q_{b.skepp} := 0.5 \cdot \rho_{air} \cdot v_{skepp}^{\ 2} = 7.946 \cdot Pa$$

$$q_{p.skepp} := c_{pe.10} \cdot q_{b.skepp} = 6.356\,Pa$$

$$v_{mean} := \begin{pmatrix} v_{m.Ta} \\ v_{m.skepp} \end{pmatrix} \qquad v := \begin{pmatrix} v_{Ta} \\ v_{skepp} \end{pmatrix} \qquad q_{pv} := \begin{pmatrix} q_{p.Ta} \\ q_{p.skepp} \end{pmatrix}$$

Mean wind velocity

Wind turbulence

$k_l := 1$ — turbulence factor, recomended value.

$L_t := 300m$ $z_t := 200m$ — reference turbulence length and height

Standard deviation of turbulence

$$\sigma_v := k_r \cdot v \cdot k_l = \begin{pmatrix} 3.336 \\ 0.556 \end{pmatrix} \frac{m}{s}$$

Gust wind size and Wind turbulence, at z=h

$$I_v := \frac{\sigma_v}{v_{mean}} = \begin{pmatrix} 0.111 \\ 0.111 \end{pmatrix}$$ turbulence intencity fators

Wind turbulence

Logarithmic decrement of structural damping

Force coefficient

$c_{f0} := 2.2$ — force coefficient for rectangualr section with sharp corners and without free endflow

$\psi_\lambda := 0.64$ — end-effect factor

$c_f := c_{f0} \cdot \psi_\lambda = 1.408$ — force coefficient for wind load

Structural damping, assumption of table F.2 in EC1-4, for timber buildings

$\delta_s := 0.04$

Aerodynamic damping, at reference height z_s=0.6h

$$\delta_a := \frac{c_f \cdot \rho_{air} \cdot v}{2 \cdot n_1 \cdot \mu_e} = \begin{pmatrix} 5.548 \times 10^{-3} \\ 9.253 \times 10^{-4} \end{pmatrix}$$

Total damping

$$\delta_{tot} := \delta_s + \delta_a = \begin{pmatrix} 0.046 \\ 0.041 \end{pmatrix}$$

Logarithmic decrement of structural damping

Structural factors

$$B := \sqrt{\exp\left[-0.05 \cdot \left(\frac{z}{z_s}\right) + \left(1 - \frac{b}{z}\right) \cdot \left[0.04 + 0.01 \cdot \left(\frac{z}{z_s}\right)\right]\right]} = 0.941$$ background factor according to national annex

$$y_c := \frac{150 \cdot n_1 \cdot m}{v} = \begin{pmatrix} 11.837 \\ 70.972 \end{pmatrix}$$

$$F_c := \frac{4 \cdot y_c}{\left(1 + 70.8 \cdot y_c^{2}\right)^{\frac{5}{6}}} = \begin{pmatrix} 0.022 \\ 6.704 \times 10^{-3} \end{pmatrix}$$

$$\phi_h := \frac{1}{\left(1 + \frac{2 \cdot n_1 \cdot h}{v}\right)} = \begin{pmatrix} 0.209 \\ 0.042 \end{pmatrix}$$

$$\phi_b := \frac{1}{\left(1 + \frac{3.2 \cdot n_1 \cdot b}{v}\right)} = \begin{pmatrix} 0.09 \\ 0.016 \end{pmatrix}$$

Resonance factor

$$R_{Ta} := \sqrt{\frac{2\cdot\pi\cdot F_{c_0}\cdot\phi_{h_0}\cdot\phi_{b_0}}{\left(\delta_s + \delta_{a_0}\right)}} = 0.24$$

$$R_{skepp} := \sqrt{\frac{2\cdot\pi\cdot F_{c_1}\cdot\phi_{h_1}\cdot\phi_{b_1}}{\left(\delta_s + \delta_{a_1}\right)}} = 0.027$$

$$R_v := \begin{pmatrix} R_{Ta} \\ R_{skepp} \end{pmatrix}$$

Structural factors

Acceleration

$$\phi_1 := \left(\frac{z}{h}\right)^{1.5} = 1$$

fundamental flexural mode, factor ξ=1.5 is given in national annex to EC1-4

$$\nu := n_1 \cdot \frac{R_v}{\sqrt{B^2 + \left(R_v\right)^2}} = \begin{pmatrix} 0.416 \\ 0.048 \end{pmatrix}\cdot Hz$$

up-crossing frequency

$$t := 600s$$

10 min peak wind

$$k_p := \sqrt{2\cdot\ln(\nu\cdot t)} + \frac{0.6}{\sqrt{2\cdot\ln(\nu\cdot t)}} = \begin{pmatrix} 3.503 \\ 2.821 \end{pmatrix}$$

peak factor

Standard deviation of acceleration

$$\sigma_{x.Ta} := \frac{3\cdot I_{v_0}\cdot R_{v_0}\cdot q_{pv_0}\cdot b\cdot c_f\cdot\phi_1}{m_e} = 6.402\times10^{-3}\,\frac{m}{s^2}$$

$$\sigma_{x.skepp} := \frac{3\cdot I_{v_1}\cdot R_{v_1}\cdot q_{pv_1}\cdot b\cdot c_f\cdot\phi_1}{m_e} = 1.973\times10^{-5}\,\frac{m}{s^2}$$

Maximum acceleration for a 50 year wind and 5 year mean wind speed

$X_{Ta} := k_{p_0} \cdot \sigma_{x.Ta} = 0.022 \dfrac{m}{s^2}$ $T_a = 5$ years

$X_{skepp} := k_{p_1} \cdot \sigma_{x.skepp} = 5.566 \times 10^{-5} \dfrac{m}{s^2}$ skeppsbron

Acceleration

Frequency according to ISO 4354:2009

Annex E, Dynamic response factors

Crosswind response

$\dfrac{h}{\sqrt{b \cdot h}} \geq 0.4 = 1$ Equation (E.9) Not OK, investigate further

$\dfrac{d}{b} = 0.4$

$v_{L.cr} := 11 \dfrac{m}{s}$ Reduced critical wind speed

$v_{h.cr} := v_{L.cr} \cdot n_1 \cdot \sqrt{b \cdot d} \cdot \dfrac{s}{m} = 469.459 \dfrac{m}{s}$ critical wind speed for crosswind or torsional aeroelastic instability

$v_{h.cr} \leq 1.5 \cdot v_{mean} = \begin{pmatrix} 0 \\ 0 \end{pmatrix}$ Equation (E.10) OK!

Terrain roughness and height exposure factors for terrain category 1, open flat sea

$z_{ref} := 10m$ reference height 10m

$z_{0.ref} := 0.003m$

$k_{tr.z.3600} := 0.87$ peak terrain roughness and height exposure factor, 1 hour peak wind

$k_{tr.z.3} := 1.11$ peak terrain roughness and height exposure factor, 3.s peak wind

$k_{tr.change} := 1$ peak terrain roughness change exposure factor

$k_{topog} := 1$ peak topography exposure factor

$c_{exp.3600} := k_{tr.z.3600} \cdot k_{tr.change} \cdot k_{topog} = 0.87$ exposure factor, 1 hour wind

$c_{exp.3} := k_{tr.z.3} \cdot k_{tr.change} \cdot k_{topog} = 1.11$ exposure factor, 3 s wind

$v_{site.3600} := v_{m.skepp} \cdot c_{exp.3600} = 4.35 \frac{m}{s}$ wind speed locally at site, 1 hour wind return period 1 year

$I_{v.3600} := 0.125$ turbulence intencity factor at 10m reference height for 3600 s wind peak

<u>For tall structures canti-levered at the base</u>

$\beta_{3600} := 0.12$ power law exponent of peak wind speed 3600s wind

$\beta_3 := 0.074$ power law exponent of peak wind speed 3s wind

$L_{vh} := 100 \cdot m \cdot \left(\frac{z_{ref}}{h}\right)^{0.5} = 64.55\,m$ turbulence scale at the reference height

$\gamma_{ISO} := 0.07$

$$B_D := \frac{1 + 0.2 \cdot \beta_{3600}}{1 + \frac{0.63 \cdot \left(\frac{\sqrt{b \cdot h}}{L_{vh}}\right)^{0.56}}{\left(\frac{h}{b}\right)^{\gamma_{ISO}}}} = 0.715$$

background factor 3600 s wind

$k := 1.5$ mode shape power exponent

$K_{ISO} := 0.27 \cdot k + 0.73 = 1.135$ mode correction factor

$$E_D := \frac{4\cdot\frac{n_1\cdot L_{vh}}{v_{site.3600}}}{\left(1+70.8\cdot\left(\frac{n_1\cdot L_{vh}}{v_{site.3600}}\right)^2\right)^{\left(\frac{5}{6}\right)}} = 0.097$$

spectrual enegry factor

$$S_{iso} := \frac{0.9}{\sqrt{\left[1+6\cdot\left(\frac{n_1\cdot h}{v_{site.3600}}\right)\right]^2\cdot\left(1+3\cdot\frac{n_1\cdot b}{v_{site.3600}}\right)}} = 8.296\times 10^{-4}$$

Size reduction factor

$$r := \frac{2\cdot\sqrt{0.053-0.042\cdot\beta_{3600}}}{1+20\cdot\left(\frac{n_1\cdot b}{v_{site.3600}}\right)} = 1.407\times 10^{-3}$$

factor for correlation effect of wind-ward pressure and lee-ward pressue

$x_D := 1.3\text{cm}$

Deflection of structure calculated for a hourly wind, $v_{.site\ 3600}$

$\zeta_{str} := 0.75\cdot 0.012 = 9\times 10^{-3}$

structural damping, 75% of value given in table E.3 should be used for evaluation of horizontal habitability comfort.

$$R_D := \left(1+0.6\cdot\beta_{3600}\right)\cdot\frac{3}{2+k}\cdot K_{ISO}\cdot\sqrt{\frac{\pi}{4\cdot\zeta_{str}}\cdot E_D\cdot S_{iso}\cdot\left(0.57-0.35\cdot\beta_{3600}+r\right)} = 0.063$$

resonace factor

<u>Along-wind repsonse</u>

$g_{DB} := 0.28$

peak factor for backgound component, which may me equated as the avarage peak factor. Assumed to be equal to a 600s wind, since other gives an acceleration of 0.

$\nu_{ISO} := n_1 = 1.687\cdot\text{Hz}$

cycling rate of vibration, approximated by the first mode natural frequency

$t_{ISO} := 3600\text{s}$

$$g_{DR} := \sqrt{2\cdot\ln\left(\nu_{ISO}\cdot t_{ISO}\right)} + \frac{0.5772}{\sqrt{2\cdot\ln\left(\nu_{ISO}\cdot t_{ISO}\right)}} = 4.312$$

peak factor

$$G_B := 2 \cdot I_{v.3600} \cdot g_{DB} \cdot B_D = 0.05$$

dynamic responce factor for background component

$$G_R := 2 \cdot I_{v.3600} \cdot g_{DR} \cdot R_D = 0.068$$

dynamic responce factor for resonance component

$$C_{dyn.m} := 1 + \sqrt{B_D^{\,2} + R_D^{\,2}} = 1.717$$

mean dynamic responce factor

$$x_{D.acc} := (2\pi \cdot n_1)^2 \cdot \frac{2 \cdot g_{DB} \cdot I_{v.3600} \cdot R_D}{1 + 2 \cdot g_{DB} \cdot I_{v.3600} \cdot \sqrt{B_D^{\,2} + R_D^{\,2}}} \cdot x_D = 6.176 \times 10^{-3} \cdot \frac{m}{s^2}$$

peak acceleration

Frequency according to ISO 4354:2009

Appendix F-British standard

Vibration dose value,VDV, defines a relationship that yields a consistent assessment of continous, intermittent, occasional and impulsive vibration and correlates well with subjective response. It is used to assess the acceptability of building vibration with respect to human response. The vibration dose is dependent of the number of occurences of the vibrations, the variation of acceleration is assumed to be constant over a 10 min period and during 8 hour exposure per day/night.

The dominant direction of vibration is horizontal, therefore as stated in the code that is the only direction that needs to be eveluated.

$t_{EC} := 10\text{min} = 600\,\text{s}$ duration of acceleration, EC =10min mean wind

$t_{ISO} := 3600\text{s}$ duration of acceleration, ISO =1 hour peak wind

$t_{day} := 57600\text{s}$ duration of exposure during day, 16 h according to BS

$t_{night} := 28800\text{s}$ duration of exposure during night, 8 h according to BS

$f := 1.687\text{Hz}$ first natural frequency, from FEM model

$X_{ISO} := 0.044\text{m}\cdot\text{s}^{-2}$ peak acceleration, 1 year wind calculated in accordance with ISO

$X_{EC} := 0.022\text{m}\cdot\text{s}^{-2}$ mean acceleration, 5 year wind calculated in accordance with EC

$X_{skepp} := 0.014\cdot\text{m}\cdot\text{s}^{-2}$ mean acceleration, wind at Skeppsbron calculated in accordance with ISO

$\text{Modulus} := 1$ modulus, frequency weighting

$$a_{rms.ISO} := \text{Modulus}\cdot X_{ISO} = 0.044\,\frac{\text{m}}{\text{s}^2}$$

$$a_{rms.EC} := \text{Modulus}\cdot X_{EC} = 0.022\,\frac{\text{m}}{\text{s}^2}$$

$$a_{rms.skepp} := \text{Modulus}\cdot X_{skepp} = 0.014\,\frac{\text{m}}{\text{s}^2}$$

Vibration dose, day/night

<u>1 year wind, ISO</u>

$$VDV_{d.\tau day.ISO} := \left(t_{ISO}\cdot a_{rms.ISO}{}^4\right)^{0.25} = 0.341\,\frac{\text{m}}{\text{s}^{1.75}}$$

$$VDV_{d.\tau night.ISO} := \left(t_{ISO} \cdot a_{rms.ISO}{}^{4}\right)^{0.25} = 0.341 \frac{m}{s^{1.75}}$$

<u>5 year wind</u>

$$VDV_{d.\tau day.EC} := \left(t_{EC} \cdot a_{rms.EC}{}^{4}\right)^{0.25} = 0.109 \frac{m}{s^{1.75}}$$

$$VDV_{d.\tau night.EC} := \left(t_{EC} \cdot a_{rms.EC}{}^{4}\right)^{0.25} = 0.109 \frac{m}{s^{1.75}}$$

<u>1 year wind, Skeppsbron</u>

$$VDV_{d.\tau day.skepp} := \left(t_{day} \cdot a_{rms.skepp}{}^{4}\right)^{0.25} = 0.217 \frac{m}{s^{1.75}}$$

$$VDV_{d.\tau night.skepp} := \left(t_{night} \cdot a_{rms.skepp}{}^{4}\right)^{0.25} = 0.182 \frac{m}{s^{1.75}}$$