A case study of a twelve-storey residential timber building

Design of structural system and control of human comfort due to oscillations.

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

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CHALMERS UNIVERSITY OF TECHNOLOGY
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ABSTRACT

There are a number of advantages of timber compared to the two other major building materials steel and concrete. Timber is the only renewable building material used in a larger scale and timber has a high strength to weight ratio which are an advantage regarding foundation properties, production and assembly. A disadvantage of a lightweight structure is the sensitivity to horizontal loads and discomfort due to wind induced vibration needs to be considered.

A multi-storey building needs a stabilizing system that can handle the horizontal loads. One way to stabilize a building is through diaphragm action.

This work aims to investigate the possibilities to stabilize a twelve stories high residential timber building by the use of diaphragm action and to study the building's dynamic response with respect to wind-induced vibrations.

An existing four-storey building were used as a reference objects and an additional eight floors were added to the structure as the calculations in this thesis are performed for a building of twelve-storeys. In this work a plastic design method is applied which is a new method to design and construct buildings with load bearing studs and diaphragm action in the walls with respect to horizontal stabilization. This method utilizes the fact that the joints exhibit some elasticity before its capacity decreases. This results in a better material utilization compared to previously applied elastic design methods.

Calculations concerning the human comfort were performed by controlling if the values of natural frequency and acceleration were within the national requirements for oscillations.

In conclusion, it is possible to stabilize a twelve-storey timber building by the use of diaphragm action in ultimate limit state. This implies large dimensions of connections, studs and sheeting. The obtained results indicate that the transversal accelerations of the building are slightly too high to meet the human comfort requirements defined for the structure if being used as a residential building.

Key words: Dynamic response, horizontal stabilization, timber, wind induced vibrations, comfort requirements, shear walls, anchoring forces, oscillations, acceleration, natural frequency.

En fallstudie av ett tolvvåningsbostadshus i trä.
Design av bärande system och kontroll av mänsklig komfort på grund av svajning orsakad av vindlaster.
Examensarbete inom Structural Engineering and Building Performance Design

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SAMMANFATTNING

Det finns ett antal fördelar med trä jämfört med de två andra stora byggnadsmaterialen stål och betong. Trä är det enda förnyelsebara byggnadsmaterialet som används i en större skala och har i förhållande till sin vikt en hög bärförmåga i jämförelse med stål och betong. Detta är fördelaktigt vad gäller transport, grundläggning, produktion och montering. En nackdel med en lätt byggnad är dock känsligheten för horisontella laster och en byggnads dynamiska respons i bruksstadiet behöver beaktas.

Ett flervåningshus kräver ett fungerande stabiliserande system som kan hantera de på byggnaden verksamma horisontella lasterna. Ett sätt att stabilisera en byggnad är genom skivverkan.

Detta arbete syftar till att undersöka möjligheterna att stabilisera ett tolv våningar högt bostadshus med träregelstomme genom skivverkan samt att studera byggnadens dynamiska respons med hänsyn till vindinducerade vibrationer.

Ett befintligt bostadshus i fyra våningar har använts som referensobjekt och ytterligare åtta våningar har adderats till konstruktionen då beräkningarna i denna avhandling är utförda för en tolv våningar hög byggnad.

I detta arbete tillämpas en plastisk dimensioneringsmetod vilket är ny metod för att utforma och dimensionera skivbeklädda träregelstommar med hänsyn till horisontalstabilisering. Denna metod utnyttjar det faktum att förbanden uppvisar viss töjbarhet innan dess kapacitet minskar avgörande. Detta medför ett bättre materialutnyttjande jämfört med tidigare tillämpade elastiska dimensioneringsmetoder.

Beräkningar gällande vindinducerade vibrationer utfördes genom att kontrollera om värdena för egenfrekvens och acceleration ligger inom de nationella kraven för svajning. För att uppnå så tillförlitliga resultat som möjligt, beräknades den naturliga frekvensen med en numerisk analys med hjälp av FEM-programmet ABAQUS.

Sammanfattningsvis är det möjligt att stabilisera ett tolvvåningshus med trästomme genom skivverkan för brottlast. Detta innebär dock stora dimensioner på anslutningar, reglar och skivor. De erhållna resultaten indikerar dock att de horisontella accelerationerna i bruksgränstillstånd är för höga för att uppfylla komfortkraven definierade för byggnaden om den används som bostadshus.

Nyckelord: Dynamisk respons, horisontalstabilisering, trä, vindinducerade vibrationer, komfortkrav, skivregelväggar, förankringskraft, svajning, acceleration, egenfrekvens.

Contents

Preface

This Master thesis has been carried out in collaboration between Derome/A-Hus and the divisions of Structural Engineering at Chalmers University of Technology with Professor Robert Kliger as the examiner.

We want to direct a special thanks to our supervisor Linus Abrahamsson at Derome/Hus-A for his great support, continuous guidance and for making this thesis work possible as well as rewarding.

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The majority of this thesis was written in the spring of 2012. For various reasons the publication was delayed and some new information related to design of timber buildings are not up to date.

Göteborg, November 2015

Sandra Watkinson

Julia Folkesson

Notations

Roman upper case letters

A_{fr}	is the area of external surface parallel to the wind
A_{ref}	is the reference area of the individual surface
A	is the area
B^2	is the background factor, allowing for the lack of full correlation of the pressure on the structure surface
B	is the width of the building
C_e	is the exposure coefficient
C_t	is the thermal coefficient
E	is the modulus of elasticity
$E1$	is the modulus of elasticity in x-direction
$E2$	is the modulus of elasticity in y-direction
$E3$	is the modulus of elasticity in z-direction
EI	is the modulus of elasticity times the moment of inertia
$E_{0.05}$	is the fifth-percentile modulus of elasticity parallel to the grain
F	is the horizontal load acting on the wall
$F(t)$	is the fluctuated wind force
$F_{hi,ekv}$	is the equivalent horizontal force
F_{p}	is the plastic capacity of stud-sheeting joints
$F_{ax,Rk}$	is the characteristic axial withdrawal capacity of the fastener
F_{fr}	is the frictional wind forces
F_{knut}	is the shear resistance of a junction unit
F_m	is the mean wind load
F_p	is the skivregel-förbandets plastic capacity
$F_{v,Rd}$	is the design load-carrying capacity
$F_{v,Rk}$	is the characteristic load-carrying capacity
$F_{w.e}$	is the external wind forces
$F_{w.i}$	is the internal wind forces
G	is the self-weight of the building, including the foundation
$G12$	is the shear modulus in the x-y plane
$G13$	is the shear modulus in the x-z plane
$G23$	is the shear modulus in the y-z plane
G_k	is the characteristic shear modules of the sheeting
H	is the horizontal load
H	is the horizontal resistance
H_d	is the design horizontal load per wall segment
$H_{i,Cx}$	is the contribution from rotation
$H_{i,Cy}$	is the contribution from rotation
H_x	is the horizontal load in x-direction
H_{xi}	is the contribution from translation
H_y	is the horizontal load in y-direction
H_{yi}	is the contribution from translation
I	is the moment of inertia
I_v	is the turbulence intensity
$I_v(z)$	is the turbulence intensity at height z

K^{MN}	is the stiffness matrix (which includes initial stiffness effects if the base state included the effects of nonlinear geometry)
$K_{ser,fin}$	is the displacement module
K_{ser}	is the slip modulus
$L(z_s)$	is the size of gusts at reference height z_s
M	is degrees of freedom
M^{MN}	is the mass matrix (which is symmetric and positive definite)
M_d	is the design moment
$M_{y,Rk}$	is the characteristic fastener yield moment
N	is degrees of freedom
Nu	is the Poisson’s ratio
$Nu12$	is the Poisson’s ratio in the x-y plane
$Nu13$	is the Poisson’s ratio in the x-z plane
$Nu23$	is the Poisson’s ratio in the y-z plane
N_d	is the normal force
P	is the load
R	is the resonant respond part
R	is the reaction force
R^2	is the resonance response factor, allowing for turbulence in resonance with the vibration mode
$R_{\text{ö}pp}$	is the reaction force for a stud at the right side of an opening
R_N	is the reaction force for a stud at the left side of an opening
S	is the snow load
Sc	is the Scruton number as defined
St	is the Strouhal number that depends on the cross section shape of the structure
$S_L(f_L)$	is the spectrum of the fluctuation wind velocity
S_T	is the global torsional stiffness calculated according to Equation()
S_i	is the spectral density function for a certain frequency
S_k	is the characteristic value of snow load on the ground
T	is the global torsional moment is calculated according to Equation ()
T_a	is the number of years
UR	is the utilisation ratio
V_0	is the shear force
$\mathrm{V_N}$	is the shear force
V_{di}	is the design value of the average vertical force for each floor on the underlying inclined walls
V_{ekv}	is the equivalent force acting on the leading stud
V_i	is the equivalent force acting on the first rule in the wall
W_y	is the section modulus

Roman lower case letters

a_G	is the factor of galloping instability
a_i	is the magnitude of the amplitude
a_i	is the distance between origo and the centre of the bracing element in x-direction
a_{max}	is the maximum acceleration
a_{rms}	is the mean acceleration
b	is the width

b_{full}	is the full board width
b_i	is the distance between origo and the centre of the bracing element in y-direction
c_d	is the dynamic building factor
c_f	is the force coefficient
c_{fr}	is the friction coefficient
c_{pe}	is the pressure coefficient for the external pressure
c_{pi}	is the pressure coefficient for the internal pressure
$c_s c_d$	is the structural factor
d	is the diameter of the fastener
d	is the plastic shear flow
e	is the eccentricity, the distance perpendicular to the load between the load resultant and the rotation centre.
e	is the vertical resultant caused by all horizontal loads
f_0	is the natural frequency
$f_{h.1,k}$	is the characteristic embedment strength in the sheeting
$f_{h.2,k}$	is the characteristic embedment strength in the timber
f_p	is the plastic shear flow
$f_L(z_s, n)$	is the non-dimensional frequency on reference height z_s
$f_{c.0.d}$	is the design compressive strength of the stud parallel to the grain
$f_{c.0.k}$	is the characteristic compression strength parallel to the grain
$f_{c.90.d}$	is the design compressive strength perpendicular to the grain
$f_{c.90.k}$	is the characteristic compressive strength perpendicular to the grain
$f_{c.k}$	is the characteristic compression strength
f_d	is the design value of strengths in different modes
f_i	is the frequency
$f_{m.y.d}$	is the design bending strength about the y-y axis
$f_{m.y.k}$	is the characteristic bending strength about the principal y-axis
$\mathrm{f_{opening}}$	is the shear flow in the sheeting under the window opening
f_p	is the plastic shear flow
$f_{t.90.d}$	is the design tension strength perpendicular to the grain
$f_{t.90.k}$	is the characteristic tension strength perpendicular to the grain
$f_{v.d}$	is the design panel shear strength
$f_{v.k}$	is the characteristic panel shear strength
g_{floor}	is the self-weight of the slabs
$g_{innerwall}$	is the self-weight of the internal walls
$g_{outerwall}$	is the self-weight of the external walls
g_{roof}	is the self-weight of the roof
h	is the height
h_{tot}	is the total height
h_{under}	is the height of the wall below the opening
k	is the stiffness of wall segment
k_h	is the depth factor
$k_{c.90}$	is a factor taking into account the load configuration, possibility of splitting and degree of compressive deformation
$k_{c.y}$	is the instability factor about the y-y axis
k_{def}	is a factor for the evaluation of creep deformation taking into account the relative service class

k_{mod}	is the modification factor for duration of load and moisture content
k_{mod1}	is the modification factor for the sheeting
k_{mod2}	is the modification factor for the timber
k_p	is the peak factor
k_r	is the terrain factor depending on the roughness length z_0
k_{sys}	is the system strength factor
l	is the length
l_1	is the length of the wall part where the full plastic shear force capacity is achieved
l_2	is the length of the wall part without openings and shear force capacity
l_3	is the length of the opening
l_{eff}	is the effective length of the wall
$l_{eff.tot}$	is the total effective length of the wall
m	is the mass per unit length of the building
m_s	is the mass per unit length
n_1	is the approximate fundamental frequency
n_{floor}	is the number of floors
n	is the number of
n_f	is the fundamental frequency
$n_{i,y}$	is the cross-wind fundamental frequency of the structure
n_{knut}	is the number of shear joints that are active for the transfer of shear force
q_b	is the is the basic velocity pressure
q_k	is the imposed load
q_m	is the mean wind velocity
$q_p(z_e)$	is the peak velocity pressure at reference height z_e
r_i	is the radial distance from the rotation centre
s	is the center spacing
s_i	is the stiffness of bracing member
s_{xi}	is the stiffness of bracing member i in x-direction
s_{yi}	is the stiffness of bracing member i in y-direction
t	is the time
t_1	is the thickness of the sheeting
t_2	is the penetration depth in the timber member
u	is the horizontal displacement
$v(t)$	is the fluctuating wind velocity
v_{50}	is the characteristic value of the reference wind velocity that is exceeded for one year with the probability of 2%, which corresponds to an average return period of 50
v_{CG}	is the onset wind velocity of galloping
v_{Ta}	is the wind velocity for different return periods
$v_{crit.i}$	is the critical wind velocity
v_m	is the mean wind velocity
$v_m(z_s)$	is the mean wind velocity at reference height z_s
w	is the angular frequency
w_e	is the wind pressure acting on the external surfaces
w_i	is the wind pressure action on the internal surfaces
x_i	is the x-coordinate
x_t	is the x-coordinate of the rotation centre

y_i	is the y-coordinate
y_t	is the y-coordinate of the rotation centre
z	is the height above ground
z_0	is the roughness length

Greek upper case letters

Δf	is the band width
Φ_1	is the fundamental along wind modal shape
Ψ_0	is the factor for combination value of a variable action
Ψ_i	is the phase angle

Greek lower case letters

δ_a	is the aerodynamic decrement of damping
δ_d	is the logarithmic decrement of damping due to special devices
δ_s	is the structural logarithmic decrement of damping
α_0	is the systematic part
α_{md}	is the average inclination
α_n	is a reduction factor for imposed loads
α_δ	is the random part
γ_M	is the partial factor for material properties
γ_m	is the partial factor for material properties and resistances
γ_{m1}	is the partial factor for material properties and resistances for the sheeting
γ_{m2}	is the partial factor for material properties and resistances for the timber
$\lambda_{rel.y}$	is the relative slenderness of a stud about the y-y axis
λ_y	is the slenderness ratio of a stud about the y-y axis
μ_1	is the snow load shape coefficient
ρ_{air}	is the density of air
ρ_{m1}	is the plastic shear flow
ρ_{m2}	is the effective wall length
$\sigma_{c.0.d}$	is the design compressive stress parallel to the grain
$\sigma_{c.90.d}$	is the design bearing stress on the plate
σ_d	is the design stresses in the material
$\sigma_{m.y.d}$	is the design bending stress about the y-y axis due to permanent and combined medium-term vertical and wind (dominant) action
ϕ^N	is the eigenvector (the mode of vibration)
β	is the ratio between the embedment strength of the members
δ	is the logarithmic decrement of damping
λ	is a dimensionless multiplier for the reduction of shear flow under the window opening
ρ	is the density

1 Introduction

1.1 Background

In line with increasing urbanization and greater population, the demand for residential and commercial buildings in the urban central parts increases. This entails densification and the development is towards higher multi-storey buildings. Regarding the material in the structural system of these buildings, the sector is dominated by steel and concrete, but the use of timber is today increasing significantly in new productions of buildings up to a certain height. Timber has a number of significant advantages in comparison to other more established construction materials and these become more and more acclaimed. The construction can, be performed cost effective, fast and with low environmental impact for example.

Timber also has a high strength to weight ratio, which makes it possible to build on land with rather poor foundation properties. This creates new opportunities for densification since buildings can be constructed more easily in places that were not previously considered.

Sweden has a strong tradition of timber construction, which to some extent can be explained by the ample supply of the raw material and the fact that Sweden has a large forestry and timber industry.

Regarding the performance of high multi-storey timber buildings the knowledge is limited, mostly due to the absence of experience. This is to a large degree the result of an earlier Swedish law that regulated the height of timber buildings. The law was introduced due to several urban fires during the 17th and 18th century where hundreds of timber buildings burned down. Since the year of 1995 it has been permitted to construct timber buildings in an unlimited number of floors as long as the requirements imposes on fire safety standards are fulfilled. (Klippberg & Fallqvist 1999)

As timber buildings now can be constructed in unlimited number of floors, an increased need for an efficient stabilizing system have emerged. The higher a building is, the greater the risk is that the transverse forces acting on the structure are causing problems with the stability. One factor that needs to be considered in design is the possibility of oscillation due to horizontal forces.

Timber is the only renewable building material used in a larger scale. It can also to a large extent be locally produced which entails reduced transport distances. The material absorbs carbon dioxide. By increasing the proportion of timber in the building sector, instead of using other construction materials that are not renewable, it will contribute to a better global climate. (Stehn, Rask, Nygern & Östman 2008)

An important factor associated with wind induced motion of buildings is the human response to vibration and the perception of motions. If the acceleration of the oscillation in a building is too high it can cause an unpleasant nausea or motion sickness among people residing the building. This is not acceptable, which means that this factor must be considered in design. Humans are sensitive to vibrations and motions may feel uncomfortable even if they correspond to relatively low levels regarding stress and strain. Therefore, there is a possibility that the serviceability considerations govern the design rather than strength issues

1.2 Aim and objectives

The aim of this thesis is to establish whether a structural building system consisting of shear walls with load bearing studs can be used for the design of a twelve-storey building. The system is currently used for design of multi-storey timber buildings up to four floors. Calculations of the building during its service life are performed with aim to determine whether the requirements for oscillation are achieved without special dampening devices added to the structure.

1.3 Limitations and assumptions

To restrict the scope of this thesis, some limitations and assumptions were established.

The report comprise essentially structural technical calculations and therefore aspects regarding fire engineering, sound requirements, building physics and production economics are not included.

Only the load-bearing and stabilizing components are included in the model and regarding global calculations the sliding and load distribution on the slab is disregarded.

No calculations or controls of the slabs and roof structure are performed since they are retrieved from the reference object and already structurally designed. Both the slabs and the roof are assumed to be rigid in its plane and fulfil required demands.

The connections between load bearing elements are not studied in great details and are approximate as rigid in the numerical FE-model.

The maximum anchorage forces are calculated and potential design solutions is proposed, but no detailed calculations are carried out in this regard.

1.4 Approach and methodology

To fulfil the aim of the thesis, both a literature study and structural calculations were made. The purpose of the literature study was to investigate what has already been performed in this field and to assimilate relevant information and knowledge regarding the specific focus of the thesis.

The literature study is to a large extent based on the existing frame system but also of research reports, technical literature and oral sources from people with considerable expertise. Relevant standards and regulations are studied in detail to get erudition of permitted values and limitations.

1.4.1 Calculations procedure

The thesis is divided into two parts. The first part focusing on design calculations in ultimate limit state and the second on serviceability limit state. The thesis begins with calculations in ultimate limit state in order to design and dimension the structural components in the building and determine the stiffness of relevant parts required in later calculations. The goal of the calculations in serviceability limit state is to establish if the house will oscillate more than the relevant standards permits and if measures needs to be performed in order to prevent nausea in form of damping of the building.

The loads acting on the building are calculated according to Eurocode and associated national annexes. The horizontal loads are designed for two different cases, load acting on the long side respective the short side of the façade.

The calculations regarding the horizontal stabilization are performed according to a plastic design method developed by Bo Källsner and Ulf Arne Girhammar and that is described in the manual “Horisontalstabilisering av skivregelstommar” (2008).

Calculations regarding the oscillation of the building are performed by numerical modelling using the FEM-program ABAQUS. Hand calculation is also performed in order to verify the results from the FEM-calculations.

2 Timber

2.1 History of timber construction

Timber is the oldest building construction material used today and the Swedish timber construction tradition is several hundred years old and includes both single-family houses and larger buildings. (Träguiden, 2 2012)

During the 17th and 18th century, a number of severe fires occurred in Swedish cities and hundreds of houses burned down. As a result of these fires regulations regarding the height of timber buildings were introduced in 1874, allowing only single- or two-storey timber buildings. (Waldenström 2008)

The law remained for more than 120 years and until the year of 1995 it was not allowed to construct multi-storey timber buildings. When Sweden joined the European Union 1994, the law was modified and the requirements on the material in the frame were changed to requirements related to function. This means that it became allowed to construct timber buildings in an unlimited number of floors, as long as the requirements imposes on fire safety standards where fulfilled. (Klippberg & Fallqvist 1999)

2.2 Timber as a construction material

Timber differs from other comely used building materials like steel and concrete in many ways and a number of factors needed to be taken into consideration regarding the design of timber structures. Timber has a low density compared to steel and concrete but needs a larger volume to achieve the same strength. The density properties of wood vary as it depends on the tree's growing conditions and genetic characteristics. (Carling 1992)

Furthermore, it has high strength to weight ratio. The material is due to its low weight, easy to transport and has a high insulation capacity. Wood has good thermal insulation properties, which entail less thermal bridges leading to the possibility of simple constructive solutions. There are also some disadvantages of timber and these include for example moisture sensitivity, the risk of rot and insects attacks and combustibility. These factors are in most cases manageable with today’s knowledge and technology.

Timber has a high strength to weight ratio. The load acting on the subgrade from the structural system is decreased by 30-50 percent with a timber structural system in comparison to other commonly used framing materials. Lightweight constructions make it possible to erect buildings on land with rather poor foundation properties. This offers new opportunities for densification since buildings can be constructed in places that were not previously possible or economically defensible.

Piling or stabilization of the subgrade is the two methods most frequently used to handle poor foundation conditions. Piling is the method that is most commonly used in Sweden. To light buildings can result in the fact that the minimum amount of piles is undercut resulting in reduced economical savings. Stabilization of the subgrade by for example fundamental compensation or lime stabilization may be of greater economic benefit regarding multi-storey timber buildings. (Sveriges Träbyggnadskansli 2012; Stehn et al. 2008)

2.2.1 The anatomy and strength properties of wood

Wood is a natural composite material mainly consisting of cellulose, hemicellulose and lignin. It has highly variable properties and is an orthotropic material, which implies that it has dissimilar properties in different directions. This entails variable material response when load is applied in different directions in relation to the grain orientation. The radial, longitudinal and tangential direction of sawn timber results in different properties. It is very important to identify the direction of the load, since timber has a very good strength and capacity parallel to the grain. This is reduced drastically perpendicular to the fibre direction. (Crocetti, Johansson, Johnsson, Kliger, Mårtensson, Norlin, Pousette & Thelandersson 2011)

Flawless timber has its highest strength in tension in the fiber direction and the lowest tensile strength appears perpendicular to the grain direction and should only be used at secondary effects. Wood is not a homogeneous materials, it always contains some defects such as knots, oblique fibers and cracks. Both strength and stiffness are affected and strongly reduced as a result of these defects. (Carling et al. 1992)

The structure of cellulose consists of both primary and secondary bonding and the technical properties of wood can be related to the variety of the bonding present. Covalent bonding is represented both within the glucose rings and linking the rings together forming molecule chains and is contributing to the high axial tensile strength of timber. (Domono & Illston 2010)

One factor that has a large influence of the behaviour and performance of timber is the moisture content. This affects both the strength and stiffness properties of the material. Decreased moisture content below the fibre saturation point entails higher strength and stiffness properties. To implement this effect in design, three different service classes are used which corresponds to the average moisture content assumed in the constructive timber parts. Wood is a hygroscopic material, which means that the moisture content depends on the relative humidity of the surrounding air. Wood is also a “living material” and the shrinkage and swelling of the timber component depends on the moisture content and the thermal conditions. A reduction of the moisture content results in shrinkage while an increase entails swelling. The influence of moisture content varies in different directions and almost all shrinkage occurs in the transverse directions. (Crocetti et al. 2011)

The wood thermal expansion coefficient is almost ten times larger transverse the fibers as in the fiber direction. However the temperature movements are often negligible compared to the movements caused by moisture. (Carling et al. 1992)

The strength and stiffness properties of wood are also depending on the temperature and are decreasing with increasing temperature. The effect is however relatively small regarding normal temperatures in a span of -30˚C to +90˚C. (Crocetti et al. 2011)

Timber loses essential strength with increased duration of load. This applies for all loading modes, but is most critical for the bending strength. To deal with this issue and facilitate the design procedure, load duration classes are used in design which comprises diverse time periods for varies loads. (Crocetti et al. 2011)

2.2.2 Timber classification

Commercial timber is divided into two types of wood, softwoods and hardwoods. The softwood has a quick growth rate resulting in low-density timber with relatively low strength while the hardwoods grow at a slower rate that generally results in a timber of higher density and strength. (Porteous & Kermani 2007)

Timber strength classes are used to simplify the design procedure of timber structures. There are a total of 18 strength classes, twelve for softwoods and six for hardwood. It ranges between the grades most often used in Europe corresponding to the weakest grade of softwood, C14, to the highest grade of hardwood, D70. The letters C and D refer to coniferous species respective deciduous species and the number in each strength class refers to its characteristic bending strength. The strength capability of timber is difficult to assess and the strength grading method of strength classification are applied to facilitate this procedure. Several design properties are associated with a strength grading including: modulus of elasticity, bending strength parallel to the grain, strength properties in tension and compression parallel and perpendicular to the grain, shear strength parallel to the grain and density. The design properties used in the strength classes of timber are determined by either visual or machine strength grading. (Porteous & Kermani 2007)

2.2.3 Environmental aspects

Timber is the only renewable building material used in a larger scale. It can also to a large extent be locally produced which entails reduced transport distances. The material absorbs carbon dioxide. By increasing the proportion of timber in the building sector, instead of using other construction materials that are not renewable, it will contribute to a better global climate. (Stehn et al. 2008)

Through the photosynthesis, growing trees are storing carbon dioxide in form of carbon compounds. Trees are also emitting some carbon dioxide, but they absorb considerably more than they emit and the faster they grow, the more carbon dioxide is captured. From an environmental aspect it is better to cultivate the forest and use the timber than to let the forest remain untouched. The carbon dioxide that the trees have absorbed remains during the whole lifetime of the product and it is therefore beneficial to use timber in large and long-lasting products such as building structures.

The production of wood-based products generally entails relative small energy demands and the by-products arising from the production is used as biofuel. Wood is a flexible material and timber buildings are easy to rebuild or extend which provides benefits and opportunities regarding a long lifetime perspective.

Another important climatic advantage is that discarded timber products are used as biofuel, replacing or reducing the use of fossil fuels. (Sveriges Träbyggnadskansli 2012; Svenskt trä 2012)

According to Niclas Svensson[1], independent studies have shown substantial environmental benefits of timber compared to other commonly used building materials. Despite this fact the increase of timber in multi-storey buildings almost exclusively depends on the benefits that the industrial wood construction entails in terms of reduced costs and construction time.

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[1] Niclas Svensson Chief Sveriges Träbyggnadskansli, phone call 26 april 2012.

3 Building systems and construction methods for multi-storey timber buildings

There are basically three types of timber construction system available, that can be combined in three construction methods.

Building Systems

- Light frame system
- Solid wood systems
- Colum-beam system

Construction Methods

- On site construction
- Prefabricated construction
- Surface elements and prefabricated volumes

3.1 Structural systems of timer building

Solid wood and light frame system are the two systems most frequently used in multi-storey timber buildings.

3.1.1 Light frame systems

A light frame system consists of beams and studs with sheeting used to transmit the horizontal loads and handle the stabilisation. Light frame systems are traditionally used in single family houses. The system can also be applied in the construction of multi-storey building, but in a more complex configuration to meet increased fire safety, sound insulation and strength requirements. (TräGuiden, 2 2012)

The technique is frequently used in the United States where the system usually is produced on site. When the system is applied in the Nordic countries the elements are generally prefabricated in a factory. The height limit using light frame technology is currently around six to seven stories. (Crocetti et al. 2011)

3.1.2 Solid wood systems

Systems consisting of solid wood are usually based on cross-laminated wall, slab and roof elements made of for example, spruce or pine. The lamellas are glued together crosswise and pressed together in an odd number of layers, forming massive timber boards. This technique is becoming more common, as it has a high degree of prefabrication. Characteristics of solid wood elements are large cross-sections which entails high load bearing capacity and rigidity making them suitable for stabilization. Insulating layers in exterior walls are continuous which implies reduction of thermal bridges. (Svensk byggtjänst – AMA 2012)

The technique are attractive for sawmills, since it is possible to use small dimension timber in the production of solid wood panels. The height limit for the type of system is at present approximate seven storeys. (Crocetti et al. 2011)

3.1.3 Column-beam system

In a column-beam system, columns and beams carries the vertical loads while the horizontal forces are commonly resisted by sheeting and struts. The stabilising elements are often positioned in stairwells or façades to minimize the disruptions of the planning. Another alternative is to have moment resisting connections which stabilize the building by means of frame action. The advantage of this design system is that it allows large spans and is therefore used mainly in buildings that require large open areas. In the column-beam system, different materials may be combined, for example, laminated wood and steel. (TräGuiden, 4 2012)

A column- beam system for multi-storey timber buildings have been developed by the company Moelven. The system has a maximum span length of eight metres, but the company have ambitions to develop the system further and increase this length. (Crocetti et al. 2011)

3.2 Construction methods

There are currently three different construction methods applied. The most traditional is the on-site construction and the other two methods are based on the industrializing development.

3.2.1 On-site construction

On-site construction means that the production of building elements is made on site. At a construction site it is desirable to reach a dense building as early in the construction stage as possible to avoid moisture problems. This factor is especially important in the construction of multi-storey building made of moisture sensitive material such as timber. The method is therefore not suitable for construction of multi-storey timber buildings since it entails time-consuming construction that leads to long periods without weather protection. The method is the most flexible but also the least productive leading to increased costs and low profit margins. (Elwing & Sjögren 2006)

3.2.2 Prefabricated construction

The most common is prefabrication of wall, ceiling and floor elements produced in a factory. The elements are usually delivered to the building site with pre-wiring for installations and only small additions are required. The elements can be almost completely finished from the factory but the most common is that they need to be supplemented with on-site insulated installation layers. The elements are containing insulation that absorbs moisture, it is therefore important that the building is weather protected during construction. The method is considerably faster than the on-site construction which reduces the costs and also the risk of moisture damage. This can be considered the biggest benefits for the construction of prefabricated houses. (Träguiden, 1 2012)

3.2.3 Surface elements and prefabricated volumes

Volume elements are an evolution of the prefab system developed in order to reduce the construction costs. Instead of prefabricate single elements entire rooms and apartments are produced in a factory and shipped to the construction site where they are lifted into place. This technique has earlier only been applied on small houses with a maximum of two storeys but recently the approach have been implemented on multi-storey buildings. This challenges the traditional on-site building method since production costs can be reduced significantly. The company Lindbäcks are the most successful company in this area in Sweden.

One problem with the method is the architectural limitation that arises when all the volume elements are transported by road, limiting its size and reduces the potential of alterations. (Boverket 2012)

3.3 Critical aspects in the design of timber buildings

Some difficult and economical decisive aspects identified for timber buildings are fire, acoustics, stability and critical moisture levels. The stability issue are treaded in detail later in the rapport and below follows a brief description of the concerns and requirements regarding the other critical aspects.

3.3.1 Fire

There are building fire regulations that need to be considered in design. This documentation should include: fire safety classes for used materials and elements, divisions into fire cells, evacuation plan, ventilation during fire and a description of used technical solutions including any additional installation like sprinklers or fire detection systems. (Crocetti et al. 2011)

Structural component are assigned one or a combination of the properties as stated below depending on their function in the building.

R – Load-bearing capacity
E – Integrity
I – Insulation

The properties are assigned a number corresponding to time in minutes under which the component should resist a certain function from the start of a fully developed fire.

Timber is a compostable material, but it burns quite slowly in a controlled manner with a charring rate in the order of 0.6 millimetre/min. Structural components with large cross sections will remain intact for a relatively long time which enables evacuations of the building. Regarding light weight structures the cross sections is small and cannot sustain a fire. Multi-storey buildings constructed as light weight structures are there for generally completed with some type of cladding to increase the fire resistance. Gypsum boards are the most common solutions to meet the fire regulations but also sprinklers and material modifications are available approaches. (Crocetti et al. 2011)

3.3.2 Acoustics

In design of multi-storey timber buildings acoustics is an important consideration and there are requirements for permitted values. Sound is transported variety in different materials and generally lightweight structures such as timber buildings are considered as more critical than heavier constructions.

According to relevant standards, building parts are categorized in four classes as follows:

Class A – A very good acoustic environment
Class B – A significantly better acoustic environment than class C
Class C – Gives satisfying acoustic environment for the majority of the inhabitants and represents the minimum requirements of the European code
Class D – Is used when class C cannot be achieved, for instance for classifying old buildings

The acceptable level for newly constructed of residential buildings needs to fulfil the requirements of Class B. (Crocetti et al. 2011)

Potential aspects concerning sounds that can emerge and needs to be considered is flanking transmission, airborne sound and impact sound at low frequencies.

Airborne sound refers to sound that primarily operates in air, such as conversations between people and the sound from electrical devises. Decisive factor for airborne sound is the density of the materials in the apartment separating elements and the insulation against airborne sound is generally good in timber buildings. When a sound source acts directly on a floor structure is known as an impact sound. The impact sound depends on the mass and stiffness of the slab and is critical for timber. The sound requirements can be decisive concerning the design and thickness of the apartment separating slabs. Slabs consisting of timber needs to be thicker then concrete slabs to fulfil the requirements witch implies a larger total building height. (Weber 2012)

Flanking transmission means that sound are transmitted via flanking parts of the building like connecting walls and slabs and are of great importance in all types of buildings. The aspect is especially critical in timber buildings and needs to be considered in the design of all structural components. (Träguiden, 3 2012)

3.3.3 Critical moisture levels

Timber is a moisture sensitive material and timber constructions needs special protection against weather exposure. Built-in moisture must be avoided during construction and weather protected conditions needs to be applied during the entire assembly phase. There are various types of moisture protection that can be applied and these depend on the choice of frame system and the area available during construction. The most common solution for weather protection in commercial buildings is protecting layers of wax or impregnation applied on the timber member. Tents are another more advance solution for weather protection. The tents can be designed to cover the whole structure or one floor at the time. (Crocetti et al. 2011)

Overall the evaluation of weather protection is good since it leads to reduced production time, higher working safety and improved quality. A disadvantage is that the moisture protection solutions often are expansive. Also an increased degree of prefabrication reduces the risk of moisture relative damages. (Crocetti et al. 2011)

3.4 High timber structures

The highest multi-storey timber buildings in Sweden is located in Växjö and was completed in 2009, see Figure 1. The housing area is called Limnologen and consists of four eight-storey residential buildings. The structural system including stairwells and elevator shafts comprised of a massive timber frame but the bottom floor is cast in concrete to provide sufficient weight to manage the stability. (Martinsons 2012; Träpriset 2012)

Figure 1 *Limnologen in Växjö, Sweden's tallest residential area constructed mainly in timber. (Smålandsposten 2012)*

The world's tallest modern multi-storey timber building is a 14 storey residential building located in Bergen, Norway called "The Tree" see Figure 2. The building are 52.8 meters high divided on 14 floors and consist mainly of timber with some concrete components. The building are constructed with four floor building modules. Every fifth floor consist of a custom-made concrete slabs, which becomes the basis for the next four floors. Bearing trusses are used both internally and externally to provide sufficient stability. (Byggvärlden 2015; Sweco, 2015)

Figure 2 *The Tree located in Bergen, Norway that today is the world's tallest multi-storey timber building. (Byggvärlden 2015)*

The world's tallest timber construction is a 118 metre high radio tower located in Gliwice, Poland. The tower was built in 1934 and is constructed of larch wood. (Worldarchitecturenews 2012; Byggvärlden 2012)

3.5 Timber construction today

Timber is the most common framing material in new buildings in Sweden today. More than 50 percent of all homes are constructed with timber frames. Regarding single-family homes over 90 percent are built with timber as structural material, while the corresponding value for multi-storey buildings are around 15 percent. Multi-storey timber buildings have increased drastically during the last decade, from one percent in 2000 to 15 percent today, 2012. (Skogsaktuellt 2012)

One influencing factor for this increase is the development of industrial wood construction. The building process can appear in a faster and more cost effective way than before. Most of the construction process then occurs in a factory and elements are prefabricated in a protected environment that is independent of weather conditions. This can reduced the construction time, up to 80 percent. A five-storey timber building can be completed in ten weeks after the slab is cast. Regarding the environmental aspects the modern timber construction technique are superior. With industrial timber construction, the transport costs can almost be reduced by 50 percent partially because timber weigh less than traditional building materials, but also because it can be near produced which entails shorter transport routes. (Svenskt trä 2012)

There is a great interest in timber construction and efforts have been performed to stimulate the development both in Sweden and abroad. About fifteen countries in Europe have over the past decades decided on different public programs to increase the use of timber in the building sector. (Träguiden, 1 2012)

The Swedish government appointed a committee called Nationella träbyggnadsstrategin that appeared between the years of 2005 and 2008 with aim to promote the use of timber in building constructions. The goal of the project was that timber should be an obvious option considered in the structural system of greater and public buildings. (Sveriges träbyggnadskansli 2012)

The project ended in 2008 and culminated in another on-going project called Trästad 2012 with aim to develop Swedish expertise as well as technology and eventually create a European and global market for modern industrial wood construction technology. (Trästad 2012)

4 Dynamic response of multi-storey buildings

A building is exposed to not only static but also dynamic loads which can generate oscillations. These oscillations may not damage the structure excessively but can cause discomfort to the occupants.

The higher a building is, the greater the risk is that transversal forces cause accelerations of the structure that reaches unacceptable values. Timber buildings are more flexible and are considered to be lightweight structures compared to traditional high-rise buildings. This entails that they will more easily be put in fluctuations and has a greater sensitivity in this matter.

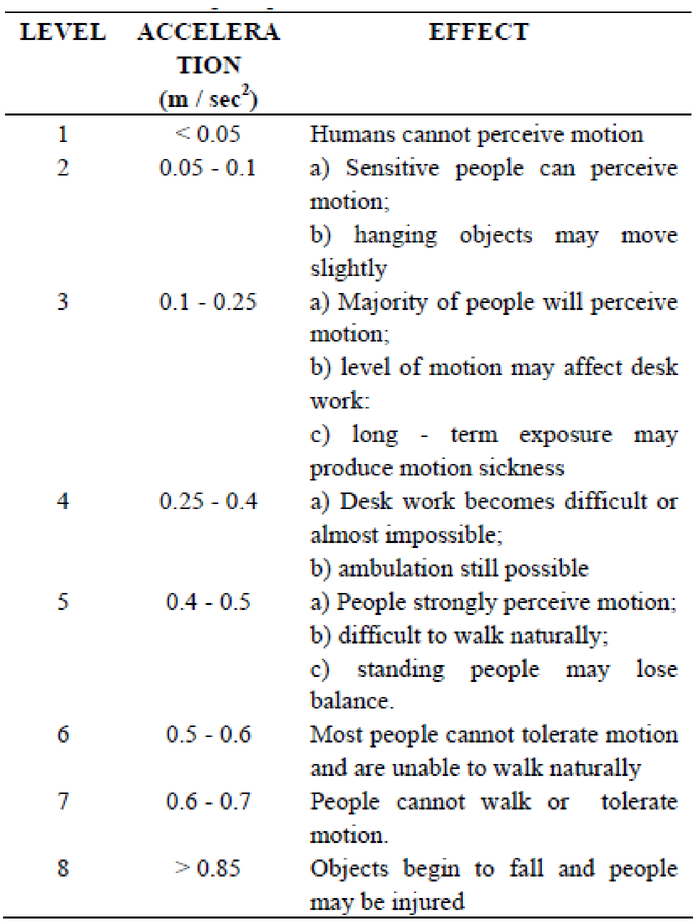
4.1 Human response

An important factor associated with wind induced motion of buildings is the human response to vibration and the perception of motions. If the acceleration of the oscillation in a building is too high it can cause an unpleasant nausea or motion sickness among people residing the building. This is not acceptable, which means that this factor must be considered in design. Humans are sensitive to vibrations and motions may feel uncomfortable even if they correspond to relatively low levels regarding stress and strain. Therefore, there is a possibility that the serviceability considerations govern the design rather than strength issues.

Acceptability criteria for the human response to building vibrations are generally expressed in terms of acceleration limits. The limits are a function of the frequency of the vibration being felt and are based on human tolerance to vibration discomfort in the upper floors of buildings. The perception of building movements depends on the degree of stimulation of the body's central nervous system. The balance sensor within the inner ears has a crucial role when determine if the accelerations is to be labelled. (Coull & Stafford 1991)

A lot of research has been performed regarding the physiological and psychological parameters that affect human perception to motion and vibration in the low frequency range encountered in tall buildings. These parameters include the occupant's expectancy and experience, their activity, body posture and orientation, visual and acoustic cues, and the amplitude, frequency, and accelerations for both the translational and rotational motions that the occupants may be exposed to. (Cheung, Haritos, Hira, Mendis, Ngo & Samali 2007)

Some guidelines concerning human perception levels are shown in Figure 3.



LEVEL	ACCELERATION (m / sec^2)	EFFECT
1	< 0.05	Humans cannot perceive motion
2	0.05 - 0.1	a) Sensitive people can perceive motion; b) hanging objects may move slightly
3	0.1 - 0.25	a) Majority of people will perceive motion; b) level of motion may affect desk work: c) long - term exposure may produce motion sickness
4	0.25 - 0.4	a) Desk work becomes difficult or almost impossible; b) ambulation still possible
5	0.4 - 0.5	a) People strongly perceive motion; b) difficult to walk naturally; c) standing people may lose balance.
6	0.5 - 0.6	Most people cannot tolerate motion and are unable to walk naturally
7	0.6 - 0.7	People cannot walk or tolerate motion.
8	> 0.85	Objects begin to fall and people may be injured

Figure 3 Human perception levels. (Coull & Stafford 2008)

4.2 Comfort requirements in buildings

There are requirements regarding accelerations in buildings and these vary depending on the intended use of the structure. The demands are based on how long people are assumed to remain in the building which generates tougher requirements for residential buildings than for commercial areas like business and offices.

The Swedish Standard 10137 (2008) “Serviceability of buildings and walkways against vibration” presents an evaluation curve regarding acceptable horizontal motions for wind-induced vibrations, see Figure 4. The curves is derived by investigations from actual buildings in general use and are defined with peak acctions at the first natural frequencies in the principal structural direction of the building and in torsion.

For acceptable levels of oscillation the value corresponding to the acceleration and the first natural frequency for the building should be in the area range governed by the graphs shown in Figure 4. The limited values for permitted acceleration are for residential buildings about two-thirds of the limitation for offices.

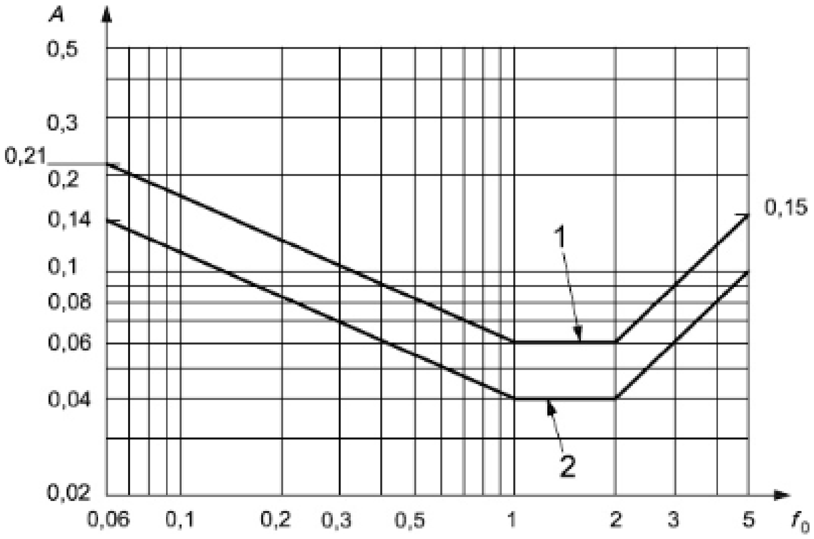


Figure 4 *Evaluation curves for wind-induced vibrations in buildings regarding peak acceleration. 1 – offices, 2 – residences, A – peak acceleration [m/s²], f_0 – first natural frequency. (Swedish Standard 2008)*

There are two different acceleration values that can be used for analyses, the mean acceleration (root-mean-square, r.m.s) and the peak acceleration. The peak value corresponds to the maximum acceleration over a given range while the mean value is the time average vibration energy during the same interval. Figure 5 shows the graphs for the different acceleration limitations.

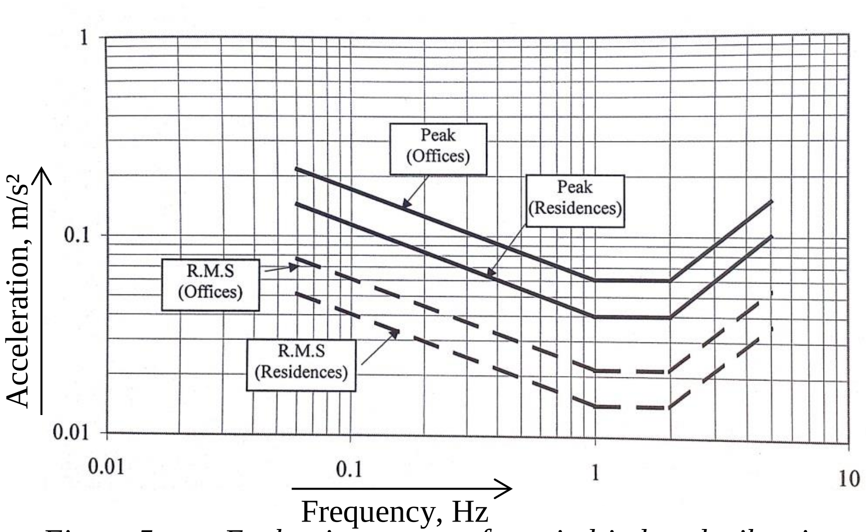


Figure 5 *Evaluations curve for wind-induced vibrations in buildings. (Handa 2008)*

4.3 Dynamic loads

Dynamic loads are loads that vary in time and can be periodic or non- periodic. A periodic load have a repetitive load pattern while a non-periodic load are more complicated to analyse with varying load configuration regarding intensity and duration. It is the dynamic load that causes oscillations of a structure and forces that give rise to dynamic loads are for example wind, earthquakes and vibrations from traffic.

4.3.1 Dynamic wind load

Wind is a complex phenomenon and a large number of flow situations can arise in the intersection of wind and a structure. Wind is composed of eddies in varying sizes and properties which are providing the gusty and turbulent character of the wind. Wind load causes both deflection and acceleration in a building which both can be essential in the serviceability design.

Wind load are a non-periodic load and vary significantly over short time intervals, with large amplitude fluctuations at high frequency intervals. The magnitude and frequency of the fluctuations is dependent on many factors associated with turbulence of the wind and local gusting effects caused by the structure and surrounding environment. The wind pressures on a structure are a function of the characteristics of the approaching wind, the geometry of the considered structure and proximity of the structures in upwind direction. The wind force pressure is not constant but highly fluctuating over time and is not uniformly distributed over the surface of the structure. The variation corresponds to differing frequency values for the pressure coefficient involved and the distribution of the wind frequencies indicates whether the wind load should be treated as a static or dynamic load. To simplify this complex wind characteristic, most international codes have adopted a simplified approach by utilising a quasi-steady assumption. This approach simply uses a single value equivalent static wind pressure, to represent the maximum peak pressure the structure would experience. (Cheung et al. 2007)

Some structures, particularly tall or slender, respond dynamically to the effects of wind. If the first natural frequency of vibrations for a structure is lower than a limited value the building should be treated as a dynamically loaded structure and a more refined analysis of the dynamic response should be performed. (Vessby 2011)

Calculation procedures described in standards is divided into static analysis and dynamic analysis methods. The static approach is based on a quasi-steady assumption, and assumes that the building is a fixed rigid body in the wind. According to Eurocode 1 the wind load can be regarded as static if the first natural frequency of a building is higher than five hertz.

The horizontal movement of a building occurs either by gusty winds or wind swirls. Gusts are causing the building to sway parallel to the wind direction while the wind swirls makes the building move in the other direction, perpendicular to the wind.

Wind load can in simple terms be divided into two components, a static and a dynamic part, see Figure 6.

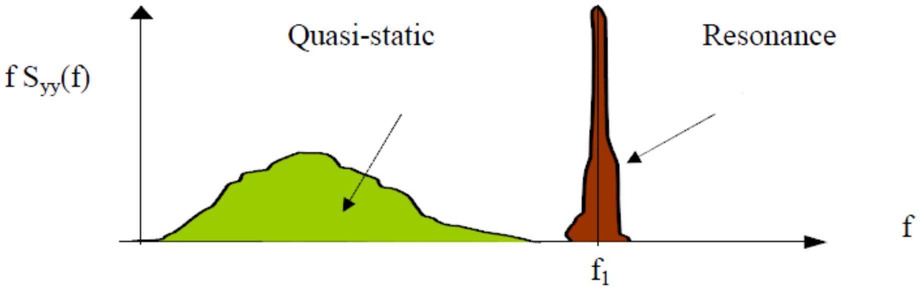


Figure 6 Response spectrum. (Handa 2008)

The horizontal acceleration of a structure is caused by the dynamic part, resonant part and the static load, quasi-static part of the response is omitted in calculations regarding human comfort. (Handa 2008)

4.4 Damping

The dynamic response of a building caused by wind load depends on its damping capability which implies the buildings capacity to absorb wind energy. The damping of a system is a measure of the degree of kinetic energy absorbed in the system and all systems have some form of damping. The damping reduces movements and makes the structure more stable which entails advantages in high buildings.

Eurocode 1 describes the damping of buildings with a coefficient δ, which is the logarithmic decrement of damping and consists of three factors see Equation (1).

$$\delta = \delta_s + \delta_a + \delta_d \tag{1}$$

where:

δ_s	is the structural logarithmic decrement of damping
δ_a	is the aerodynamic decrement of damping
δ_d	is the logarithmic decrement of damping due to special devices

The first part δ_s, is called structural damping and depends on how the material is affected by the movements of the building. Eurocode 1 gives approximate values for this factor depending on what kind of material that are used in the structural system.

The second part δ_a, is the aerodynamic damping, depending on the building aerodynamic shape. This damping is created by the air resistance against the building at oscillation and gives rise to vibrations due to the shape of the building.

The last part δ_d, is the damping due to special devices. These devices are used to control and increase the damping and are sometimes the only practical and economical solution for reducing vibrations in buildings.

4.5 Wind tunnel tests

There are many situations when analytical methods cannot be applied to estimate certain types of wind loads and associated structural response. Wind loading governs the design of some types of structures such as for example tall buildings and slender towers and the standards do not apply to structures that are of uncommon shape or rare location. In such situations a more accurate estimation of wind effects on buildings can be obtained through an aero elastic model in a wind tunnel test. Wind tunnel testing is a powerful tool that allows engineers to determine the nature and intensity of wind forces acting on complex structures and the testing becomes more and more common. Wind tunnel testing implies that air is simulated to emulate wind load and a model of the considered building and its surrounding are loaded by the air at various angles relative to the building orientation representing the wind directions. (Cheung et al. 2007)

4.6 Measures to reduce the acceleration

To decrease the oscillatory motions and to reduce the accelerations of a structure requires a complex solution. Increased stiffness, mass and dampers are measures that can be applied to reduce the acceleration.

4.6.1 Increased mass

Increased mass contributes to a larger structural damping and thus a reduction in acceleration. It is often necessary to significantly increase the mass to achieve the desired effect. This can cause problems with load carrying capacity and appearance.

In most cases, increased stiffness entails higher natural frequency and a reduction of acceleration. This often leads to greater cross-section dimensions and more mass, which is uneconomical and will restrict architectural freedom.

4.6.2 Mechanical dampers

The vibrations of a structure can be controlled by a so called mechanical damping system. The method is a cost effective solution and there are four types of mechanical damping systems that can be implement, passive, active, hybrid and semi-active dampers. (Cheung et al. 2007)

The passive damping system does not require any external energy. There is no opportunity to control the system to provide an adapted damping to the system at different vibrations.

For the active system a sensor are used for detecting the movements of the structure which entails that the damping can be controlled highly accurate. The system requires much energy to provide divergent force in a predetermined manner and the damping of this system can be adapted depending on the dynamic effects.

Hybrid damping system is a combination of passive and active damping system where different types of damping devices can be combined.

Semi-active dampers are used when the vibrations have low deviation of vibration ant the system are low energy consuming.

5 Case study, structural system and reference object

As a basis for the project a pre-dimensioned building are used as a reference object regarding building system, geometry and geographic location.

The construction system that the reference object is based on is designed according to a technical platform for industrial timber construction that Derome and Tyrens have developed, but with some modifications. The system is flexible and can be applied to most buildings that are within the technical limits regarding floor spans. Module customization is not necessary but is useful to facilitate and rationalizing the construction procedure. The system is designed for quick assembly and in good weather conditions one storey consisting of load bearing walls and slabs with an approximate area about 250 m^2 can be mounted in two days. The walls are delivered without scaling walls, which makes them easy to merge and the most parts are joined together with nails.

5.1 Description of the structural system

The stabilization system comprises of shear walls that are constructed like frames. The frames are consisting of vertical timber studs and horizontal header and sole plates with intermediate battens and sheeting nailed to the frame from both sides, see Figure 7.

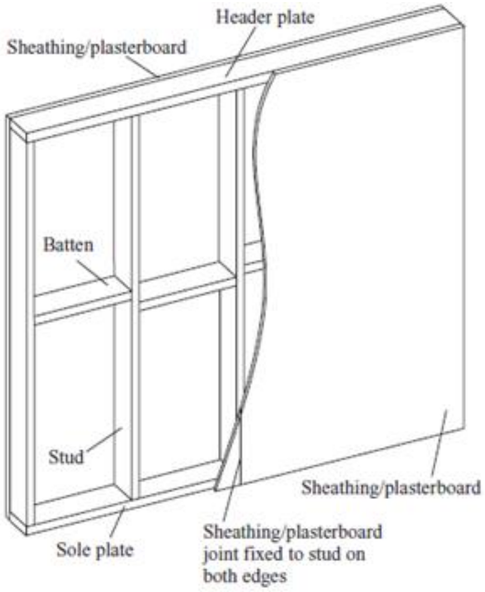


Figure 7 Details of a typical stud wall. (Porteous & Kermani 2007)

The exterior and apartment separating walls are used as stabilizing members and the studs are made of C24 timber with a centre spacing of 600 millimetre. The material of the sheeting varies depending on desired capacity and the dimensions of the sheeting are 1200 millimetre times 3000 millimetre.

The risk of settlements due to deformations in the timber parts is significant as the only wood component that are loaded perpendicular to the grain direction is the sill, sole plates and header plates. The sole and header plates are constructed in medium density fibre boards (MDF) and the sill consists of glulam.

A plan view of the buildings structural system is illustrated in Figure 8. The walls in the figure are used both as vertical load bearing members as well as components in the horizontal stabilization system.

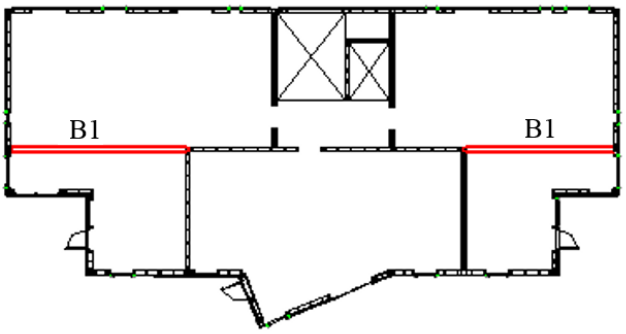


Figure 8 A plan view of the buildings structural system. The steel beams are marked B1.

The building is constructed almost exclusively in timber, except for the stairwell and steel beams used due to large floors spans, see Figure 8. The supporting steel beams make the system more flexible and customizable floor plans of the apartments. The stairwell and elevator shaft is constructed in steel and concrete and is in this thesis not considered as stabilizing members.

The load bearing walls are stacked on top of each other fixing the sheathing from the wall above to plywood strips located under the sole plate that are anchored by nails to the studs in the underlying walls, see Figure 9.

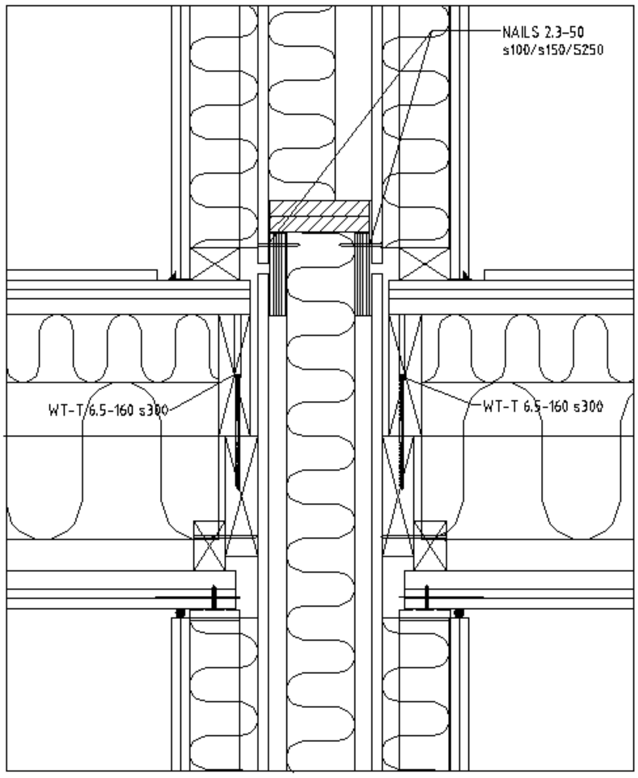


Figure 9 The connection between the exterior wall and slab.

On the bottom floor, the walls are anchored to the sole plate by nails and the sole plate is in turn attached to the foundation with expander bolts, see Figure 10. To take advantage of the walls regarding the horizontal stabilization the walls can also be anchor with brackets directly to the ground slab.

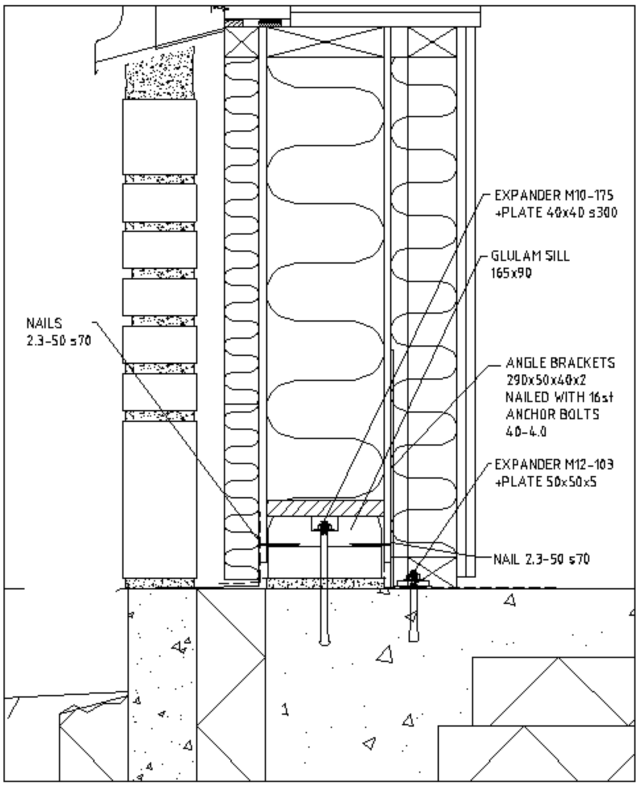


Figure 10 The connection between the sole plate and bottom concrete slab.

The slabs are constructed of Kerto S beams with intermediate isolation supporting horizontal particle boards covered with gypsum. With respect to acoustics problems, profiles are added to the structure placed under the beams with function to reduce the structure- borne sound, as seen in Figure 11. The slabs are fixed with bolts to battens along the load bearing walls that are fastened to the studs in underlying walls, see Figure 11.

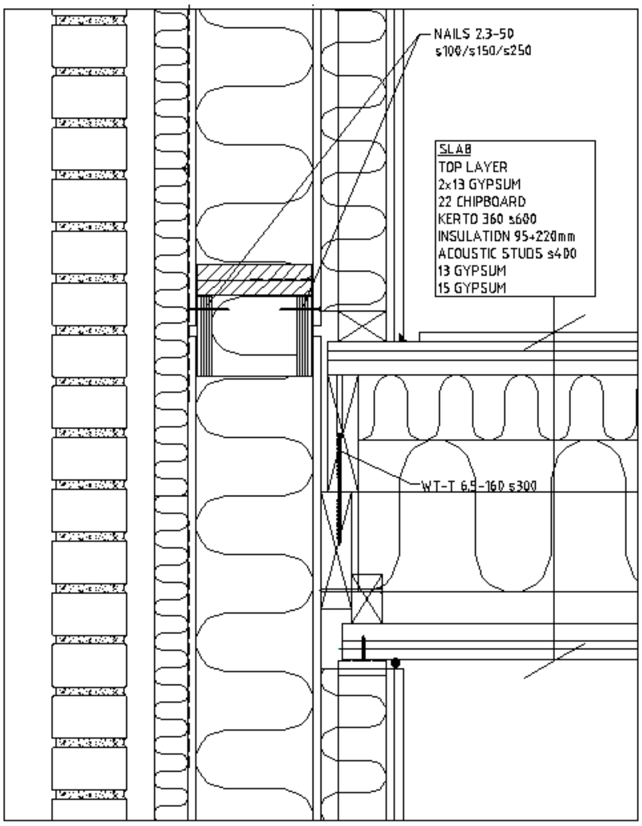


Figure 11 Overview of slab structure.

The slabs spans between the long side of the building's façade, the steel beams and the internal load bearing walls. The slabs are considered to work as one way slabs, as shown in Figure 12. The maximum slab span is for the system 6000 millimetre.

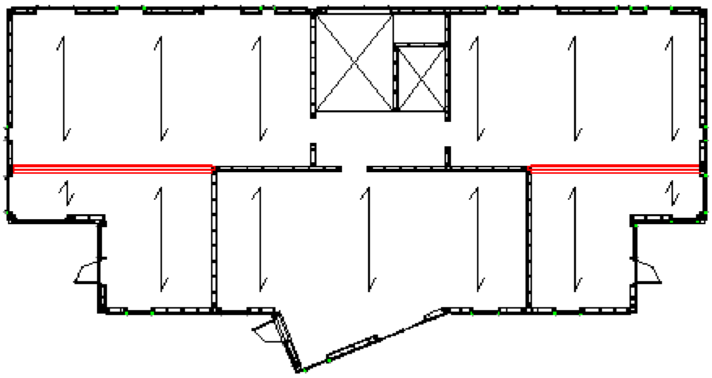


Figure 12 The load distribution and slab span direction.

The height for one floor is 2975 millimetre, and the interior headroom 2500 millimetre, see Figure 13.

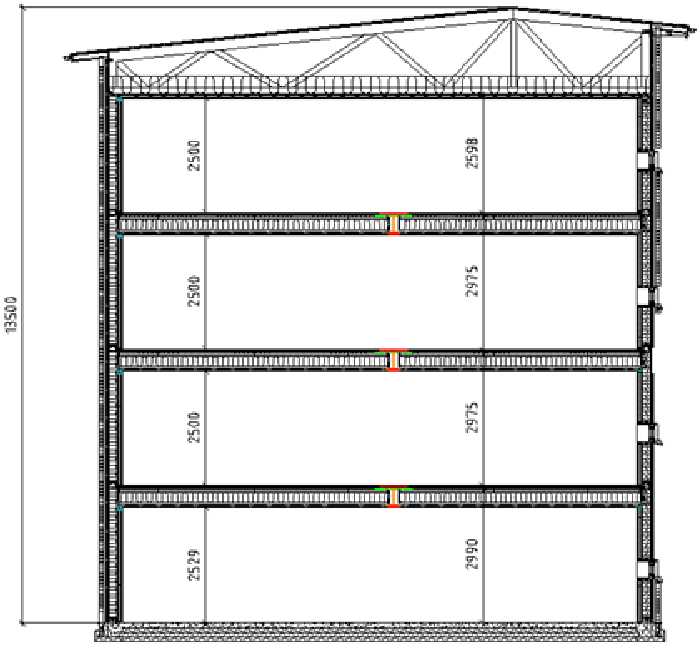


Figure 13   A section of the building.

The system is designed to be constructed in up to four storeys and holds fire class R (EI) -60 in the load bearing and separating structure. A good sound standard is achieved by separate scaling walls, and carefully designed structures to minimize resonances in the system.

5.2 Reference object

The reference object is a residential building that are a part of an on-going construction projects called Göingegården located in Varberg , see Figure 14.

Figure 14   The reference object of the project, Göingegården located in Varberg. (Derome Mark & Bostad 2012)

The building is constructed in four stories with a total height of 13.5 m and all storeys have the same execution and plan design.

The roof of the building is asymmetric with an inclination of six degrees on both sides of the ridges and is only supported by the external walls.

The studs in all load bearing walls are made of C24 timber and are placed with a centre distance of 600 millimetre and the dimensions of the studs in the exterior and interior walls are 170 millimetres, 145 millimetres respectively. The sheeting board has a width of 1200 millimetre and 11 millimetres OSB is used in the exterior walls and 15 millimetres plywood panels are used in the interior walls. The anchoring of the walls is performed with distributed anchor by the sole plate in both the external and internal walls.

All load bearing members in the building such as walls, slabs and roof trusses was prefabricated at factory. The total construction time for the building excluding the foundation was 28 weeks where the assembly of the frame took about three weeks. The maximum dimensions for production in the factory are three metre high and eight metre long elements.

5.2.1 Foundation properties

The foundation construction is designed as a traditional bottom slab with stiffened edges. The slab is 100 millimetre thick and is amplified along all the loadbearing walls by an additional depth of 200 millimetre. The design of the foundation is performed in geotechnical class 2 and the depth to firm bottom or hill is in the ranges between 1.4 and 5.2 metres. At a depth of about two metres from existing ground level is a sensitive layer of clayey silt. To minimize future settlements a preload of the ground was performed during two months before the construction process initiated and the top layer of soil was removed and filled out with compressible friction material.

5.2.2 Classes

The reference object is located in an environment corresponding to ***Terrain category 2*** which implies low vegetation and isolated obstacles with separations of at least 20 obstacle heights. The service class for the framework of the building is set to ***Service class 1*** corresponding to moisture content in the materials at a temperature of 20˚C and assuming that the relative humidity of the surrounding air only exceeding 65 percent for a few weeks every year.

5.2.3 Simplifications of the reference object

In comparison to the reference object, eight floors are added to the structure which implies that the building in total consists of 12 floors and are 36 metres high, see Figure 15.

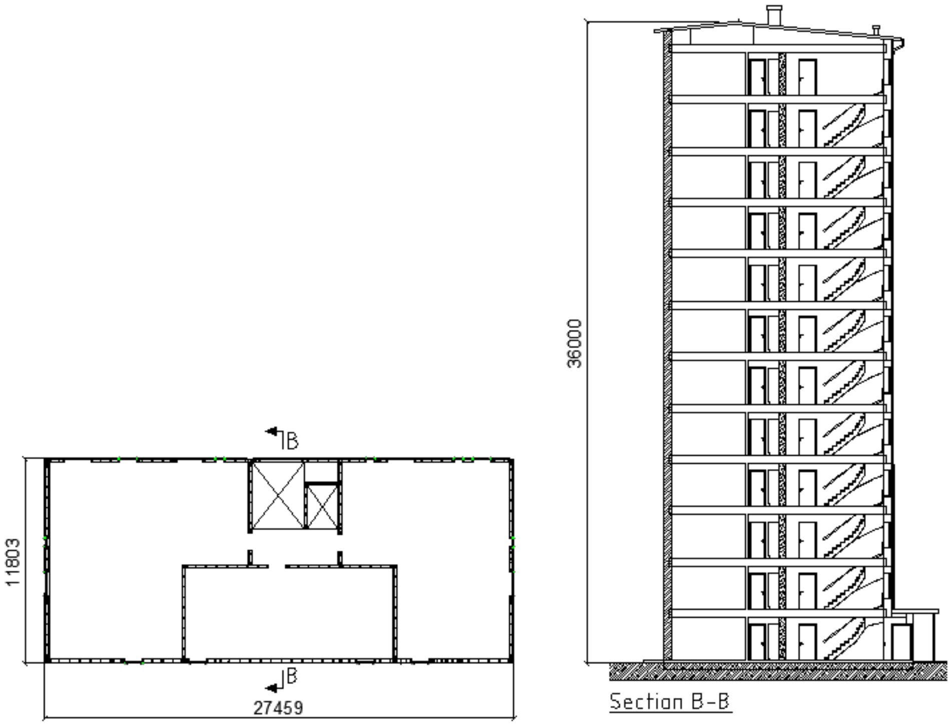


Figure 15 Simplification of the reference objects geometry and a section of the twelve-storey building.

Simplifications regarding the geometry of the reference objects are made to make the calculations more manageable and the impact from the balconies are neglected. The geometry and external dimensions of the building that are used in the project are shown in Figure 15.

The contribution from the stairwell and elevator shaft regarding the stability is in this project disregarded. No controls and dimensioning of the roof and slabs are made since these are dimensioned in the reference object and assumed to fulfil required demands.

6 Frame stabilisation

Since timber buildings are categories as light structures one of the main issues that have to be taken into consideration when designing is the stabilization against horizontal loads. The problem becomes more severe with each increasing floor.

AlFl structural components in a stabilizing system need to be dimensioned against horizontal actions and an anchoring system must be designed to transmit the loads acting on the building to the foundation.

6.1 Diaphragm action

Stabilization by diaphragm action implies that the stabilizing system consists of sheeting connected to a three-dimensional system through diverse configurations of fasteners and anchor connections. The sheeting provides a stiffness which is used for stabilization when the board is loaded in direction of its own plane.

The horizontal loads are transferred floor by floor through the external walls perpendicular to the wind direction on the buildings leeward and windward side to the slabs. The load is acting as line loads along the slab edges and is transmitted by shear forces in the slab ends to the underlying walls which are parallel to the wind direction, see Figure 16. It is important that the intersection between the slabs and walls are designed correctly so that the mode of action can be ensured.

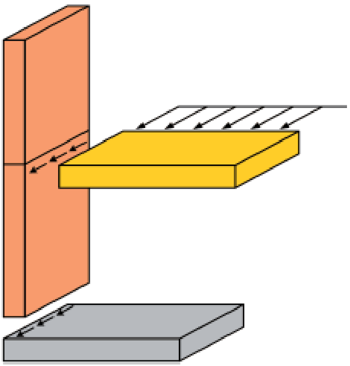


Figure 16 Showing how the horizontal slab loads are transmitted through the wall sheeting to the ground. (Källsner & Girhammar 2008)

Figure 17 indicates how diaphragm action in the roof and walls are used to stabilize a single-storey building against horizontal loads. When the load is acting on the long side of the façade the sheeting in the ceiling transfer the load acting on the upper part of wall to the stabilizing walls parallel to the wind direction in form of shear flow. These walls are then transferring the load down to the foundation. The load acting on the lower part of the wall is transmitted directly to the foundation provided that the sheeting has sufficient rigid capacity.

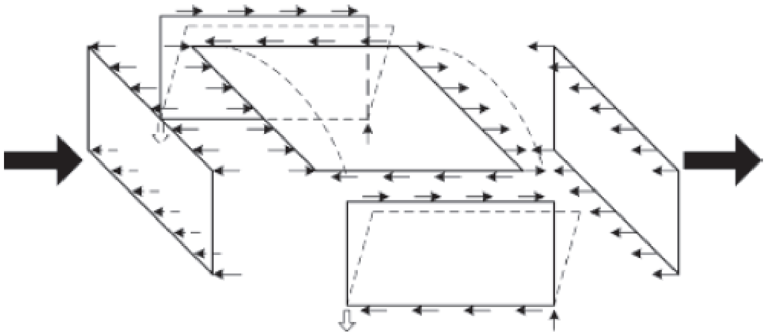


Figure 17 *Transfer of horizontal loads via the slab to the wall sheeting. (Källsner & Girhammar 2008)*

6.1.1 Load case division

The forces on each floor are added downwards in the structure and since the horizontal loads are acting on top of the walls bending and shear will occur, see Figure 18. How the distribution of these modes of action impact the individual wall unit depends on the relationship between the slab and wall stiffness. For rigid wall panels, the bending mode is crucial, while shear is dominant regarding the behaviour of timber shear walls.

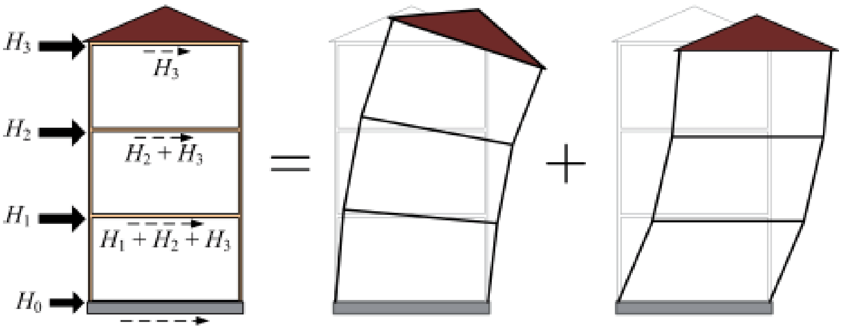


Figure 18 *Frame stability regarding bending- and shear deformations. (Källsner & Girhammar 2008)*

Regarding design of multi-storey buildings with weak shear wall panels it is suitable to treat each horizontal line load acting along the slab edge on each floor as partial load acting on the entire building, see Figure 19. When the wall is in complete anchored there are advantages to divide the anchoring force by a long length and the impact from all partial loads from each floor is added from the top of the building down to the foundation.

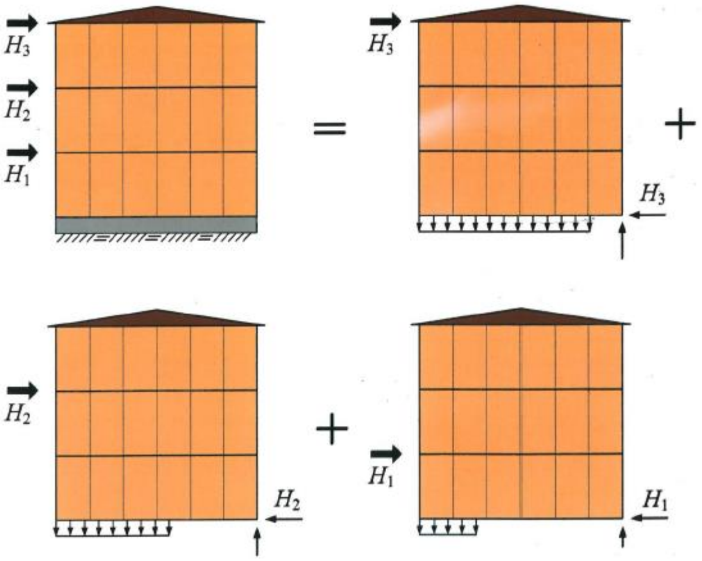


Figure 19 Load case division for multi-storey buildings with assumption of plastic condition. (Källsner & Girhammar 2008)

6.1.2 Design method

Traditionally a simplified plastic design method has been used to evaluate the resistance of the shear walls, based on shear capacity of an individual fastener according to EC5 (Method A). The handbook "Horisontalstabilisering av träregelstommar" by Källsner and Girhammar (2008) that the calculations in this thesis are based on, presents a new plastic method for design and dimension of sheeted timber stud frames with respect to horizontal stabilization. The plastic method implies that the mechanical connections have plastic properties and exploits the fact that many of the units have sufficient ductility before its capacity is reduced in a decisive way which leads to a better material utilization. It is also possible to control the flow of force in an effective way and the method provides the opportunity to utilize the contribution to the resistance of wall segments with openings in form of windows and doors.

The plastic method gives the designer new opportunities, especially for tall buildings since the designer do not have to use simplified and constructive limited assumptions. The method thereby contributing to increased competitiveness of timber frames with respect to other structural materials.

According to the traditionally elastic design method, the leading and trailing studs in the stabilizing walls needs to be sufficiently anchored against uplift forces but according to the plastic method it is possible to use several different options for the anchoring of the walls. In the elastic method complete anchor of the leading stud can be used, but it is also possible to transmit tensile forces by using the sheets via the sheeting stud joints to bring down the forces to the substructure by anchoring the sole plate completely or partially.

Figure 20 shows three different methods to vertically anchor a horizontally loaded sheeted timber stud wall.

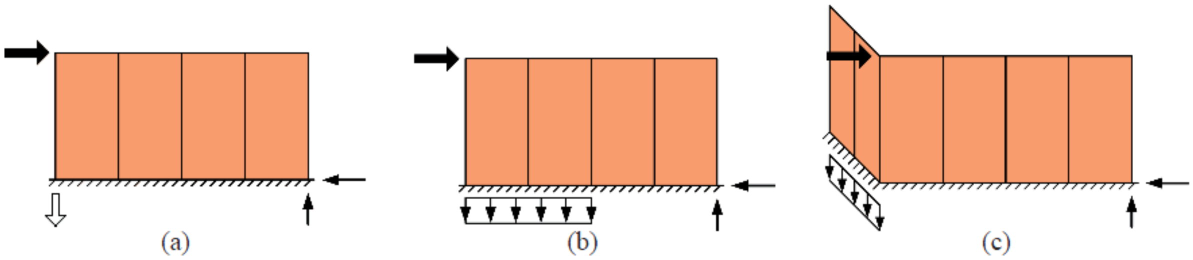


Figure 20 Different methods to vertically anchor a horizontal loaded sheeted timber stud wall. (a) Concentrated anchor at leading stud. (b) Distributed anchor by the sole plate. (c) Anchoring via transverse walls. (Källsner & Girhammar 2008)

In this thesis, it is assumed that the interior walls have a fully anchored leading stud (a) while the exterior walls have a distributed anchoring by the sole plate (b).

A constructive aspect that needs to be taken into consideration is the major anchoring forces that occur in the leading wall stud and the stabilizing pressure forces in the trailing stud in the walls. This applies particularly to the large compressive forces in the sole plate and buckling of the studs. Openings in form of windows and doors also give rise to vertical pressures and anchor forces that must be considered.

7 Loads

The loads considered in this thesis are self-weight, wind load, snow load, imposed load and unintended inclination. Detailed calculations can be found in Appendix B.

7.1 Self-weight

The self-weight is the load from dead weight of building parts and is considered as a permanent action. The self-weight of the structural components in the building are in this thesis estimated to:

Inner walls $g_{innerwall} = 1.7\ kN/m$

Outer walls $g_{outerwall} = 1.3\ kN/m$

Slabs $g_{floor} = 0.6\ \ kN/m^2$

Roof $g_{roof} = 0.6\ kN/m^2$

7.2 Calculation of snow load

The roof of the building is affected by a uniformly distributed snow load that are calculated according to Eurocode 1 part 1-3 (European Standard 2003) and the national annex (EKS 8 2011).

The snow load acting on the roof is calculated according to Equation (2).

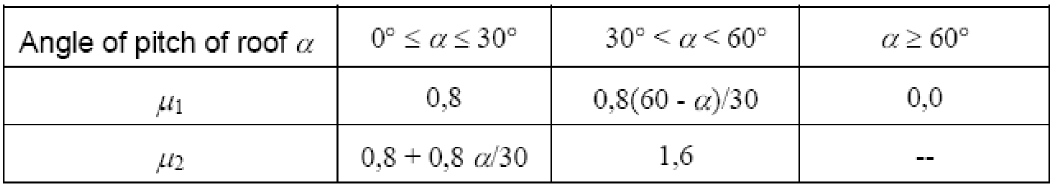
$$S = \mu_1 \cdot C_e \cdot C_t \cdot S_k \tag{2}$$

where:

μ_1 is the snow load shape coefficient
C_e is the exposure coefficient
C_t is the thermal coefficient
S_k is the characteristic value of snow load on the ground

The value of the roof shape coefficient, μ_1, for undrafted and drifted snow load depends on the building shape and the angel of the roof, see Table 1.

Table 1 Snow load shape coefficients. (European Standard 2003)



Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	0,8(60 - α)/30	0,0
μ_2	0,8 + 0,8 α/30	1,6	--

The snow load shape coefficients that should be used for pitched roofs are divided into three different cases and are given in Figure 21.

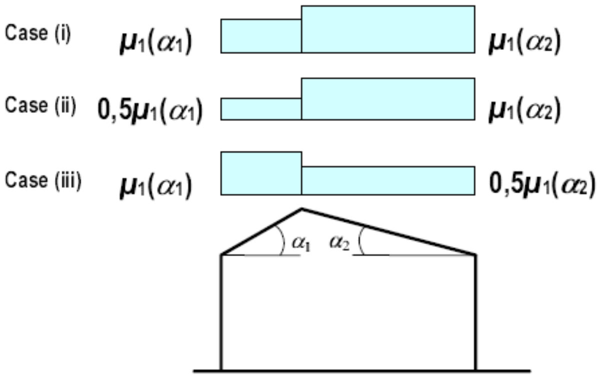


Figure 21 Snow load shape coefficients - pitched roofs. (European Standard 2003)

The exposure coefficient, C_e, consider the future development around the site and should for normal topography be taken as 1.0.

The thermal coefficient, C_t, are used to account for the reduction of snow loads on roofs with high thermal transmittance over 1 W/m^{2}K, for all other cases $C_t = 1$.

S_k is the characteristic snow load on the ground with a probability of 0.98 that it is not being exceeded once a year with a return period of 50 years. For the relevant building location in this thesis the characteristic snow load are estimated to $S_k = 1.5kN/m^2$.

7.3 Calculation of wind load

The wind load acting on the building are calculated according to Eurocode 1 part 1-4 (European Standard 2005) and the national annex (EKS 8 2011). Here follows a brief summary of the design procedure, for detailed calculation description see Appendix B.

The wind force, F_w, acting on the building are determined by vectorial summation of the forces calculated from the external pressure, internal pressure and the friction forces using the following Equations (3), (4) and (5).

External forces:

$$F_{w.e} = c_s c_d \cdot \sum w_e \cdot A_{ref} \tag{3}$$

Internal forces:

$$F_{w.i} = \sum w_i \cdot A_{ref} \tag{4}$$

Friction forces:

$$F_{fr} = c_{fr} \cdot q_p(z_e) \cdot A_{fr} \tag{5}$$

where:

$c_s c_d$	is the structural factor
w_e	is the wind pressure acting on the external surfaces
w_i	is the wind pressure acting on the internal surfaces
A_{ref}	is the reference area of the individual surface
c_{fr}	is the friction coefficient
$q_p(z_e)$	is the peak velocity pressure at reference height z_e

A_{fr} is the area of external surface parallel to the wind

According to EC1 part 1-4, the wind force acting on a structure with a height over 15 metres, or a natural frequency below 5Hz, must take into account the fact that wind is a dynamic load. This means the emergence of non-simultaneous gust wind pressure and fluctuations in construction as a result of turbulence. To simplify these calculation EC1 part 1-4 indicates that the external wind pressure can be multiplied by a structural factor, $c_s c_d$, which are calculated according to Equation (6).

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot I_v(z) \cdot \sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z)} \tag{6}$$

where:

k_p is the peak factor

$I_v(z)$ is the turbulence intensity at height z

B^2 is the background factor, allowing for the lack of full correlation of the pressure on the structure surface

R^2 is the resonance response factor, allowing for turbulence in resonance with the vibration mode

The external and internal pressure is considered to act at the same time and the worst combination of these pressures is considered in the calculations.

The wind pressure acting on the external surfaces of the building, w_e, are obtained from Equation (7).

$$w_e = q_p(z) \cdot c_{pe} \tag{7}$$

where:

q_p is the is the peak velocity pressure

c_{pe} is the pressure coefficient for the external pressure

The wind pressure acting on the internal surfaces of the building, w_i, are obtained from Equation (8).

$$w_i = q_p(z) \cdot c_{pi} \tag{8}$$

where:

q_p is the is the peak velocity pressure

c_{pi} is the pressure coefficient for the internal pressure

The peak velocity pressure, $q_p(z_e)$, includes mean and short-term velocity fluctuations and is calculated according to Equation (9).

$$q_p(z_e) = [1 + 6 \cdot I_v(z)] \left[k_r \cdot \ln\left(\frac{\mathrm{z}}{\mathrm{z}_0}\right)\right]^2 \cdot \mathrm{q_b} \tag{9}$$

where:

$I_v(z)$ is the turbulence intensity at height z

k_r is the terrain factor depending on the roughness length z_0

z is the height above ground

z_0 is the roughness length

q_b is the is the basic velocity pressure

The reference heights, z_e, depend on the ratio between the building face and the height of the building (h/b). The reference heights and the corresponding velocity pressure profile differs in this thesis depending on which side of the house that is loaded by the wind and the model for the different cases are shown in Figure 22 and Figure 23 below.

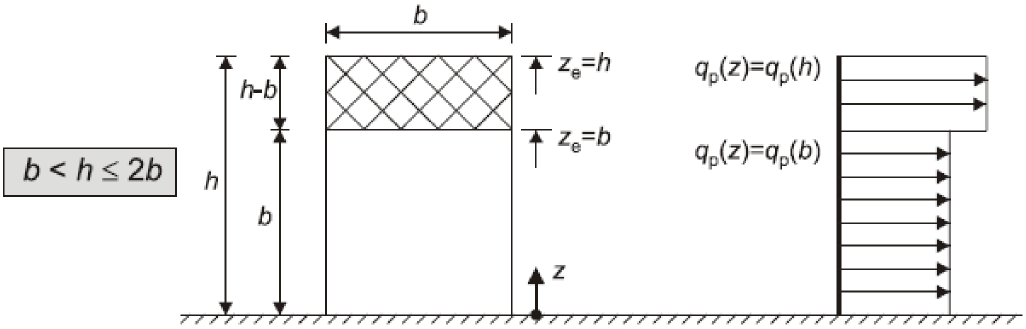


Figure 22 *The reference heights and the corresponding velocity pressure profile when the wind are acting on the long side of the façade on the building. (European Standard 2003)*

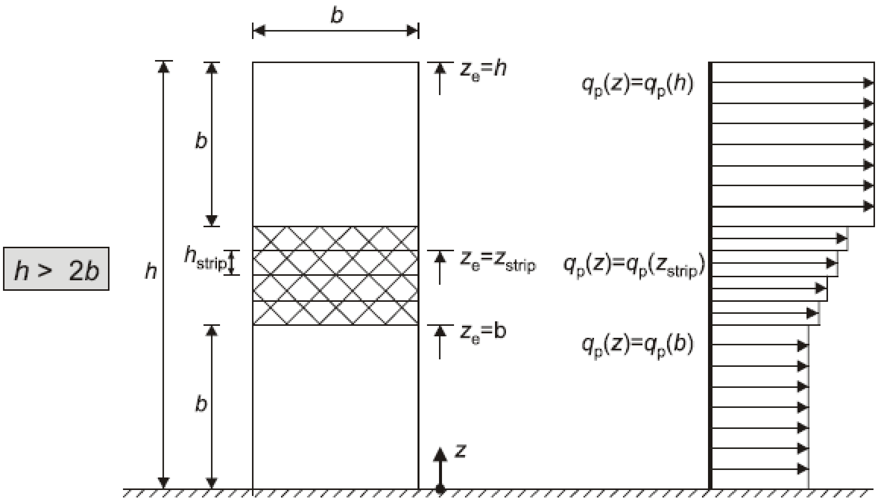


Figure 23 *The reference heights and the corresponding velocity pressure profile when the wind are acting on the short side of the façade on the building. (European Standard 2003)*

7.4 Calculations of imposed load

Imposed loads on buildings are those arising from occupancy and include for example normal use by persons, furniture and moveable objects. The load is calculated according to Eurocode 1 (European Standard 2002) and the national annex (EKS 8 2011).

Areas in buildings are divided into categories according to their specific intended use. In this thesis the load for category A is used which corresponds to areas for domestic and residential buildings.

$$q_k = 2\ kN/m^2$$

The self-weight of movable partitions are also taken into account by a uniformly distributed load which is added to the imposed loads for floors. This defined uniformly distributed load is dependent on the self-weight of the partitions and in this thesis a value of $q_k = 0.5\ kN/m^2$ are used which corresponds to movable partitions with a self-weight of $\leq 1\ kN/m$ wall length.

The total imposed load from several storeys are reduced and multiplied by a factor, α_n, which is calculated according to Equation (10).

$$\alpha_n = \frac{2 + (n-2)\Psi_0}{n} \tag{10}$$

where:

n	is the number of storeys (>2) above the loaded structural element from the same category
Ψ_0	is the factor for combination value of a variable action

7.5 Calculations of unintended inclination

The unintended inclination is calculated according to a method described in “Massivträhandboken” (2006).

For the control of the stabilizing system the load from unintended inclination is converted to an equivalent horizontal line load along the slab edges, and is added to the wind loads acting on each floor, see Figure 24.

The inclination is assumed to be composed of a systematic- and a random part and an average inclination can be calculated according to Equation (11).

$$\alpha_{md} = \alpha_0 + \frac{\alpha_\delta}{\sqrt{n}} \tag{11}$$

where:

α_0	is the systematic part
α_δ	is the random part
n	is the number of structural elements

For the system used in this thesis interaction between all load bearing walls, perpendicular to the wind direction, within a single storey are assumed.

The equivalent horizontal force due to the inclination of each floor is calculated according to Equation (12).

$$F_{hi,ekv} = V_{di} \cdot n \cdot \alpha_{md} \tag{12}$$

where:

V_{di}	is the design value of the average vertical force for each floor on the underlying inclined walls
n	is the number of structural elements

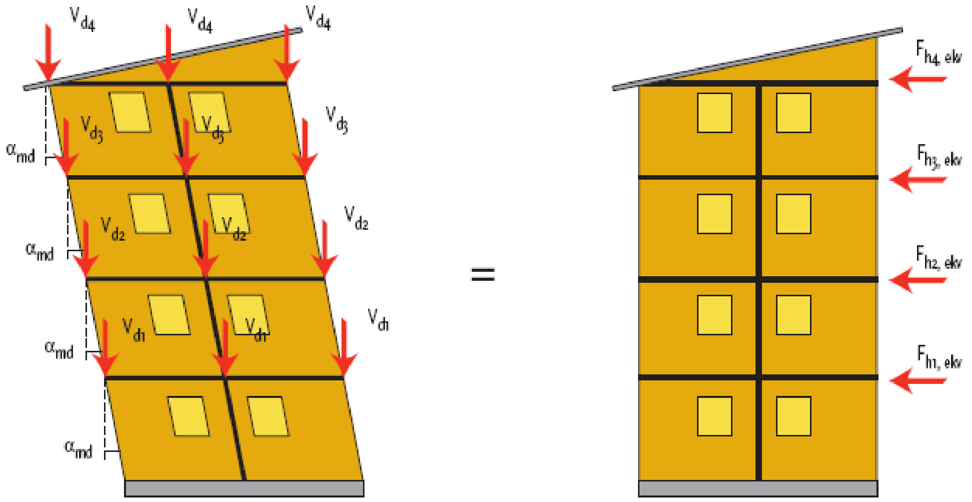


Figure 24 Equivalent horizontal forces on the floor structure due to misalignment of the bearing walls. (Massivträhandboken 2006)

8 Design in Ultimate limit state

Calculations in the ultimate limit states correspond to the most unfavourable load case that might occur during a constructions lifetime, approximately 50-100 years. The design in the ultimate limit states are performed to satisfy certain safety requirements, which are formulated on the basis of risk of personal injury in case of collapse.

8.1 Calculation procedure

The dimensions of the wall studs are estimated by calculations considering vertical loads with a contribution from the wind load regarding the external walls .The design of joints and sheeting dimensions is performed by calculations including both vertical and horizontal loads acting on the building and are calculated with a plastic design method based on the manual “Horisontalstabilisering av skivregelstommar” (2008).

When all loads acting on the building are calculated, the vertical loads are included in relevant load combinations and appropriate dimensions of structural components are estimated. The building must be controlled for both local and global stability and in terms of global stability this work only considers tilting.

The horizontal loads are distributed on the stabilizing walls and both the external and apartment separating walls are used as stabilizing members. A detail study is performed of the interior and the external walls that carry most load per unit length and the calculations that are included are:

- Horizontal load bearing capacity
- Anchoring force
- Contact pressure
- Control of front studs
- Control of end studs

8.2 Load distribution

The distribution of the horizontal load depends on the location of the stabilizing walls and the rigidity ratio between the floor and walls. In this thesis the floor slab board is considered to be infinitely stiff in relation to the wallboards. The walls of the building is placed asymmetrically which entails a rotating moment.

The horizontal loads acting on a structure are causing a deformation that can be divided in a translation and rotation. The bracing element has a stiffness with regard to translation in x- respective y-direction. The horizontal loads are divided in components in x- and y-directions and these load cases are solved separately.

In the stabilising system it is possible to determine a rotation centre that depends on the arrangement of the bracing units and the stiffness distribution. When the resultant of the horizontal loads acts besides the rotation centre it will cause a translation and a rotation of the system which both has to be considered. The coordinates of the rotation centre is found by Equation (13) and (14).

$$x_t = \frac{\sum a_i \cdot s_{yi}}{\sum s_{yi}} \tag{13}$$

$$y_t = \frac{\sum b_i \cdot s_{xi}}{\sum s_{xi}} \tag{14}$$

where:

s_{yi}	is the stiffness of bracing member i in y-direction
a_i	is the distance between origo and the centre of the bracing element in x-direction
s_{xi}	is the stiffness of bracing member i in x-direction
b_i	is the distance between origo and the centre of the bracing element in y-direction

The bracing force in each direction corresponding to translation is found by Equation (15) and (16) respectively.

$$H_{xi} = H_x \frac{s_{xi}}{\sum s_{xi}} \tag{15}$$

$$H_{yi} = H_y \frac{s_{yi}}{\sum s_{yi}} \tag{16}$$

where:

H_x	is the horizontal load in x-direction
H_y	is the horizontal load in y-direction

The impact from rotation is calculated according to Equation (17) and (18).

$$H_{i,Cx} = T \frac{s_{xi} \cdot y_i}{S_T} \tag{17}$$

$$H_{i,Cy} = T \frac{s_{yi} \cdot x_i}{S_T} \tag{18}$$

where:

T	is the global torsional moment is calculated according to Equation (19)
r	is the radial distance from the rotation center
S_T	is the global torsional stiffness calculated according to Equation (20)
x_i	is the x-coordinate
y_i	is the y-coordinate

The global torsional moment is calculated according to Equation (19).

$$T = H \cdot e \tag{19}$$

where:

H	is the horizontal force
e	is the eccentricity, the distance perpendicular to the load between the load resultant and the rotation centre.

For the global torsional stiffness, see Equation (20).

$$S_T = \sum (s_i \cdot {r_i}^2) = \sum (s_{xi} \cdot {y_i}^2) + \sum (s_{yi} \cdot {x_i}^2) \tag{20}$$

where:

s_i	is the stiffness of bracing member

r_i is the radial distance from the rotation centre

The total load acting on the bracing elements are then calculated by addition of the rotation and translation contribution, see Equation (21) and (22).

$$H_{xi} = H_{i,Cx} + H_{xi} \tag{21}$$

$$H_{yi} = H_{i,Cy} + H_{yi} \tag{22}$$

where:

$H_{i,Cx}$ is the contribution from rotation

H_{xi} is the contribution from translation

$H_{i,Cy}$ is the contribution from rotation

H_{yi} is the contribution from translation

8.3 Calculations and sizing

Buildings are subjected to both horizontal and vertical loads that need to be transferred down through the building to the foundation in a secure manner. The vertical loads are transmitted to the foundation by the slabs to the studs while the horizontal loads are transferred down to the foundation as shear forces in the wall sheeting, so-called diaphragm action.

8.3.1 Tilting

The horizontal loads acting on the stabilizing system may result in tilting of the building. This load needs to be taken care of in terms of pressure and shear stresses between the foundation and the subgrade, as can be seen in Figure 25. The spread of compressive stresses on the foundation is only schematically displayed. The foundation is assumed to be stiff when controlling tilting. The vertical loads such counteract the tilting moment of horizontal loads and increases the contact pressure and therefore also the friction with the sub grade.

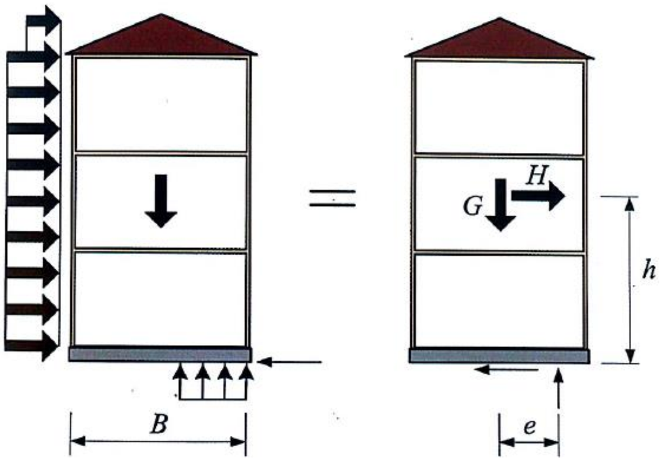


Figure 25 Framework stability for tilting and sliding of the building as a whole (self-weight G for both the building and base plate). (Källsner & Girhammar 2008)

The safety against tilting is checked by showing that the stabilizing moment is greater than the tilting moment, see Equation (23).

$$G \cdot e > H \cdot h \tag{23}$$

where:

G	is the self-weight of the building, including the foundation
H	is the horizontal loads
h	is the height to the horizontal load

To make sure that the building is safe from tilting the vertical resultant caused by all horizontal loads are controlled so that they are within the border of the core for the rectangular plan of the foundation, Equation (24). (Massivträhandboken 2006)

$$e \leq \frac{B}{6} \tag{24}$$

where:

e	is the vertical resultant caused by all horizontal loads
B	is the width of the building

Regarding tilting controls, the total vertical design load are reduced with regard to favourable loading effect.

8.3.2 Sizing of studs

The requirements on the studs are calculated and controlled according to Eurocode 5 part 1-1 (2004). To obtain the design condition for the stud design the worst load combination is used.

The load bearing walls are subjected to vertical load, horizontal longitudinal loads and the external walls are also subjected to horizontal load perpendicular to the plane. The load is distributed on each wall stud, with an estimated load width. The transmission of vertical loads from overlying floor is performed by the slab construction that spreads the load to the wall studs in form of line loads along the load-bearing walls. The vertical load on each individual wall stud is calculated by multiplying the vertical load effects along the height of the building with a width of the centre distance between the studs. The loads are transferred via the studs down to the sole and header plate in the underlying wall and this connection must be controlled regarding local pressure perpendicular to the grain.

The calculations regarding vertical load capacity including sizing of the studs and control of the sole plate are performed according to Eurocode 5.

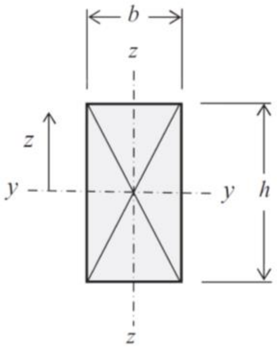


Figure 26 Cross-section of a rectangular beam. (Porteous & Kermani 2007, pp. 105)

When the relative slenderness, $\lambda_{rel.y}$, see Equation (25) exceeds the value of 0.3, axial load buckling effects has to be taken into account. No benefit is taken of any plastic behaviour in the member and the ultimate load is achieved when the material reaches its failure strength in the extreme fibre. If the relative slenderness of members regarding bending around the major axis is lower or equal to 0.75, lateral (tensional) torsional instability cannot occur.

$$\lambda_{rel.y} = \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} \tag{25}$$

where:

λ_y is the slenderness ratio of a stud about the y-y axis

$f_{c.0.k}$ is the characteristic compression strength parallel to the grain

$E_{0.05}$ is the fifth-percentile modulus of elasticity parallel to the grain

The internal walls are only subjected to axial compression and should comply with Equation (26).

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} \leq 1 \tag{26}$$

where:

$\sigma_{c.0.d}$ is the design compressive stress parallel to the grain

$f_{c.0.d}$ is the design compressive strength of the stud parallel to the grain

$k_{c.y}$ is the instability factor about the y-y axis

The design compression strength about the y-y axis is calculated according to Equation (27).

$$f_{c.d} = \frac{k_{mod} \cdot k_{sys} \cdot f_{c.k}}{\gamma_M} \tag{27}$$

where:

k_{mod} is the modification factor for duration of load and moisture content

k_{sys} is the system strength factor

$f_{c.k}$ is the characteristic compression strength

γ_M is the partial factor for material properties

The external walls are subjected to both axial compression and out of plane bending due to horizontal loads. This requires that the compressive stress in the member is less than the compressive strength of the member. In the case where there is axial loading and bending only about the major y-y axis, the strength validation equation is reduced to Equation (28).

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} \leq 1 \tag{28}$$

where:

$\sigma_{c.0.d}$	is the design compressive stress parallel to the grain
$f_{c.0.d}$	is the design compressive strength of the stud parallel to the grain
$k_{c.y}$	is the instability factor about the y-y axis
$\sigma_{m.y.d}$	is the design bending stress about the y-y axis due to permanent and combined medium-term vertical and wind (dominant) action
$f_{m.y.d}$	is the design bending strength about the y-y axis

The design bending stress at the extreme fibre is calculated according to Equation (29).

$$\sigma_{m.y.d} = \frac{M_d}{W_y} \tag{29}$$

where:

M_d	is the design moment
W_y	is the section modulus

The design bending strength about the y-y axis is calculated according to Equation (30).

$$f_{m.y.d} = \frac{k_{mod} \cdot k_{sys} \cdot k_h \cdot f_{m.y.k}}{\gamma_M} \tag{30}$$

where:

k_{mod}	is the modification factor for duration of load and moisture content
k_{sys}	is the system strength factor
k_h	is the depth factor
$f_{m.y.k}$	is the characteristic bending strength about the principal y-axis
γ_M	is the partial factor for material properties

8.3.3 Horizontal capacity

The horizontal loads are transmitted through diaphragm action by the slab to the stabilizing walls. The slabs are assumed to have sufficient bearing capacity and rigidity to serve as a board and work as a rigid body. Both the interior and external walls are used as stabilizing members and the sheeting is nailed to the timber frame in the walls to ensure sufficient resistance against horizontal loads. The internal and external walls that are carrying most load have been identified and are studied in detail concerning horizontal capacity, anchor and reaction forces.

In this thesis the contribution from walls with openings are taken into account. To be able to use this method it needs to be assumed that all floors have the same height and that the aperture size and placement is identical at each floor.

8.3.3.1 Design of shear walls in several floors with openings according to Källsner & Girhammar (2008)

Here follows a brief summary of the design procedure regarding horizontal stabilisation for shear walls in several floors with openings. For detailed calculation descriptions see the handbook "Horisontalstabilisering av träregelstommar" that the calculations are based on.

A wall with apertures is divided into different wall segments bounded by the openings. The calculations of each wall part are performed separately and the total effective length of the entire wall is obtained by summation of the values from the different parts.

The calculations are performed repeatedly for each floor downwards in the building.

8.3.3.2 Calculation procedure according to Källsner & Girhammar (2008)

The procedure begins with determination of the loads that are acting on the vertical studs. There is a substantial difference between rigid and shear weak walls concerning how the vertical loads impact the stabilization. For a rigid wall all vertical forces can be utilized to counteract the tilting moment of the wall considered as a rigid body while for shear weak walls, only a part of the vertical forces are used as stabilizer, see Figure 27. The right part of the wall is completely plasticized while the left portion is in the elastic range which entails that only the self-weight acting on the left side of the wall will prevent uplift while the vertical forces acting on the right part of the wall will pass right through the vertical stud down to the underlying structural members.

The wall is assumed to consist of three fictive elements, l_1, l_2 and l_3, see Figure 27. The length l_1 is determined from the condition that the full plastic shear force capacity of the wall is achieved in this vertical section.

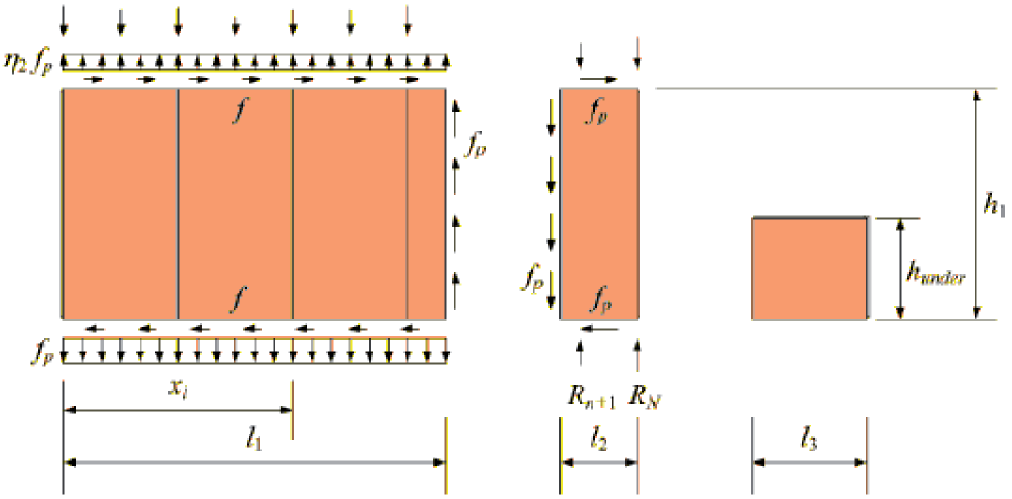


Figure 27 Division of a wall with opening in to different fictive parts. (Källsner & Girhammar 2008)

The left part of the wall are used primarily for anchoring against lifting forces, while the right part is used only for transmission of shear forces to the sole plate. The length l_1 is calculated according to Equation (31).

$$l_1 = h_{tot}\left(1 - \frac{\sum_{i=k}^{n} V_i}{f_p \cdot h_{tot}}\right) \tag{31}$$

where:

f_p	is the plastic shear flow
h_{tot}	is the total height of the wall panel for the current load case
V_i	is the equivalent force
n	is the number of studs

The remaining part of the wall is corresponding to the distance l_2, see Equation (32), and l_3 are the width of the opening.

$$l_2 = l - l_1 \tag{32}$$

where:

l	is the length of the wall part

If the wall is short, it is not certain that the plastic shear resistance is reached within the wall length which provides that $l_1 = l$.

In the left wall part the displacements in the vertical joints between the boards is located in the elastic range, which indicates small movements and the outer vertical forces will be transmitted through the vertical stud members to the boards. In the right part on the other hand the displacements are large which means that the external vertical loads cannot be transferred to this part of the board but is transmitted through the joists directly to the sole plate. The horizontal resistance of the wall H, corresponding to the both wall parts are calculated using Equation (33).

$$H = f_p \cdot l_{eff} \tag{33}$$

where:

f_p	is the plastic shear flow
l_{eff}	is the effective wall length according to Equation (35)

The plastic shear flow in the wall is calculated using Equation (34).

$$f_p = \frac{F_p}{s} \tag{34}$$

where:

F_p	is the plastic capacity of the connections between studs and sheeting
s	is the center distance between the joints

A wall that includes apertures is divided into fictive parts that are bounded by the openings and that are analysed separately. The approach is to obtain maximum shear flow transfer in the sheeting which connects to the aperture and a contribution to the resistance from the sheeting under the opening are considered.

Two main cases are treated. The first one corresponds to the case when the board under the opening archives full plastic shear flow and the second indicates that the plastic shear flow are reached a part in to the board under the opening. The latter case may occur when the height under the opening is greater than the length of the wall to the left of the aperture times the factor η_j, $h_{under} > \eta_j(l_1 + l_2)$ where η_j corresponds to the ratio between the local and total height.

The effective length of the wall, l_{eff}, is calculated differently for main case one and two.

For main case one the effective length are calculated according to Equation (35).

$$l_{eff} = \left(\frac{l_1}{2 \cdot h_{tot}} + \frac{V_{ekv}}{f_p \cdot h_{tot}}\right) l_1 + l_2 + \frac{h_{under}}{h} \lambda \cdot l_3 \tag{35}$$

where:

l_3 is the length of the opening
V_{ekv} is the equivalent force acting on the leading stud
h is the height of one floor
h_{under} is the height of the wall below the opening
λ is a dimensionless multiplier for the reduction of shear flow under the window opening

The factor for the reduction of shear flow under the window is calculated according to Equation (36). The first case corresponds to no reduction of shear flow and the second and third case corresponds to the situation when the contact pressure entails large shear forces in the header and sole plate on the bottom floor respectively. The final case is indicating that the contact forces are too large with respect to the potential of fragmentation of the sheeting.

$$\lambda = min \begin{cases} 1 \\ \frac{h}{h_{under}} \left[\left(\sqrt{1 - (1 - \frac{1}{n_{floor}})^2} - \frac{l_1}{2 \cdot h_{tot}} - \frac{V_{ekv}}{f_p \cdot h_{tot}} \right) l_1 + \frac{n_{knut} \cdot F_{knut}}{f_p} \right. \\ \frac{2 \cdot h}{l_3 \cdot (h - h_{under})} \left[\left(\frac{l_1}{2 \cdot h_{tot}} + \frac{V_{ekv}}{f_p \cdot h_{tot}} \right) l_1 + l_2 \right] \\ \frac{b_{full}}{l_3} \end{cases} \tag{36}$$

where:

n_{floor} is the number of floors
n_{knut} is the number of shear joints that are active for the transfer of shear force
F_{knut} is the shear resistance of a junction unit
b_{full} is the full board width

The effective length of main case two is calculated according to Equation (37).

$$l_{eff} = \left(\frac{l_1}{2 \cdot h_{tot}} + \frac{V_{ekv}}{f_p \cdot h_{tot}}\right) l_1 + l_2 + \frac{f_{opening} \cdot h_{under}}{f_p \cdot h} \lambda \cdot l_3 \tag{37}$$

where:

$\mathrm{f}_{\mathrm{opening}}$

$$f_{opening} = \frac{f_p \cdot l_1 + \sum_{i=0}^{n} V_i}{n_{floor} \cdot h_{under}} \tag{38}$$

$$l_1 = h_{tot} \left(1 - \frac{\sum_{i=k}^{n} V_i}{f_p \cdot h_{tot}}\right) \tag{39}$$

$$\lambda = min\begin{cases} \dfrac{f_p \cdot h}{f_{opening} \cdot h_{under}}\left[\left(\sqrt{1-(1-\dfrac{1}{n_{floor}})^2}^{\,1} - \dfrac{l_1}{2 \cdot h_{tot}} - \dfrac{V_{ekv}}{f_p \cdot h_{tot}}\right) l_1 + \dfrac{n_{knut} \cdot F_{knut}}{f_p}\right. \\ \dfrac{f_p \cdot 2 \cdot h}{f_{opening} \cdot l_3 \cdot (h - h_{under})}\left[\left(\dfrac{l_1}{2 \cdot h_{tot}} + \dfrac{V_{ekv}}{f_p \cdot h_{tot}}\right) l_1 + l_2\right] \\ \dfrac{f_p \cdot b_{full}}{f_{opening} \cdot l_3} \end{cases} \tag{40}$$

V_{ekv} represents the equivalent force acting on the leading stud in the wall corresponding to the fact that the left wall sheeting acts as a rigid body which is simply supported at both ends and are calculated according to Equation (41). The variable x_i denotes the x-coordinate of the stud number i where the stud number 0 is located at origin.

$$V_{ekv} = \sum_{i=0}^{n} V_i \frac{l_1 - x_i}{l_1} \tag{41}$$

where:

x_i is the x-coordinate for the stud number i where the stud number 0 is located at the origo.

The total effective length of the wall is calculated by summation of the results from the different fictive wall parts according to Equation (42).

$$l_{eff.tot} = l_{eff.1} + l_{eff.2} + l_{eff.3} \tag{42}$$

The total horizontal resistance of the entire wall corresponding to all the fictive wall parts are calculated using Equation (43).

$$H = f_p \cdot l_{eff.tot} \tag{43}$$

8.3.4 Connections, sheeting capacity and stiffness of wall segments

The boundary conditions and properties of the connections are essential for the mode of action, the horizontal load capacity and rigidity of the building. The mechanical properties of the stud to sheeting connection are crucial for the wall load bearing capabilities. In order to apply the plastic method the connection needs to exhibit plastic behaviour with sufficient deformation capability; brittle fracture behaviour cannot be applied.

8.3.4.1 Joints between sheeting and timber frame

The design method applied in this thesis is based on the assumption that the joints between the sheeting and the frame structure have a force-displacement characteristic associated with ideal plastics behaviour as shown in Figure 28.

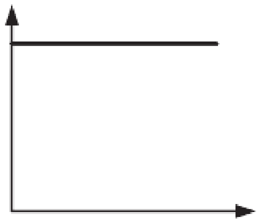


Figure 28 Force-displacement relation of stud sheeting joints according to an ideal plastic model. (Källsner & Girhammar 2008)

For the dimensioning of the panels, a so-called plastic lower bound approach are applied. The basic conditions for this method are that the static equilibrium of the structure is achieved and that the joists are showing a plastic behaviour with sufficient ductility before the capacity decrease significantly.

There are different types of failure modes that can appear in a panel to timber connection which can involve failure of the timber as well as the dowel. To be able to use the plastic design method failure mode, f should be crucial, see Equation (45). This mode are corresponding to the development of two yield points in the nail, see Figure 29. At this location the nail is curled locally as a result of the idealized working curve, but remains in otherwise straight.

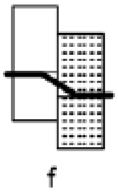


Figure 29 Failure mode, f for timber and panel connections (European Standard 2004)

The design load-carrying capacity per fastener is calculated according to Equation (44).

$$F_{v,Rd} = F_{v,Rk} \frac{\sqrt{k_{mod1} \cdot k_{mod2}}}{min(\gamma_{m1}, \gamma_{m2})} \tag{44}$$

where:

$F_{v,Rk}$	is the characteristic load-carrying capacity for each fastener
k_{mod1}	is the modification factor for the sheeting
k_{mod2}	is the modification factor for the timber
γ_{m1}	is the partial factor for material properties and resistances for the sheeting
γ_{m2}	is the partial factor for material properties and resistances for the timber

The characteristic load-carrying capacity for each fastener is calculated according to Equation (45).

$$F_{v,Rk} = min \begin{cases} f_{h.1,k} t_1 d & (a) \\ f_{h.2,k} t_2 d & (b) \\ \frac{f_{h.1,k} t_1 d}{1+\beta} \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)\right] + \beta^3 \left(\frac{t_2}{t_1}\right)} - \beta \left(1 + \frac{t_2}{t_1}\right) \right] + \frac{F_{ax,Rk}}{4} & (c) \\ 1.05 \frac{f_{h.1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta) M_{y,Rk}}{f_{h.1,k} d * {t_1}^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & (d) \\ 1.05 \frac{f_{h.1,k} t_2 d}{1+2\beta} \left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta) M_{y,Rk}}{f_{h.1,k} d * {t_2}^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & (e) \\ 1.15 \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2 M_{y,Rk} f_{h.1,k} d} + \frac{F_{ax,Rk}}{4} & (f) \end{cases} \tag{45}$$

where:

$f_{h.1,k}$	is the characteristic embedment strength in the sheeting
t_1	is the thickness of the sheeting
$f_{h.2,k}$	is the characteristic embedment strength in the timber
t_2	is the penetration depth in the timber member
d	is the fastener diameter
β	is the ratio between the embedment strength of the members
$M_{y,Rk}$	is the characteristic fastener yield moment
$F_{ax,Rk}$	is the characteristic axial withdrawal capacity of the fastener

The lateral load-carrying capacity for the nails along the edges of the sheets is increased by a factor of 1.2 according to Eurocode 5 part 9.2.4.2.

8.3.4.2 Stiffness of wall segments

The stiffness property, k, of the wall segment is calculated by the assumption of a concentrated load according to Equation (46).

$$k = \frac{F}{u} \tag{46}$$

where:

F	is the horizontal load acting on the wall
u	is the displacement of the wall

The horizontal displacement of the header plate in a one storey-high wall can be calculated according to Equation (47).

$$u = 4.5 \frac{s}{b} \cdot \frac{H_d}{K_{ser,fin}} + \frac{H_d \cdot h}{G_k \cdot b \cdot t} \tag{47}$$

where:

s	is the spacing between the nails along the periphery of the board
b	is the width of the wall segment
H_d	is the design horizontal load per wall segment
$K_{ser,fin}$	is the displacement module
G_k	is the characteristic shear modules of the sheeting
h	is the height of the wall segment
t	is the thickness of the sheeting

The design load for a fully anchored wall segment is calculated by dividing the horizontal load acting on the wall by the effective length, see Equation (48).

$$H_d = \frac{H}{l_{eff}} \tag{48}$$

where:

H is the horizontal load acting on the wall

l_{eff} is the effective length of the wall, see Chapter 8.3.3.2.

The final mean value of the slip modules for Serviceability limit state are used which consider the load duration and moisture influence on deformations.

$$K_{ser,fin} = \frac{K_{ser}}{(1 + k_{def})} \tag{49}$$

where:

K_{ser} is the slip modulus

k_{def} is a factor for the evaluation of creep deformation taking into account the relative service class

The slip modules are calculated according to Equation (50).

$$K_{ser} = [\sqrt{\rho_{m1} \cdot \rho_{m2}}]^{1.5} \frac{d^{0.8}}{30} \tag{50}$$

where:

ρ_{m1} is the plastic shear flow

ρ_{m2} is the effective wall length

d is the plastic shear flow

8.3.5 Reaction and anchor forces

When analysing a shear wall, a constructive aspect that needs to be taken into consideration is the major anchoring forces that occur in the leading wall stud and the stabilizing pressure forces in the trailing stud in the walls, see Figure 30. This applies particularly to the large compressive forces in the sole plate and buckling of the studs and lifting forces. Openings in form of windows and doors also give rise to vertical pressures and anchor forces that must be considered and the transmission of uplift forces between units of shear walls built on top of each other is a critical.

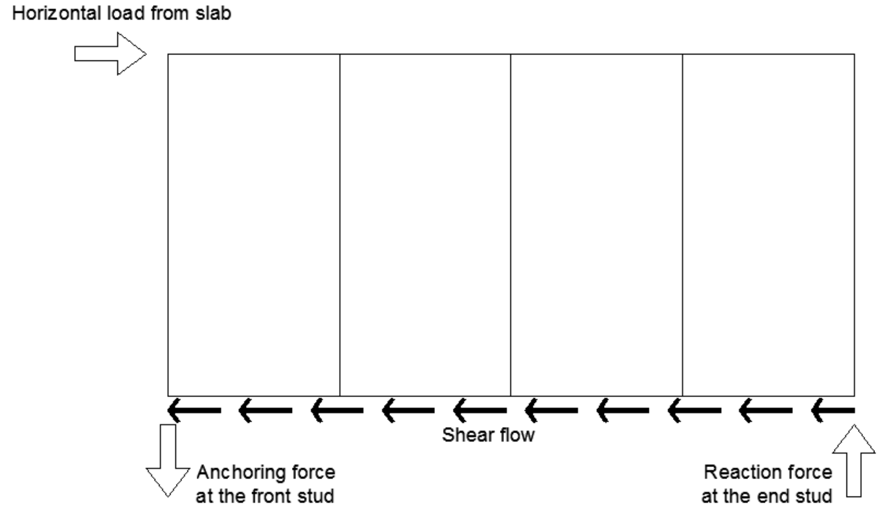


Figure 30 Forces on a single wall when the front stud is completely anchored against lifting.

The wall should be checked against wind in both directions, causing the leading and trailing stud to shifts notations depending on wind direction. In the calculations the most unfavourable case are considered when the leading stud are located in the exterior wall. In this thesis, it is assumed that the interior walls have a fully anchored leading stud while the exterior walls have a distributed anchoring by the sole plate.

8.3.5.1 Reaction forces

The reaction forces arising in the end stud of a wall can generally be determined by calculations of the vertical equilibrium according to Equation (51).

$$R = f_p \cdot h \tag{51}$$

where:

f_p is the plastic shear flow in the wall
h is the height of the wall

The reaction force for a stud at the left side of an opening is calculated by summation of the force contribution from the fictive parts and the sheeting under the opening. The reaction force depends on if the plastic shear capacity are reached before the opening or not, see Equation (52).

$$R_N = \begin{cases} f_p\left(l_1 - \lambda \cdot n_{floor} \cdot h_{under}\right) + \sum_{i=0}^{n} V_i & \text{if } \mathrm{l}_2 = 0 \\ f_p\left(h - \lambda \cdot n_{floor} \cdot h_{under}\right) + V_N & \text{if } \mathrm{l}_2 > 0 \end{cases} \tag{52}$$

where:

f_p is the plastic shear flow in the wall
h is the height of one floor
l_1 is the length of the wall part where the full plastic shear force capacity is achieved
λ is a dimensionless multiplier for the reduction of shear flow under the window opening

n_{floor} is the number of floors
h_{under} is the height of the wall below the opening
n is the number of studs
V_i is the equivalent force acting on the first rule in the wall
V_N is the shear force

The reaction force for the stud at the right side of an opening is calculated according to Equation (53).

$$R_{\ddot{o}pp} = f_p \cdot \lambda \cdot n_{floor} \cdot h_{under} \tag{53}$$

where:

f_p is the plastic shear flow in the wall
λ is a dimensionless multiplier for the reduction of shear flow under the window opening
n_{floor} is the number of floors
h_{under} is the height of the wall below the opening

8.3.5.2 Anchor forces

The internal walls are assumed to have a concentrated anchor at the leading stud while the external walls have a distributed anchor by the sole plate. The location of the stud with concentrated anchor is determined in relation to apertures to obtain maximum utilization. For wall 3 the anchored stud are placed after the door opening. For wall placements see Figure 31.

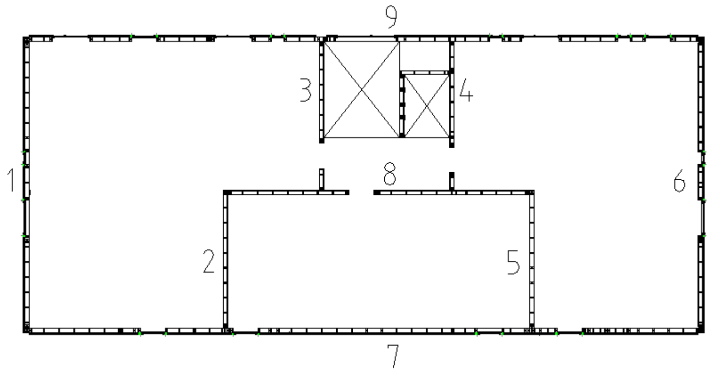


Figure 31 Plan view and numbering of the stabilising walls.

8.3.5.2.1 Concentrated anchor at leading stud

When a wall have a concentrated anchor at the leading stud the lifting forces are calculated in the same manner as reaction forces but with favourable load condition considered, see Chapter 8.3.5.1.

8.3.5.2.2 Distributed anchor by the sole plate

Anchoring of the sole plate occurs in practice, quite often, in order to avoid complex and expensive anchoring devices which fully anchored front studs implies.

If the vertical load acting on a wall is not sufficient to resist the lifting force occurring in the sole plate it needs to be anchored. In practice, the sole plate should always be anchored along its entire length with respect to shear. The sole plate should be anchored to resist a vertical line load, f_p. The anchorage must also cope with a horizontal line load, f_p but not in combination with the vertical line load, see Figure 32. (Källsner & Girhammar 2008)

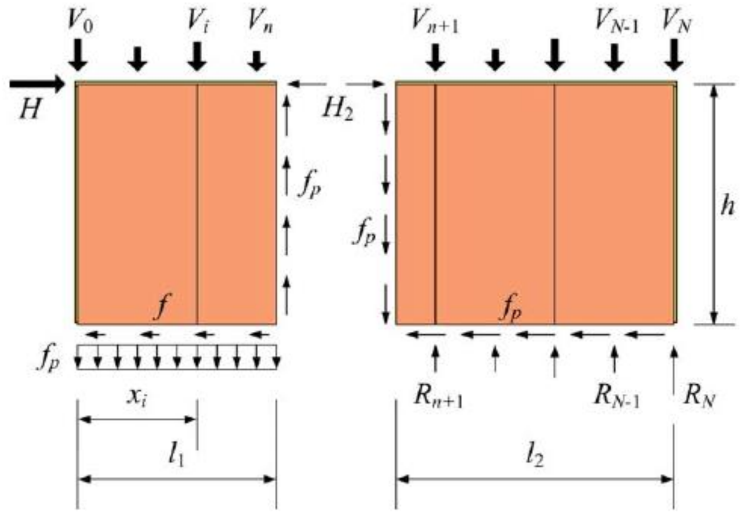


Figure 32 A schematic view of the vertical and horizontal line loads acting on a wall that the anchoring needs to resist. (Källsner & Girhammar 2008)

8.3.6 Sole plate

When timber is loaded perpendicular to the grain direction, the wood fibres will withstand increased loading as they are squeezed together.

The sole and header plates are subjected to compression perpendicular to the grain. They are controlled in ultimate limit state according to Eurocode 5 and the condition to be met can be seen in Equation (54). However, the deformations in these plates are calculated in serviceability limit state.

$$\sigma_{c.90.d} \leq f_{c.90.d} \tag{54}$$

where:

$\sigma_{c.90.d}$ is the design bearing stress on the plate
$f_{c.90.d}$ is the design compressive strength perpendicular to the grain. See Equation (55)

The design compressive strength perpendicular to the grain is calculated according to Equation (55).

$$f_{c.90.d} = \frac{k_{mod} \cdot k_{c.90} \cdot f_{c.90.k}}{\gamma_M} \tag{55}$$

where:

k_{mod} is the modification factor for duration of load and moisture content.
$k_{c.90}$ is a factor taking into account the load configuration, possibility of splitting and degree of compressive deformation.
$f_{c.90.k}$ is the characteristic compressive strength perpendicular to the grain
γ_M is the partial factor for material properties.

8.4 Results from calculations in ultimate limit state (ULS)

The results of calculations carried out in accordance with Chapter 8 can be viewed below.

8.4.1 Load distribution

The horizontal loads are distributed on the stabilizing walls and both the external and apartment separating walls are used as stabilizing members. For a plan view and numbering of the stabilizing members see Figure 31.

External walls are built with 11 millimetre thick, 1.2 metre wide and 2.975 metre high OSB boards with a shear capacity of 700 N. The internal walls are built with 15 millimetre thick, 1.2 metre wide and 2.975 metre high plywood with a shear capacity of 700 N. The horizontal loads seen in Table 2 are considered to act as line loads along the slab edges at each building level and the results are presented as load per unit length. Table 3 and Table 4 indicate the load distribution on the stabilising walls that are based on Equation (13) to Equation (22). For detailed calculations see Appendix C.

Table 2 Horizontal design load acting on the long side of the façade.

Floor	H_y [kN]
12	153.536
11	303.159
10	303.159
9	290.073
8	290.073
7	290.073
6	290.073
5	290.073
4	290.073
3	290.073
2	290.073
1	290.073
0	191.812

Table 3 Load distribution on the stabilising walls when the horizontal loads acting on the long side of the façade.

Wall Floor	1 [kN/m]	2 [kN/m]	3 [kN/m]	4 [kN/m]	5 [kN/m]	6 [kN/m]
12	2.209	4.703	5.228	4.305	4.626	2.129
11	4.362	9.285	10.322	8.5	9.134	4.203
10	4.362	9.285	10.322	8.5	9.134	4.203

9	4.174	8.885	9.877	8.133	8.739	4.021
8	4.174	8.885	9.877	8.133	8.739	4.021
7	4.174	8.885	9.877	8.133	8.739	4.021
6	4.174	8.885	9.877	8.133	8.739	4.021
5	4.174	8.885	9.877	8.133	8.739	4.021
4	4.174	8.885	9.877	8.133	8.739	4.021
3	4.174	8.885	9.877	8.133	8.739	4.021
2	4.174	8.885	9.877	8.133	8.739	4.021
1	4.174	8.885	9.877	8.133	8.739	4.021
0	2.76	5.875	6.531	5.378	5.779	2.659

Table 4 Load distribution on the stabilising walls when the horizontal loads acting on the short side of the façade.

Wall	7	8	9
Floor	[kN/m]	[kN/m]	[kN/m]
12	1.497	3.417	1.501
11	1.497	3.417	1.501
10	2.994	6.834	3.001
9	2.994	6.834	3.001
8	2.994	6.834	3.001
7	2.789	6.365	2.795
6	2.789	6.365	2.795
5	2.789	6.365	2.795
4	2.789	6.365	2.795
3	2.789	6.365	2.795
2	2.44	5.568	2.445
1	2.44	5.568	2.445
0	1.576	3.598	1.58

The worst case occur when the load is acting on the long side of the façade and wall number 1 and 3 are identified as the most loaded internal respective external walls and these walls are further investigated.

8.4.2 Tilting

Since a timber building of this height is a light weight structure, but with the same exposure to wind as for example concrete structures. Something needs to anchor the building to resist tilting. For the building to resist the wind pressure the foundation needs to weigh at least 74 kN/m². For detailed calculations see Appendix D.

8.4.3 Sizing of stud

The same stud dimension of 45x245 millimetres is used over the entire building height in both internal and external walls. This stud dimension is the largest standard stud produced by Derome. The studs consist of C24 timber with a centre distance of 600 millimetres. The studs are dimensioned regarding the worst load case arising at the bottom floor of the building and the calculations are based on Equation (25) to Equation (30).

8.4.4 Horizontal capacity

The most loaded internal and external walls have been identified and correspond to wall number 1 and 3, see Figure 33. Wall 1 is divided into three fictive parts and wall 3 into two fictive parts both bounded by the openings.

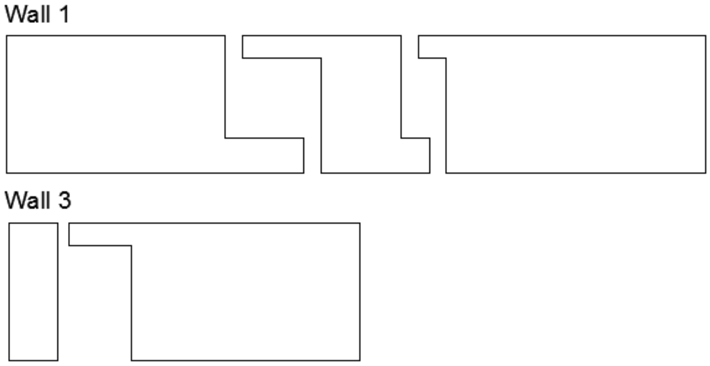


Figure 33 The appearance of wall 1 and wall 3.

The geometry of wall 1 and wall 3 can be seen in Table 5 and Table 7 respectively. Table 6 and Table 8 present the results concerning horizontal capacity for wall 1 respective wall 3. The tables includes the results of plastic capacity and shear flow of the stud-sheeting joints, nail spacing, horizontal capacity and utilisation ratio for the wall over the entire height of the building. The results are based on the calculation procedure described in Chapter 8.3.3.2. For detailed calculations see Appendix E and F.

Table 5 Geometry of wall 1, See Figure 33.

Width of boards	1.2 m
Distance below windows	0.643 m
Distance over windows	0.888 m
Width of window one	1.4 m
Width of window two	0.5 m
Centre spacing between studs	0.6 m
Total length of wall 1	11.803 m
Shear resistance of a junction unit	700 N
Length of wall part 1	3.878 m

Length of wall part 2	1.423 m
Length of wall part 3	4.602 m

Table 6 Input data and results for wall 1. Where F_p is the plastic capacity of stud-sheeting joints, s is the centrum distance between joints, f_p is the plastic shear flow in the stud-sheeting joints, H is the horizontal resistance capacity, UR is the utilisation ratio.

	Input				Result	
Floor	F_p [kN]	s [m]	Δf_p [kN/m]	f_p [kN/m]	H [kN]	UR
11	1.4	0.15	9.333	9.333	26.076	0.772
10	1.4	0.05	10.667	20	51.488	0.811
9	1.8	0.05	16	36	51.488	0.898
8	2.5	0.05	14	50	49.265	0.93
7	3.2	0.05	14	64	49.265	0.891
6	3.9	0.05	14	78	49.265	0.852
5	4.6	0.05	14	92	49.265	0.812
4	5.3	0.05	14	106	49.265	0.772
3	5.9	0.05	12	118	49.265	0.806
2	6.4	0.05	10	128	49.265	0.816
1	6.9	0.05	10	138	49.265	0.726
0	7.2	0.05	6	144	49.265	0.883

Table 7 Geometry of wall 3. See Figure 33.

Width of boards	1.2 m
Interior door width	1.010 m
Distance above door	0.992 m
Centre spacing between studs	0.6 m
Total length of wall 3	5.919 m
Shear resistance of a junction unit	700 N
Length of wall part 1	0.855 m
Length of wall part 2	4.054 m

Table 8 Input data and results for wall 3. Where F_p is the plastic capacity of stud-sheeting joints, s is the centrum distance between joints, f_p is the plastic shear flow in the stud-sheeting joints, H is the horizontal resistance capacity, UR is the utilisation ratio.

	Input				Result	
Floor	F_p [kN]	s [m]	Δf_p [kN/m]	f_p [kN/m]	H [kN]	UR
11	1.4	0.18	7.778	7.778	25.663	0.8

10	1.4	0.06	15.556	23.333	50.671	0.797
9	1.8	0.05	12.667	36	50.671	0.98
8	2.4	0.05	12	48	48.484	0.989
7	3	0.05	12	60	48.484	0.988
6	3.6	0.05	12	72	48.484	0.987
5	4.2	0.05	12	84	48.484	0.986
4	4.8	0.05	12	96	48.484	0.984
3	5.4	0.05	12	108	48.484	0.98
2	6	0.05	12	120	48.484	0.975
1	6.6	0.05	12	132	48.484	0.965
0	7.2	0.05	12	144	48.484	0.935

With the given plastic capacity of the stud-sheeting joints and the centrum distance between joints the walls utilisation ratio is within an acceptable limit.

8.4.5 Connections, sheeting capacity and stiffness of wall segments

The required plastic capacity of the stud-sheeting joints is almost identical in the internal and external walls, resulting in equal sheeting and nail dimensions. The needed resisting forces vary substantially over the height and the building are therefore divided in three components with varying sheeting properties and nail dimensions. The characteristic tensile strength of the nails is 800 N/mm². OSB boards are used in the walls on the three top levels and Kerto Q on the remaining floors.

Table 9 shows the connection properties and capacity as well as the utilisation ratio and stiffness of the wall segments. The results are based on Equation (44) to Equation (50). For detailed calculations see Appendix G.

Table 9 *Results for wall 1 and 3 regarding sheeting thickness, nail dimensions and stiffness. Where t_1 is the thickness of the sheeting, d is the diameter of the nails, l is the length of the nails, $F_{v.Rd}$ is the design load-carrying capacity per shear plane per fastener, UR is the utilisation ratio, k is the stiffness of wall segment.*

		Input				Result	
Floor	t_1 [mm]	d [mm]	l [mm]	$F_{v.Rd}$ [kN]	UR – Wall 1	UR – Wall 3	k 10^3 [kN/m]
11	25	3	120	0.905	0.774	0.774	3.913
10	25	3	120	0.905	0.774	0.774	3.913
9	25	3	120	0.905	0.995	0.995	3.913
8	55	5.7	125	2.361	0.53	0.508	4.932
7	55	5.7	125	2.361	0.678	0.635	4.932
6	55	5.7	125	2.361	0.826	0.762	4.932
5	55	5.7	125	2.361	0.974	0.89	4.932

4	65	7.5	150	3.718	0.713	0.646	6.059
3	65	7.5	150	3.718	0.793	0.726	6.059
2	65	7.5	150	3.718	0.861	0.807	6.059
1	65	7.5	150	3.718	0.928	0.888	6.059
0	65	7.5	150	3.718	0.968	0.968	6.059

With the given nail dimensions the walls utilisation ratio is within an acceptable limit.

8.4.6 Reaction forces

The walls are dimensioned for equal reaction forces in both ends calculated for the worst case corresponding to reaction forces on the end stud of the wall. The calculations of the reaction forces are performed using unfavourable load condition since the vertical loads are contributing to increased values.

Table 10 and Table 11 indicate the reaction forces generated through the building and the maximum required number of studs for wall 1 and 3 respectively. The results are based on Equation (51) to (53) regarding reaction forces and Equation (25) to (30) for the required numbers of studs. Fore detailed calculations see Appendix K and L. For location of the maximum reaction forces, see Figure 34 and Figure 35.

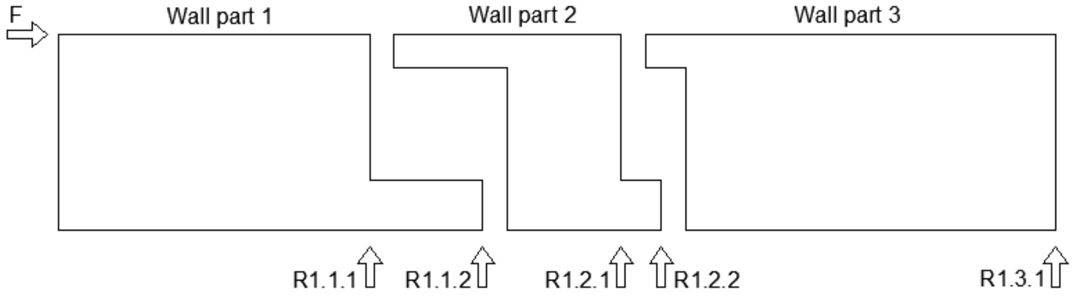


Figure 34 The locations of the reaction and anchoring forces in wall 1.

Table 10 Results for wall 1 regarding reaction forces and required number of studs. For the position of the forces see Figure 34.

Floor	R.1.1.1 [kN]	R.1.1.2 [kN]	R.1.2.1 [kN]	R.1.2.2 [kN]	R.1.3.1 [kN]	Required number of studs
11	1.575	42.62	42.62	-54.735	51.952	2
10	2.089	91.471	91.471	-95.049	110.04	2
9	7.488	156.12	156.12	-74.021	192.672	2
8	12.087	7.03	213.813	-96.362	266.1	5
7	17.202	264.208	270.99	-133.784	339.528	5
6	25.482	318.22	325.002	-168.876	412.956	5
5	41.478	364.516	371.298	-194.966	486.384	5

4	65.19	403.096	409.878	-212.054	559.812	8
3	93.271	429.55	436.333	-217.842	624.036	8
2	123.517	446.085	452.867	-214.902	679.056	8
1	166.641	449.74	456.523	-205.532	727.767	8
0	181.633	451.568	460.381	-192.852	746.617	8

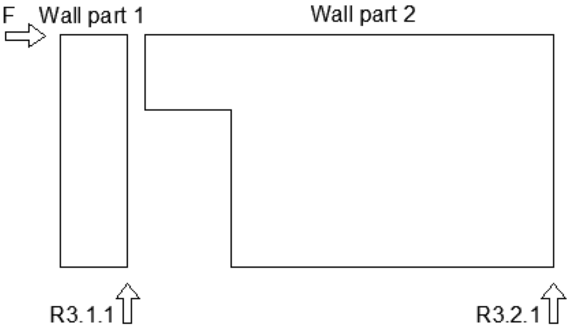


Figure 35 The locations of the reaction and anchoring forces in wall 3

Table 11 Results for wall 3 regarding reaction forces and required number of studs. For the position of the forces see Figure 35.

Floor	R.3.1.1 [kN]	R.3.2.1 [kN]	Required number of studs
11	9.71	289.907	13
10	39.462	799.982	13
9	66.744	1182	13
8	92.376	1505	22
7	117.468	1791	22
6	142.236	2042	22
5	166.788	2257	22
4	191.186	2437	28
3	215.468	2581	28
2	239.66	2689	28
1	263.78	2761	28
0	300.5	2799	28

The required number of studs is placed together at the wall endings to take of the reaction forces.

8.4.7 Anchoring forces

The walls are dimensioned for equal anchoring forces in both ends calculated for the worst case corresponding to anchoring forces in the leading stud of the wall. The calculations of the anchoring forces are performed using favourable load condition since the vertical loads are beneficial.

Table 12 indicates the anchor forces that appear in the building floor by floor generated for wall 1 and 3. The location of the worst anchoring forces is illustrated in Figure 36.

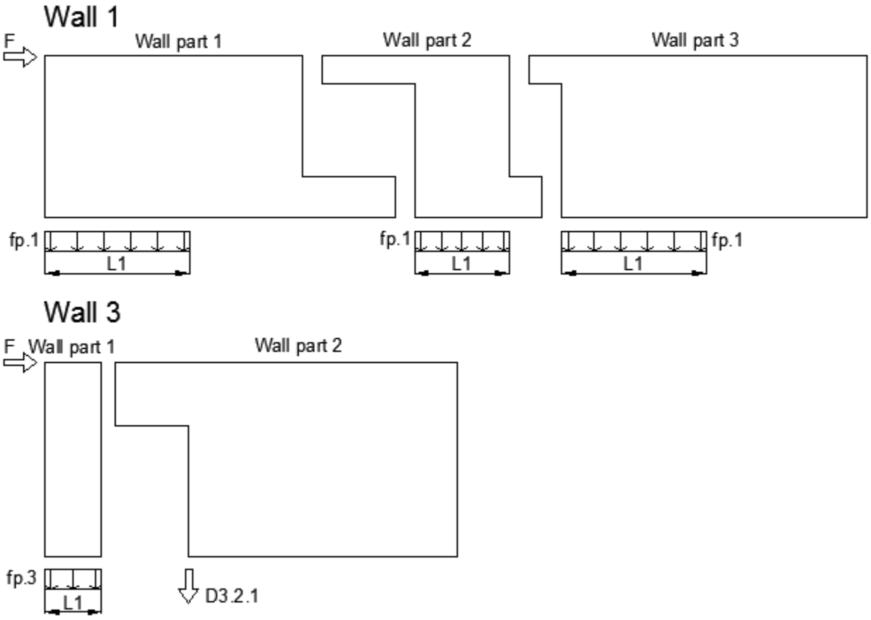


Figure 36 The locations and appearance of the anchoring forces in wall 1 and wall 3

Table 12 Results for wall 1 and 3 regarding anchoring forces. For the position of the force see Figure 36.

	Wall 1	Wall 3	
Floor	fp -1 [kN/m]	fp -3 [kN/m]	D3.2.1 10^3[kN]
11	9.333	7.778	278
10	20	23.333	787
9	36	36	1164
8	50	48	1486
7	64	60	1771
6	78	72	2021
5	92	84	2235
4	106	96	2414
3	118	108	2557

2	128	120	2664
1	138	132	2735
0	144	144	2771

These forces need to be taken care of and there are a number of different methods to do so, however this has not been investigated in this thesis.

8.4.8 Sizing of sole, heater plate and sill

Due to the large reaction forces generated, it is not possible to use an either timber or glulam in the sole plates, heater plates or the sill because it will be crushed. Medium density fibre boards can withstand the forces and are used as sole and header plates. The sole and header plates have the same appearance over the building and comprise of 22 millimetre thick MDF boards.

Regarding the sill MDF boards can resist the reaction forces but is a poor solution due to its moisture sensitivity and manufacturing sizing. A better solution is to use a steel sill.

9 Design in Serviceability limit state

Calculations in serviceability limit states correspond to the conditions that a construction is exposed to during its service life. The design in the serviceability limit state is made to ensure that the building works as expected and meets the performance requirements.

It has been observed that large deformations can occur in slender timber buildings before the ultimate load is reached. This indicates that the serviceability limit states are important and can be essential in the design of tall timber structures. (Vessby 2011)

9.1 Calculation procedure

The acceleration of structures due to dynamic wind load needs to be checked against given standard limitation values for comfort, see Chapter 4.2. A spectral analysis has been used to calculate the time history of wind loading and then using this load to calculate the structures natural frequency. The natural frequency calculations have been performed both by hand as well as with a FEM-program.

The aero elastic effects are checked against given limitations to investigate if further analysis is needed.

9.2 Calculations concerning human comfort

All equations in Chapter 9.2 follow Eurocode 1 part 1-4 (2005) unless otherwise referenced.

9.2.1 Return period

The return period is an estimation of how often an event occurs. For a wind with a return period of 50 years the value are the maximum estimated wind load during this entire period. The return period of one year result in a lower estimated value. The limitations in SS-ISO 10137 seen in Figure 4, indicates values for loads with a recurrence interval of one year, while the calculations in Eurocode 1 part 1-4 are based on a 50 years return period. The explanation of this dissimilarity is that there are softer requirements regarding the regulations concerning the motions sickness in compassion to other limitations that entails larger risks and possible permanent damage. In a 50-year storm there may be necessary to evacuate the upper floors with regard to unpleasant vibrations which according to the standard is considered to be reasonable.

The wind velocity for different return periods can be calculated using Equation (56). The equation is based on the characteristic value of a reference wind velocity that is exceeded for one year with the probability of two percent, which corresponds to an average return period of 50 years.

$$v_{Ta} = 0.75 \cdot v_{50} \sqrt{1 - 0.2\ln\left(-\ln\left(1 - \frac{1}{T_a}\right)\right)} \tag{56}$$

where:

T_a is the number of years

v_{50} is the characteristic value of the reference wind velocity that is exceeded for one year with the probability of 2%, which corresponds to an average return period of 50 years.

9.2.2 Spectral analyses

By a spectral analysis it is possible to obtain a magnitude of wind loading as a function of time which is required to perform a dynamic response analyses. A spectrum of the fluctuating wind velocity $S_L(f_L)$ can be calculated by data of wind velocity used as initial values, see Figure 37.

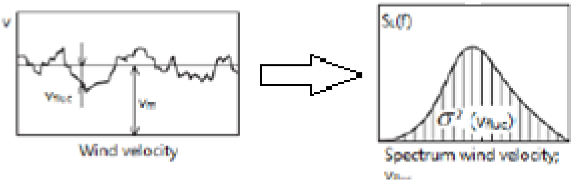


Figure 37 Design process of spectral analyses for fluctuating wind velocity. (Coull & Stafford 2008)

A spectrum describes a variable conversion from a time domain into a spectrum, a frequency domain. The spectrum of fluctuating wind velocity is described in EC1 and calculated according to Equation (57).

$$S_L(f_L) = \frac{6.8 \cdot f_L(z_s, n)}{(1 + 10.2 \cdot f_L(z_s, n))^{5/3}} \tag{57}$$

where:

$f_L(z_s, n)$ is the non-dimensional frequency on reference height z_s

The frequency used is non-dimensional and a function of the fundamental frequency and the reference height of the building, see Equation (58). The reference height is according to EC1 $0.6h$ where h is the total height of the building. The magnitude of the fluctuating wind is thereby determined by the fundamental vibration mode of the structure and the location on the building's façade, where fluctuations decrease upwards on the building according to EC1.

$$f_L(z_s, n) = \frac{n_f \cdot L(z_s)}{v_m(z_s)} \tag{58}$$

where:

n_f is the fundamental frequency
$L(z_s)$ is the size of gusts at reference height z_s
$v_m(z_s)$ is the mean wind velocity at reference height z_s

9.2.3 Time history of wind loading

Horizontal accelerations are representative for the behaviour of the structure under fluctuating wind loads as defined by the spectrum, $S_L(f_L)$. In order to clarify the accuracy of these values it is desired to excite the structure directly by a time-varying wind load, so that the dynamic response over a certain time period can be analysed.

By the spectral analysis it is possible to obtain an approximate time signal of the wind velocity.

Wind forces are a stationary stochastic loading process with a fluctuating behaviour without any repeating patterns. A stochastic processes can be calculated as a summation of infinite sinusoidal waves, with each a unique amplitude, phase angle and frequency. The time signal corresponding to the fluctuating part of the wind is calculated as a summation of the sinusoidal waves according to Equation (59).

$$v(t) = \sum_{i=1}^{N} a_i \cdot \sin[(f_i t) + \Psi_i] \tag{59}$$

where:

a_i	is the magnitude of the amplitude
f_i	is the frequency
t	is the time
Ψ_i	is the phase angle

The magnitude of the amplitudes is originated in the spectrum of fluctuating wind velocity and can be expressed as seen in Equation (60).

$$a_i = \sqrt{2 \cdot S_i \cdot \Delta f} \tag{60}$$

where:

S_i	is the spectral density function for a certain frequency
Δf	is the band width

Assuming that the wind fluctuation is equal in magnitude over the current area the wind velocity is converted to a fluctuated wind forces by Equation (61).

$$F(t) = \rho_{air} \cdot c_d \cdot A \cdot v_m \cdot v(t) \tag{61}$$

where:

ρ_{air}	is the density of air
c_d	is the dynamic building factor
A	is the area
v_m	is the mean wind velocity
$v(t)$	is the fluctuating wind velocity

The mean part of the wind load is calculated according to Equation (62).

$$F_m = \frac{\rho_{air}}{2} c_d \cdot A \cdot {v_m}^2 \tag{62}$$

where:

ρ_{air}	is the density of air
c_d	is the dynamic building factor
A	is the area
v_m	is the mean wind velocity

The dynamic building factor is calculated according to Equation (63).

$$c_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s)\sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)\sqrt{B^2}} \tag{63}$$

where:

k_p	is the peak factor
$I_v(z_s)$	is the turbulence intensity at reference height z_s
B^2	is the background response part
R^2	is the resonant respond part

9.2.4 Horizontal acceleration

An expression for calculating the maximum acceleration or the peak value used when determine the fluctuations in the first mode of a console structure with constant mass along the main axis of the structural are described in the National annex, see Equation (64).

$$a_{max} = k_p \cdot a_{rms} \tag{64}$$

where:

k_p	is the peak factor
a_{rms}	is the mean acceleration

The mean acceleration (root-mean-square, r.m.s) are calculated according to Equation (65).

$$a_{rms} = \frac{3 \cdot I_v \cdot R \cdot \; q_m \; \cdot b \cdot c_f \; \cdot \Phi_1}{m} \tag{65}$$

where:

I_v	is the turbulence intensity
R^2	is the resonant respond part
q_m	is the mean wind velocity
b	is the width of the building
c_f	is the force coefficient
Φ_1	is the fundamental along wind modal shape
m	is the mass per unit length of the building

In general, the acceleration is strongly associated with the natural frequency of the building, which in turn mainly depends on the mass and stiffness distribution.

9.2.5 Natural frequency

An important parameter regarding dynamic response analysis and calculations is the system's natural frequency. The natural frequency corresponds to the number of cycles per second of a structure in free oscillations. Structures have an unlimited number of frequencies where the lowest are called the fundamental.

Literature describes several diverse methods to estimate the fundamental frequency, both simple approximations and more complex approaches in order to improve the accuracy.

Eurocode 1 indicates a number of different equations regarding the fundamental frequency. A common approximation in an early stage is only based on the height of the building but is only valid for buildings from 50 metres and up, see Equation (66).

$$n_1 = \frac{46}{h_{tot}} \tag{66}$$

where:

h_{tot} is the total height of the building

This formula is based on fundamental frequencies of buildings designed in concrete or steel which material parameters differ considerably in comparison to timber.

Another method to calculate the natural frequency for a system with one degree of freedom are presented in "Kompendium i Byggnadsaerodynamik" by Kamal Handa (1982) and include substantially additional parameters that entail a more reliably result, see Equation (67).

$$f_0 = \frac{1}{2\pi}\sqrt{\frac{3 \cdot EI}{m_s \cdot {h_{tot}}^4}} \tag{67}$$

where:

E	is the modulus of elasticity
I	is the moment of inertia
m_s	is the mass per unit length
h_{tot}	is the total height if the building

9.2.6 Aero elastic effects

When a building is exposed to wind, it can in addition to transverse oscillations also cause a fluctuating load perpendicular to the wind direction, see Figure 38. This is due to vortex shedding or galloping which occur when the frequency of the wind approaches one of the natural frequencies of the building. The criteria in Eurocode are therefore based on keeping the critical wind velocity higher than the mean wind velocity.

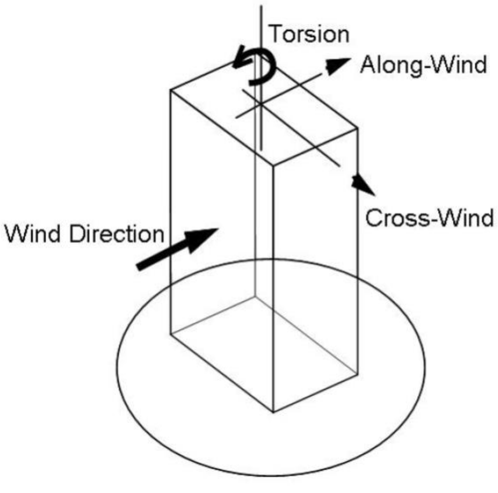


Figure 38 Wind response directions. (Cheung et al. 2007)

Structural vibrations may occur if the frequency of vortex–shedding is the same as a natural frequency of the structure. This condition occurs when the wind velocity is equal to the critical wind velocity.

9.2.6.1 Vortex shedding

Vortex shedding occurs when vortices arise on opposite sides of a building and can affect any structure regardless of configuration but is special critical for structures with circular cross sections.

The size of these phenomena is dependent of the building shape and its susceptibility to fluctuating wind loading, determined by its fundamental frequency. The effects from vortex shedding needs to be taken into consideration if the relationship between the maximum and minimum dimension in a plane perpendicular to the wind direction is greater than six and if the critical wind velocity is larger than 1.25 times the characteristic mean wind velocity described in EC1, annex E, see Equation (68).

$$v_{crit.i} > 1.25 \cdot v_m \tag{68}$$

where:

$v_{crit.i}$	is the critical wind velocity
v_m	is the characteristic ten minutes mean wind velocity at the cross section where vortex shedding occurs

The critical wind velocity for vortex shedding are calculated according to Equation (69).

$$v_{crit.i} = \frac{b \cdot n_{i,y}}{St} \tag{69}$$

where:

b	is the reference width of the cross-section
$n_{i,y}$	is the natural frequency
St	is the Strouhal number that depends on the cross section shape of the structure

9.2.6.2 Galloping

Galloping is a self-induced vibration of a flexible structure that occur when the wind attack the building in a particular direction. All building shapes are prone to galloping except structures with circular cross-sections. Oscillation caused by galloping starts at a specific wind velocity, $\mathrm{v_{CG}}$, see Equation (71) and the amplitude rises in generally rapidly with increasing wind velocity.

The criteria that need to be fulfilled in order to neglect the influence from galloping are described in EC1, annex E, see Equation (70).

$$v_{CG} > 1.25 \cdot v_m \tag{70}$$

where:

v_{CG}	is the onset wind velocity of galloping
v_m	is the characteristic 10 minutes mean wind velocity at the cross section where vortex shedding occurs

$$v_{CG} = \frac{2 \cdot Sc}{a_G} \cdot n_{i,y} \cdot b \tag{71}$$

where:

Sc	is the Scruton number as defined
b	is the reference width of the cross-section
a_G	is the factor of galloping instability
$n_{i,y}$	is the cross-wind fundamental frequency of the structure

9.2.7 Displacement

There are limited recommendations for allowable top displacements of multi-storey timber buildings in Swedish design codes. In Eurocode 5 the top displacement, u, is restricted to u≤h/300, where h are the building height. An example of a corresponding value is presented in German DIN- code where the recommendations are u≤h/500, allowing less displacement. (Vessby 2011)

An estimation of the top displacement of a structure considered to be fixed to the foundation and regarded as a cantilever beam subjected to a point load in the buildings upper edge can be performed according to “Kompendium i Byggnadsaerodynamik” by Kamal Handa (1982), see Equation (72)

$$u = \frac{P}{w^2 \cdot m} \tag{72}$$

where:

P	is the load
w	is the angular frequency
m	is the mass per unit length

9.3 Results from calculations concerning human comfort

The results of calculations carried out in accordance with calculations concerning human comfort can be viewed below.

9.3.1 Time history of wind loading

The results of the fluctuating wind velocity are shown in Figure 39.

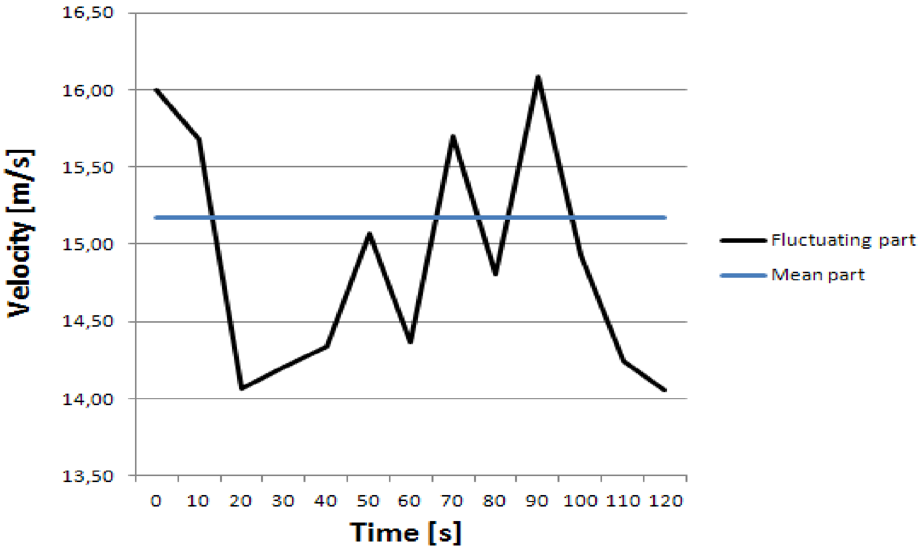


Figure 39 Time history of the fluctuating wind velocity.

The results of the fluctuated wind forces are shown in Figure 40.

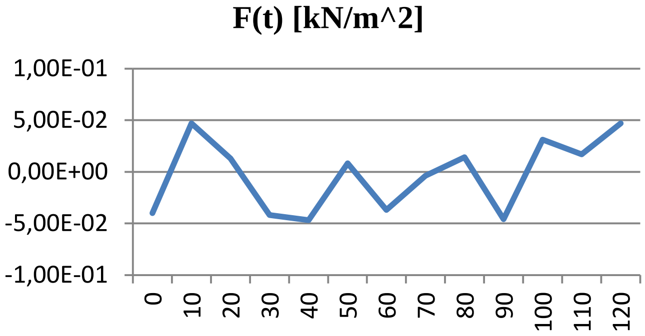


Figure 40 Fluctuated wind forces.

For detailed calculations on time history of wind loading see Appendix M.

9.3.2 Horizontal acceleration

Calculations show that $a_{max} = 0.296 \frac{m}{s^2}$ and that $a_{rms} = 0.083 \frac{m}{s^2}$.

For detailed calculations see Appendix O.

9.3.3 Natural frequency

The natural frequency of the building is calculated according to Equation (67) since the building is less than 50 m high. The calculations show a natural frequency of approximately 3 Hz. For detailed calculations see Appendix N.

9.3.4 Vortex shedding

The effects of vortex shedding does not needs to be considered, see Appendix O.

9.3.5 Galloping

Recommended condition are fulfilled. For detailed calculations see Appendix O.

9.4 FE-module

The numerical analysis model is created in ABACUS/CAE. In the Graphical User Interface different parts are created, assembled and assigned different material properties. All elements used in ABAQUS are divided into different categories depending on the modelling space. The elements used in the model are shell and solid elements modulated in the 3D space. The thick shell elements used are based on the Mindlin shell theory, which includes shear deformations. The solid elements are first-order linear interpolation, which essentially are constant strain elements.

To model all the structural parts of the building such as studs, sheets, nails etcetera would make a very complex system. To simplify the model to a manageable size equivalent materials have been develop to replace several connected parts, such as:

- a wall segments
- the connection between wall and slab
- the steel beam holding up the great floor spans

To simplify the wall segments the displacement, u, of the header plate have been hand calculated for one external wall and one internal wall on three different storeys (first, sixth and tenth). In ABAQUS a shell element is modulated to represent a wall segment and to get the same properties for the wall in ABAQUS as the real wall the material properties are defined as for the sheeting. A unit load is applied along the top shell edge, see Figure 41.

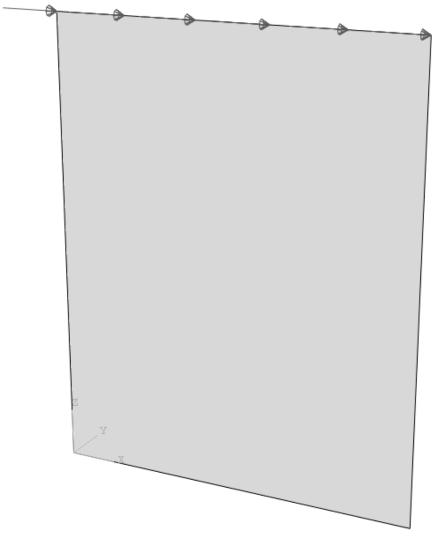


Figure 41 Load placement on an equivalent wall segment.

The boundary conditions for the top edge is free to move in x direction and rotation around the y axis while the bottom edge only is free to rotate around the y axis. A static step is run and the displacement at the top of the shell element shown in Figure 42 is compared to the hand calculations. The thickness of the shell element is adjusted until the same result is reached in Abaqus as in the hand calculations.

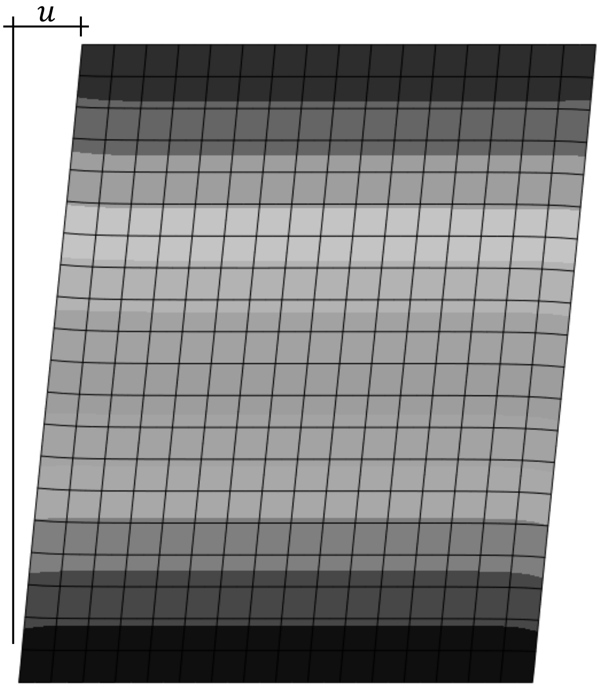


Figure 42 Top displacement of equivalent wall segment.

To simplify the model of the steel girders and timber beams in the middle of the large floor spans to one girder Equations (73) and (74) have been used to find an equivalent element with square cross section, see Figure 43. This is possible due to that the materials do not plasticise under the applied load and is there for linear.

$$2(EI)_1 + (EI)_2 = (EI)_{new} \tag{73}$$

$$b = \frac{12 \cdot (EI)_{new}}{E \cdot h^3} \tag{74}$$

where:

$(EI)_1$	is the modulus of elasticity times the moment of inertia for the timber beams
$(EI)_2$	is the modulus of elasticity times the moment of inertia for the steel girders
E	is the modulus of elasticity
b	is the width of the equivalent element
h	is **the** hight of the equivalent element

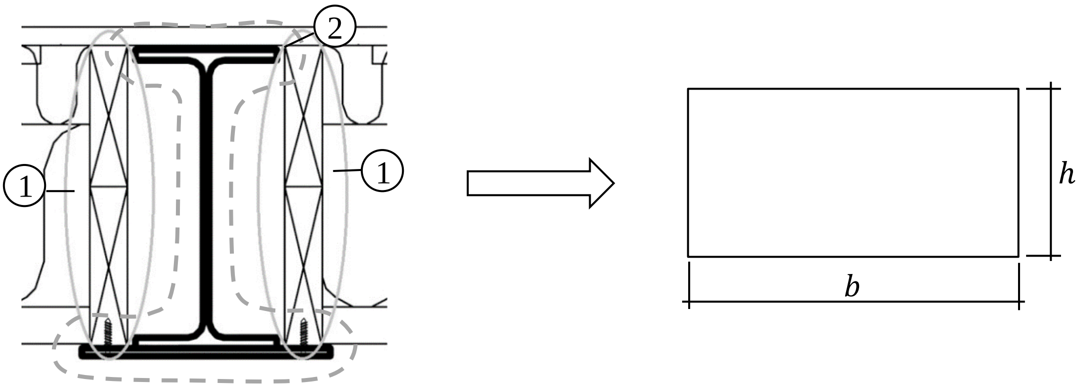


Figure 43 Equivalent steel-timber beams.

In the same manner an equivalent material have been calculated for the girders along the internal and external walls.

The number of studs needed at the end of each wall due to horizontal loads is hand calculated for one internal and one external wall at three different stories (first, sixth and tenth).

9.4.1 Analysis type

The source used to describe the different analysis types is Abaqus 6.10, Analysis User's Manual, vol 2 (2010).

General static analysis

The general static analysis can involve both linear and nonlinear effects and is performed to analyse static behaviour such as deflection due to a static load. In this case a linear analysis is used and the default time period of 1.0 time units, representing 100 percent of the applied load.

Linear eigenvalue analysis

Abaqus provides three different eigenvalue extraction methods. The one used in this thesis is the Lanczos eigensolver due to its general capabilities and is faster when a large number of egienmodes are required.

Linear eigenvalue analysis performs eigenvalue extraction to calculate the natural frequencies and the corresponding mode shapes of a system. The eigenvalue problem for the natural frequencies of an undamped finite element model is calculated according to Equation (75)

$$(-\omega^2 M^{MN} + K^{MN})\phi^N = 0 \tag{75}$$

where:

M^{MN}	is the mass matrix (which is symmetric and positive definite)
K^{MN}	is the stiffness matrix (which includes initial stiffness effects if the base state included the effects of nonlinear geometry)
ϕ^N	is the eigenvector (the mode of vibration)
$M\ and\ N$	are degrees of freedom

Dynamic analysis (Implicit analysis)

The dynamic implicit analysis method is used to calculate the transient dynamic response of a system. When nonlinear dynamic responses are studied, a direct time integration of the system must be used. In linear analysis, modal methods can be used to predict the response of the system, using eigenmode extraction and is generally less expensive than direct integration.

In this thesis the dynamic analysis is used to study the deflection and acceleration due to dynamic loads on the model.

9.4.2 Model

The different floor segments have been modulated using shell elements with the same design as the slabs in the reference object. LVL Kerto S-beams going across the span and timber beams (class C24) around the segment with timber stiffeners (class C24) in between some of the Kerto S beams as shown in Figure 44.

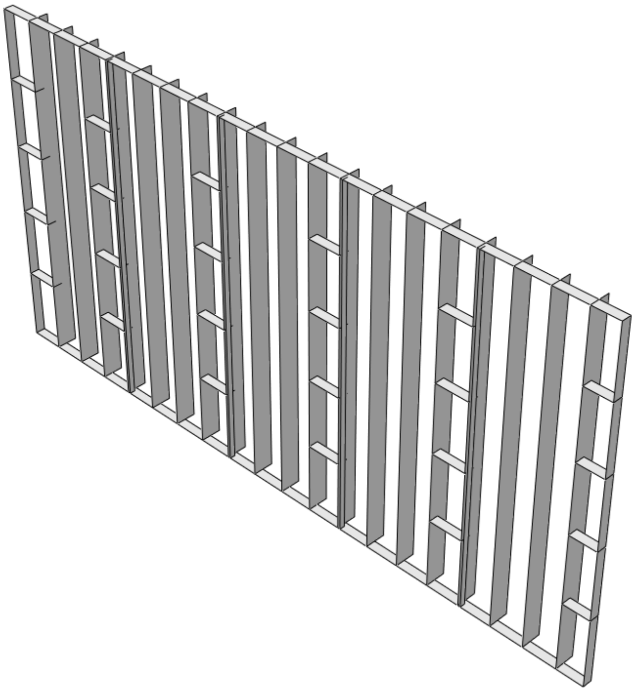


Figure 44 One floor segment without sheeting.

The different floor segments are assembled with the floor sheets also modulated as shell elements, see Figure 45. A hole in the slab is modulated to represent the elevator shaft but no consideration has been taken into account for the stairwell.

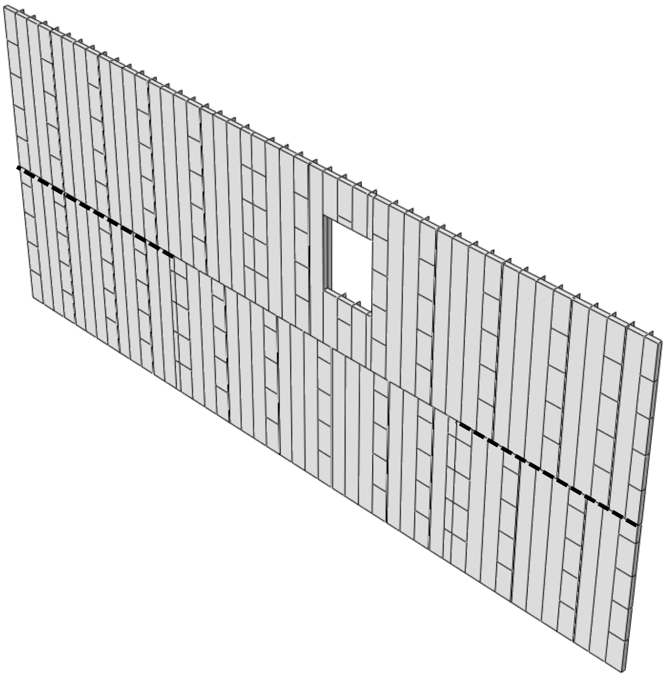


Figure 45 One floor.

The studs are modulated with solid elements and placed at the end of all the walls. Extra studs are placed under the steel beam to take care of the increased load due to the large span.

The external and internal walls, studs and the floor are assembled and merged to one storey, see Figure 46. This is repeated for the three different storey heights.

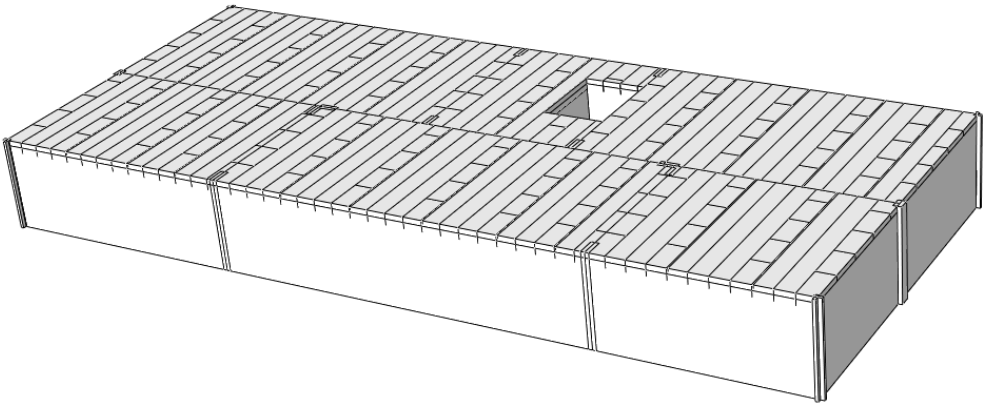


Figure 46 One storey.

The various storeys are placed on top of each other and merged, see Figure 47. For simplification, a floor is used at the top to represent the roof due to the similarities in structure and weight.

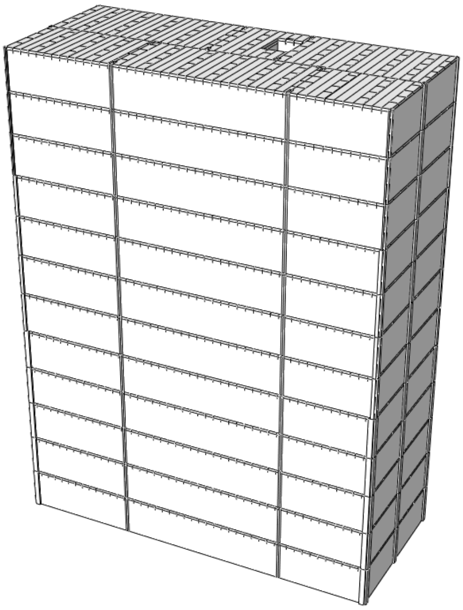


Figure 47 The model of the 12 storey building.

Boundary conditions are used at the bottom of the building to represent the connection to the foundation which is assumed to be fully fixed.

9.4.3 Material Data

The derived thicknesses for the different wall segments in the model can be seen in Table 13.

Table 13 Thicknesses of the equivalent walls.

Storey	Thickness [mm]

External	1 - 5	143
	6 - 9	52
	10 - 12	37
Internal	1 - 5	68
	6 - 9	52
	10 - 12	44

Indata for the different materials in different directions in the model is shown in Table 14.

Table 14 *Material*

Engineering	ρ [kg/m³]	E1 [GPa]	E2 [GPa]	E3 [GPa]	Nu12	Nu13	Nu23	G12 [GPa]	G13 [GPa]	G23 [GPa]
C24 slab x-dir	350	11	0.6	0.37	0.04	0.03	0.35	0.69	0.3	0.03
C24 slab y-dir	350	0.6	11	0.37	0.04	0.35	0.03	0.69	0.03	0.3
C24 stud x-dir	350	0.6	0.37	11.0	0.35	0.04	0.03	0.03	0.69	0.3
C24 stud x-dir	350	0.37	0.6	11.0	0.35	0.03	0.04	0.03	0.3	0.69
Kerto S	510	0.13	13.8	0.43	0.02	0.5	0.02	0.15	0.05	0.6
Kerto Q	510	2.0	2.4	10.5	0.5	0.02	0.02	0.05	0.6	0.15
Wall x-dir	510	2.0	2.4	10.5	0.5	0.02	0.02	0.05	0.6	0.15
Wall y-dir	510	2.4	2.0	10.5	0.5	0.02	0.02	0.05	0.15	0.6

Isotropic	ρ [kg/m³]	E [GPa]	Nu
Steel	780	2100	0.3

ρ	is the density
E	is the modulus of elasticity
$E1$	is the modulus of elasticity in x-direction
$E2$	is the modulus of elasticity in y-direction
$E3$	is the modulus of elasticity in z-direction
Nu	is the Poisson's ratio
$Nu12$	is the Poisson's ratio in the x-y plane
$Nu13$	is the Poisson's ratio in the x-z plane
$Nu23$	is the Poisson's ratio in the y-z plane
$G12$	is the shear modulus in the x-y plane
$G13$	is the shear modulus in the x-z plane
$G23$	is the shear modulus in the y-z plane

9.4.4 Mesh

The model is meshed with an edge seed of 0.2, see Figure 48. The reason for the fine mesh is the use of both shell and solid elements in a dynamic analysis. The shell elements are meshed with quad-dominated element shapes and the solid elements with hex element shapes.

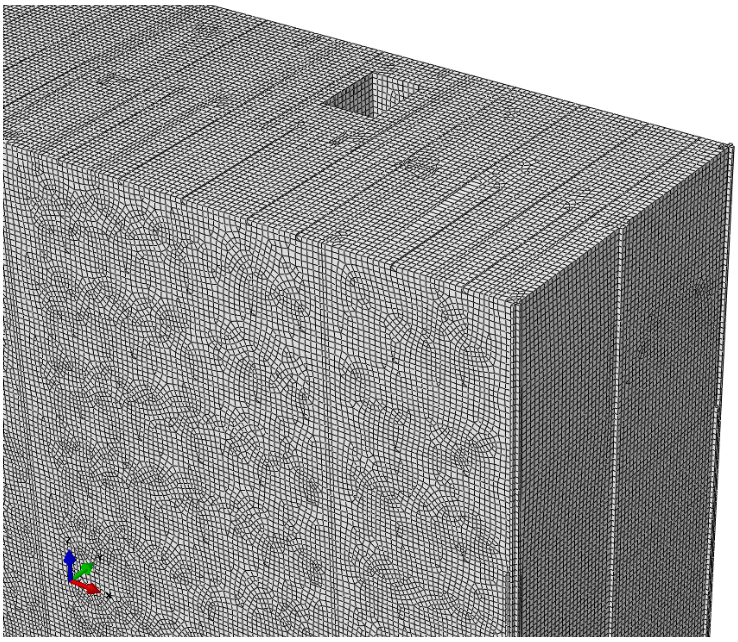


Figure 48 A meshed model.

9.4.5 Application of load

The wind load is calculated by hand and applied as a pressure load along the long side of the façade, see Figure 49. Using different loads depending on where the lode is applied, over or under the reference height at 27.6 metres.

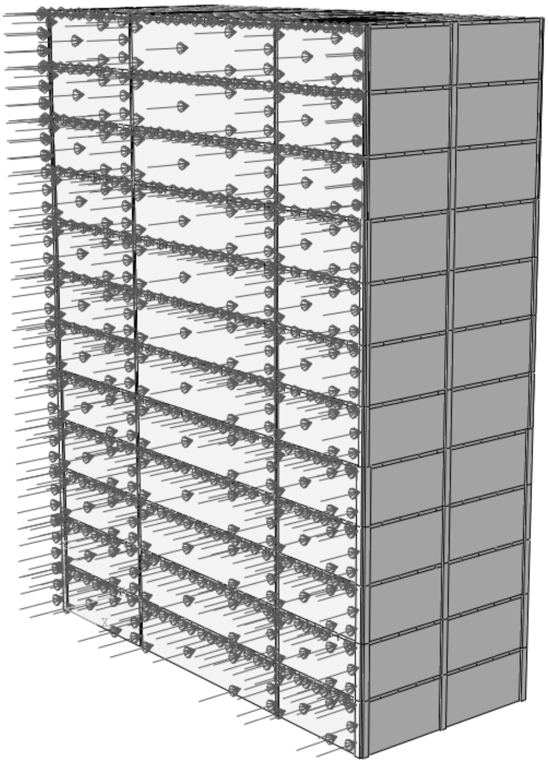


Figure 49 The load placement.

Table 15 Applied dynamic loads.

Year	Ref. height 1 [kN/m²]	Ref. height 2 [kN/m²]
1.1	0.838	0.899
5	0.867	0.931
10	0.875	0.939
50	0.891	0.957

Table 16 Applied static load.

Year	Ref. height 1 [kN/m²]	Ref. height 2 [kN/m²]
50	1.547	1.664

9.5 Results from the FE-module

The linear eigenvalue analysis gives the frequencies for the different mode shapes. The natural frequency for the numerical model is 1.28 Hz and the first mode shape is shown in Figure 50. The frequencies for the first teen mode shapes can be seen in Table 17.

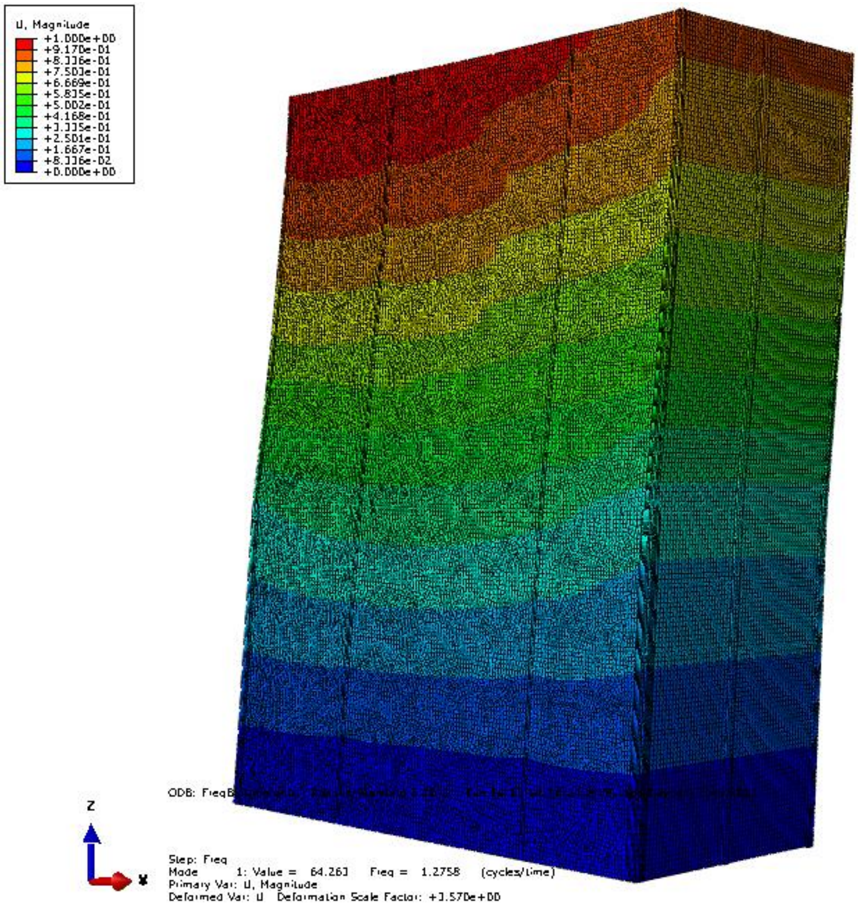


Figure 50 The first mode of the frequency analysis.

Table 17 The first ten modes of the frequency analysis.

Mode	Frequency [Hz]
1	1.2758
2	1.4891
3	1.7090
4	3.3819
5	3.6226
6	3.9873
7	5.1700
8	5.2752
9	5.3754
10	5.3918

A dynamic analysis was preformed but could not be validated. Therefor the result from the dynamic analysis is not presented in this thesis.

9.6 Results of comfort requirements

The natural frequency from the FEM-analysis together with the result from the horizontal acceleration, see Chapter 9.3.2, is plotted in the diagram concerning human comfort requirements explained in Chapter 4.2. The plotted result is shown in Figure 51.

The calculated results are plotted between the limits for residential and office buildings. This shows that the building is acceptable to be used as an office building but not as a residential building.

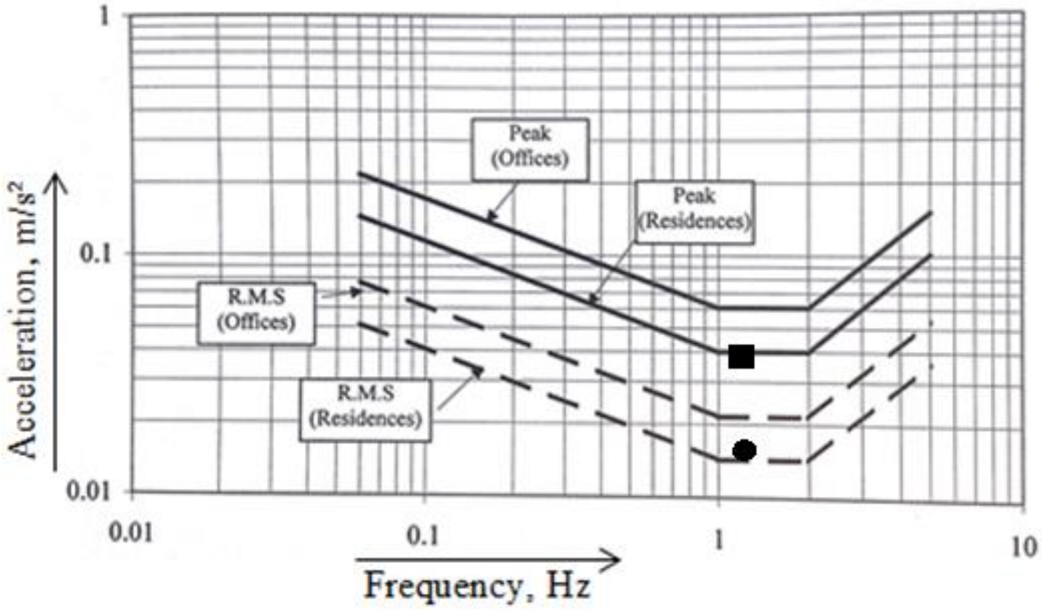


Figure 51 *The result inserted into the evaluation curve for wind-induced vibrations in buildings. The rectangle represents the peak-value and the dot represents the r.m.s.-value.*

10 Discussion

Besides the natural frequency results from the FEM-analysis, the hope was to also achieve a dynamic analysis from the FEM-model, but because of some difficulties a reasonable result was not achieved within the frame of this thesis.

When comparing the natural frequency from the hand calculations with the frequency from the FEM-analysis the result is as expected. The approximate hand calculation results in a bigger value than the result from the FEM-analysis. This is due to the simplifications in the hand calculations and the fact that the hand calculations are mainly based on the reference building but applied to twelve-storey mean while the FEM-model is recalculated to withstand the extra loads from the extra storeys.

The FE model is based on a significant approximations which have large effect on the final result. Some of these approximations relates to semi-rigid connections between “walls and walls” and “walls and slabs”. Also anchoring effects were not included in the model.

The result concerning human comfort requirements showing that the building is acceptable as an office building but not as a residential building and shows the possibilities but also the limitations of multi-storey timber buildings.

Other parts that have not been taken into consideration in this thesis are acoustics, fire and economy. These aspects also have a great impact on the possibilities of building multi-storey timber buildings and also needs to be checked.

This thesis had the goal to build a twelve-storey building completely in timber which resulted in some load bearing parts of the building being unreasonably disproportionate for a sustainable building. This can be improved by choosing steel instead of timber to reduce the dimensions in some of the critical load bearing parts.

Oscillations of multi-storey timber buildings are an area that is not well researched. It is not surprising, since it previously was not allowed to build high-rise buildings made of timber and where the method of building multi-storey buildings in timber is not highly developed. In today's society where the buildings constructed in timbre gets taller and taller the dynamic influence on timber buildings needs to be further explored since timber is a lightweight building and therefore does not have the same conditions as the more established building materials for multi-storey buildings.

The results in this study is based on full interaction between all elements.

11 Conclusion

In conclusion it is possible to stabilize a twelve-story timber building in ultimate limit state by the use of diaphragm action in the walls. This implies however large dimensions of connections and sheeting and a high contact pressure between the sill and the end studs in the walls arises. Consideration should be given the possibility of using other materials then timber in some critical structural parts and an additional number of stabilizing components are to be recommended. The plastic design method that are applied in this thesis utilizes the fact that the joints exhibit some extensibility before its capacity decreases crucial. This results in a better material utilization compared to previously applied elastic design methods.

The obtained results indicate that the transversal accelerations of the building are slightly too high to meet the human comfort requirements defined for the structure. The human perception of vibrations depends on many variables and it is difficult to decide an exact limit value of what is considered habitable. More research and calculations can provide interesting information and possible improvements to provide more understanding of the system's mode of action. The dynamic effects are very complex and therefore more accurate studies in this area can produce large profits, from a financial and material perspective. Wind tunnel test are probably the best solution to study the buildings behavior, but with the disadvantage that they are costly and highly time consuming.

Further studies of damping devices and tension rods can be highly interesting as it is a common way to solve vibration problems in different parts of the world where many tall buildings are constructed.

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12.2 Oral sources

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[2] Rune Abrahamsen Sweco, mail contact 7 may 2012

A Input data, description and information of the building

Location	Varberg	
Terrain category	2	
Service class	1	
Partial factor - National Annex p.15	Safety class 1	$\gamma_{d.1} := 0.83$
	Safety class 2	$\gamma_{d.2} := 0.91$
	Safety class 3	$\gamma_{d.3} := 1$

Geometry of the building

Height of each storey	$h := 2.975m$
Depth of the structure	$b := 11.803m$
Width of the structure	$d := 27.459m$
Total hight of building	$h_{tot} := 36m$
Number of storeys	$n_{storey} := 12$
Roof slope	$\alpha := 6deg$
The same roof angle on both sides of the ridge	$\alpha_{Roof1} := \alpha = 0.105$ $\alpha_{Roof2} := \alpha = 0.105$
Length of roof	$L_{roof} := \frac{b}{\cos(\alpha)} = 11.868\,m$
Span of floor between wall 8 and 9	$L_{floor.8\_9} := \frac{5.919m}{2} = 2.959\,m$

Areas

Roof area	$A_{roof} := L_{roof} \cdot d = 325.884\,m^2$
Area of floor	$A_{floor} := d \cdot b = 324.099\,m^2$
Area of floor acting on wall 9	$A_{Load.wall.9} := d \cdot L_{floor.8\_9} = 81.265\,m^2$
C-C distance between studs	$s_{stud} := 0.6m$

Self weights

Self weight of the roof $G_{roof} := 0.6\frac{kN}{m^2}$

Self weight of the floor $G_{floor} := 0.6\frac{kN}{m^2}$

Self weight of the inner walls $G_{innerwall} := 1.7\frac{kN}{m}$

Self weight of the outer walls $G_{outerwall} := 1.3\frac{kN}{m}$

Vertikal loads

Total weight of roof $G_{roof.tot} := G_{roof} \cdot A_{roof} = 195.53 \cdot kN$

Weight from roof acting on one wall $G_{tak} := G_{roof} \cdot \frac{A_{roof}}{2} = 97.765 \cdot kN$

Selfweight from roof acting as a line load on the outer walls

$$G_{roof.LL} := \frac{G_{tak}}{d} = 3.56 \cdot \frac{kN}{m}$$

Selfweight from roof acting on one stud in the outer walls

$$G_{roof.stud} := G_{roof.LL} \cdot s_{stud} = 2.136 \cdot kN$$

Total load from self weight acting on one stud in wall 9 for one floor

$$G_{wall.9.stud} := 1.626kN$$

Total load from self weight acting on one stud in wall 8 for one floor

$$G_{wall.8.stud} := 2.859kN$$

Length of the different walls in the building

$$\begin{pmatrix} L_{wall.1} \\ L_{wall.2} \\ L_{wall.3} \\ L_{wall.4} \\ L_{wall.5} \\ L_{wall.6} \\ L_{wall.7} \\ L_{wall.8} \\ L_{wall.9} \end{pmatrix} := \begin{pmatrix} b \\ 5.488m \\ 5.919m \\ 5.919m \\ 5.488m \\ b \\ d \\ 12.044m \\ d \end{pmatrix} = \begin{pmatrix} 11.803 \\ 5.488 \\ 5.919 \\ 5.919 \\ 5.488 \\ 11.803 \\ 27.459 \\ 12.044 \\ 27.459 \end{pmatrix} m \qquad L_{walls} := \begin{pmatrix} L_{wall.1} \\ L_{wall.2} \\ L_{wall.3} \\ L_{wall.4} \\ L_{wall.5} \\ L_{wall.6} \\ L_{wall.7} \\ L_{wall.8} \\ L_{wall.9} \end{pmatrix} = \begin{pmatrix} 11.803 \\ 5.488 \\ 5.919 \\ 5.919 \\ 5.488 \\ 11.803 \\ 27.459 \\ 12.044 \\ 27.459 \end{pmatrix} m$$

$L_{tot.innerwalls} := L_{wall.2} + L_{wall.3} + L_{wall.4} + L_{wall.5} + L_{wall.8} = 34.858\,m$

$L_{tot.outerwalls} := L_{wall.1} + L_{wall.6} + L_{wall.7} + L_{wall.9} = 78.524\,m$

Thicknes of the walls $T_{outerwall} := 0.242m$ $T_{innerwall} := 0.175m$

Selfweight from one floor

Total length of outer walls $L_{outer.walls} := L_{wall.1} + L_{wall.6} + L_{wall.7} + L_{wall.9} = 78.524\,m$

Total length of inner walls $L_{inner.walls} := L_{wall.2} + L_{wall.3} + L_{wall.4} + L_{wall.5} + L_{wall.8} = 34.858\,m$

Total selfweight from one floor

$G_{one.floor} := G_{floor} \cdot A_{floor} + G_{outerwall} \cdot L_{outer.walls} + G_{innerwall} \cdot L_{inner.walls} = 355.799 \cdot kN$

Total height at the different floors

Floor 12 $h_{tot.12} := 12 \cdot h = 35.7\,m$

Floor 11 $h_{tot.11} := 11 \cdot h = 32.725\,m$

Floor 10 $h_{tot.10} := 10 \cdot h = 29.75\,m$

Floor 9 $h_{tot.9} := 9 \cdot h = 26.775\,m$

Floor 8 $h_{tot.8} := 8 \cdot h = 23.8\,m$

Floor 7 $h_{tot.7} := 7 \cdot h = 20.825\,m$

Floor 6 $h_{tot.6} := 6 \cdot h = 17.85\,m$

Floor 5 $h_{tot.5} := 5 \cdot h = 14.875\,m$

Floor 4 $h_{tot.4} := 4 \cdot h = 11.9\,m$

Floor 3 $h_{tot.3} := 3 \cdot h = 8.925\,m$

Floor 2 $h_{tot.2} := 2 \cdot h = 5.95\,m$

Floor 1 $h_{tot.1} := 1 \cdot h = 2.975\,m$

Floor 0 $h_{tot.0} := 0h = 0$

Number of floors

Floor 12	$n_{fl12} := 12$
Floor 11	$n_{fl11} := 11$
Floor 10	$n_{fl10} := 10$
Floor 9	$n_{fl9} := 9$
Floor 8	$n_{fl8} := 8$
Floor 7	$n_{fl7} := 7$
Floor 6	$n_{fl6} := 6$
Floor 5	$n_{fl5} := 5$
Floor 4	$n_{fl4} := 4$
Floor 3	$n_{fl3} := 3$
Floor 2	$n_{fl2} := 2$
Floor 1	$n_{fl1} := 1$
Floor 0	$n_{fl0} := 0$

B Loads acting on the building

Imposed loads - Section 6 EN 1991-1-1:2002 / National Annex

Category A - Imposed loads for residental buildings

Characteristic value of a uniformly distributed load (Table 6.2 Table C-1)

General $q_{kimpG} := 2.0\frac{kN}{m^2}$

Stairs $q_{kimpS} := 2.0\frac{kN}{m^2}$

Balconies $q_{kimpB} := 3.5\frac{kN}{m^2}$

Uniformly distributed load for movable partitions with a self-weight < 1,0 kN/m wall length (8)

$$q_k := 0.5\frac{kN}{m^2}$$

Total uniformly distributed load for both imposed load and movable partitions (Table 6.2)

$$q_{kimpGtot} := q_{kimpG} + q_k = 2.5\cdot\frac{kN}{m^2}$$

Representative values for imposed loads - Category A (Table B-1)

Combination value $\psi_0 := 0.7$

Frequent value $\psi_1 := 0.5$

Quasi - permanent value $\psi_2 := 0.3$

Reduction factor for structural members that carry imposed loads from several stories

Calculations of the reduction factor α.n for the different floors that account for the fact that it is unlikely that loads on several floors attain high values at the same time. n are the number of storeys (> 2) above the loaded structural elements from the same category. (Formula 6.2)

Floor 11 $n_{11} := 0$ $\alpha_{n11} := 1$

Floor 10 $n_{10} := 1$ $\alpha_{n10} := 1$

Floor 9 $n_9 := 2$ $\alpha_{n9} := \frac{\left[2 + \left(n_9 - 2\right) \cdot \psi_0\right]}{n_9} = 1$

Floor 8 $n_8 := 3$ $\alpha_{n8} := \frac{\left[2 + \left(n_8 - 2\right) \cdot \psi_0\right]}{n_8} = 0.9$

Floor 7 $n_7 := 4$ $\alpha_{n7} := \frac{\left[2 + \left(n_7 - 2\right) \cdot \psi_0\right]}{n_7} = 0.85$

Floor 6 $n_6 := 5$ $\alpha_{n6} := \frac{\left[2 + \left(n_6 - 2\right) \cdot \psi_0\right]}{n_6} = 0.82$

Floor 5 $n_5 := 6$ $\alpha_{n5} := \frac{\left[2 + \left(n_5 - 2\right) \cdot \psi_0\right]}{n_5} = 0.8$

Floor 4 $n_4 := 7$ $\alpha_{n4} := \frac{\left[2 + \left(n_4 - 2\right) \cdot \psi_0\right]}{n_4} = 0.786$

Floor 3 $n_3 := 8$ $\alpha_{n3} := \frac{\left[2 + \left(n_3 - 2\right) \cdot \psi_0\right]}{n_3} = 0.775$

Floor 2 $n_2 := 9$ $\alpha_{n2} := \frac{\left[2 + \left(n_2 - 2\right) \cdot \psi_0\right]}{n_2} = 0.767$

Floor 1 $n_1 := 10$ $\alpha_{n1} := \frac{\left[2 + \left(n_1 - 2\right) \cdot \psi_0\right]}{n_1} = 0.76$

Floor 0 $n_0 := 11$ $\alpha_{n0} := \frac{\left[2 + \left(n_0 - 2\right) \cdot \psi_0\right]}{n_0} = 0.755$

Characteristic imposed loads acting on each floor

The value of the imposed load for each floor including the reduction factor α.n.
The same characteristic value of the uniformly distributed imposed load are valid for all floors.

Floor 11 $q_{imp11} := \alpha_{n11} \cdot q_{kimpGtot} \cdot 0 = 0 \cdot \frac{kN}{m^2}$

Floor 10 $q_{imp10} := \alpha_{n10} \cdot \left(q_{kimpGtot} \cdot 1\right) = 2.5 \cdot \frac{kN}{m^2}$

Floor 9 $q_{imp9} := \alpha_{n9} \cdot \left(q_{kimpGtot} \cdot 2\right) = 5 \cdot \frac{kN}{m^2}$

Floor 8 $q_{imp8} := \alpha_{n8} \cdot \left(q_{kimpGtot} \cdot 3\right) = 6.75 \cdot \frac{kN}{m^2}$

Floor 7 $q_{imp7} := \alpha_{n7} \cdot \left(q_{kimpGtot} \cdot 4\right) = 8.5 \cdot \frac{kN}{m^2}$

Floor 6 $q_{imp6} := \alpha_{n6} \cdot \left(q_{kimpGtot} \cdot 5\right) = 10.25 \cdot \frac{kN}{m^2}$

Floor 5 $q_{imp5} := \alpha_{n5} \cdot \left(q_{kimpGtot} \cdot 6\right) = 12 \cdot \frac{kN}{m^2}$

Floor 4 $q_{imp4} := \alpha_{n4} \cdot \left(q_{kimpGtot} \cdot 7\right) = 13.75 \cdot \frac{kN}{m^2}$

Floor 3 $q_{imp3} := \alpha_{n3} \cdot \left(q_{kimpGtot} \cdot 8\right) = 15.5 \cdot \frac{kN}{m^2}$

Floor 2 $q_{imp2} := \alpha_{n2} \cdot \left(q_{kimpGtot} \cdot 9\right) = 17.25 \cdot \frac{kN}{m^2}$

Floor 1 $q_{imp1} := \alpha_{n1} \cdot \left(q_{kimpGtot} \cdot 10\right) = 19 \cdot \frac{kN}{m^2}$

Floor 0 $q_{imp0} := \alpha_{n0} \cdot \left(q_{kimpGtot} \cdot 11\right) = 20.75 \cdot \frac{kN}{m^2}$

Snow loads - EN 1991-1-3 / National Annex

Weight density for snow $\gamma_{snow} := 2 \frac{kN}{m^3}$

Characteristic value of snow load on the ground in Varberg (Table C-9) $S_k := 1.5 \frac{kN}{m^2}$

Exposure coefficient - Normal topography (Table 5.1) $C_e := 1$

Thermal coefficient $C_t := 1$

Representative values for snow load on the roof (Table B-1)

Combination value $\psi S_0 := 0.6$

Frequent value $\psi S_1 := 0.3$

Quasi - permanent value $\psi S_2 := 0.1$

Main roof - Pitched roof - Section 5

The same roof angle on both sides of the ridge

$\alpha_{Roof1} := \alpha_{Roof2} = 0.105$

Snow load shape coefficient - Duopitch/Monopitch roof α=6 (Tabel 5.2) $\mu_1 := 0.8$

Snow loads on main roof (Equation 5.1)

Pitched roof

Case (i)

$$S_{i.1} := \mu_1 \cdot C_e \cdot C_t \cdot S_k = 1.2 \cdot \frac{kN}{m^2}$$

$$S_{i.2} := \mu_1 \cdot C_e \cdot C_t \cdot S_k = 1.2 \cdot \frac{kN}{m^2}$$

Case (ii)

$$S_{ii.1} := 0.5 \cdot \left(\mu_1 \cdot C_e \cdot C_t \cdot S_k \right) = 0.6 \cdot \frac{kN}{m^2}$$

$$S_{ii.2} := \mu_1 \cdot C_e \cdot C_t \cdot S_k = 1.2 \cdot \frac{kN}{m^2}$$

Case (ii)

$$S_{iii.1} := \mu_1 \cdot C_e \cdot C_t \cdot S_k = 1.2 \cdot \frac{kN}{m^2}$$

$$S_{iii2} := 0.5 \cdot \left(\mu_1 \cdot C_e \cdot C_t \cdot S_k \right) = 0.6 \cdot \frac{kN}{m^2}$$

<u>Design value for snow load - Main roof</u> $q_{snow} := 1.2 \frac{kN}{m^2}$

Roof over entrance - Section 5

Distance between roofs

$$h_{B4.roof} := 19.923m - 11.465m + 1m = 9.458\,m$$

b value for main roof part 1 $b_{roof1} := 8.224m$

b value for main roof part 2 $b_{roof2} := 3.579m$

The drift length $l_{s4} := \min(2\cdot h_{B4.roof}, 10m) = 10m$

Recommended restriction $5m \le l_s \le 10m$

$b_2 \le l_s$ entails

Case (i)

Snow load shape coefficients - flat roof $\mu_{1.4} := 0.8$

Snow loads on Roof balcony 2 $S_{1.4} := S_k\cdot\mu_{1.4} = 1.2\cdot\frac{kN}{m^2}$

Case (ii)

Snow load shape coefficients - roofs abutting to taller construction works $\mu_{1.4} = 0.8$

$\mu_{2.4} = \mu_s + \mu_w$

Snow load shape coefficient due to sliding of snow from the upper roof μ_s

$\alpha \le 15°$ entails $\mu_{s4} := 0$

Snow load shape coefficient due to wind

$$\mu_{w4} := \max\left[\left[\frac{(b_{roof1}+b_{roof2})}{2\cdot h_{B4.roof}}\right], \left(\gamma_{snow}\cdot\frac{h_{B4.roof}}{S_k}\right)\right] = 12.611$$

$$\mu_{w4} := \min\left[\left[\frac{(b_{roof1}+b_{roof2})}{2\cdot h_{B4.roof}}\right], \left(\gamma_{snow}\cdot\frac{h_{B4.roof}}{S_k}\right)\right] = 0.624$$

Recommended range $0.8 \le \mu_w \le 4$ entails $\mu_{w4final} := \max(\mu_{w4}, 0.8) = 0.8$

$$\mu_{2.4} := \mu_{s4} + \mu_{w4final} = 0.8$$

Snow loads over entrance

Case (i) $S_{1.4} = 1.2\cdot kPa$

Case (ii) $S_{2.4.left} := \mu_{1.4}\cdot S_k = 1.2\cdot kPa$

$$S_{2.4.right} := S_k\cdot\left[\mu_{1.4} + \frac{1}{10}\cdot(\mu_{2.4} - \mu_{1.4})\right] = 1.2\cdot kPa$$

<u>Design value for snow load - Entrance roof</u> $q_{snow.ER} := 1.2\,\frac{kN}{m^2}$

Wind loads - EN 1991-1-4

The wind actions calculated using EN 1991-1-4 are characteristic values (See EN 1990, 4.1.2). They are determined from the basic values of wind velocity or the velocity pressure. In accordance with EN 1990 4.1.2 (7)P the basic values are characteristic values having annual probabilities of exceedence of 0,02, which is equivalent to a mean return period of 50 years.

Terrain category	2
Reference wind speed (Table C-10)	$v_b := 25\frac{m}{s}$
Air density	$\rho := 1.25\frac{kg}{m^3}$
Basic velocity pressure (Equation 4.8)	$q_b := \frac{1}{2}\cdot\rho\cdot v_b^{\,2} = 0.391\cdot kPa$
Height above ground	$z := h_{tot} = 36\,m$
Minimum height (Table 4.1)	$z_{min} := 2m$
Maximum height	$z_{max} := 200m$
Roughness length (Table 4.1)	$z_0 := 0.05m$
	$z_{0.2} := 0.05m$
Pressure coefficient for the external pressure	C_{pe}
Pressure coefficient for the internal pressure	C_{pi}
External wind pressure (Equation 5.1)	$w_e = q_p\cdot c_{pe}$
Internal wind pressure (Equation 5.2)	$w_i = q_p\cdot c_{pi}$

Representative values for wind loads (Table B-1)

Combination value	$\psi W_0 := 0.3$
Frequent value	$\psi W_1 := 0.2$
Quasi - permanent value	$\psi W_2 := 0$

Basic values

The fundamental value of the basic wind velocity, $v_{b,0}$, is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights.

The fundamental value of the basic wind velocity $v_{b.0} := 25 \frac{m}{s}$

The directional factor $c_{dir} := 1$

The season factor $c_{season} := 1$

The basic wind velocity (Equation 4.1) $v_b := c_{dir} \cdot c_{season} \cdot v_{b.0} = 25 \frac{m}{s}$

Mean wind - Variation with height

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity.

Mean velocity at height z (Equation 4.3) $v_m = c_r \cdot c_0 \cdot v_b$

Terrain roughness

The roughness factor, $c_r(z)$, accounts for the variability of the mean wind velocity at the site of the structure due to the height above ground level, the ground roughness of the terrain upwind of the structure in the wind direction considered

Terrain factor (Equation 4.5) $k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.2}} \right)^{0.07} = 0.19$

Roughness factor (Equation 4.4) $c_r := k_r \cdot \ln\left(\frac{z}{z_0} \right)$ $z_{min} \le z \le z_{max}$

$c_{r.z.min} = c_{r.min}$ $z \le z_{min}$

Terrain orography

Where orography (e.g. hills, cliffs etc.) increases wind velocities by more than 5% the effects should be taken into account using the orography factor c_O.
The effects of orography may be neglected when the average slope of the upwind terrain is less than 3°. The upwind terrain may be considered up to a distance of 10 times the height of the isolated orographic feature.

Orography factor, taken as 1,0 $c_0 := 1$

Mean velocity at height z (Equation 4.3) $v_m := c_r \cdot c_0 \cdot v_b$

Wind turbulence

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

Turbulence factor, taken as 1 $k_l := 1$

Standard deviation of the turbulent component of wind velocity (Equation 4.6)

$$\sigma_v := k_r \cdot v_b \cdot k_l = 4.75 \frac{m}{s}$$

Turbulence intensity (Equation 4.7) $I_v := \frac{\sigma_v}{v_m}$

Peak velocity pressure

The peak velocity pressure $q_p(z)$ at height z, includes mean and short-term velocity fluctuations

Peak velocity pressure (Equation 4.5(1)) $q_p = \left(1 + 6 \cdot I_v\right)\left(k_r \cdot \ln\left(\frac{z}{z_0}\right)\right)^2 \cdot q_b$

Reference hights

$d < h_{tot} \le 2d$ entails two parts

Reference hight 1 $h_1 := d = 27.459\,m$

Reference hight 2 $h_2 := h_{tot} = 36\,m$

Reference hight 1

Roughness factor $c_{r1} := k_r \cdot \ln\left(\frac{h_1}{z_0}\right) = 1.199$ $z_{min} \le z \le z_{max}$

Mean velocity at height z (Equation 4.3) $v_{m1} := c_{r1} \cdot c_0 \cdot v_b = 29.965\,\frac{m}{s}$

Standard deviation of the turbulent component of wind velocity (Equation 4.6)

$$\sigma_v := k_r \cdot v_b \cdot k_l = 4.75\,\frac{m}{s}$$

Turbulence intensity (Equation 4.7) $I_{v1} := \frac{\sigma_v}{v_{m1}} = 0.159$

Peak velocity pressure (Formula 4.5(1)) $q_{p1} := \left[\left(1 + 6 \cdot I_{v1}\right)\left(k_r \cdot \ln\left(\frac{h_1}{z_0}\right)\right)^2\right] \cdot q_b = 1.095 \cdot \frac{kN}{m^2}$

Reference hight 2

Roughness factor $c_{r2} := k_r \cdot \ln\left(\frac{h_2}{z_0}\right) = 1.25$ $z_{min} \le z \le z_{max}$

Mean velocity at height z (Equation 4.3) $v_{m2} := c_{r2} \cdot c_0 \cdot v_b = 31.251\,\frac{m}{s}$

Standard deviation of the turbulent component of wind velocity (Equation 4.6)

$$\sigma_v := k_r \cdot v_b \cdot k_l = 4.75\,\frac{m}{s}$$

Turbulence intensity(Equation 4.7) $I_{v2} := \frac{\sigma_v}{v_{m2}} = 0.152$

Peak velocity pressure (Formula 4.5(1)) $q_{p2} := \left[\left(1 + 6 \cdot I_{v2}\right)\left(k_r \cdot \ln\left(\frac{h_2}{z_0}\right)\right)^2\right] \cdot q_b = 1.167 \cdot \frac{kN}{m^2}$

Wind load acting on the walls

$e_w := \min\left(d, 2h_{tot}\right) = 27.459\,m$

$e > b$

entails two zones, zone A and zone B

$\frac{h_{tot}}{b} = 3.05$

Values for the design of the overall load bearing structure of the building

Values for cpe,10 may be used for the design of the overall load bearing structure of buildings

Pressure coefficient for the external pressure

Zone A $c_{Ape10} := -1.2$

Zone B $c_{Bpe10} := -0.8$

Zone D $c_{Dpe10} := 0.8$

Zone E $c_{Epe10} := -0.5 + \left[\frac{(3.05 - 1)}{(5 - 1)}\right] \cdot \left[-0.7 - (-0.5)\right] = -0.602$

Net pressure in the different zones

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

Pressure coefficient for the internal pressure

Zone A $C_{Api10} := (0.2, -0.3)$

Zone B $C_{Bpi10} := (0.2, -0.3)$

Zone D $c_{Dpi10} := (0.2, -0.3)$

Zone E $c_{Epi10} := (0.2, -0.3)$

External wind pressure (Equation 5.1) $w_e = q_p \cdot c_{pe}$

Zone A $\frac{e_w}{5} = 5.492\,m$ Zone B $b - \frac{e_w}{5} = 6.311\,m$

Design value for wind load in the different zones

Reference hight 1 (h.max=27.458m)

$w_{e10.1A} := q_{p1} \cdot (c_{Ape10} - 0.2) = -1.533 \cdot \frac{kN}{m^2}$ $w_{e10.1B} := q_{p1} \cdot (c_{Bpe10} - 0.2) = -1.095 \cdot \frac{kN}{m^2}$

$w_{e10.1D} := q_{p1} \cdot [c_{Dpe10} - (-0.3)] = 1.204 \cdot \frac{kN}{m^2}$ $w_{e10.1E} := q_{p1} \cdot (c_{Epe10} - 0.2) = -0.879 \cdot \frac{kN}{m^2}$

Reference hight 2 (h.max=36m)

$w_{e10.2A} := q_{p2} \cdot (c_{Ape10} - 0.2) = -1.634 \cdot \frac{kN}{m^2}$ $w_{e10.2B} := q_{p2} \cdot (c_{Bpe10} - 0.2) = -1.167 \cdot \frac{kN}{m^2}$

$w_{e10.2D} := q_{p2} \cdot [c_{Dpe10} - (-0.3)] = 1.284 \cdot \frac{kN}{m^2}$ $w_{e10.2E} := q_{p2} \cdot (c_{Epe10} - 0.2) = -0.937 \cdot \frac{kN}{m^2}$

The wind forces for the whole structure or a structural component

Pressure $c_{Dpe10} = 0.8$

Suction $c_{Epe10} = -0.602$

Total wind pressure coefficient for the building $C_{pe10} := c_{Dpe10} - c_{Epe10} = 1.402$

Values for the design of small elements and fixings in the building

Values for *c*pe,1 are intended for the design of small elements and fixings with an area per element of 1 m^2 or less such as cladding elements and roofing elements.

Pressure coefficient for the external pressure

Zone A $c_{Ape1} := -1.4$

Zone B $c_{Bpe1} := -1.1$

Zone D $c_{Dpe1} := 1$

Zone E $c_{Epe1} := -0.5 + \left[\frac{(3.05 - 1)}{(5 - 1)}\right] \cdot [-0.7 - (-0.5)] = -0.602$

Internal pressure coefficient for uniformly dustributed openings $\mu_{cpi} := 1$ $\frac{h}{d} = 0.108$

Pressure coefficient for the internal pressure

Zone A $c_{Api1} := 0.2, -0.3$

Zone B $c_{Bpi1} := 0.2, -0.3$

Zone D $c_{Dpi1} := 0.2, -0.3$

Zone E $c_{Epi1} := 0.2, -0.3$

Net pressure in the different zones

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

$c_{pe1A} := c_{Ape1} - 0.2 = -1.6$

$c_{pe1B} := c_{Bpe1} - 0.2 = -1.3$

$c_{pe1D} := c_{Dpe1} - (-0.3) = 1.3$

$c_{pe1E} := c_{Epe1} - 0.2 = -0.803$

External wind pressure (Equation 5.1) $w_e = q_p \cdot c_{pe}$

Zone A $\frac{e_w}{5} = 5.492\,m$ Zone B $b - \frac{e_w}{5} = 6.311\,m$

<u>Design value for wind load in the different zones</u>

Reference hight 1 (h.max=27.458m)

$w_{e1A} := q_{p1} \cdot c_{pe1A} = -1.752 \cdot \frac{kN}{m^2}$ $w_{e1B} := q_{p1} \cdot c_{pe1B} = -1.423 \cdot \frac{kN}{m^2}$

$w_{e1D} := q_{p1} \cdot c_{pe1D} = 1.423 \cdot \frac{kN}{m^2}$ $w_{e1E} := q_{p1} \cdot c_{pe1E} = -0.879 \cdot \frac{kN}{m^2}$

Reference hight 2 (h.max=36m)

$w_{e2A} := q_{p2} \cdot c_{pe1A} = -1.867 \cdot \frac{kN}{m^2}$ $w_{e1.2B} := q_{p2} \cdot c_{pe1B} = -1.517 \cdot \frac{kN}{m^2}$

$w_{e2D} := q_{p2} \cdot c_{pe1D} = 1.517 \cdot \frac{kN}{m^2}$ $w_{e1.2E} := q_{p2} \cdot c_{pe1E} = -0.937 \cdot \frac{kN}{m^2}$

The wind forces for the whole structure or a structural component

Pressure $c_{Dpe1} = 1$

Suction $c_{Epe1} = -0.602$

Total wind pressure coefficient for the building

$C_{pe1} := c_{Dpe1} - c_{Epe1} = 1.603$

Friction force

Friction force (Equation 5.7) $F_{fr} = c_{fr} \cdot q_p \cdot A_{fr}$

Frictional coefficients, Smooth (Table 7.10)

$c_{fr} := 0.01$

Reference area $A_{fr} := \min(2 \cdot d, 4 \cdot h_{tot}) \cdot 12 \cdot h \cdot 2 = 3.921 \times 10^3\,m^2$

Structural factor *cscd*

The structural factor *cscd* should take into account the effect on wind actions from the non-simultaneous occurrence of peak wind pressures on the surface together with the effect of the vibrations of the structure due to turbulence.

The size factor *cs* takes into account the reduction effect on the wind action due to the non-simultaneity of occurrence of the peak wind pressures on the surface.

The dynamic factor *cd* takes into account the increasing effect from vibrations due to turbulence in resonance with the structure.

$$\mathrm{cscd} = \frac{1 + 2\cdot k_p\cdot I_v\cdot\sqrt{B^2 + R_{cscd.1}{}^2}}{1 + 7\cdot I_v}$$

Reference length scale $L_t := 300\mathrm{m}$

Reference height $z_t := 200\mathrm{m}$

Roughness length in meters $z_{0.z} := \frac{z_0}{m} = 0.05$

$$\alpha_Z := 0.67 + 0.05\cdot\ln\left(z_{0.z}\right) = 0.52$$

The turbulent length scale (Equation B.1) $L_Z := L_t\cdot\left(\frac{z}{z_t}\right)^{\alpha_Z} = 122.943\,\mathrm{m}$

The background factor (Equation B.3)
It is on the safe side to use B^2=1

$$B := \sqrt{\frac{1}{1 + 0.9\cdot\left(\frac{b + h_{tot}}{L_Z}\right)^{0.63}}} = 0.817$$

$$h_{ref} := 200\mathrm{m}$$

$$B := \sqrt{\exp\left[-0.05\cdot\left(\frac{h_{tot}}{h_{ref}}\right) + \left(1 - \frac{d}{h_{tot}}\right)\cdot\left[0.04 + 0.01\cdot\left(\frac{h_{tot}}{h_{ref}}\right)\right]\right]} = 1$$

The mass per unit length $m_s := \frac{G_{one.floor}}{h\cdot g} = 1.22\times 10^4\,\frac{kg}{m}$

Fundamental flexural frequency $n_{1x} := 3.06\mathrm{Hz}$

The logarithmic decrement of structural damping (Table F.2) $\delta_s := 0.1$

The logarithmic decrement of damping due to special devices δ_d

The force coefficient of rectangular sections with sharp corners and without free-end flow. (Figure 7.23)

$$c_{f.0} := 2.2$$

The slenderness ratio (Table 7.16)

$$\lambda := \frac{2h_{tot}}{b} - \left[\frac{(50-36)}{(50-15)}\right]\cdot\left(\frac{2h_{tot}}{b} - \frac{1.4h_{tot}}{b}\right) = 5.368$$

The sum of the projected areas of the members

$$A_{cscd} := L_{tot.innerwalls}\cdot T_{innerwall} + L_{tot.outerwalls}\cdot T_{outerwall} = 25.103\,m^2$$

The overall envelope area $A_c := b\cdot d = 324.099\,m^2$

The solidity ratio (Equation 7.28) $\varphi := \frac{A_{cscd}}{A_c} = 0.077$

The end-effect factor for elements with free-end flow as defined (Figure 7.36)

$$\psi_\lambda := 0.855$$

The air density $\rho = 1.25\frac{kg}{m^3}$

The force coefficient for wind action in the wind direction (Equation 7.12) $c_f := c_{f.0}\cdot\psi_\lambda = 1.881$

The fundamental flexural mode (Equation F.13) $\phi = \left(\frac{z}{h_{tot}}\right)^{\zeta}$

$$\zeta := 1$$

The mass per unit length $m_s := \frac{G_{one.floor}}{h\cdot g} = 1.22\times 10^4\,\frac{kg}{m}$

The averaging time for the mean wind velocity $T_{600.sec} := 600.s$

Reference hight 1

The fundamental flexural mode (Equation F.13) $\phi_1 := \left(\frac{h_1}{h_{tot}}\right)^{\zeta} = 0.763$

The equivalent mass per unit length of the fundamental mode (Equation F.14).

$$m_{e.1} := \frac{\int_0^{h_1} m_s\cdot\phi_1^{\,2}\,ds}{\int_0^{h_1} \phi_1^{\,2}\,ds} = 12.195\,\frac{s^2}{m}\cdot\frac{kN}{m}$$

General recommendations Part 6.3.1(1)

$$y_{c1} := \frac{150m \cdot n_{1x}}{v_{m1}} = 15.318$$

$$F_{cscd.1} := \frac{4 \cdot y_{c1}}{\left(1 + 70.8 \cdot y_{c1}^{2}\right)^{\frac{5}{6}}} = 0.019$$

$$\phi_{h.1} := \frac{1}{1 + \frac{2 \cdot n_{1x} \cdot h_{tot}}{v_{m1}}} = 0.12$$

$$\phi_{b.1} := \frac{1}{1 + \frac{3.2 \cdot n_{1x} \cdot b}{v_{m1}}} = 0.206$$

The logarithmic decrement of aerodynamic damping (Equation F.16)

$$\delta_{a.1} := \frac{c_f \cdot \rho \cdot b \cdot v_{m1}}{2 \cdot n_{1x} \cdot m_{e.1}} = 0.011$$

The resonance responce factor (Equation C.2)

$$R_{cscd.1} := \sqrt{\frac{2\pi \cdot F_{cscd.1} \cdot \phi_{b.1} \cdot \phi_{h.1}}{\delta_s + \delta_{a.1}}} = 0.161$$

The averaging time for the mean wind velocity

$$T_{600.sec} := 600.s$$

The natural frequency of the structure

$$n_{1.x} := 3.06Hz$$

The up-crossing frequency (Equation B.5)

$$v_1 := n_{1.x} \cdot \frac{R_{cscd.1}}{\sqrt{B^2 + R_{cscd.1}^{2}}} = 0.487 \frac{1}{s}$$

The peak factor (Equation B.4)

$$k_{p.1} := \max\left(3, \sqrt{2 \cdot \ln\left(v_1 \cdot T_{600.sec}\right)} + \frac{0.6}{\sqrt{2 \cdot \ln\left(v_1 \cdot T_{600.sec}\right)}}\right) = 3.547$$

Structural factor (Equation 6.1)

$$cscd_1 := \frac{1 + 2 \cdot k_{p.1} \cdot I_v \cdot \sqrt{B^2 + R_{cscd.1}^{2}}}{1 + 7 \cdot I_v} = 1.014$$

Reference hight 2

The equivalent mass per unit length of the fundamental mode (Equation F.14).

$$m_{e.2} := \frac{\int_{h_1}^{h_2} m_s \cdot \phi_1^{\,2}\, ds}{\int_{h_1}^{h_2} \phi_1^{\,2}\, ds} = 12.195\frac{s^2}{m}\cdot\frac{kN}{m}$$

General recommendations Part 6.3.1(1)

$$y_{c2} := \frac{150m \cdot n_{1x}}{v_{m2}} = 14.687$$

$$F_{cscd.2} := \frac{4 \cdot y_{c2}}{\left(1 + 70.8 \cdot y_{c2}^{\,2}\right)^{\frac{5}{6}}} = 0.019$$

$$\phi_{h.2} := \frac{1}{1 + \frac{2 \cdot n_{1x} \cdot h_{tot}}{v_{m2}}} = 0.124$$

$$\phi_{b.2} := \frac{1}{1 + \frac{3.2 \cdot n_{1x} \cdot b}{v_{m2}}} = 0.213$$

The logarithmic decrement of aerodynamic damping (Equation F.16)

$$\delta_{a.2} := \frac{c_f \cdot \rho \cdot b \cdot v_{m2}}{2 \cdot n_{1x} \cdot m_{e.2}} = 0.012$$

The resonance responce factor (Equation C.2)

$$R_{cscd.2} := \sqrt{\frac{2\pi \cdot F_{cscd.2} \cdot \phi_{b.2} \cdot \phi_{h.2}}{\delta_s + \delta_{a.2}}} = 0.169$$

The up-crossing frequency given

$$v_2 := n_{1.x} \cdot \frac{R_{cscd.2}}{\sqrt{B^2 + R_{cscd.2}^{\,2}}} = 0.509 \cdot Hz$$

The peak factor (Equation B.4)

$$k_{p.2} := \max\left(3, \sqrt{2 \cdot \ln\left(v_2 \cdot T_{600.sec}\right)} + \frac{0.6}{\sqrt{2 \cdot \ln\left(v_2 \cdot T_{600.sec}\right)}}\right) = 3.56$$

Structural factor (Equation 6.1)

$$cscd_2 := \frac{1 + 2 \cdot k_{p.2} \cdot I_{v2} \cdot \sqrt{B^2 + R_{cscd.2}^{\,2}}}{1 + 7 \cdot I_{v2}} = 1.017$$

Design value for the design of the overall load bearing structure of the building

Design value for wind load for the different reference heights

Reference hight 1

Total wind pressure acting on the building $w_{e10.1} := q_{p1} \cdot C_{pe10} = 1.536 \cdot \frac{kN}{m^2}$

External force (Equation 5.5) $F_{we.10.1} := cscd_1 \cdot w_{e10.1} = 1.557 \cdot \frac{kN}{m^2}$

Friction force (Equation 5.7) $F_{fr.10.1} := c_{fr} \cdot q_{p2} \cdot A_{fr} = 45.763 \cdot kN$

$$q_{wind1} := F_{we.10.1} + \frac{F_{fr.10.1}}{12 \cdot h \cdot d} = 1.604 \cdot \frac{kN}{m^2}$$

Reference hight 2

Total wind pressure acting on the building $w_{e10.2} := q_{p2} \cdot C_{pe10} = 1.637 \cdot \frac{kN}{m^2}$

External force (Equation 5.5) $F_{we.10.2} := cscd_2 \cdot w_{e10.2} = 1.664 \cdot \frac{kN}{m^2}$

Friction force (Equation 5.7) $F_{fr.10.2} := c_{fr} \cdot q_{p2} \cdot A_{fr} = 45.763 \cdot kN$

$$q_{wind2} := F_{we.10.2} + \frac{F_{fr.10.2}}{12 \cdot h \cdot d} = 1.711 \cdot \frac{kN}{m^2}$$

Design value for the design of small elements and fixings in the building

Reference hight 1

Total wind pressure acting on the building $w_{e1.1} := q_{p1} \cdot C_{pe1} = 1.755 \cdot \frac{kN}{m^2}$

External force (Equation 5.5) $F_{we.1} := cscd_1 \cdot w_{e1.1} = 1.779 \cdot \frac{kN}{m^2}$

Friction force (Equation 5.7) $F_{fr.1} := c_{fr} \cdot q_{p2} \cdot A_{fr} = 45.763 \cdot kN$

$$q_{wind.1.1} := F_{we.1} + \frac{F_{fr.1}}{12 \cdot h \cdot d} = 1.826 \cdot \frac{kN}{m^2}$$

Reference hight 2

Total wind pressure acting on the building $w_{e1.2} := q_{p2} \cdot C_{pe1} = 1.87 \cdot \frac{kN}{m^2}$

External force (Equation 5.5) $F_{we.1.2} := cscd_2 \cdot w_{e1.2} = 1.901 \cdot \frac{kN}{m^2}$

Friction force (Equation 5.7) $F_{fr.1.2} := c_{fr} \cdot q_{p2} \cdot A_{fr} = 45.763 \cdot kN$

$$q_{wind.1.2} := F_{we.1.2} + \frac{F_{fr.1.2}}{12 \cdot h \cdot d} = 1.948 \cdot \frac{kN}{m^2}$$

Roof

External pressure coefficients for duopitch roofs

At θ = 0° the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of θ = -5° to +45°, so both positive and negative values are given**.** For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face**.**

$\theta := 0$ $\qquad e_{R0} := \min\left(d, 2h_{tot}\right) = 27.459\,m$

Values for the design of the overall load bearing structure of the building

Values for cpe,10 may be used for the design of the overall load bearing structure of buildings.

Pressure coefficient for the external pressure

$$C_{pe10F} := -1.7 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-1.7-(-0.9)\right] = -1.62 \qquad C_{peF} := 0 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[0-(0.2)\right] = 0.02$$

$$C_{pe10G} := -1.2 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-1.2-(-0.8)\right] = -1.16 \qquad C_{peG} := 0 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[0-(0.2)\right] = 0.02$$

$$C_{pe10H} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-0.6-(-0.3)\right] = -0.57 \qquad C_{peH} := 0 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[0-(0.2)\right] = 0.02$$

$$C_{pe10I} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-0.6-(-0.4)\right] = -0.58 \qquad C_{peI} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-0.6-(0)\right] = -0.54$$

$$C_{pe10J} := 0.2 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[0.2-(-1)\right] = 0.08 \qquad C_{peJ} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right]\cdot\left[-0.6-(0)\right] = -0.54$$

Pressure coefficient for the internal pressure

Zone F $\qquad C_{pi10F} := 0.2, -0.3$

Zone G $\qquad C_{pi10G} := 0.2, -0.3$

Zone H $\qquad C_{pi10H} := 0.2, -0.3$

Zone I $\qquad c_{pi1oI} := 0.2, -0.3$

Zone J $\qquad c_{pi10} := 0.2, -0.3$

Net pressure in the different zones

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

$$C_{p10F} := C_{pe10F} - 0.2 = -1.82 \qquad C_{pF} := C_{peF} - (-0.3) = 0.32$$

$$C_{p10G} := C_{pe10G} - 0.2 = -1.36 \qquad C_{pG} := C_{peG} - (-0.3) = 0.32$$

$$C_{p10H} := C_{pe10H} - 0.2 = -0.77 \qquad C_{pH} := C_{peH} - (-0.3) = 0.32$$

$C_{p10I} := C_{pe10I} - 0.2 = -0.78$ $\qquad$ $C_{pI} := C_{peI} - 0.2 = -0.74$

$C_{p10J} := C_{pe10J} - (-0.3) = 0.38$ $\qquad$ $C_{pJ} := C_{peJ} - 0.2 = -0.74$

The pressure changes rapidly between positive and negative values on the windward face around a pitch angle of α'E1 = -5° to +45°, so both positive and negative values are given. For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face.

Roof areas

$e_{roof} := \min(d, 2h_{tot}) = 27.459\,m$

$$Area_F := \frac{e_{roof}}{4}\cdot\frac{e_{roof}}{10}\cdot 2 = 37.7\,m^2$$

$$Area_G := \left(\frac{d\cdot e_{roof}}{10}\right) - Area_F = 37.7\,m^2$$

$$Area_H := \frac{A_{roof}}{2} - Area_F - Area_G = 87.542\,m^2$$

$$Area_J := \frac{e_{roof}}{10}\cdot d = 75.4\,m^2$$

$$Area_I := \frac{A_{roof}}{2} - Area_J = 87.542\,m^2$$

$W_{F.1} := Area_F\cdot C_{pF} = 12.064\,m^2$ $\qquad$ $W_{F.2} := Area_F\cdot C_{p10F} = -68.614\,m^2$

$W_{G.1} := Area_G\cdot C_{pG} = 12.064\,m^2$ $\qquad$ $W_{G.2} := Area_G\cdot C_{p10G} = -51.272\,m^2$

$W_{H.1} := Area_H\cdot C_{pH} = 28.014\,m^2$ $\qquad$ $W_{H.2} := Area_H\cdot C_{p10H} = -67.408\,m^2$

$W_{J.1} := Area_J\cdot C_{pJ} = -55.796\,m^2$ $\qquad$ $W_{J.2} := Area_J\cdot C_{p10J} = 28.652\,m^2$

$W_{I.1} := Area_I\cdot C_{pI} = -64.781\,m^2$ $\qquad$ $W_{I.2} := Area_I\cdot C_{p10I} = -68.283\,m^2$

$$W_{tot.pre} := \max(W_{F.1}, W_{F.2}) + \max(W_{G.1}, W_{G.2}) + \max(W_{H.1}, W_{H.2}) \;...\; = 16.012\,m^2$$
$$+ \max(W_{J.1}, W_{J.2}) + \max(W_{I.1}, W_{I.2})$$

$$W_{tot.suc} := \min(W_{F.1}, W_{F.2}) + \min(W_{G.1}, W_{G.2}) + \min(W_{H.1}, W_{H.2}) \;...\; = -311.372\,m^2$$
$$+ \min(W_{J.1}, W_{J.2}) + \min(W_{I.1}, W_{I.2})$$

Design value for the design of the overall load bearing structure of the building

Total wind pressure acting on the roof

Pressure

Wind pressure (Equation 5.1) $w_{10roof} := q_{p2} \cdot \frac{W_{tot.pre}}{d \cdot b} = 0.058 \cdot \frac{kN}{m^2}$

Suction

Wind suction (Equation 5.1) $w_{10.suction} := q_{p2} \cdot \frac{W_{tot.suc}}{d \cdot b} = -1.121 \cdot \frac{kN}{m^2}$

Design values for wind load acting on the roof

Pressure

Horizontal wind load on the roof θ=0 $q_{wH} := w_{10roof} \cdot \sin(\alpha) = 6.027 \times 10^{-3} \cdot \frac{kN}{m^2}$

Vertical wind load on the roof θ=0 $q_{wV} := w_{10roof} \cdot \cos(\alpha) = 0.057 \cdot \frac{kN}{m^2}$

Suction

Horizontal wind load on the roof θ=0 $q_{wH.suction} := w_{10.suction} \cdot \sin(\alpha) = -0.117 \cdot \frac{kN}{m^2}$

Vertical wind load on the roof θ=0 $q_{wV.suction} := w_{10.suction} \cdot \cos(\alpha) = -1.115 \cdot \frac{kN}{m^2}$

Values for the design of small elements and fixings in the building

Values for *c*pe,1 are intended for the design of small elements and fixings with an area per element of 1 m2 or less such as cladding elements and roofing elements.

$C_{pe1F} := -2.5 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-2.5 - (-2)] = -2.45$ $C_{peF} := 0 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [0 - (0.2)] = 0.02$

$C_{pe1G} := -2 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-2 - (-1.5)] = -1.95$ $C_{peG} := 0 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [0 - (0.2)] = 0.02$

$C_{pe1H} := -1.2 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-1.2 - (-0.3)] = -1.11$ $C_{peH} := 0 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [0 - (0.2)] = 0.02$

$C_{pe1I} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-0.6 - (-0.4)] = -0.58$ $C_{peI} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-0.6 - (0)] = -0.54$

$C_{pe1J} := 0.2 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [0.2 - (-1.5)] = 0.03$ $C_{peJ} := -0.6 - \left[\frac{(6-5)}{(15-5)}\right] \cdot [-0.6 - (0)] = -0.54$

Pressure coefficient for the internal pressure

Zone F $C_{pi1F} := 0.2, -0.3$

Zone G $C_{pi1G} := 0.2, -0.3$

Zone H $C_{pi1H} := 0.2, -0.3$

Zone I $c_{pi1I} := 0.2, -0.3$

Zone J $c_{pi1J} := 0.2, -0.3$

Net pressure in the different zones

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative.

$C_{p1F} := C_{pe1F} - 0.2 = -2.65$

$C_{p1G} := C_{pe1G} - 0.2 = -2.15$

$C_{p1H} := C_{pe1H} - 0.2 = -1.31$

$C_{p1I} := C_{pe1I} - 0.2 = -0.78$

$C_{p1J} := C_{pe1J} - (-0.3) = 0.33$

$C_{pF} := C_{peF} - (-0.3) = 0.32$

$C_{pG} := C_{peG} - (-0.3) = 0.32$

$C_{pH} := C_{peH} - (-0.3) = 0.32$

$C_{pI} := C_{peI} - 0.2 = -0.74$

$C_{pJ} := C_{peJ} - 0.2 = -0.74$

The pressure changes rapidly between positive and negative values on the windward face around a pitch angle of α'E1 = -5° to +45°, so both positive and negative values are given. For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face.

In this thesis we use the value from the most loaded area for the entire roof

Pressure

Design value of the net pressure

$$C_{p1.roof} := C_{pF} + C_{p1J} = 0.65$$

External wind pressure (Equation 5.1)

$$w_{1roof} := q_{p2} \cdot C_{p1.roof} = 0.759 \cdot \frac{kN}{m^2}$$

Suction

Design value ofthe net suction

$$C_{p1.roof.suction} := C_{p1F} + C_{p1I} = -3.43$$

External wind suction (Equation 5.1)

$$w_{1.suction} := q_{p2} \cdot C_{p1.roof.suction} = -4.003 \cdot \frac{kN}{m^2}$$

<u>Design values for wind load acting on the roof</u>

Pressure

<u>Horizontal wind load on the roof θ=0</u>

$$q_{wH1} := w_{10roof} \cdot \sin(\alpha) = 6.027 \times 10^{-3} \cdot \frac{kN}{m^2}$$

<u>Vertical wind load on the roof θ=0</u>

$$q_{wV1} := w_{10roof} \cdot \cos(\alpha) = 0.057 \cdot \frac{kN}{m^2}$$

Suction

<u>Horizontal wind load on the roof θ=0</u>

$$q_{wH1.suction} := w_{10.suction} \cdot \sin(\alpha) = -0.117 \cdot \frac{kN}{m^2}$$

<u>Vertical wind load on the roof θ=0</u>

$$q_{wV1.suction} := w_{10.suction} \cdot \cos(\alpha) = -1.115 \cdot \frac{kN}{m^2}$$

Unintended inclination - Massivträhandboken 2006 Chapter 2 page 66-68

The unfavourable effects of possible deviations in the geometry of the structure and the position of loads shall be taken into account in the analysis of members and structures.

The horizontal force due to inclination of each floor $F_{UI} = V \cdot n \cdot \alpha_{md}$

Systematic part of inclination angle $\alpha_0 := 0.003$

Load acting on the long side of the facade $\alpha_d := 0.012$

Number of supporting walls in the system loaded with the vertical loads

$n := 6$

Unintended inclinations $\alpha_{md} := \alpha_0 + \frac{\alpha_d}{\sqrt{n}} = 7.899 \times 10^{-3}$

Design value of the average vertical force on each floor on the underlying inclined walls

The horizontal force due to inclination of each floor $F_{UI} := V \cdot n \cdot \alpha_{md}$

Vertikal loads

Loads acting on the roof

Total windload acting on the roof

Unfavorable $q_{Wind} := q_{wV} \cdot A_{roof} = 18.687 \cdot kN$

Favorable $q_{Wind.suction} := q_{wV.suction} \cdot A_{roof} = -363.394 \cdot kN$

Total snowload acting on the roof $q_{Snow} := q_{snow} \cdot A_{roof} = 391.061 \cdot kN$

Selfweight of roof $G_{roof.tot} = 195.53 \cdot kN$

Loads acting one floor

Imposed load $q_{imp.one.floor} := q_{kimpGtot} \cdot A_{floor} = 810.246 \cdot kN$

Selfweight $G_{one.floor} = 355.799 \cdot kN$

Load combinations (Table B-3)

Floor 11

With variable main load

Wind leading variabel action

$$Q_{d11.wind} := \gamma_{d.3} \cdot 0.89 \cdot 1.35 \cdot G_{roof.tot} + \gamma_{d.3} \cdot 1.5 \cdot \left(q_{Wind} + \psi S_0 \cdot q_{Snow}\right) = 614.915 \cdot kN$$

Snow load leading variabel action

$$Q_{d11.Snow} := \gamma_{d.3} \cdot 0.89 \cdot 1.35 \cdot G_{roof.tot} + \gamma_{d.3} \cdot 1.5 \cdot \left(q_{Wind} \cdot \psi W_0 + q_{Snow}\right) = 829.93 \cdot kN$$

Without variable main load

$$Q_{d11.no} := \gamma_{d.3} \cdot 1.35 \cdot G_{roof} \cdot A_{roof} + \gamma_{d.3} \cdot 1.5\left(q_{Wind} \cdot \psi W_0 + \psi S_0 \cdot q_{Snow}\right) = 624.33 \cdot kN$$

Vertical load acting on the walls at floor 11

$$V_{d11} := \max\left(Q_{d11.wind}, Q_{d11.Snow}, Q_{d11.no}\right) = 829.93 \cdot kN$$

Floor 0-10

With the imposed load as a variable main load

$$Q_{dfl.0\_10} := \gamma_{d.3} \cdot 0.89 \cdot 1.35 \cdot G_{one.floor} + \gamma_{d.3} \cdot 1.5 \cdot \left(q_{imp.one.floor}\right) = 1.643 \times 10^{3} \cdot kN$$

Without variable main load

$$Q_{dfl.0\_10.no} := \gamma_{d.3} \cdot 1.35 \cdot G_{one.floor} + \gamma_{d.3} \cdot 1.5\left(\psi_0 \cdot q_{imp.one.floor}\right) = 1.331 \times 10^{3} \cdot kN$$

Vertical load acting on the walls at floor 0-10 from one floor

$$V_{d.0\_10} := \max\left(Q_{dfl.0\_10}, Q_{dfl.0\_10.no}\right) = 1.643 \times 10^{3} \cdot kN$$

The horizontal force due to inclination of each floor

$$\begin{pmatrix} F_{UI.12} \\ F_{UI.11} \\ F_{UI.10} \\ F_{UI.9} \\ F_{UI.8} \\ F_{UI.7} \\ F_{UI.6} \\ F_{UI.5} \\ F_{UI.4} \\ F_{UI.3} \\ F_{UI.2} \\ F_{UI.1} \\ F_{UI.0} \end{pmatrix} := \begin{pmatrix} V_{d11} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \\ V_{d.0\_10} \end{pmatrix} \cdot n \cdot \alpha_{md} = \begin{array}{|c|c|} \hline & 0 \\ \hline 0 & 39.334 \\ \hline 1 & 77.862 \\ \hline 2 & 77.862 \\ \hline 3 & 77.862 \\ \hline 4 & 77.862 \\ \hline 5 & 77.862 \\ \hline 6 & 77.862 \\ \hline 7 & 77.862 \\ \hline 8 & 77.862 \\ \hline 9 & 77.862 \\ \hline 10 & 77.862 \\ \hline 11 & 77.862 \\ \hline 12 & 77.862 \\ \hline \end{array} \cdot kN$$

The wind line loads acting on each floor at the long side of the facade

Reference height 1

$$q_{wH0} := q_{wind1} \cdot \frac{h}{2} = 2.386 \cdot \frac{kN}{m}$$

$$q_{wH1} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH2} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH3} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH4} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH5} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH6} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH7} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH8} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

$$q_{wH9} := q_{wind1} \cdot \frac{h}{2} \cdot 2 = 4.771 \cdot \frac{kN}{m}$$

Reference height 2

$$q_{wH10} := q_{wind2} \cdot \frac{h}{2} \cdot 2 = 5.089 \cdot \frac{kN}{m}$$

$$q_{wH11} := q_{wind2} \cdot \frac{h}{2} \cdot 2 = 5.089 \cdot \frac{kN}{m}$$

$$q_{wH12} := q_{wind2} \cdot \frac{h}{2} + \frac{q_{wH}}{d} \cdot \frac{A_{roof}}{2} = 2.58 \cdot \frac{kN}{m}$$

Total horizontal force from wind and unintended inclination acting on each floor

$$\begin{pmatrix} H_{12} \\ H_{11} \\ H_{10} \\ H_9 \\ H_8 \\ H_7 \\ H_6 \\ H_5 \\ H_4 \\ H_3 \\ H_2 \\ H_1 \\ H_0 \end{pmatrix} := \gamma_{d.3} \cdot 0.89 \cdot 1.35 \cdot \begin{pmatrix} F_{UI.12} \\ F_{UI.11} \\ F_{UI.10} \\ F_{UI.9} \\ F_{UI.8} \\ F_{UI.7} \\ F_{UI.6} \\ F_{UI.5} \\ F_{UI.4} \\ F_{UI.3} \\ F_{UI.2} \\ F_{UI.1} \\ F_{UI.0} \end{pmatrix} + \gamma_{d.3} \cdot 1.5 \left[\begin{pmatrix} q_{wH12} \\ q_{wH11} \\ q_{wH10} \\ q_{wH9} \\ q_{wH8} \\ q_{wH7} \\ q_{wH6} \\ q_{wH5} \\ q_{wH4} \\ q_{wH3} \\ q_{wH2} \\ q_{wH1} \\ q_{wH0} \end{pmatrix} \cdot d \right] = \begin{array}{|c|c|} \hline & 0 \\ \hline 0 & 153.536 \\ \hline 1 & 303.159 \\ \hline 2 & 303.159 \\ \hline 3 & 290.073 \\ \hline 4 & 290.073 \\ \hline 5 & 290.073 \\ \hline 6 & 290.073 \\ \hline 7 & 290.073 \\ \hline 8 & 290.073 \\ \hline 9 & 290.073 \\ \hline 10 & 290.073 \\ \hline 11 & 290.073 \\ \hline 12 & 191.812 \\ \hline \end{array} \cdot kN$$

$$\begin{pmatrix} H_{12no} \\ H_{11no} \\ H_{10no} \\ H_{9no} \\ H_{8no} \\ H_{7no} \\ H_{6no} \\ H_{5no} \\ H_{4no} \\ H_{3no} \\ H_{2no} \\ H_{1no} \\ H_{0no} \end{pmatrix} := \gamma_{d.3} \cdot 1.35 \cdot \begin{pmatrix} F_{UI.12} \\ F_{UI.11} \\ F_{UI.10} \\ F_{UI.9} \\ F_{UI.8} \\ F_{UI.7} \\ F_{UI.6} \\ F_{UI.5} \\ F_{UI.4} \\ F_{UI.3} \\ F_{UI.2} \\ F_{UI.1} \\ F_{UI.0} \end{pmatrix} + \gamma_{d.3} \cdot 1.5 \left[\begin{pmatrix} q_{wH12} \\ q_{wH11} \\ q_{wH10} \\ q_{wH9} \\ q_{wH8} \\ q_{wH7} \\ q_{wH6} \\ q_{wH5} \\ q_{wH4} \\ q_{wH3} \\ q_{wH2} \\ q_{wH1} \\ q_{wH0} \end{pmatrix} \cdot d \cdot \psi W_0 \right] = \begin{array}{|c|c|} \hline & 0 \\ \hline 0 & 84.983 \\ \hline 1 & 167.996 \\ \hline 2 & 167.996 \\ \hline 3 & 164.07 \\ \hline 4 & 164.07 \\ \hline 5 & 164.07 \\ \hline 6 & 164.07 \\ \hline 7 & 164.07 \\ \hline 8 & 164.07 \\ \hline 9 & 164.07 \\ \hline 10 & 164.07 \\ \hline 11 & 164.07 \\ \hline 12 & 134.591 \\ \hline \end{array} \cdot kN$$

Unfavorable loads

$$\begin{pmatrix} V_{i.stud.wall.1.fl12} \\ V_{i.stud.wall.1.fl11} \\ V_{i.stud.wall.1.fl10} \\ V_{i.stud.wall.1.fl9} \\ V_{i.stud.wall.1.fl8} \\ V_{i.stud.wall.1.fl7} \\ V_{i.stud.wall.1.fl6} \\ V_{i.stud.wall.1.fl5} \\ V_{i.stud.wall.1.fl4} \\ V_{i.stud.wall.1.fl3} \\ V_{i.stud.wall.1.fl2} \\ V_{i.stud.wall.1.fl1} \end{pmatrix} := 1.2m \cdot s_{stud} \cdot \begin{bmatrix} \frac{V_{d11}}{A_{roof}} \\ G_{floor} + \left(\alpha_{n10} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n9} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n8} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n7} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n6} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n5} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n4} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n3} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n2} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n1} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n0} \cdot q_{kimpGtot}\right) \end{bmatrix} + s_{stud} \cdot G_{outerwall} =$$

	0
0	2.614
1	3.012
2	3.012
3	2.832
4	2.742
5	2.688
6	2.652
7	2.626
8	2.607
9	2.592
10	2.58
11	2.57

$\cdot kN$

$$\begin{pmatrix} Q_{tot.fl.12} \\ Q_{tot.fl.11} \\ Q_{tot.fl.10} \\ Q_{tot.fl.9} \\ Q_{tot.fl.8} \\ Q_{tot.fl.7} \\ Q_{tot.fl.6} \\ Q_{tot.fl.5} \\ Q_{tot.fl.4} \\ Q_{tot.fl.3} \\ Q_{tot.fl.2} \\ Q_{tot.fl.1} \end{pmatrix} := 1.2m \cdot \begin{bmatrix} \frac{V_{d11}}{A_{roof}} \\ G_{floor} + \left(\alpha_{n10} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n9} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n8} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n7} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n6} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n5} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n4} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n3} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n2} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n1} \cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n0} \cdot q_{kimpGtot}\right) \end{bmatrix} + G_{outerwall} =$$

	0
0	4.356
1	5.02
2	5.02
3	4.72
4	4.57
5	4.48
6	4.42
7	4.377
8	4.345
9	4.32
10	4.3
11	4.284

$\cdot \frac{kN}{m}$

$$\begin{pmatrix} V_{i.stud.wall.3.fl12} \\ V_{i.stud.wall.3.fl11} \\ V_{i.stud.wall.3.fl10} \\ V_{i.stud.wall.3.fl9} \\ V_{i.stud.wall.3.fl8} \\ V_{i.stud.wall.3.fl7} \\ V_{i.stud.wall.3.fl6} \\ V_{i.stud.wall.3.fl5} \\ V_{i.stud.wall.3.fl4} \\ V_{i.stud.wall.3.fl3} \\ V_{i.stud.wall.3.fl2} \\ V_{i.stud.wall.3.fl1} \end{pmatrix} := 2\cdot 1.2\text{m}\cdot s_{stud}\cdot \begin{bmatrix} 0 \\ G_{floor} + \left(\alpha_{n10}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n9}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n8}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n7}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n6}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n5}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n4}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n3}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n2}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n1}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n0}\cdot q_{kimpGtot}\right) \end{bmatrix} + s_{stud}\cdot G_{innerwall} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 1.02 \\ \hline 1 & 5.484 \\ \hline 2 & 5.484 \\ \hline 3 & 5.124 \\ \hline 4 & 4.944 \\ \hline 5 & 4.836 \\ \hline 6 & 4.764 \\ \hline 7 & 4.713 \\ \hline 8 & 4.674 \\ \hline 9 & 4.644 \\ \hline 10 & 4.62 \\ \hline 11 & 4.6 \\ \hline \end{array} \cdot \text{kN}$$

$$\begin{pmatrix} Q_{tot.wall3.fl.12} \\ Q_{tot.wall3.fl.11} \\ Q_{tot.wall3.fl.10} \\ Q_{tot.wall3.fl.9} \\ Q_{tot.wall3.fl.8} \\ Q_{tot.wall3.fl.7} \\ Q_{tot.wall3.fl.6} \\ Q_{tot.wall3.fl.5} \\ Q_{tot.wall3.fl.4} \\ Q_{tot.wall3.fl.3} \\ Q_{tot.wall3.fl.2} \\ Q_{tot.wall3.fl.1} \end{pmatrix} := 1.2\text{m}\cdot \begin{bmatrix} \dfrac{V_{d11}}{A_{roof}} \\ G_{floor} + \left(\alpha_{n10}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n9}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n8}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n7}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n6}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n5}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n4}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n3}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n2}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n1}\cdot q_{kimpGtot}\right) \\ G_{floor} + \left(\alpha_{n0}\cdot q_{kimpGtot}\right) \end{bmatrix} + G_{innerwall} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 4.756 \\ \hline 1 & 5.42 \\ \hline 2 & 5.42 \\ \hline 3 & 5.12 \\ \hline 4 & 4.97 \\ \hline 5 & 4.88 \\ \hline 6 & 4.82 \\ \hline 7 & 4.777 \\ \hline 8 & 4.745 \\ \hline 9 & 4.72 \\ \hline 10 & 4.7 \\ \hline 11 & 4.684 \\ \hline \end{array} \cdot \frac{\text{kN}}{\text{m}}$$

C Load distribution

Stiffnes of walls

ULS

Shear capacity of inner walls $F_{pd.plywood} := 700N$

Shear capacity of outer walls $F_{pd.OSB} := 700N$

Relationship between the outer and inner walls regarding the stiffnes

Stiffnes outer walls =1 $S_{outerwall} := 1$

$$\frac{F_{pd.plywood}}{F_{pd.OSB}} = 1$$

Stiffnes inner walls =1.757

The inner walls are 1.757 times stiffer then the outerwalls $S_{innerwall} := 1$

SLS

Inner walls

Mean modulus of rigidity for plywood $G_{plywood} := 500$

Thickness of plywood $t_{plywood} := 15$

$h_{plywood} := 2.975$

$s_{plywood} := 0.3$

$b_{plywood} := 1.2$

$H_{d.plywood} := 100$

$\rho_{meanC24} := 420$

$\rho_{meanplywood} := 460$

$d_{fastener} := 2.3$

$\kappa_{plywood} := 1$

$$K_{ser.plywood} := \left(\sqrt{\rho_{meanC24} \cdot \rho_{meanplywood}}\right)^{1.5} \cdot \frac{d_{fastener}^{0.8}}{30} = 598.09$$

$$k_{plywood} := \kappa_{plywood} \cdot K_{ser.plywood} = 598.09$$

$$u_{plywood} := 4.5 \cdot \frac{s_{plywood}}{b_{plywood}} \cdot \frac{H_{d.plywood}}{k_{plywood}} + \frac{H_{d.plywood} \cdot h_{plywood}}{G_{plywood} \cdot b_{plywood} \cdot t_{plywood}} = 0.221$$

Outer walls

Mean modulus of rigidity for OSB $G_{OSB} := 1080$

Thickness of OSB $t_{OSB} := 11$

$h_{osb} := 2.975$

$s_{OSB} := 0.3$

$b_{OSB} := 1.2$

$H_{d.OSB} := 100$

$\rho_{meanC24} := 420$

$\rho_{meanOSB} := 550$

$d_{fastener} := 2.3$

$\kappa_{OSB} := 1$

$$K_{ser.OSB} := \left(\sqrt{\rho_{meanC24} \cdot \rho_{meanOSB}}\right)^{1.5} \cdot \frac{d_{fastener}^{\;0.8}}{30} = 683.864$$

$$k_{OSB} := \kappa_{OSB} \cdot K_{ser.OSB} = 683.864$$

$$u_{OSB} := 4.5 \cdot \frac{s_{OSB}}{b_{OSB}} \cdot \frac{H_{d.OSB}}{k_{OSB}} + \frac{H_{d.OSB} \cdot h_{osb}}{G_{OSB} \cdot b_{OSB} \cdot t_{OSB}} = 0.185$$

Relationship between the outer and inner walls regarding the stiffnes

Stiffnes inner walls =1 $S_{innerwall.SLS} := 1$

$\frac{u_{plywood}}{u_{OSB}} = 1.193$ $\frac{u_{OSB}}{u_{plywood}} = 0.838$

Stiffnes outer walls =1.193

The inner walls are 1.193 times stiffer then the outerwalls

$$S_{outerwall.SLS} := \frac{u_{plywood}}{u_{OSB}} = 1.193$$

Load distribution

Relationsship between inner and outer walls regarding the stiffnes

Thicknes of the walls $T_{outerwall} := 0.242m$ $T_{innerwall} := 0.175m$

Distance between the walls and origo in x-direction

Wall 1 $a_{i.1} := \frac{T_{outerwall}}{2} = 0.121\,m$

Wall 2 $a_{i.2} := 7.869m - \frac{T_{innerwall}}{2} = 7.781\,m$

Wall 3 $a_{i.3} := 11.8395m + \frac{T_{innerwall}}{2} = 11.927\,m$

Wall 4 $a_{i.4} := 17.05m + \frac{T_{innerwall}}{2} = 17.137\,m$

Wall 5 $a_{i.5} := 7.869m + 5.930m + 6.114m - \frac{T_{innerwall}}{2} = 19.826\,m$

Wall 6 $a_{i.6} := d - \frac{T_{outerwall}}{2} = 27.338\,m$

Wall 7 $a_{i.7} := \frac{d}{2} = 13.729\,m$

Wall 8 $a_{i.8} := 7.869m + \frac{(5.930m + 6.114m)}{2} = 13.891\,m$

Wall 9 $a_{i.9} := \frac{d}{2} = 13.729\,m$

Distance between the walls and origo in y-direction

Wall 1 $b_{i.1} := \frac{b}{2} = 5.902\,m$

Wall 2 $b_{i.2} := \frac{5.488m}{2} + T_{outerwall} = 2.986\,m$

Wall 3 $b_{i.3} := 5.488m + T_{outerwall} - \frac{5.919m}{2} = 2.771\,m$

Wall 4 $b_{i.4} := 5.488m + T_{outerwall} - \frac{5.919m}{2} = 2.771\,m$

Wall 5 $b_{i.5} := \frac{5.488m}{2} + T_{outerwall} = 2.986\,m$

Wall 6 $b_{i.6} := \frac{b}{2} = 5.902\,m$

Wall 7 $b_{i.7} := \frac{T_{outerwall}}{2} = 0.121\,m$

Wall 8 $b_{i.8} := 5.488m + T_{outerwall} - \frac{T_{innerwall}}{2} = 5.643\,m$

Wall 9 $b_{i.9} := b - \frac{T_{outerwall}}{2} = 11.682\,m$

Stiffness of the walls

$$\begin{pmatrix} S_{xi.1} \\ S_{xi.2} \\ S_{xi.3} \\ S_{xi.4} \\ S_{xi.5} \\ S_{xi.6} \\ S_{xi.7} \\ S_{xi.8} \\ S_{xi.9} \end{pmatrix} := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1 \\ 1 \\ 1 \end{pmatrix} \qquad \begin{pmatrix} S_{yi.1} \\ S_{yi.2} \\ S_{yi.3} \\ S_{yi.4} \\ S_{yi.5} \\ S_{yi.6} \\ S_{yi.7} \\ S_{yi.8} \\ S_{yi.9} \end{pmatrix} := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

$$S_{xi} := \begin{pmatrix} S_{xi.1} \\ S_{xi.2} \\ S_{xi.3} \\ S_{xi.4} \\ S_{xi.5} \\ S_{xi.6} \\ S_{xi.7} \\ S_{xi.8} \\ S_{xi.9} \end{pmatrix} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 1 \\ 1 \\ 1 \end{pmatrix} \qquad \sum S_{xi} = 3 \qquad S_{yi} := \begin{pmatrix} S_{yi.1} \\ S_{yi.2} \\ S_{yi.3} \\ S_{yi.4} \\ S_{yi.5} \\ S_{yi.6} \\ S_{yi.7} \\ S_{yi.8} \\ S_{yi.9} \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 0 \\ 0 \\ 0 \end{pmatrix} \qquad \sum S_{yi} = 6$$

$$a_i := \begin{pmatrix} a_{i.1} \\ a_{i.2} \\ a_{i.3} \\ a_{i.4} \\ a_{i.5} \\ a_{i.6} \\ a_{i.7} \\ a_{i.8} \\ a_{i.9} \end{pmatrix} = \begin{pmatrix} 0.121 \\ 7.781 \\ 11.927 \\ 17.137 \\ 19.826 \\ 27.338 \\ 13.729 \\ 13.891 \\ 13.729 \end{pmatrix} m \qquad b_i := \begin{pmatrix} b_{i.1} \\ b_{i.2} \\ b_{i.3} \\ b_{i.4} \\ b_{i.5} \\ b_{i.6} \\ b_{i.7} \\ b_{i.8} \\ b_{i.9} \end{pmatrix} = \begin{pmatrix} 5.902 \\ 2.986 \\ 2.771 \\ 2.771 \\ 2.986 \\ 5.902 \\ 0.121 \\ 5.643 \\ 11.682 \end{pmatrix} m$$

Coordinates of the rotation center

$$x_i := \frac{a_i \cdot S_{yi}}{\sum S_{yi}} = 14.022\,m \qquad y_i := \frac{b_i \cdot S_{xi}}{\sum S_{xi}} = 5.815\,m$$

Horizontal load acting on the long side of the facade

$H_x := 0$

$$H_y := \begin{pmatrix} H_{12} \\ H_{11} \\ H_{10} \\ H_9 \\ H_8 \\ H_7 \\ H_6 \\ H_5 \\ H_4 \\ H_3 \\ H_2 \\ H_1 \\ H_0 \end{pmatrix} =$$

	0
0	153.536
1	303.159
2	303.159
3	290.073
4	290.073
5	290.073
6	290.073
7	290.073
8	290.073
9	290.073
10	290.073
11	290.073
12	191.812

$\cdot kN$

$e_x := \frac{d}{2} - x_i = -0.292\,m$ $\qquad$ $e_y := \frac{b}{2} - y_i = 0.086\,m$

$$\begin{pmatrix} T_{12} \\ T_{11} \\ T_{10} \\ T_9 \\ T_8 \\ T_7 \\ T_6 \\ T_5 \\ T_4 \\ T_3 \\ T_2 \\ T_1 \\ T_0 \end{pmatrix} := H_y \cdot e_x + H_x \cdot e_y =$$

	0
0	-44.871
1	-88.598
2	-88.598
3	-84.774
4	-84.774
5	-84.774
6	-84.774
7	-84.774
8	-84.774
9	-84.774
10	-84.774
11	-84.774
12	-56.057

$\cdot kN \cdot m$

$$\begin{pmatrix} x_{i.1} \\ x_{i.2} \\ x_{i.3} \\ x_{i.4} \\ x_{i.5} \\ x_{i.6} \\ x_{i.7} \\ x_{i.8} \\ x_{i.9} \end{pmatrix} := \begin{bmatrix} -\left(x_i - \frac{T_{outerwall}}{2}\right) \\ -\left(x_i - a_{i.2}\right) \\ -\left(x_i - a_{i.3}\right) \\ \left(a_{i.4} - x_i\right) \\ \left(a_{i.5} - x_i\right) \\ \left(a_{i.6} - x_i\right) \\ \left(\frac{d}{2} - x_i\right) \\ \left(a_{i.8} - x_i\right) \\ \left(\frac{d}{2} - x_i\right) \end{bmatrix} = \begin{pmatrix} -13.901 \\ -6.24 \\ -2.095 \\ 3.116 \\ 5.804 \\ 13.316 \\ -0.292 \\ -0.131 \\ -0.292 \end{pmatrix} m \qquad x_{i.tot} := \begin{pmatrix} x_{i.1} \\ x_{i.2} \\ x_{i.3} \\ x_{i.4} \\ x_{i.5} \\ x_{i.6} \\ x_{i.7} \\ x_{i.8} \\ x_{i.9} \end{pmatrix}$$

$$\begin{pmatrix} y_{i.1} \\ y_{i.2} \\ y_{i.3} \\ y_{i.4} \\ y_{i.5} \\ y_{i.6} \\ y_{i.7} \\ y_{i.8} \\ y_{i.9} \end{pmatrix} := \begin{bmatrix} \left(\frac{b}{2} - y_i\right) \\ -\left(y_i - b_{i.2}\right) \\ \left(b_{i.3} - y_i\right) \\ \left(b_{i.4} - y_i\right) \\ -\left(y_i - b_{i.5}\right) \\ \left(\frac{b}{2} - y_i\right) \\ -\left(y_i - b_{i.7}\right) \\ -\left(y_i - b_{i.8}\right) \\ \left(b_{i.9} - y_i\right) \end{bmatrix} = \begin{pmatrix} 0.086 \\ -2.829 \\ -3.045 \\ -3.045 \\ -2.829 \\ 0.086 \\ -5.694 \\ -0.173 \\ 5.867 \end{pmatrix} m \qquad y_{i.tot} := \begin{pmatrix} y_{i.1} \\ y_{i.2} \\ y_{i.3} \\ y_{i.4} \\ y_{i.5} \\ y_{i.6} \\ y_{i.7} \\ y_{i.8} \\ y_{i.9} \end{pmatrix}$$

Torsional stiffnes $S_T := \sum \left(S_{yi} \cdot x_i^2\right) + \sum \left(S_{xi} \cdot y_i^2\right) = 1.281 \times 10^3 \, m^2$

$$H_{i.Bx} := \frac{S_{xi}}{\sum S_{xi}} \cdot H_x = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{pmatrix}$$

$$H_{i.By.12} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_{12} = \begin{pmatrix} 25.589 \\ 25.589 \\ 25.589 \\ 25.589 \\ 25.589 \\ 25.589 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.12} = 153.536 \cdot kN$$

$$H_{12} = 153.536 \cdot kN$$

$$H_{i.By.11} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_{11} = \begin{pmatrix} 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.11} = 303.159 \cdot kN$$

$$H_{11} = 303.159 \cdot kN$$

$$H_{i.By.10} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_{10} = \begin{pmatrix} 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 50.526 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.10} = 303.159 \cdot kN$$

$$H_{10} = 303.159 \cdot kN$$

$$H_{i.By.9} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_{9} = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.9} = 290.073 \cdot kN$$

$$H_{9} = 290.073 \cdot kN$$

$$H_{i.By.8} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_8 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.8} = 290.073 \cdot kN$$

$$H_8 = 290.073 \cdot kN$$

$$H_{i.By.7} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_7 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.7} = 290.073 \cdot kN$$

$$H_7 = 290.073 \cdot kN$$

$$H_{i.By.6} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_6 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.6} = 290.073 \cdot kN$$

$$H_6 = 290.073 \cdot kN$$

$$H_{i.By.5} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_5 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.By.5} = 290.073 \cdot kN$$

$$H_5 = 290.073 \cdot kN$$

$$H_{i.By.4} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_4 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.By.4} = 290.073 \cdot kN$$

$$H_4 = 290.073 \cdot kN$$

$$H_{i.By.3} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_3 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.By.3} = 290.073 \cdot kN$$

$$H_3 = 290.073 \cdot kN$$

$$H_{i.By.2} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_2 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.By.2} = 290.073 \cdot kN$$

$$H_2 = 290.073 \cdot kN$$

$$H_{i.By.1} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_1 = \begin{pmatrix} 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 48.345 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.By.1} = 290.073 \cdot kN$$

$$H_1 = 290.073 \cdot kN$$

$$H_{i.By.0} := \frac{S_{yi}}{\sum S_{yi}} \cdot H_0 = \begin{pmatrix} 31.969 \\ 31.969 \\ 31.969 \\ 31.969 \\ 31.969 \\ 31.969 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.By.0} = 191.812 \cdot kN$$

$$H_0 = 191.812 \cdot kN$$

$$H_{i.cy.12} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_{12} = \begin{pmatrix} 0.487 \\ 0.219 \\ 0.073 \\ -0.109 \\ -0.203 \\ -0.466 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.12} = 0 \cdot kN$$

$$H_{i.cy.11} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_{11} = \begin{pmatrix} 0.961 \\ 0.432 \\ 0.145 \\ -0.215 \\ -0.401 \\ -0.921 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.11} = 1.137 \times 10^{-15} \cdot kN$$

$$H_{i.cy.10} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_{10} = \begin{pmatrix} 0.961 \\ 0.432 \\ 0.145 \\ -0.215 \\ -0.401 \\ -0.921 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.10} = 1.137 \times 10^{-15} \cdot kN$$

$$H_{i.cy.9} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_9 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.9} = 0 \cdot kN$$

$$H_{i.cy.8} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_8 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.8} = 0 \cdot kN$$

$$H_{i.cy.7} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_7 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.7} = 0 \cdot kN$$

$$H_{i.cy.6} := \frac{\overrightarrow{\left(S_{yi} \cdot x_{i.tot}\right)}}{S_T} \cdot T_6 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.6} = 0 \cdot kN$$

$$H_{i.cy.5} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_5 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.5} = 0 \cdot kN$$

$$H_{i.cy.4} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_4 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.4} = 0 \cdot kN$$

$$H_{i.cy.3} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_3 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.3} = 0 \cdot kN$$

$$H_{i.cy.2} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_2 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN \qquad \sum H_{i.cy.2} = 0 \cdot kN$$

$$H_{i.cy.1} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_1 = \begin{pmatrix} 0.92 \\ 0.413 \\ 0.139 \\ -0.206 \\ -0.384 \\ -0.881 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.cy.1} = 0 \cdot kN$$

$$H_{i.cy.0} := \overrightarrow{\frac{\left(S_{yi} \cdot x_{i.tot}\right)}{S_T}} \cdot T_0 = \begin{pmatrix} 0.608 \\ 0.273 \\ 0.092 \\ -0.136 \\ -0.254 \\ -0.583 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{i.cy.0} = 0 \cdot kN$$

$$H_{iy.12} := H_{i.By.12} + H_{i.cy.12} = \begin{pmatrix} 26.076 \\ 25.808 \\ 25.663 \\ 25.48 \\ 25.386 \\ 25.123 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.12} = 153.536 \cdot kN$$

$$H_{11} = 303.159 \cdot kN$$

$$H_{iy.11} := H_{i.By.11} + H_{i.cy.11} = \begin{pmatrix} 51.488 \\ 50.958 \\ 50.671 \\ 50.311 \\ 50.125 \\ 49.606 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.11} = 303.159 \cdot kN$$

$$H_{11} = 303.159 \cdot kN$$

$$H_{iy.10} := H_{i.By.10} + H_{i.cy.10} = \begin{pmatrix} 51.488 \\ 50.958 \\ 50.671 \\ 50.311 \\ 50.125 \\ 49.606 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.10} = 303.159 \cdot kN$$

$$H_{10} = 303.159 \cdot kN$$

$$H_{iy.9} := H_{i.By.9} + H_{i.cy.9} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.9} = 290.073 \cdot kN$$

$$H_{9} = 290.073 \cdot kN$$

$$H_{iy.8} := H_{i.By.8} + H_{i.cy.8} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.8} = 290.073 \cdot kN$$

$$H_{8} = 290.073 \cdot kN$$

$$H_{iy.7} := H_{i.By.7} + H_{i.cy.7} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.7} = 290.073 \cdot kN$$

$$H_{7} = 290.073 \cdot kN$$

$$H_{iy.6} := H_{i.By.6} + H_{i.cy.6} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.6} = 290.073 \cdot kN$$

$$H_6 = 290.073 \cdot kN$$

$$H_{iy.5} := H_{i.By.5} + H_{i.cy.5} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.5} = 290.073 \cdot kN$$

$$H_5 = 290.073 \cdot kN$$

$$H_{iy.4} := H_{i.By.4} + H_{i.cy.4} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.4} = 290.073 \cdot kN$$

$$H_4 = 290.073 \cdot kN$$

$$H_{iy.3} := H_{i.By.3} + H_{i.cy.3} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.3} = 290.073 \cdot kN$$

$$H_3 = 290.073 \cdot kN$$

$$H_{iy.2} := H_{i.By.2} + H_{i.cy.2} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.2} = 290.073 \cdot kN$$

$$H_2 = 290.073 \cdot kN$$

$$H_{iy.1} := H_{i.By.1} + H_{i.cy.1} = \begin{pmatrix} 49.265 \\ 48.758 \\ 48.484 \\ 48.139 \\ 47.961 \\ 47.464 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.1} = 290.073 \cdot kN$$

$$H_1 = 290.073 \cdot kN$$

$$H_{iy.0} := H_{i.By.0} + H_{i.cy.0} = \begin{pmatrix} 32.577 \\ 32.242 \\ 32.06 \\ 31.832 \\ 31.715 \\ 31.386 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot kN$$

$$\sum H_{iy.0} = 191.812 \cdot kN$$

$$H_0 = 191.812 \cdot kN$$

D Tilting

Excentricity of the resulting force that corresponds to the tilting moment

$$e_{tilt} := \frac{b}{6} = 1.967\,m$$

Equivalent higt of distributed wind load

$$h_{wind.ref1} := \frac{h_1}{2} = 13.729\,m$$

$$h_{wind.ref2} := \frac{h_2 - h_1}{2} + h_1 = 31.73\,m$$

Wind load acting on the facade

$$q_{wind.fac.ref1} := w_{e10.1D} \cdot d = 33.073 \cdot \frac{kN}{m}$$

$$q_{wind.fac.ref2} := w_{e10.2D} \cdot d = 35.251 \cdot \frac{kN}{m}$$

Wind load acting on the roof

Pressure

$$q_{wind.roofp.v} := q_{wV} \cdot d = 1.575 \cdot \frac{kN}{m}$$

$$q_{wind.roofp.h} := q_{wH} \cdot d = 0.165 \cdot \frac{kN}{m}$$

Suction

$$q_{wind.roofs.v} := q_{wV.suction} \cdot d = -30.62 \cdot \frac{kN}{m}$$

$$q_{wind.roofs.h} := q_{wH.suction} \cdot d = -3.218 \cdot \frac{kN}{m}$$

Self weights

Roof $q_{roof} := G_{roof} \cdot d = 16.475 \cdot \frac{kN}{m}$

Floor $q_{floor} := G_{floor} \cdot d = 16.475 \cdot \frac{kN}{m}$

Interior walls $q_{inwall} := G_{innerwall} = 1.7 \cdot \frac{kN}{m}$

Exterior walls $q_{exwall} := G_{outerwall} = 1.3 \cdot \frac{kN}{m}$

Counteracting moment

$$M_C := \left(q_{roof} + q_{floor} \cdot 11 + q_{inwall} \cdot 12 + q_{exwall} \cdot 12\right) \cdot b \cdot e_{tilt} = 5.426 \times 10^3 \cdot kN \cdot m$$

Driving moment

$$M_D := q_{wind.fac.ref1} \cdot h_1 \cdot h_{wind.ref1} + q_{wind.fac.ref2} \cdot h_2 \cdot h_{wind.ref2} = 5.273 \times 10^4 \cdot kN \cdot m$$

Safety against tilting

For the building to be safe against tilting the foundation need to weigh at least:

$$q_{fund} := \frac{M_D - M_C}{b \cdot e_{tilt} \cdot d} = 74.203 \cdot \frac{kN}{m^2}$$

E Horizontal capacity of wall 1

Loads acting on wall 1

Total load from self weight from the roof acting on one stud in wall 1

Load acting on one stud $G_{wall.1.stud} := G_{outerwall} \cdot s_{stud} = 0.78 \cdot kN$

Load acting on one stud (Load favorable Table B-3,Equation 6.10a)

$V_{i.stud.wall.1} := 1kN$

Geometry of wall 1

Width of boards $L_{plates} := 1.2m$

Interior door width $b_{innerdoor} := 1.010m$

Full board width $b_{full} := L_{plates} = 1.2\,m$

Distance under window $h_{under.wall.1} := 0.643m$

Distance over window $h_{over.wall.1} := 0.888m$

Width of window one $b_{window.1.wall.1} := 1.4m$

Width of window two $b_{window.2.wall.1} := 0.5m$

Center spacing between joists $s_{studs} := 0.6m$

Number of floors $n_{fl12} := 12$

Total length of wall 1 $L_{wall.1} = 11.803\,m$

Anchorage and joint properties

Shear resistance of a junction unit (Table 7.2) $F_{knut} := 700N$

Plate stud anchor plastic capacity (Table 7.1)

$F_{p.OSB} := 2 \cdot 700N = 1.4 \cdot kN$

The wall are divided into three different parts

Wall part 1 $L_{wall.1.1} := 3.878m$

Wall part 2 $L_{wall.1.2} := 3.323m - b_{window.1.wall.1} - b_{window.2.wall.1} = 1.423\,m$

Wall part 3 $L_{wall.1.3} := 4.602m$

Wall part 1

Length of the wall part 1 $L_{Dim.wall.1.1} := L_{wall.1.1} = 3.878\,m$

Number of studs in the wall $n_{studs.wall.3.1} := \dfrac{L_{Dim.wall.1.1}}{s_{studs}} = 6.463$

Number of node connections which are active for the transfer of shear force

$n_{knut.wall.1.1} := 5$

Wall part 2

Length of the wall part 2 $L_{Dim.wall.1.2} := L_{wall.1.2} = 1.423\,m$

Number of studs in the wall $n_{studs.wall.1.2} := \dfrac{L_{Dim.wall.1.2}}{s_{studs}} = 2.372$

Number of node connections which are active for the transfer of shear force

$n_{knut.wall.1.2} := 4$

Wall part 3

Length of the wall part 3 $L_{Dim.wall.1.3} := L_{wall.1.3} = 4.602\,m$

Number of studs in the wall $n_{studs.wall.1.3} := \dfrac{L_{Dim.wall.1.3}}{s_{studs}} = 7.67$

Number of node connections which are active for the transfer of shear force

$n_{knut.wall.1.3} := 6$

Horizontal load-bearing capacity

Floor 12

Centre distance between joints $s_{s.wall.1.fl12} := 0.15m$

Load acting on one stud $V_{i.stud.wall.1.fl12} = 2.614 \cdot kN$

Plate stud anchor plastic capacity $F_{p.wall.1.fl12} := 1.4kN$

Plastic shear flow force per unit length (Formula 4.1)

$$f_{p.wall.1.fl12} := \frac{F_{p.wall.1.fl12}}{s_{s.wall.1.fl12}} = 9.333 \cdot \frac{kN}{m}$$

Plastic wall capacity $f_{p.wall.1.fl12} \cdot h_{tot} = 336 \cdot kN$

Plastic capacity over door $f_{p.wall.1.fl12} \cdot n_{fl12} \cdot h_{over.wall.1} = 99.456 \cdot kN$

Part 1

Vertical load $V_{i.wall.1.1.fl12} := 1 \cdot Q_{tot.fl.12} \cdot L_{Dim.wall.1.1} = 16.893 \cdot kN$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.1.fl12} := \min\left[h_{tot.12} \cdot \left(1 - \frac{V_{i.wall.1.1.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right), L_{Dim.wall.1.1}\right] = 3.878\,m$$

$L_{Dim.wall.1.1} = 3.878\,m$ entains

$$l_{2.wall.1.1.fl12} := \max\left[\left(L_{Dim.wall.1.1} - l_{1.wall.1.1.fl12}\right), 0\right] = 0\,m$$

$$l_{3.wall.1.1.fl12} := b_{window.1.wall.1} = 1.4\,m$$

Number of studs in the wall $n_{studs.wall.1.1.fl12} := \frac{l_{1.wall.1.1.fl12}}{s_{studs}} = 6.463$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.1.fl12} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 6.463 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$V_{ekv.wall.1.1.fl12} := \sum \left(\frac{l_{1.wall.1.1.fl12} - x_{i.wall.1.1.fl12} \cdot s_{studs}}{l_{1.wall.1.1.fl12}} \cdot V_{i.stud.wall.1.fl12}\right) = 9.804 \cdot kN$$

Control of main case $h_{under} > \eta_j(l_1 + l_2)$ entains main case 2

Dimensionless load factor (Formula 6.5) $\eta_{j.wall.1.1.fl12} := \dfrac{h_{tot.12}}{h_{tot}} = 0.992$

$h_{under.wall.1} = 0.643\,m$ $\eta_{j.wall.1.1.fl12} \cdot (l_{1.wall.1.1.fl12} + l_{2.wall.1.1.fl12}) = 3.846\,m$

Okej with main case 1

Factor for reduction of the shear flow in the boards (Formula 6.14)

$$\lambda_{wall.1.1.fl12.a} := 1$$

$$\lambda_{wall.1.1.fl12.b} := \left[\frac{h}{l_{3.wall.1.1.fl12} \cdot h_{under.wall.1}} \left[\left[\sqrt{1 - \left(1 - \frac{1}{n_{fl12}}\right)^2} \cdots + -\frac{l_{1.wall.1.1.fl12}}{2 \cdot h_{tot.12}} - \frac{V_{ekv.wall.1.1.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right] \cdot l_{1.wall.1.1.fl12} \cdots + \frac{n_{knut.wall.1.1} \cdot F_{knut}}{f_{p.wall.1.fl12}}\right]\right]$$

$$\lambda_{wall.1.1.fl12.c} := \left[\frac{2 \cdot h}{l_{3.wall.1.1.fl12} \cdot (h - h_{under.wall.1})} \left[\left(\frac{l_{1.wall.1.1.fl12}}{2 \cdot h_{tot.12}} \cdots + \frac{V_{ekv.wall.1.1.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right) \cdot l_{1.wall.1.1.fl12} \cdots + l_{2.wall.1.1.fl12}\right]\right]$$

$$\lambda_{wall.1.1.fl12.d} := \left(\frac{b_{full}}{l_{3.wall.1.1.fl12}}\right)$$

$$\lambda_{wall.1.1.fl12} := \min(\lambda_{wall.1.1.fl12.a}, \lambda_{wall.1.1.fl12.b}, \lambda_{wall.1.1.fl12.c}, \lambda_{wall.1.1.fl12.d}) = 0.592$$

The effective length of transferring shear forces (Formula 6.13)

$$l_{eff.wall.1.1.fl12} := \min\left[\left[\left(\frac{l_{1.wall.1.1.fl12}}{2h_{tot.12}} \cdots + \frac{V_{ekv.wall.1.1.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right) \cdot l_{1.wall.1.1.fl12} \cdots + l_{2.wall.1.1.fl12} \cdots + \frac{h_{under.wall.1}}{h} \cdot \lambda_{wall.1.1.fl12} \cdot l_{3.wall.1.1.fl12}\right], \left(l_{1.wall.1.1.fl12} \cdots + l_{2.wall.1.1.fl12}\right)\right] = 0.504\,m$$

Horizontal load capacity of wall (Formula 4.2)

$$H_{1.wall.1.1.fl12} := f_{p.wall.1.fl12} \cdot l_{eff.wall.1.1.fl12} = 4.702 \cdot kN$$

Reaction forces l.2=0 (Formula 6.15 and 6.16)

$$R_{N.wall.1.1.fl12} := f_{p.wall.1.fl12} \cdot (l_{1.wall.1.1.fl12} - \lambda_{wall.1.1.fl12} \cdot n_{fl12} \cdot h_{under.wall.1}) \cdots + 1 \cdot 8 \cdot V_{i.stud.wall.1} = 1.575 \cdot kN$$

$$R_{opening.wall.1.1.fl12} := f_{p.wall.1.fl12} \cdot \lambda_{wall.1.1.fl12} \cdot n_{fl12} \cdot h_{under.wall.1} = 42.62 \cdot kN$$

Part 2

Vertical load $V_{i.wall.1.2.fl12} := 1 \cdot Q_{tot.fl.12} \cdot L_{Dim.wall.1.2} = 6.199 \cdot kN$

Maximum shear force to be transmitted through the consoles

$$V_{max.wall.1.2.fl12} := f_{p.wall.1.fl12} \cdot n_{fl12} \cdot h_{over.wall.1} = 99.456 \cdot kN$$

The resultant forces acting on the two consoles

$$V_{\_1.wall1.2.fl12} := 1 \cdot V_{i.stud.wall.1.fl12} = 2.614 \cdot kN$$

$$V_{\_2.wall1.2.fl12} := V_{max.wall.1.2.fl12} - V_{\_1.wall1.2.fl12} = 96.842 \cdot kN$$

$$V_{0.wall.1.2.fl12} := R_{opening.wall.1.1.fl12} + 1 \cdot V_{i.stud.wall.1.fl12} = 45.233 \cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.2.fl12} := \min\left[h_{tot.12} \cdot \left(1 - \frac{V_{i.wall.1.2.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right), L_{Dim.wall.1.2}\right] = 1.423\,m$$

$l_{1.wall.1.2.fl12} = 1.423\,m$ entains

$$l_{2.wall.1.2.fl12} := \max\left[\left(L_{Dim.wall.1.2} - l_{1.wall.1.2.fl12}\right), 0\right] = 0\,m$$

$$l_{3.wall.1.2.fl12} := b_{window.2.wall.1} = 0.5\,m$$

Number of studs in the wall $n_{studs.wall.1.2.fl12} := \dfrac{l_{1.wall.1.2.fl12}}{s_{studs}} = 2.372$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.2.fl12} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 2.372 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$\begin{aligned} V_{ekv.wall.1.2.fl12} := & \left(\frac{l_{1.wall.1.2.fl12} + s_{stud} \cdot 2}{l_{1.wall.1.2.fl12}} \cdot V_{\_2.wall1.2.fl12}\right) \cdots \\ & + \left(\frac{l_{1.wall.1.2.fl12} + s_{stud} \cdot 1}{l_{1.wall.1.2.fl12}} \cdot V_{\_1.wall1.2.fl12}\right) \cdots \\ & + \sum \left(\frac{l_{1.wall.1.2.fl12} - x_{i.wall.1.2.fl12} \cdot s_{studs}}{l_{1.wall.1.2.fl12}} \cdot V_{i.stud.wall.1.fl12}\right) \end{aligned} = 186.759 \cdot kN$$

Control of main case $h_{under} > \eta_j\left(l_1 + l_2\right)$ entains main case 2

Dimensionless load factor (Formula 6.5) $\eta_{j.wall.1.2.fl12} := \dfrac{h_{tot.12}}{h_{tot}} = 0.992$

$h_{under.wall.1} = 0.643\,m$ $\eta_{j.wall.1.2.fl12}\cdot\left(l_{1.wall.1.2.fl12} + l_{2.wall.1.2.fl12}\right) = 1.411\,m$

Okej with main case 1

Factor for reduction of the shear flow in the boards (Formula 6.14)

$\lambda_{wall.1.2.fl12.a} := 1$

$$\lambda_{wall.1.2.fl12.b} := \left[\frac{h}{l_{3.wall.1.2.fl12}\cdot h_{under.wall.1}}\left[\left[\sqrt{1-\left(1-\frac{1}{n_{fl12}}\right)^{2}} - \frac{l_{1.wall.1.2.fl12}}{2\cdot h_{tot.12}} \cdots + -\frac{V_{ekv.wall.1.2.fl12}}{f_{p.wall.1.fl12}\cdot h_{tot.12}}\right]\cdot l_{1.wall.1.2.fl12} \cdots + \frac{n_{knut.wall.1.1}\cdot F_{knut}}{f_{p.wall.1.fl12}}\right]\right]$$

$$\lambda_{wall.1.2.fl12.c} := \left[\frac{2\cdot h}{l_{3.wall.1.2.fl12}\cdot\left(h - h_{under.wall.1}\right)}\left[\left(\frac{l_{1.wall.1.2.fl12}}{2\cdot h_{tot.12}} \cdots + \frac{V_{ekv.wall.1.2.fl12}}{f_{p.wall.1.fl12}\cdot h_{tot.12}}\right)\cdot l_{1.wall.1.2.fl12} \cdots + l_{2.wall.1.2.fl12}\right]\right]$$

$$\lambda_{wall.1.2.fl12.d} := \left(\frac{b_{full}}{l_{3.wall.1.2.fl12}}\right)$$

$\lambda_{wall.1.2.fl12} := \min\left(\lambda_{wall.1.2.fl12.a}, \lambda_{wall.1.2.fl12.b}, \lambda_{wall.1.2.fl12.c}, \lambda_{wall.1.2.fl12.d}\right) = 1$

The effective length of transferring shear forces (Formula 6.13)

$$l_{eff.wall.1.2.fl12} := \min\left[\left[\left(\frac{l_{1.wall.1.2.fl12}}{2h_{tot.12}} \cdots + \frac{V_{ekv.wall.1.2.fl12}}{f_{p.wall.1.fl12}\cdot h_{tot.12}}\right)\cdot l_{1.wall.1.2.fl12} \cdots + l_{2.wall.1.2.fl12} \cdots + \frac{h_{under.wall.1}}{h}\cdot\lambda_{wall.1.2.fl12}\cdot l_{3.wall.1.2.fl12}\right], \left(l_{1.wall.1.2.fl12} \cdots + l_{2.wall.1.2.fl12}\right)\right] = 0.934\,m$$

Horizontal capacity of wall part 2 (Formula 4.2)

$$H_{1.wall.1.2.fl12} := f_{p.wall.1.fl12}\cdot l_{eff.wall.1.2.fl12} = 8.718\cdot kN$$

Reaction forces l.2=0 (Formula 6.15 and 6.16)

$$R_{N.wall.1.2.fl12} := f_{p.wall.1.fl12}\cdot\left(l_{1.wall.1.2.fl12} - \lambda_{wall.1.2.fl12}\cdot n_{fl12}\cdot h_{under.wall.1}\right) \cdots + 1\cdot 4\cdot V_{i.stud.wall.1} = -54.735\cdot kN$$

$$R_{opening.wall.1.2.fl12} := f_{p.wall.1.fl12}\cdot\lambda_{wall.1.1.fl12}\cdot n_{fl12}\cdot h_{under.wall.1} = 42.62\cdot kN$$

Part 3

Vertical load $V_{i.wall.1.3.fl12} := 1 \cdot Q_{tot.fl.12} \cdot L_{Dim.wall.1.3} = 20.047 \cdot kN$

Maximum shear force to be transmitted through the consoles

$$V_{max.wall.1.3.fl12} := f_{p.wall.1.fl12} \cdot n_{fl12} \cdot h_{over.wall.1} = 99.456 \cdot kN$$

$$V_{\_1.wall1.3.fl12} := 1 \cdot V_{i.stud.wall.1.fl12} = 2.614 \cdot kN$$

$$V_{\_2.wall.1.3.fl12} := V_{max.wall.1.3.fl12} - V_{\_1.wall1.3.fl12} = 96.842 \cdot kN$$

$$V_{0.wall.1.3.fl12} := R_{opening.wall.1.2.fl12} + 1 \cdot V_{i.stud.wall.1.fl12} = 45.233 \cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.3.fl12} := \min\left[h_{tot.12} \cdot \left(1 - \frac{V_{i.wall.1.3.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right), L_{Dim.wall.1.3}\right] = 4.602 \, m$$

$L_{Dim.wall.1.2} = 1.423 \, m$ entains $l_{2.wall.1.3.fl12} := \max\left[\left(L_{Dim.wall.1.3} - l_{1.wall.1.3.fl12}\right), 0\right] = 0 \, m$

Number of studs in the wall $n_{studs.wall.1.3.fl12} := \frac{l_{1.wall.1.3.fl12}}{s_{studs}} = 7.67$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.3.fl12} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \\ 7.67 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$\begin{aligned} V_{ekv.wall.1.3.fl12} := & \left(\frac{l_{1.wall.1.3.fl12} + s_{stud} \cdot 2}{l_{1.wall.1.3.fl12}} \cdot V_{\_2.wall.1.3.fl12}\right) \cdots \\ & + \left(\frac{l_{1.wall.1.3.fl12} + s_{stud} \cdot 1}{l_{1.wall.1.3.fl12}} \cdot V_{\_1.wall1.3.fl12}\right) \cdots \\ & + \sum \left(\frac{l_{1.wall.1.3.fl12} - x_{i.wall.1.3.fl12} \cdot s_{studs}}{l_{1.wall.1.3.fl12}} \cdot V_{i.stud.wall.1.fl12}\right) \end{aligned} = 136.417 \cdot kN$$

The effective length of transferring shear forces (Formula 6.3)

$$l_{eff.wall.1.3.fl12} := \min\left[\left[\left(\frac{l_{1.wall.1.3.fl12}}{2h_{tot.12}} \cdots + \frac{V_{ekv.wall.1.3.fl12}}{f_{p.wall.1.fl12} \cdot h_{tot.12}}\right) \cdot l_{1.wall.1.3.fl12} \cdots + l_{2.wall.1.3.fl12}\right], \left(l_{1.wall.1.3.fl12} \cdots + l_{2.wall.1.3.fl12}\right)\right] = 2.181\,m$$

Horizontal capacity of wall part 2 (Formula 4.2)

$$H_{1.wall.1.3.fl12} := f_{p.wall.1.fl12} \cdot l_{eff.wall.1.3.fl12} = 20.354 \cdot kN$$

Reaction force l.2=0 (Formula 6.8)

$$R_{N.wall.1.3.fl12} := f_{p.wall.1.fl12} \cdot l_{1.wall.1.3.fl12} + 1 \cdot 9 \cdot V_{i.stud.wall.1} = 51.952 \cdot kN$$

Total design resistance of wall 1

$$R_{d.wall.1.fl12} := H_{1.wall.1.1.fl12} + H_{1.wall.1.2.fl12} + H_{1.wall.1.3.fl12} = 33.773 \cdot kN$$

Design load from horizontal forces

$$H_{d.wall.1.fl12} := H_{iy.12_{0}} = 26.076 \cdot kN$$

Utilization ratio

$$UR_{wall.1fl12} := \frac{H_{d.wall.1.fl12}}{R_{d.wall.1.fl12}} = 0.772$$

Floor 11

Centre distance between joints $s_{s.wall.1.fl11} := 0.05m$

Load acting on one stud $V_{i.stud.wall.1.fl11} = 3.012 \cdot kN$

Plate stud anchor plastic capacity $F_{p.wall.1.fl11} := 1.4kN$

Plastic shear flow force per unit length (Formula 4.1)

$$f_{p.wall.1.fl11} := \frac{F_{p.wall.1.fl11}}{s_{s.wall.1.fl11}} - f_{p.wall.1.fl12} = 18.667 \cdot \frac{kN}{m}$$

Plastic wall capacity $f_{p.wall.1.fl11} \cdot h_{tot} = 672 \cdot kN$

Plastic capacity over door $f_{p.wall.1.fl11} \cdot n_{fl11} \cdot h_{over.wall.1} = 182.336 \cdot kN$

Part 1

Vertical load $V_{i.wall.1.1.fl11} := 1 \cdot Q_{tot.fl.11} \cdot L_{Dim.wall.1.1} = 19.468 \cdot kN$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.1.fl11} := \min\left[h_{tot.11} \cdot \left(1 - \frac{V_{i.wall.1.1.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}}\right), L_{Dim.wall.1.1}\right] = 3.878\,m$$

$L_{Dim.wall.1.1} = 3.878\,m$ entains $l_{2.wall.1.1.fl11} := \max\left[\left(L_{Dim.wall.1.1} - l_{1.wall.1.1.fl11}\right), 0\right] = 0\,m$

$l_{3.wall.1.1.fl11} := b_{window.1.wall.1} = 1.4\,m$

Number of studs in the wall $n_{studs.wall.1.1.fl11} := \dfrac{l_{1.wall.1.1.fl11}}{s_{studs}} = 6.463$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.1.fl11} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 6.463 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$V_{ekv.wall.1.1.fl11} := \sum \left(\frac{l_{1.wall.1.1.fl11} - x_{i.wall.1.1.fl11} \cdot s_{studs}}{l_{1.wall.1.1.fl11}} \cdot V_{i.stud.wall.1.fl11}\right) = 11.298 \cdot kN$$

Control of main case $h_{under} > \eta_j \left(l_1 + l_2\right)$ entains main case 2

Dimensionless load factor (Formula 6.5) $\eta_{j.wall.1.1.fl11} := \frac{h_{tot.11}}{h_{tot}} = 0.909$

$h_{under.wall.1} = 0.643\,m$ $\quad$ $\eta_{j.wall.1.1.fl11} \cdot \left(l_{1.wall.1.1.fl11} + l_{2.wall.1.1.fl11}\right) = 3.525\,m$

Okej with main case 1

Factor for reduction of the shear flow in the boards (Formula 6.14)

$\lambda_{wall.1.1.fl11.a} := 1$

$$\lambda_{wall.1.1.fl11.b} := \left[\frac{h}{l_{3.wall.1.1.fl11} \cdot h_{under.wall.1}} \left[\left[\sqrt{1 - \left(1 - \frac{1}{n_{fl11}}\right)^2} \cdots + \frac{l_{1.wall.1.1.fl11}}{2 \cdot h_{tot.11}} - \frac{V_{ekv.wall.1.1.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \right] \cdot l_{1.wall.1.1.fl11} \cdots + \frac{n_{knut.wall.1.1} \cdot F_{knut}}{f_{p.wall.1.fl11}} \right] \right]$$

$$\lambda_{wall.1.1.fl11.c} := \left[\frac{2 \cdot h}{l_{3.wall.1.1.fl11} \cdot \left(h - h_{under.wall.1}\right)} \left[\left(\frac{l_{1.wall.1.1.fl11}}{2 \cdot h_{tot.11}} \cdots + \frac{V_{ekv.wall.1.1.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \right) \cdot l_{1.wall.1.1.fl11} \cdots + l_{2.wall.1.1.fl11} \right] \right]$$

$$\lambda_{wall.1.1.fl11.d} := \left(\frac{b_{full}}{l_{3.wall.1.1.fl11}} \right)$$

$$\lambda_{wall.1.1.fl11} := \min\left(\lambda_{wall.1.1.fl11.a}, \lambda_{wall.1.1.fl11.b}, \lambda_{wall.1.1.fl11.c}, \lambda_{wall.1.1.fl11.d}\right) = 0.549$$

The effective length of transferring shear forces (Formula 6.13)

$$l_{eff.wall.1.1.fl11} := \min\left[\left[\left(\frac{l_{1.wall.1.1.fl11}}{2h_{tot.11}} \cdots + \frac{V_{ekv.wall.1.1.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \right) \cdot l_{1.wall.1.1.fl11} \cdots + l_{2.wall.1.1.fl11} \cdots + \frac{h_{under.wall.1}}{h} \cdot \lambda_{wall.1.1.fl11} \cdot l_{3.wall.1.1.fl11} \right], \left(l_{1.wall.1.1.fl11} \cdots + l_{2.wall.1.1.fl11} \right) \right] = 0.468\,m$$

Horizontal load capacity of wall (Formula 4.2)

$$H_{1.wall.1.1.fl11} := f_{p.wall.1.fl11} \cdot l_{eff.wall.1.1.fl11} = 8.732 \cdot kN$$

Reaction forces l.2=0 (Formula 6.15 and 6.16)

$$R_{N.wall.1.1.fl11} := f_{p.wall.1.fl11} \cdot \left(l_{1.wall.1.1.fl11} - \lambda_{wall.1.1.fl11} \cdot n_{fl11} \cdot h_{under.wall.1}\right) \cdots + 1 \cdot 8 \cdot V_{i.stud.wall.1} = 7.843 \cdot kN$$

$$R_{opening.wall.1.1.fl11} := f_{p.wall.1.fl11} \cdot \lambda_{wall.1.1.fl11} \cdot n_{fl11} \cdot h_{under.wall.1} = 72.547 \cdot kN$$

Part 2

Vertical load $V_{i.wall.1.2.fl11} := 1 \cdot Q_{tot.fl.11} \cdot L_{Dim.wall.1.2} = 7.143 \cdot kN$

Maximum shear force to be transmitted through the consoles

$$V_{max.wall.1.2.fl11} := f_{p.wall.1.fl11} \cdot n_{fl11} \cdot h_{over.wall.1} = 182.336 \cdot kN$$

The resultant forces acting on the two consoles

$$V_{\_1.wall1.2.fl11} := 1 \cdot V_{i.stud.wall.1.fl11} = 3.012 \cdot kN$$

$$V_{\_2.wall1.2.fl11} := V_{max.wall.1.2.fl11} - V_{\_1.wall1.2.fl11} = 179.324 \cdot kN$$

$$V_{0.wall.1.2.fl11} := R_{opening.wall.1.1.fl11} + 1 \cdot V_{i.stud.wall.1.fl11} = 75.559 \cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.2.fl11} := \min\left[h_{tot.11} \cdot \left(1 - \frac{V_{i.wall.1.2.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}}\right), L_{Dim.wall.1.2}\right] = 1.423\,m$$

$L_{Dim.wall.1.2} = 1.423\,m$ entains $l_{2.wall.1.2.fl11} := \max\left[\left(L_{Dim.wall.1.2} - l_{1.wall.1.2.fl11}\right), 0\right] = 0\,m$

$$l_{3.wall.1.2.fl11} := b_{window.2.wall.1} = 0.5\,m$$

Number of studs in the wall $n_{studs.wall.1.2.fl11} := \dfrac{l_{1.wall.1.2.fl11}}{s_{studs}} = 2.372$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.2.fl11} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 2.372 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$\begin{aligned} V_{ekv.wall.1.2.fl11} := &\left(\frac{l_{1.wall.1.2.fl11} + s_{stud} \cdot 2}{l_{1.wall.1.2.fl11}} \cdot V_{\_2.wall1.2.fl11}\right) \dots \\ &+ \left(\frac{l_{1.wall.1.2.fl11} + s_{stud} \cdot 1}{l_{1.wall.1.2.fl11}} \cdot V_{\_1.wall1.2.fl11}\right) \dots \\ &+ \sum\left(\frac{l_{1.wall.1.2.fl11} - x_{i.wall.1.2.fl11} \cdot s_{studs}}{l_{1.wall.1.2.fl11}} \cdot V_{i.stud.wall.1.fl11}\right) \end{aligned} = 340.054 \cdot kN$$

Control of main case $h_{under} > \eta_j\left(l_1 + l_2\right)$ entains main case 2

Dimensionless load factor (Formula 6.5) $\eta_{j.wall.1.2.fl11} := \dfrac{h_{tot.11}}{h_{tot}} = 0.909$

$h_{under.wall.1} = 0.643\,m$ $\eta_{j.wall.1.2.fl11} \cdot \left(l_{1.wall.1.2.fl11} + l_{2.wall.1.2.fl11}\right) = 1.294\,m$

Okej with main case 1

Factor for reduction of the shear flow in the boards (Formula 6.14)

$$\lambda_{wall.1.2.fl11.a} := 1$$

$$\lambda_{wall.1.2.fl11.b} := \left[\frac{h}{l_{3.wall.1.2.fl11} \cdot h_{under.wall.1}} \left[\left[\begin{array}{l} \sqrt{1 - \left(1 - \frac{1}{n_{fl11}}\right)^{2}} \cdots \\ + -\frac{l_{1.wall.1.2.fl11}}{2 \cdot h_{tot.11}} \cdots \\ + -\frac{V_{ekv.wall.1.2.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \end{array}\right] \cdot l_{1.wall.1.2.fl11} \cdots \atop + \frac{n_{knut.wall.1.1} \cdot F_{knut}}{f_{p.wall.1.fl11}}\right]\right]$$

$$\lambda_{wall.1.2.fl11.c} := \left[\frac{2 \cdot h}{l_{3.wall.1.2.fl11} \cdot \left(h - h_{under.wall.1}\right)} \left[\left(\begin{array}{l} \frac{l_{1.wall.1.2.fl11}}{2 \cdot h_{tot.11}} \cdots \\ + \frac{V_{ekv.wall.1.2.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \end{array}\right) \cdot l_{1.wall.1.2.fl11} \cdots \atop + l_{2.wall.1.2.fl11}\right]\right]$$

$$\lambda_{wall.1.2.fl11.d} := \left(\frac{b_{full}}{l_{3.wall.1.2.fl11}}\right)$$

$$\lambda_{wall.1.2.fl11} := \min\left(\lambda_{wall.1.2.fl11.a}, \lambda_{wall.1.2.fl11.b}, \lambda_{wall.1.2.fl11.c}, \lambda_{wall.1.2.fl11.d}\right) = -0.396$$

The effective length of transferring shear forces (Formula 6.13)

$$l_{eff.wall.1.2.fl11} := \min\left[\left[\begin{array}{l} \left(\begin{array}{l} \frac{l_{1.wall.1.2.fl11}}{2h_{tot.11}} \cdots \\ + \frac{V_{ekv.wall.1.2.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}} \end{array}\right) \cdot \left(l_{1.wall.1.2.fl11}\right) \cdots \\ + l_{2.wall.1.2.fl11} \cdots \\ + \frac{h_{under.wall.1}}{h} \cdot \lambda_{wall.1.2.fl11} \cdot l_{3.wall.1.2.fl11} \end{array}\right], \left(\begin{array}{l} l_{1.wall.1.2.fl11} \cdots \\ + l_{2.wall.1.2.fl11} \end{array}\right)\right] = 0.78\,m$$

Horizontal capacity of wall part 2 (Formula 4.2)

$$H_{1.wall.1.2.fl11} := f_{p.wall.1.fl11} \cdot l_{eff.wall.1.2.fl11} = 14.566 \cdot kN$$

Reaction forces l.2=0 (Formula 6.15 and 6.16)

$$R_{N.wall.1.2.fl11} := f_{p.wall.1.fl11} \cdot \left(l_{1.wall.1.2.fl11} - \lambda_{wall.1.2.fl11} \cdot n_{fl11} \cdot h_{under.wall.1}\right) \cdots + 1 \cdot 8 \cdot V_{i.stud.wall.1} = 86.812 \cdot kN$$

$$R_{opening.wall.1.2.fl11} := f_{p.wall.1.fl11} \cdot \lambda_{wall.1.1.fl11} \cdot n_{fl11} \cdot h_{under.wall.1} = 72.547 \cdot kN$$

Part 3

Vertical load $V_{i.wall.1.3.fl11} := 1\cdot Q_{tot.fl.11}\cdot L_{Dim.wall.1.3} = 23.102\cdot kN$

Maximum shear force to be transmitted through the consoles

$$V_{max.wall.1.3.fl11} := f_{p.wall.1.fl11}\cdot n_{fl11}\cdot h_{over.wall.1} = 182.336\cdot kN$$

The resultant forces acting on the two consoles

$$V_{\_1.wall1.3.fl11} := 1\cdot V_{i.stud.wall.1.fl11} = 3.012\cdot kN$$

$$V_{\_2.wall.1.3.fl11} := V_{max.wall.1.3.fl11} - V_{\_1.wall1.3.fl11} = 179.324\cdot kN$$

$$V_{0.wall.1.3.fl11} := R_{opening.wall.1.2.fl11} + 1\cdot V_{i.stud.wall.1.fl11} = 75.559\cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.1.3.fl11} := \min\left[h_{tot.11}\cdot\left[1 - \frac{\left(V_{i.wall.1.3.fl11}\right)}{f_{p.wall.1.fl11}\cdot h_{tot.11}}\right], L_{Dim.wall.1.3}\right] = 4.602\,m$$

$L_{Dim.wall.1.2} = 1.423\,m$ entains $l_{2.wall.1.3.fl11} := \max\left[\left(L_{Dim.wall.1.3} - l_{1.wall.1.3.fl11}\right), 0\right] = 0\,m$

Number of studs in the wall $n_{studs.wall.1.3.fl11} := \dfrac{l_{1.wall.1.3.fl11}}{s_{studs}} = 7.67$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.1.3.fl11} := \begin{pmatrix} 0 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \\ 7.67 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$\begin{aligned} V_{ekv.wall.1.3.fl11} := &\left(\frac{l_{1.wall.1.3.fl11} + s_{stud}\cdot 2}{l_{1.wall.1.3.fl11}}\cdot V_{\_2.wall.1.3.fl11}\right) \dots \\ &+ \left(\frac{l_{1.wall.1.3.fl11} + s_{stud}\cdot 1}{l_{1.wall.1.3.fl11}}\cdot V_{\_1.wall1.3.fl11}\right) \dots \\ &+ \sum\left(\frac{l_{1.wall.1.3.fl11} - x_{i.wall.1.3.fl11}\cdot s_{studs}}{l_{1.wall.1.3.fl11}}\cdot V_{i.stud.wall.1.fl11}\right) \end{aligned} = 242.589\cdot kN$$

The effective length of transferring shear forces (Formula 6.3)

$$l_{eff.wall.1.3.fl11} := \min\left[\left[\left(\frac{l_{1.wall.1.3.fl11}}{2h_{tot.11}} \cdots + \frac{V_{ekv.wall.1.3.fl11}}{f_{p.wall.1.fl11} \cdot h_{tot.11}}\right) \cdot l_{1.wall.1.3.fl11} \cdots + l_{2.wall.1.3.fl11}\right], \left(l_{1.wall.1.3.fl11} \cdots + l_{2.wall.1.3.fl11}\right)\right] = 2.151\,m$$

Horizontal capacity of wall part 2 (Formula 4.2)

$$H_{1.wall.1.3.fl11} := f_{p.wall.1.fl11} \cdot l_{eff.wall.1.3.fl11} = 40.155 \cdot kN$$

Reaction force l.2=0 (Formula 6.8)

$$R_{N.wall.1.3.fl11} := f_{p.wall.1.fl11} \cdot l_{1.wall.1.3.fl11} + 1 \cdot 9 \cdot V_{i.stud.wall.1} = 94.904 \cdot kN$$

Total design resistance of wall 1

$$R_{d.wall.1.fl11} := H_{1.wall.1.1.fl11} + H_{1.wall.1.2.fl11} + H_{1.wall.1.3.fl11} = 63.452 \cdot kN$$

Design load from horizontal forces

$$H_{d.wall.1.fl11} := H_{iy.11_0} = 5.149 \times 10^4\,N$$

<u>Utilization ratio</u>

$$UR_{wall.1fl11} := \frac{H_{d.wall.1.fl11}}{R_{d.wall.1.fl11}} = 0.811$$

Repeated for the remaining floors

Values wall 1

$$UR_{wall.1} := \begin{pmatrix} UR_{wall.1fl12} \\ UR_{wall.1fl11} \\ UR_{wall.1fl10} \\ UR_{wall.1fl9} \\ UR_{wall.1fl8} \\ UR_{wall.1fl7} \\ UR_{wall.1fl6} \\ UR_{wall.1fl5} \\ UR_{wall.1fl4} \\ UR_{wall.1fl3} \\ UR_{wall.1fl2} \\ UR_{wall.1fl1} \end{pmatrix} =$$

	0
0	0.772
1	0.811
2	1.625
3	0.93
4	0.891
5	0.852
6	0.812
7	0.772
8	0.806
9	0.816
10	0.726
11	0.883

$$F_{p.wall.1} := \begin{pmatrix} F_{p.wall.1.fl12} \\ F_{p.wall.1.fl11} \\ F_{p.wall.1.fl10} \\ F_{p.wall.1.fl9} \\ F_{p.wall.1.fl8} \\ F_{p.wall.1.fl7} \\ F_{p.wall.1.fl6} \\ F_{p.wall.1.fl5} \\ F_{p.wall.1.fl4} \\ F_{p.wall.1.fl3} \\ F_{p.wall.1.fl2} \\ F_{p.wall.1.fl1} \end{pmatrix} =$$

	0
0	1.4
1	1.4
2	1.8
3	2.5
4	3.2
5	3.9
6	4.6
7	5.3
8	5.9
9	6.4
10	6.9
11	7.2

$\cdot kN$

$$s_{s.wall.1} := \begin{pmatrix} s_{s.wall.1.fl12} \\ s_{s.wall.1.fl11} \\ s_{s.wall.1.fl10} \\ s_{s.wall.1.fl9} \\ s_{s.wall.1.fl8} \\ s_{s.wall.1.fl7} \\ s_{s.wall.1.fl6} \\ s_{s.wall.1.fl5} \\ s_{s.wall.1.fl4} \\ s_{s.wall.1.fl3} \\ s_{s.wall.1.fl2} \\ s_{s.wall.1.fl1} \end{pmatrix} =$$

	0
0	0.15
1	0.05
2	0.05
3	0.05
4	0.05
5	0.05
6	0.05
7	0.05
8	0.05
9	0.05
10	0.05
11	0.05

m

$$f_{p.wall.1} := \frac{F_{p.wall.1}}{s_{s.wall.1}} =$$

	0
0	9.333
1	28
2	36
3	50
4	64
5	78
6	92
7	106
8	118
9	128
10	138
11	144

$\cdot \frac{kN}{m}$

$$f_{p.wall1} := \begin{pmatrix} f_{p.wall.1.fl12} \\ f_{p.wall.1.fl11} \\ f_{p.wall.1.fl10} \\ f_{p.wall.1.fl9} \\ f_{p.wall.1.fl8} \\ f_{p.wall.1.fl7} \\ f_{p.wall.1.fl6} \\ f_{p.wall.1.fl5} \\ f_{p.wall.1.fl4} \\ f_{p.wall.1.fl3} \\ f_{p.wall.1.fl2} \\ f_{p.wall.1.fl1} \end{pmatrix} =$$

	0
0	9.333
1	18.667
2	8
3	14
4	14
5	14
6	14
7	14
8	12
9	10
10	10
11	6

$\cdot \frac{kN}{m}$

$$H_{d.wall.1} := \begin{pmatrix} H_{d.wall.1.fl12} \\ H_{d.wall.1.fl11} \\ H_{d.wall.1.fl10} \\ H_{d.wall.1.fl9} \\ H_{d.wall.1.fl8} \\ H_{d.wall.1.fl7} \\ H_{d.wall.1.fl6} \\ H_{d.wall.1.fl5} \\ H_{d.wall.1.fl4} \\ H_{d.wall.1.fl3} \\ H_{d.wall.1.fl2} \\ H_{d.wall.1.fl1} \end{pmatrix} =$$

	0
0	26.076
1	51.488
2	51.488
3	49.265
4	49.265
5	49.265
6	49.265
7	49.265
8	49.265
9	49.265
10	49.265
11	49.265

$\cdot kN$

F Horizontal capacity of wall 3

Loads acting on wall 3

Total load from self weight acting on one stud in wall 3 for one floor

Load acting on one stud $G_{wall.3.stud} := G_{innerwall} \cdot s_{stud} = 1.02 \cdot kN$

Load acting on one stud (Load favorable Table B-3,Equation 6.10a)

$V_{i.stud.wall.3} := 1.5kN$

Load acting as a line load $g_{Fav.innerwalls} := G_{innerwall} = 1.7 \cdot \frac{kN}{m}$

Geometry of wall 3

Width of boards $L_{plates} := 1.2m$

Interior door width $b_{innerdoor} := 1.010m$

Center spacing between joists $s_{studs} := 0.6m$

Number of floors $n_{fl1} := 12$

Full board width $b_{full} := L_{plates} = 1.2\,m$

Distance above the door $h_{over.wall.3} := 0.992m$

Anchorage and joint properties

Shear resistance of a junction unit (Table 7.2) $F_{knut} := 700N$

Plate stud anchor plastic capacity (Table 7.1) $F_p := 2 \cdot 700N = 1.4 \cdot kN$

Wall part 1

Length of the wall part 1 $L_{Dim.wall.3.1} := 0.855m$

$L_{wall.3.1} := 0.855m$

Number of studs in the wall $n_{studs.wall.3.1} := \frac{L_{Dim.wall.3.1}}{s_{studs}} = 1.425$

Number of node connections which are active for the transfer of shear force

$n_{knut.wall.3.1} := 1$

Wall part 2

Length of the wall $L_{wall.3.2} := 4.054m$

$L_{Dim.wall.3.2} := L_{wall.3.2} = 4.054\,m$

Number of studs in the wall $n_{studs.wall.3.2} := \frac{L_{Dim.wall.3.2}}{s_{studs}} = 6.757$

Number of node connections which are active for the transfer of shear force

$n_{knut.wall.3.2} := 6$

Horizontal load-bearing capacity

Floor 12

Centre distance between joints $s_{s.wall.3fl12} := 0.18m$

Load acting on one stud $V_{i.stud.wall.3.fl12} = 1.02 \cdot kN$

Plate stud anchor plastic capacity $F_{p.wall.3.fl12} := 2 \cdot 700N = 1.4 \cdot kN$

Plastic shear flow force per unit length (Formula 4.1)

$$f_{p.wall.3.fl12} := \frac{F_{p.wall.3.fl12}}{s_{s.wall.3fl12}} = 7.778 \cdot \frac{kN}{m}$$

Plastic capacity over door $f_{p.wall.3.fl12} \cdot n_{fl12} \cdot h_{over.wall.3} = 92.587 \cdot kN$

Vertical force at the front stud $V_{0.wall.3.1.fl12} := f_{p.wall.3.fl12} \cdot h = 23.139 \cdot kN$

Vertical load acting at the end stud $V_{N.wall.3.1.fl12} := V_{i.stud.wall.3.fl12} = 1.02 \cdot kN$

$$V_{N.wall.3.2.fl12} := V_{i.stud.wall.3.fl12} = 1.02 \cdot kN$$

Part 1

Vertical load

$$V_{i.wall.3.1.fl12} := 1 \cdot Q_{tot.wall3.fl.12} \cdot L_{Dim.wall.3.1} = 4.066 \cdot kN$$

$$V_{0.wall.3.1.fl12} := V_{i.stud.wall.3.fl12} = 1.02 \cdot kN$$

$$V_{max.wall3.1.fl12} := f_{p.wall.3.fl12} \cdot 1 \cdot h_{over.wall.3} = 7.716 \cdot kN$$

$$V_{\_1.wall.3.1.fl12} := V_{i.stud.wall.3.fl12} = 1.02 \cdot kN$$

$$V_{\_2.wall.3.1.fl12} := V_{max.wall3.1.fl12} - V_{\_1.wall.3.1.fl12} = 6.696 \cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.3.1.fl12} := \min\left[h_{tot.12} \cdot \left(1 - \frac{V_{i.wall.3.1.fl12}}{f_{p.wall.3.fl12} \cdot h_{tot.12}}\right), L_{wall.3.1}\right] = 0.855\,m$$

$L_{Dim.wall.3.1} = 0.855\,m$ entains $l_{2.wall.3.1.fl12} := \max\left[\left(L_{Dim.wall.3.1} - l_{1.wall.3.1.fl12}\right), 0\right] = 0\,m$

Number of studs in the wall $n_{studs.l.1wall.3.1.fl12} := \frac{l_{1.wall.3.1.fl12}}{s_{studs}} = 1.425$

Distance from the front stud to the other studs in the wall

$$x_{i.wall.3.1.fl12} := \begin{pmatrix} 0 \\ 1 \\ 1.425 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$\begin{aligned} V_{ekv.wall.3.1.fl12} := & \left(\frac{l_{1.wall.3.1.fl12} + s_{stud} \cdot 2}{l_{1.wall.3.1.fl12}} \cdot V_{\_2.wall.3.1.fl12} \right) \cdots \\ & + \left(\frac{l_{1.wall.3.1.fl12} + s_{stud} \cdot 1}{l_{1.wall.3.1.fl12}} \cdot V_{\_1.wall.3.1.fl12} \right) \cdots \\ & + \sum \left(\frac{l_{1.wall.3.1.fl12} - x_{i.wall.3.1.fl12} \cdot s_{studs}}{l_{1.wall.3.1.fl12}} \cdot V_{i.stud.wall.3.fl12} \right) \end{aligned} = 19.153 \cdot kN$$

The effective length of transferring shear forces (Formula 6.3)

$$l_{eff.wall.3.1.fl12} := \min\left[\left[\left(\frac{l_{1.wall.3.1.fl12}}{2h_{tot.12}} \cdots + \frac{V_{ekv.wall.3.1.fl12}}{f_{p.wall.3.fl12} \cdot h_{tot.12}} \right) \cdot l_{1.wall.3.1.fl12} \cdots + l_{2.wall.3.1.fl12} \right], \left(l_{1.wall.3.1.fl12} \cdots + l_{1.wall.3.1.fl12} \right) \right] = 0.069\,m$$

Horizontal wall capacity (Formula 4.2)

$$H_{1.wall.3.1.fl12} := f_{p.wall.3.fl12} \cdot l_{eff.wall.3.1.fl12} = 0.538 \cdot kN$$

Reaction force at the end stud l.2=0 (Formula 6.8)

$$R_{N.wall.3.1.fl12} := f_{p.wall.3.fl12} \cdot l_{1.wall.3.1.fl12} + 1 \cdot 3V_{N.wall.3.1.fl12} = 9.71 \cdot kN$$

Part 2

Vertical load $V_{i.wall.3.2.fl12} := 1 \cdot Q_{tot.wall3.fl.12} \cdot L_{Dim.wall.3.1} = 4.066 \cdot kN$

The walls entire shear capacity is reached immediately at the front stud

$$l_{1.wall.3.1fl12} := 0$$

$$l_{2.wall.3.1fl12} := L_{wall.3.2} = 4.054\,m$$

The effective length of transferring shear forces

$$l_{eff.wall.3.2.fl12} := l_{2.wall.3.1fl12} = 4.054\,m$$

Horizontal load capacity of wall (Formula 4.2)

$$H_{1.wall.3.2.fl12} := f_{p.wall.3.fl12} \cdot l_{eff.wall.3.2.fl12} = 31.531 \cdot kN$$

Reaction force at end stud l.2=0.855m (Formula 6.7)

$$R_{N.wall.3.2.fl12} := f_{p.wall.3.fl12} \cdot h_{tot.12} + 12V_{N.wall.3.2.fl12} = 289.907 \cdot kN$$

Total design resistance of wall 3 $R_{d.wall.3.fl12} := H_{1.wall.3.1.fl12} + H_{1.wall.3.2.fl12} = 32.069 \cdot kN$

Design load from horizontal forces $H_{d.wall.3.fl12} := H_{iy.12_2} = 2.566 \times 10^4 N$

<u>Utilization ratio</u> $UR_{wall.3.fl12} := \frac{H_{d.wall.3.fl12}}{R_{d.wall.3.fl12}} = 0.8$

Floor 11

Centre distance between joints $s_{s.wall.3fl11} := 0.06m$

Load acting on one stud $V_{i.stud.wall.3.fl11} = 5.484 \cdot kN$

Plate stud anchor plastic capacity $F_{p.wall.3.fl11} := 1.4kN$

Plastic shear flow force per unit length (Formula 4.1)

$$f_{p.wall.3.fl11} := \frac{F_{p.wall.3.fl11}}{s_{s.wall.3fl11}} - f_{p.wall.3.fl12} = 15.556 \cdot \frac{kN}{m}$$

Plastic capacity over door $f_{p.wall.3.fl11} \cdot n_{fl11} \cdot h_{over.wall.3} = 169.742 \cdot kN$

Vertical force at the front stud $V_{0.wall.3.1.fl11} := f_{p.wall.3.fl11} \cdot h = 46.278 \cdot kN$

Vertical load acting at the end stud $V_{N.wall.3.1.fl11} := V_{i.stud.wall.3.fl11} = 5.484 \cdot kN$

Part 1

Vertical load

$$V_{i.wall.3.1.fl11} := 1 \cdot Q_{tot.wall3.fl.11} \cdot L_{Dim.wall.3.1} = 4.634 \cdot kN$$

$$V_{0.wall.3.1.fl11} := V_{i.stud.wall.3.fl11} = 5.484 \cdot kN$$

$$V_{max.wall3.1.fl11} := f_{p.wall.3.fl11} \cdot 1 \cdot h_{over.wall.3} = 15.431 \cdot kN$$

$$V_{\_1.wall.3.1.fl11} := V_{i.stud.wall.3.fl11} = 5.484 \cdot kN$$

$$V_{\_2.wall.3.1.fl11} := V_{max.wall3.1.fl11} - V_{\_1.wall.3.1.fl11} = 9.947 \cdot kN$$

The length of the fictitious section (Formula 6.24)

$$l_{1.wall.3.1.fl11} := \min\left[h_{tot.11} \cdot \left(1 - \frac{V_{i.wall.3.1.fl11}}{f_{p.wall.3.fl11} \cdot h_{tot.11}}\right), L_{wall.3.1}\right] = 0.855m$$

$L_{wall.3.1} = 0.855m$ entains $l_{2.wall.3.1.fl11} := \max\left[\left(L_{wall.3.1} - l_{1.wall.3.1.fl11}\right), 0\right] = 0m$

Number of studs in the wall $n_{studs.l.1wall.3.1.fl11} := \frac{l_{1.wall.3.1.fl11}}{s_{studs}} = 1.425$

$$x_{i.wall.3.1.fl11} := \begin{pmatrix} 0 \\ 1 \\ 1.425 \end{pmatrix}$$

The equivalent force acting on the first stud in the wall (Formula 6.25)

$$V_{ekv.wall.3.1.fl11} := \sum\left(\frac{l_{1.wall.3.1.fl11} - x_{i.wall.3.1.fl11} \cdot s_{studs}}{l_{1.wall.3.1.fl11}} \cdot V_{i.stud.wall.3.fl11}\right) = 7.12 \cdot kN$$

The effective length of transferring shear forces (Formula 6.3)

$$l_{eff.wall.3.1.fl11} := \min\left[\left[\left(\frac{l_{1.wall.3.1.fl11}}{2h_{tot.11}} \cdots + \frac{V_{ekv.wall.3.1.fl11}}{f_{p.wall.3.fl12} \cdot h_{tot.11}}\right) \cdot l_{1.wall.3.1.fl11} \cdots + l_{2.wall.3.1.fl11}\right], \left(l_{1.wall.3.1.fl11} \cdots + l_{2.wall.3.1.fl11}\right)\right] = 0.035\,m$$

Horizontal wall capacity (Formula 4.2)

$$H_{1.wall.3.1.fl11} := f_{p.wall.3.fl11} \cdot l_{eff.wall.3.1.fl11} = 0.546 \cdot kN$$

Reaction force at the end stud

$$R_{N.wall.3.1.fl11} := f_{p.wall.3.fl11} \cdot l_{1.wall.3.1.fl11} + 1 \cdot 3V_{N.wall.3.1.fl11} = 29.752 \cdot kN$$

Part 2

Vertical load $V_{i.wall.3.2.fl11} := 1 \cdot Q_{tot.wall3.fl.11} \cdot L_{Dim.wall.3.2} = 21.973 \cdot kN$

The walls entire shear capacity is reached immediately at the front stud

$$l_{1.wall.3.1fl11} := 0$$

$$l_{2.wall.3.1fl11} := L_{Dim.wall.3.2} = 4.054\,m$$

The effective length of transferring shear forces

$$l_{eff.wall.3.2.fl11} := l_{2.wall.3.1fl11} = 4.054\,m$$

Horizontal load capacity of wall (Formula 4.2)

$$H_{1.wall.3.2.fl11} := f_{p.wall.3.fl11} \cdot l_{eff.wall.3.2.fl11} = 63.062 \cdot kN$$

Reaction force at end stud

$$R_{N.wall.3.2.fl11} := f_{p.wall.3.fl11} \cdot h_{tot.11} + 1V_{N.wall.3.2.fl12} = 510.076 \cdot kN$$

Design resistance $R_{d.wall.3.fl11} := H_{1.wall.3.1.fl11} + H_{1.wall.3.2.fl11} = 63.608 \cdot kN$

Design load from horizontal forces $H_{d.wall.3.fl11} := H_{iy.11_2} = 50.671 \cdot kN$

<u>Utilization ratio</u> $UR_{wall.3.fl11} := \frac{H_{d.wall.3.fl11}}{R_{d.wall.3.fl11}} = 0.797$

▶ Repeated for the remaining floors

Values wall 3

$$UR_{wall.3} := \begin{pmatrix} UR_{wall.3.fl12} \\ UR_{wall.3.fl11} \\ UR_{wall.3.fl10} \\ UR_{wall.3.fl9} \\ UR_{wall.3.fl8} \\ UR_{wall.3.fl7} \\ UR_{wall.3.fl6} \\ UR_{wall.3.fl5} \\ UR_{wall.3.fl4} \\ UR_{wall.3.fl3} \\ UR_{wall.3.fl2} \\ UR_{wall.3.fl1} \end{pmatrix} =$$

	0
0	0.8
1	0.797
2	0.98
3	0.989
4	0.988
5	0.987
6	0.986
7	0.984
8	0.98
9	0.975
10	0.965
11	0.935

$$F_{p.wall.3} := \begin{pmatrix} F_{p.wall.3.fl12} \\ F_{p.wall.3.fl11} \\ F_{p.wall.3.fl10} \\ F_{p.wall.3.fl9} \\ F_{p.wall.3.fl8} \\ F_{p.wall.3.fl7} \\ F_{p.wall.3.fl6} \\ F_{p.wall.3.fl5} \\ F_{p.wall.3.fl4} \\ F_{p.wall.3.fl3} \\ F_{p.wall.3.fl2} \\ F_{p.wall.3.fl1} \end{pmatrix} =$$

	0
0	1.4
1	1.4
2	1.8
3	2.4
4	3
5	3.6
6	4.2
7	4.8
8	5.4
9	6
10	6.6
11	7.2

$\cdot kN$

$$s_{s.wall.3} := \begin{pmatrix} s_{s.wall.3fl12} \\ s_{s.wall.3fl11} \\ s_{s.wall.3fl10} \\ s_{s.wall.3fl9} \\ s_{s.wall.3fl8} \\ s_{s.wall.3fl7} \\ s_{s.wall.3fl6} \\ s_{s.wall.3fl5} \\ s_{s.wall.3fl4} \\ s_{s.wall.3fl3} \\ s_{s.wall.3fl2} \\ s_{s.wall.3fl1} \end{pmatrix} =$$

	0
0	0.18
1	0.06
2	0.05
3	0.05
4	0.05
5	0.05
6	0.05
7	0.05
8	0.05
9	0.05
10	0.05
11	0.05

m

$$f_{p.wall.3} := \frac{F_{p.wall.3}}{s_{s.wall.3}} =$$

	0
0	7.778
1	23.333
2	36
3	48
4	60
5	72
6	84
7	96
8	108
9	120
10	132
11	144

$\cdot \frac{kN}{m}$

$$f_{p.wall.3} := \begin{pmatrix} f_{p.wall.3.fl12} \\ f_{p.wall.3.fl11} \\ f_{p.wall.3.fl10} \\ f_{p.wall.3.fl9} \\ f_{p.wall.3.fl8} \\ f_{p.wall.3.fl7} \\ f_{p.wall.3.fl6} \\ f_{p.wall.3.fl5} \\ f_{p.wall.3.fl4} \\ f_{p.wall.3.fl3} \\ f_{p.wall.3.fl2} \\ f_{p.wall.3.fl1} \end{pmatrix} =$$

	0
0	7.778
1	15.556
2	12.667
3	12
4	12
5	12
6	12
7	12
8	12
9	12
10	12
11	12

$\cdot \frac{kN}{m}$

$$H_{d.wall.3} := \begin{pmatrix} H_{d.wall.3.fl12} \\ H_{d.wall.3.fl11} \\ H_{d.wall.3.fl10} \\ H_{d.wall.3.fl9} \\ H_{d.wall.3.fl8} \\ H_{d.wall.3.fl7} \\ H_{d.wall.3.fl6} \\ H_{d.wall.3.fl5} \\ H_{d.wall.3.fl4} \\ H_{d.wall.3.fl3} \\ H_{d.wall.3.fl2} \\ H_{d.wall.3.fl1} \end{pmatrix} =$$

	0
0	25.663
1	50.671
2	50.671
3	48.484
4	48.484
5	48.484
6	48.484
7	48.484
8	48.484
9	48.484
10	48.484
11	48.484

$\cdot kN$

G Connections and sheeting capacity of wall segments

Wall 1 and 3 - OSB

Floor 0-4

Thickness of Kerto Q $t_{kertoQ.fl.0\_4} := 65mm$

Nail diameter $d_{nail.fl.0\_4} := 7.5mm$

Minimum centre distance between nails (Table 8.2)

$$a_{wall.fl.0\_4} := \left(4 + \cos(90)\right)\cdot d_{nail.fl.0\_4} = 0.027\,m$$

$s_{s.wall.1.fl1} = 0.05\,m$ OK

The pointside penetration length should be at least 8d

Length of the nail $l_{nail.fl.0\_4.min} := 8d_{nail.fl.0\_4} + t_{kertoQ.fl.0\_4} = 0.125\,m$

$l_{nail.fl.0\_4} := 150mm$

Caracteristic density for C24 $\rho_{k.C24} := 350\frac{kg}{m^3}$

Density for KertoS $\rho_{k.keroQ} := 510\frac{kg}{m^3}$

According to Figure 8.4 EC5 $t_{timb.fl0\_4} := l_{nail.fl.0\_4} - t_{kertoQ.fl.0\_4} = 85\cdot mm$

Characteristic tensile strength of the wire $f_u := 800\frac{N}{mm^2}$

Unit correction factors for mathcad

$m_{emb} := \frac{m^{0.3}m^3}{kg}\cdot MPa$ $my := mm^{0.4}$

$d_{nail.fl0\_4} := \frac{d_{nail.fl.0\_4}}{m}\cdot 1000 = 7.5$ $t_{KeroQ.ow.fl.0\_4} := \frac{t_{kertoQ.fl.0\_4}}{m}\cdot 1000 = 65$

$\rho_{k.C24.ow} := \rho_{k.C24}\cdot\frac{m^3}{kg} = 350$ $\rho_{k.kertoQ.ow} := \rho_{k.keroQ}\cdot\frac{m^3}{kg} = 510$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.fl0\_4} := 0.3\cdot f_u\cdot d_{nail.fl.0\_4}{}^{2.6}\cdot my = 0.045\cdot kN\cdot m$$

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

Caracteristic embedment strength for LVL (Equation 8.15)

$$f_{h.kertoQ.k.fl.0\_4} := \left(0.082 d_{nail.fl0\_4}{}^{-0.3} \cdot \rho_{k.kertoQ.ow}\right) \frac{N}{mm^2} = 22.849 \cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.32)

$$f_{h.timb.k.fl.0\_4} := \left(0.082 d_{nail.fl0\_4}{}^{-0.3} \cdot \rho_{k.C24.ow}\right) \cdot \frac{N}{mm^2} = 15.681 \cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{fl.0\_4} := \frac{f_{h.timb.k.fl.0\_4}}{f_{h.kertoQ.k.fl.0\_4}} = 0.686$$

Withdrawal and pull penetration-through strength

$$f_{axk.fl.0\_4} := 20 \cdot 10^{-6} \cdot \left(\rho_{k.C24.ow}\right)^2 \cdot MPa = 2.45 \cdot MPa$$

Contribution from rope effect According to 8.24

$$F_{ax.Rk.fl.0\_4} := f_{axk.fl.0\_4} \cdot d_{nail.fl.0\_4} \cdot t_{timb.fl0\_4} = 1.562 \cdot kN$$

Characteristic load-carrying capacity per shear plane per fastner (Equation 8.6)

$$F_{v.Rk.fl0\_4_0} := f_{h.kertoQ.k.fl.0\_4} \cdot t_{kertoQ.fl.0\_4} \cdot d_{nail.fl.0\_4}$$

$$F_{v.Rk.fl0\_4_1} := f_{h.timb.k.fl.0\_4} \cdot t_{timb.fl0\_4} \cdot d_{nail.fl.0\_4}$$

$$F_{v.Rk.fl0\_4_2} := \frac{f_{h.kertoQ.k.fl.0\_4} \cdot t_{kertoQ.fl.0\_4} \cdot d_{nail.fl.0\_4}}{1 + \beta_{fl.0\_4}} \left[\sqrt{\beta_{fl.0\_4} + 2 \cdot \beta_{fl.0\_4}{}^2 \left[1 + \frac{t_{timb.fl0\_4}}{t_{kertoQ.fl.0\_4}} \ldots + \left(\frac{t_{timb.fl0\_4}}{t_{kertoQ.fl.0\_4}}\right)^2\right] \ldots \ldots + \beta_{fl.0\_4}{}^3 \cdot \left(\frac{t_{timb.fl0\_4}}{t_{kertoQ.fl.0\_4}}\right)^2} \ldots + -\beta_{fl.0\_4} \cdot \left(1 + \frac{t_{timb.fl0\_4}}{t_{kertoQ.fl.0\_4}}\right)\right] \ldots + \frac{F_{ax.Rk.fl.0\_4}}{4}$$

$$F_{v.Rk.fl0\_4_3} := 1.05 \cdot \frac{f_{h.kertoQ.k.fl.0\_4} \cdot t_{kertoQ.fl.0\_4} \cdot d_{nail.fl.0\_4}}{2 + \beta_{fl.0\_4}} \cdot \left[\sqrt{2 \cdot \beta_{fl.0\_4} \cdot \left(1 + \beta_{fl.0\_4}\right) \ldots + \frac{4 \cdot \beta_{fl.0\_4} \cdot \left(2 + \beta_{fl.0\_4}\right) \cdot M_{y.Rk.fl0\_4}}{f_{h.kertoQ.k.fl.0\_4} \cdot d_{nail.fl.0\_4} \cdot t_{kertoQ.fl.0\_4}{}^2}} + -\beta_{fl.0\_4}\right] \ldots + \frac{F_{ax.Rk.fl.0\_4}}{4}$$

$$F_{v.Rk.fl0\_4_4} := 1.05 \cdot \frac{f_{h.kertoQ.k.fl.0\_4} \cdot t_{timb.fl0\_4} \cdot d_{nail.fl.0\_4}}{1 + 2\beta_{fl.0\_4}} \cdot \left[\sqrt{2 \cdot \beta_{fl.0\_4}^{2} \cdot \left(1 + \beta_{fl.0\_4}\right) \dots + \frac{4 \cdot \beta_{fl.0\_4} \cdot \left(1 + 2\beta_{fl.0\_4}\right) \cdot M_{y.Rk.fl0\_4}}{f_{h.kertoQ.k.fl.0\_4} \cdot d_{nail.fl.0\_4} \cdot t_{timb.fl0\_4}^{2}}} \dots + -\beta_{fl.0\_4} \right] \dots + \frac{F_{ax.Rk.fl.0\_4}}{4}$$

$$F_{v.Rk.fl0\_4_5} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta_{fl.0\_4}}{1 + \beta_{fl.0\_4}}} \cdot \sqrt{2 \cdot M_{y.Rk.fl0\_4} \cdot f_{h.kertoQ.k.fl.0\_4} \cdot d_{nail.fl.0\_4}} + \frac{F_{ax.Rk.fl.0\_4}}{4}$$

$$F_{v.Rk.fl0\_4} = \begin{pmatrix} 11.139 \\ 9.996 \\ 4.734 \\ 4.656 \\ 4.678 \\ 4.475 \end{pmatrix} \cdot kN$$

For fasteners along the edges of an individual sheet, design lateral load-carrying capacity Should be increased by a factor of 1.2 over the corresponding values.

$$F_{v.Rk.fl.0\_4} := \min\left(F_{v.Rk.fl0\_4}\right) \cdot 1.2 = 5.37 \cdot kN$$

Partial factor for timber C24 (Table 2.3) $\gamma_{m.timb} := 1.3$

Partial factor for OSB (Table 2.3) $\gamma_{m.OSB} := 1.2$

Modification factor taking into account the effect of the duration of load and moisture content (Table 3.1)

$k_{mod.OSB.short} := 0.9$

$k_{mod.timb.short} := 0.9$

$$F_{v.Rd.fl.0\_4} := F_{v.Rk.fl.0\_4} \cdot \frac{\sqrt{k_{mod.timb.short} \cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb}, \gamma_{m.OSB}\right)} = 3.718 \cdot kN$$

$$F_{p.MAX} := \max\left(\frac{F_{p.wall.1}}{2}, \frac{F_{p.wall.3}}{2}\right) = 3.6 \cdot kN$$

$$UR_{ow.fl.0\_4} := \frac{F_{p.MAX}}{F_{v.Rd.fl.0\_4}} = 0.968$$ $F_{v.Rd.fl.0\_4} = 3.718 \cdot kN$

$\frac{F_{p.wall.1}}{2} =$

	0
0	0.7
1	0.7
2	0.9
3	1.25
4	1.6
5	1.95
6	2.3
7	2.65
8	2.95
9	3.2
10	3.45
11	3.6

$\cdot kN$

$\frac{F_{p.wall.3}}{2} =$

	0
0	0.7
1	0.7
2	0.9
3	1.2
4	1.5
5	1.8
6	2.1
7	2.4
8	2.7
9	3
10	3.3
11	3.6

$\cdot kN$

$H_{d.wall.1} =$

	0
0	$2.608\cdot10^4$
1	$5.149\cdot10^4$
2	$5.149\cdot10^4$
3	$4.927\cdot10^4$
4	$4.927\cdot10^4$
5	$4.927\cdot10^4$
6	$4.927\cdot10^4$
7	$4.927\cdot10^4$
8	$4.927\cdot10^4$
9	$4.927\cdot10^4$
10	$4.927\cdot10^4$
11	$4.927\cdot10^4$

N

$UR_{ow.fl.0} := \frac{\left(\frac{F_{p.wall.1}}{2}\right)}{F_{v.Rd.fl.0\_4}} =$

	0
0	0.188
1	0.188
2	0.242
3	0.336
4	0.43
5	0.524
6	0.619
7	0.713
8	0.793
9	0.861
10	0.928
11	0.968

$UR_{ow.fl.0\_4wall3} := \frac{\left(\frac{F_{p.wall.3}}{2}\right)}{F_{v.Rd.fl.0\_4}} =$

	0
0	0.188
1	0.188
2	0.242
3	0.323
4	0.403
5	0.484
6	0.565
7	0.646
8	0.726
9	0.807
10	0.888
11	0.968

Floor 4-7

Thickness of KertoS

$$t_{kertoQ.fl.4\_7} := 55mm$$

Nail diameter

$$d_{nail.fl.4\_7} := 5.7mm$$

Minimum centre distance between nails (Table 8.2)

$$a_{wall.fl4\_7} := \left(4 + \cos(90)\right)\cdot d_{nail.fl.4\_7} = 0.02\,m$$

$$s_{s.wall.1.fl1} = 0.05\,m \qquad OK$$

The pointside penetration length should be at least 8d

$$l_{nail.fl.4\_7.min} := 8d_{nail.fl.4\_7} + t_{kertoQ.fl.4\_7} = 0.101\,m$$

Length of the nail

$$l_{nail.fl.4\_7} := 125mm$$

According to Figure 8.4 EC5

$$t_{timb.fl.4\_7} := l_{nail.fl.4\_7} - t_{kertoQ.fl.4\_7} = 70\cdot mm$$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.fl.4.7} := 0.3\cdot f_u\cdot d_{nail.fl.4\_7}{}^{2.6}\cdot my = 0.022\cdot kN\cdot m$$

$$d_{nail.ow.fl.4\_7} := \frac{d_{nail.fl.4\_7}}{m}\cdot 1000 = 5.7 \qquad t_{kertoS.ow.fl.4\_7} := \frac{t_{kertoQ.fl.4\_7}}{m}\cdot 1000 = 55$$

Caracteristic embedment strength for KertoS (Equation 8.15)

$$f_{h.KertoQ.k.fl.4\_7} := \left(0.082d_{nail.ow.fl.4\_7}{}^{-0.3}\cdot \rho_{k.kertoQ.ow}\right)\frac{N}{(mm)^2} = 24.81\cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.15)

$$f_{h.timb.k.fl.4\_7} := \left(0.082d_{nail.ow.fl.4\_7}{}^{-0.3}\cdot \rho_{k.C24.ow}\right)\cdot\frac{N}{(mm)^2} = 17.026\cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{fl.4\_7} := \frac{f_{h.timb.k.fl.4\_7}}{f_{h.KertoQ.k.fl.4\_7}} = 0.686$$

Withdrawal and pull penetration-through strength

$$f_{axk.fl.4\_7} := 20\cdot 10^{-6}\cdot\left(\rho_{k.C24.ow}\right)^2\cdot MPa = 2.45\cdot MPa$$

Contribution from rope effect According to 8.24

$$F_{ax.Rk.fl.4\_7} := f_{axk.fl.4\_7}\cdot d_{nail.fl.4\_7}\cdot t_{timb.fl.4\_7} = 0.978\cdot kN$$

Characteristic load-carrying capacity per shear plane per fastner (Equation 8.6)

$$F_{\mathrm{v.Rk.fl4\_7}_0} := f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot t_{\mathrm{kertoQ.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}$$

$$F_{\mathrm{v.Rk.fl4\_7}_1} := f_{\mathrm{h.timb.k.fl.4\_7}} \cdot t_{\mathrm{timb.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}$$

$$F_{\mathrm{v.Rk.fl4\_7}_2} := \frac{f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot t_{\mathrm{kertoQ.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}}{1+\beta_{\mathrm{fl.4\_7}}} \left[\sqrt{ \begin{array}{l} \beta_{\mathrm{fl.4\_7}} + 2 \cdot \beta_{\mathrm{fl.4\_7}}^{2} \left[\begin{array}{l} 1 + \dfrac{t_{\mathrm{timb.fl.4\_7}}}{t_{\mathrm{kertoQ.fl.4\_7}}} \ldots \\ + \left(\dfrac{t_{\mathrm{timb.fl.4\_7}}}{t_{\mathrm{kertoQ.fl.4\_7}}} \right)^{2} \end{array} \right] \ldots \ldots \\ + \beta_{\mathrm{fl.4\_7}}^{3} \cdot \left(\dfrac{t_{\mathrm{timb.fl.4\_7}}}{t_{\mathrm{kertoQ.fl.4\_7}}} \right)^{2} \end{array} } \ldots \atop + -\beta_{\mathrm{fl.4\_7}} \cdot \left(1 + \dfrac{t_{\mathrm{timb.fl.4\_7}}}{t_{\mathrm{kertoQ.fl.4\_7}}} \right) \right] \ldots$$

$$+ \frac{F_{\mathrm{ax.Rk.fl.4\_7}}}{4}$$

$$F_{\mathrm{v.Rk.fl4\_7}_3} := 1.05 \cdot \frac{f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot t_{\mathrm{kertoQ.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}}{2+\beta_{\mathrm{fl.4\_7}}} \cdot \left[\sqrt{ \begin{array}{l} 2 \cdot \beta_{\mathrm{fl.4\_7}} \cdot \left(1 + \beta_{\mathrm{fl.4\_7}}\right) \ldots \\ + \dfrac{4 \cdot \beta_{\mathrm{fl.4\_7}} \cdot \left(2 + \beta_{\mathrm{fl.4\_7}}\right) \cdot M_{\mathrm{y.Rk.fl.4.7}}}{f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}} \cdot t_{\mathrm{kertoQ.fl.4\_7}}^{2}} \end{array} } \atop + -\beta_{\mathrm{fl.4\_7}} \right]$$

$$+ \frac{F_{\mathrm{ax.Rk.fl.4\_7}}}{4}$$

$$F_{\mathrm{v.Rk.fl4\_7}_4} := 1.05 \cdot \frac{f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot t_{\mathrm{timb.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}}{1+2\beta_{\mathrm{fl.4\_7}}} \cdot \left[\sqrt{ \begin{array}{l} 2 \cdot \beta_{\mathrm{fl.4\_7}}^{2} \cdot \left(1 + \beta_{\mathrm{fl.4\_7}}\right) \ldots \\ + \dfrac{4 \cdot \beta_{\mathrm{fl.4\_7}} \cdot \left(1 + 2\beta_{\mathrm{fl.4\_7}}\right) \cdot M_{\mathrm{y.Rk.fl.4.7}}}{f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}} \cdot t_{\mathrm{timb.fl.4\_7}}^{2}} \end{array} } \ldots \atop + -\beta_{\mathrm{fl.4\_7}} \right]$$

$$+ \frac{F_{\mathrm{ax.Rk.fl.4\_7}}}{4}$$

$$F_{\mathrm{v.Rk.fl4\_7}_5} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta_{\mathrm{fl.4\_7}}}{1+\beta_{\mathrm{fl.4\_7}}}} \cdot \sqrt{2 \cdot M_{\mathrm{y.Rk.fl.4.7}} \cdot f_{\mathrm{h.KertoQ.k.fl.4\_7}} \cdot d_{\mathrm{nail.fl.4\_7}}} + \frac{F_{\mathrm{ax.Rk.fl.4\_7}}}{4}$$

$$F_{\mathrm{v.Rk.fl4\_7}} = \begin{pmatrix} 7.778 \\ 6.793 \\ 3.236 \\ 3.15 \\ 3.11 \\ 2.842 \end{pmatrix} \cdot \mathrm{kN}$$

For fasteners along the edges of an individual sheet, design lateral load- carrying capacity
Should be increased by a factor of 1.2 over the corresponding values.

$$F_{\mathrm{v.Rk.fl.4\_7}} := \min\left(F_{\mathrm{v.Rk.fl4\_7}}\right) \cdot 1.2 = 3.41 \cdot \mathrm{kN}$$

$$F_{v.Rd.fl.4\_7} := F_{v.Rk.fl.4\_7} \cdot \frac{\sqrt{k_{mod.timb.short} \cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb}, \gamma_{m.OSB}\right)} = 2.361 \cdot kN$$

$$UR_{ow.fl.4\_7} := \frac{2.3kN}{F_{v.Rd.fl.4\_7}} = 0.974$$

$\frac{F_{p.wall.1}}{2} =$

	0
0	0.7
1	0.7
2	0.9
3	1.25
4	1.6
5	1.95
6	2.3
7	2.65
8	2.95
9	3.2
10	3.45
11	3.6

$\cdot kN$

$\frac{F_{p.wall.3}}{2} =$

	0
0	0.7
1	0.7
2	0.9
3	1.2
4	1.5
5	1.8
6	2.1
7	2.4
8	2.7
9	3
10	3.3
11	3.6

$\cdot kN$

$UR_{ow.fl4} := \frac{\left(\frac{F_{p.wall.1}}{2}\right)}{F_{v.Rd.fl.4\_7}} =$

	0
0	0.297
1	0.297
2	0.381
3	0.53
4	0.678
5	0.826
6	0.974
7	1.123
8	1.25
9	1.356
10	1.461
11	1.525

$UR_{ow.fl4.wall3} := \frac{\left(\frac{F_{p.wall.3}}{2}\right)}{F_{v.Rd.fl.4\_7}} =$

	0
0	0.297
1	0.297
2	0.381
3	0.508
4	0.635
5	0.762
6	0.89
7	1.017
8	1.144
9	1.271
10	1.398
11	1.525

Floor 8-11

Thickness of OSB $t_{OSB.fl.8\_11} := 25mm$

Nail diameter $d_{nail.fl.8\_11} := 3mm$

The timber should be pre-drilled whwn the diameter of the nail exceeds 8mm

The timber does not need to be predrilled

Minimum centre distance between nails (Table 8.2) $a_{fl.8\_11} := \left(4 + \cos(90)\right) \cdot d_{nail.fl.8\_11} = 10.656 \cdot mm$

$s_{s.wall.1.fl8} = 0.05\,m$ OK

The pointside penetration length should be at least 8d

$$l_{nail.fl.8\_11.min} := 8d_{nail.fl.8\_11} + t_{OSB.fl.8\_11} = 0.049\,m$$

Length of the nail $l_{nail.fl.8\_11} := 120mm$

Density for OSB $\rho_{k.OSB} := 550\frac{kg}{m^3}$

$$\rho_{k.OSB.ow} := \rho_{k.OSB} \cdot \frac{m^3}{kg} = 550$$

According to Figure 8.4 EC5 $t_{timb.fl.8\_11} := l_{nail.fl.8\_11} - t_{OSB.fl.8\_11} = 95 \cdot mm$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.fl.8\_11} := 0.3 \cdot f_u \cdot d_{nail.fl.8\_11}{}^{2.6} \cdot my = 4.176 \times 10^{-3} \cdot kN \cdot m$$

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

$$d_{nail.ow.fl.8\_11} := \frac{d_{nail.fl.8\_11}}{m} \cdot 1000 = 3 \qquad t_{OSB.ow.fl.8\_11} := \frac{t_{OSB.fl.8\_11}}{m} \cdot 1000 = 25$$

Caracteristic embedment strength for OSB (Equation 8.15)

$$f_{h.OBS.k.fl.8\_11} := \left(0.082d_{nail.ow.fl.8\_11}{}^{-0.3} \cdot \rho_{k.OSB.ow}\right) \frac{N}{(mm)^2} = 32.437 \cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.15)

$$f_{h.timb.k.fl.8\_11} := \left(0.082d_{nail.ow.fl.8\_11}{}^{-0.3} \cdot \rho_{k.C24.ow}\right) \cdot \frac{N}{(mm)^2} = 20.642 \cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{\text{OSB.fl.8\_11}} := \frac{f_{\text{h.timb.k.fl.8\_11}}}{f_{\text{h.OBS.k.fl.8\_11}}} = 0.636$$

Withdrawal and pull penetration-through strength

$$f_{\text{axk.fl.8\_11}} := 20 \cdot 10^{-6} \cdot \left(\rho_{\text{k.C24.ow}}\right)^{2} \cdot \text{MPa} = 2.45 \cdot \text{MPa}$$

Contribution from rope effect According to 8.24

$$F_{\text{ax.Rk.fl.8\_11}} := f_{\text{axk.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}} \cdot t_{\text{timb.fl.8\_11}} = 0.698 \cdot \text{kN}$$

Characteristic load-carrying capacity per shear plane per fastner (equation 8.8)

$$F_{\text{v.Rkfl.8\_11}_{0}} := f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}}$$

$$F_{\text{v.Rkfl.8\_11}_{1}} := f_{\text{h.timb.k.fl.8\_11}} \cdot t_{\text{timb.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}}$$

$$F_{\text{v.Rkfl.8\_11}_{2}} := \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}}}{1 + \beta_{\text{OSB.fl.8\_11}}} \left[\sqrt{\begin{array}{l} \beta_{\text{OSB.fl.8\_11}} \cdots \\ + 2 \cdot \beta_{\text{OSB.fl.8\_11}}^{2} \left[1 + \dfrac{t_{\text{timb.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}} \cdots \\ \quad + \left(\dfrac{t_{\text{timb.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}} \right)^{2} \right] \cdots \\ + \beta_{\text{OSB.fl.8\_11}}^{3} \cdot \left(\dfrac{t_{\text{timb.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}} \right)^{2} \end{array}} \cdots \\ + -\beta_{\text{OSB.fl.8\_11}} \cdot \left(1 + \frac{t_{\text{timb.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}} \right) \right] \cdots$$

$$+ \frac{F_{\text{ax.Rk.fl.8\_11}}}{4}$$

$$F_{\text{v.Rkfl.8\_11.a}} := 1.05 \cdot \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}}}{2 + \beta_{\text{OSB.fl.8\_11}}}$$

$$F_{\text{v.Rkfl.8\_11.b}} := \left[\sqrt{\begin{array}{l} 2 \cdot \beta_{\text{OSB.fl.8\_11}} \cdot \left(1 + \beta_{\text{OSB.fl.8\_11}}\right) \cdots \\ + \dfrac{4 \cdot \beta_{\text{OSB.fl.8\_11}} \cdot \left(2 + \beta_{\text{OSB.fl.8\_11}}\right) \cdot M_{\text{y.Rk.fl.8\_11}}}{f_{\text{h.OBS.k.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}}^{2}} \end{array}} - \beta_{\text{OSB.fl.8\_11}} \right]$$

$$F_{\text{v.Rkfl.8\_11.c}} := \frac{F_{\text{ax.Rk.fl.8\_11}}}{4}$$

$$F_{\text{v.Rkfl.8\_11}_{3}} := F_{\text{v.Rkfl.8\_11.a}} \cdot F_{\text{v.Rkfl.8\_11.b}} + F_{\text{v.Rkfl.8\_11.c}}$$

$$F_{\text{v.Rkfl.8\_11.d}} := 1.05 \cdot \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{timb.fl.8\_11}} \cdot d_{\text{nail.fl.8\_11}}}{2 + \beta_{\text{OSB.fl.8\_11}}}$$

$$F_{v.Rkfl.8\_11.e} := \left[\sqrt{\begin{array}{l}2\cdot\beta_{OSB.fl.8\_11}{}^{2}\cdot\left(1+\beta_{OSB.fl.8\_11}\right)\ \ldots \\ +\dfrac{4\cdot\beta_{OSB.fl.8\_11}\cdot\left(1+2\beta_{OSB.fl.8\_11}\right)\cdot M_{y.Rk.fl.8\_11}}{f_{h.OBS.k.fl.8\_11}\cdot d_{nail.fl.8\_11}\cdot t_{timb.fl.8\_11}{}^{2}}\end{array}} - \beta_{OSB.fl.8\_11}\right]$$

$$F_{v.Rkfl.8\_11.f} := \frac{F_{ax.Rk.fl.8\_11}}{4}$$

$$F_{v.Rkfl.8\_11_4} := F_{v.Rkfl.8\_11.d}\cdot F_{v.Rkfl.8\_11.e} + F_{v.Rkfl.8\_11.f}$$

$$F_{v.Rkfl.8\_11_5} := 1.15\cdot\sqrt{\frac{2\cdot\beta_{OSB.fl.8\_11}}{1+\beta_{OSB.fl.8\_11}}}\cdot\sqrt{2\cdot M_{y.Rk.fl.8\_11}\cdot f_{h.OBS.k.fl.8\_11}\cdot d_{nail.fl.8\_11}} + \frac{F_{ax.Rk.fl.8\_11}}{4}$$

$$F_{v.Rkfl.8\_11} = \begin{pmatrix}2.433\\5.883\\2.272\\1.103\\2.114\\1.089\end{pmatrix}\cdot kN$$

For fasteners along the edges of an individual sheet, design lateral load- carrying capacity Should be increased by a factor of 1.2 over the corresponding values.

$$F_{v.Rk.fl.8\_11} := \min\left(F_{v.Rkfl.8\_11}\right)\cdot 1.2 = 1.307\cdot kN$$

$$F_{v.Rd.outerwall.fl.8\_11} := F_{v.Rk.fl.8\_11}\cdot\frac{\sqrt{k_{mod.timb.short}\cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb},\gamma_{m.OSB}\right)} = 0.905\cdot kN$$

$$UR_{ow.fl.8\_11} := \frac{0.9kN}{F_{v.Rd.outerwall.fl.8\_11}} = 0.995$$

$$UR_{ow.fl8} := \frac{\left(\dfrac{F_{p.wall.1}}{2}\right)}{F_{v.Rd.outerwall.fl.8\_11}} =$$

	0
0	0.774
1	0.774
2	0.995
3	1.382
4	1.769
5	2.156
6	2.543
7	2.93
8	3.261
9	3.538
10	3.814
11	3.98

$$UR_{ow.fl8.wall3} := \frac{\left(\dfrac{F_{p.wall.3}}{2}\right)}{F_{v.Rd.outerwall.fl.8\_11}} =$$

	0
0	0.774
1	0.774
2	0.995
3	1.327
4	1.658
5	1.99
6	2.322
7	2.653
8	2.985
9	3.316
10	3.648
11	3.98

H Joints

Wall 1 and 3 - OSB

Floor 0-4

Thickness of OSB $t_{OSB} := 20mm$

Nail diameter $d_{nail.outerwall} := 14mm$

The timber should be pre-drilled whwn the diameter of the nail exceeds 8mm

The timber should be predrilled

Minimum centre distance between nails (Table 8.2)

$$a_{wall.1} := (4 + \cos(90))\cdot d_{nail.outerwall} = 0.05\,m$$

$$s_{s.wall.1.fl1} = 0.05\,m \qquad \text{OK}$$

The pointside penetration length should be at least 8d

Length of the nail $l_{nail.outerwall} := 8d_{nail.outerwall} + t_{OSB} = 0.132\,m$

Caracteristic density for C24 $\rho_{k.C24} := 350\dfrac{kg}{m^3}$

Density for OSB $\rho_{k.OSB} := 550\dfrac{kg}{m^3}$

According to Figure 8.4 EC5 $t_{timb.outerwall} := l_{nail.outerwall} - t_{OSB} = 112\cdot mm$

Characteristic tensile strength of the wire $f_u := 800\dfrac{N}{mm^2}$

Unit correction factors for mathcad

$$m_{emb} := \frac{m^{0.3}m^3}{kg}\cdot MPa \qquad my := mm^{0.4}$$

$$d_{nail.ow} := \frac{d_{nail.outerwall}}{m}\cdot 1000 = 14 \qquad t_{OSB.ow} := \frac{t_{OSB}}{m}\cdot 1000 = 20$$

$$\rho_{k.C24.ow} := \rho_{k.C24}\cdot\frac{m^3}{kg} = 350$$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.outerwall} := 0.3\cdot f_u\cdot d_{nail.outerwall}^{\;2.6}\cdot my = 0.229\cdot kN\cdot m$$

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

Caracteristic embedment strength for OSB (Equation 8.37)

$$f_{h.OBS.k} := \left(50 \cdot d_{nail.ow}{}^{-0.6} \cdot t_{OSB.ow}{}^{0.2}\right) \frac{N}{(mm)^2} = 18.685 \cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.32)

$$f_{h.timb.k.outerwall} := \left[0.082\left(1 - 0.01 \cdot d_{nail.ow}\right) \cdot \rho_{k.C24.ow}\right] \cdot \frac{N}{(mm)^2} = 24.682 \cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{OSB} := \frac{f_{h.timb.k.outerwall}}{f_{h.OBS.k}} = 1.321$$

$F_{ax.Rk} := 0$ According to 8.2.2 (2)

Characteristic load-carrying capacity per shear plane per fastner (Equation 8.6)

$$F_{v.Rk.outerwall_0} := f_{h.OBS.k} \cdot t_{OSB} \cdot d_{nail.outerwall}$$

$$F_{v.Rk.outerwall_1} := f_{h.timb.k.outerwall} \cdot t_{timb.outerwall} \cdot d_{nail.outerwall}$$

$$F_{v.Rk.outerwall.a} := \frac{f_{h.OBS.k} \cdot t_{OSB} \cdot d_{nail.outerwall}}{1 + \beta_{OSB}}$$

$$F_{v.Rk.outerwall.b} := \sqrt{\beta_{OSB} + 2 \cdot \beta_{OSB}{}^2 \left[\left(1 + \frac{t_{timb.outerwall}}{t_{OSB}}\right) \ldots + \left(\frac{t_{timb.outerwall}}{t_{OSB}}\right)^2\right] \ldots + \beta_{OSB}{}^3 \cdot \left(\frac{t_{timb.outerwall}}{t_{OSB}}\right)^2} \ldots - \beta_{OSB} \cdot \left(1 + \frac{t_{timb.outerwall}}{t_{OSB}}\right)$$

$$F_{v.Rk.outerwall.c} := \frac{F_{ax.Rk}}{4}$$

$$F_{v.Rk.outerwall_2} := F_{v.Rk.outerwall.a} \cdot F_{v.Rk.outerwall.b} + F_{v.Rk.outerwall.c}$$

$$F_{v.Rk.outerwall.d} := 1.05 \cdot \frac{f_{h.OBS.k} \cdot t_{OSB} \cdot d_{nail.outerwall}}{2 + \beta_{OSB}}$$

$$F_{v.Rk.outerwall.e} := \left[\sqrt{2 \cdot \beta_{OSB} \cdot \left(1 + \beta_{OSB}\right) \ldots + \frac{4 \cdot \beta_{OSB} \cdot \left(2 + \beta_{OSB}\right) \cdot M_{y.Rk.outerwall}}{f_{h.OBS.k} \cdot d_{nail.outerwall} \cdot t_{OSB}{}^2}} - \beta_{OSB}\right]$$

$$F_{v.Rk.outerwall_3} := F_{v.Rk.outerwall.d} \cdot F_{v.Rk.outerwall.e} + F_{v.Rk.outerwall.c}$$

$$F_{v.Rk.outerwall.f} := 1.05\cdot\frac{f_{h.OBS.k}\cdot t_{timb.outerwall}\cdot d_{nail.outerwall}}{2+\beta_{OSB}}$$

$$F_{v.Rk.outerwall.g} := \left[\sqrt{2\cdot\beta_{OSB}^{2}\cdot\left(1+\beta_{OSB}\right)\ ... + \frac{4\cdot\beta_{OSB}\cdot\left(1\ ...\ + 2\beta_{OSB}\right)\cdot M_{y.Rk.outerwall}}{f_{h.OBS.k}\cdot d_{nail.outerwall}\cdot t_{timb.outerwall}^{2}}} - \beta_{OSB}\right]$$

$$F_{v.Rk.outerwall_4} := F_{v.Rk.outerwall.f}\cdot F_{v.Rk.outerwall.g} + F_{v.Rk.outerwall.c}$$

$$F_{v.Rk.outerwall_5} := 1.15\cdot\sqrt{\frac{2\cdot\beta_{OSB}}{1+\beta_{OSB}}}\cdot\sqrt{2\cdot M_{y.Rk.outerwall}\cdot f_{h.OBS.k}\cdot d_{nail.outerwall}} + \frac{F_{ax.Rk}}{4}$$

$$F_{v.Rk.outerwall} = \begin{pmatrix} 5.232 \\ 38.701 \\ 12.707 \\ 8.857 \\ 16.23 \\ 13.435 \end{pmatrix}\cdot kN$$

$$F_{v.Rk.outerwall} = \begin{pmatrix} 5.232 \\ 38.701 \\ 12.707 \\ 8.857 \\ 16.23 \\ 13.435 \end{pmatrix}\cdot kN$$

$$F_{v.Rk.outerwall} := \min\left(F_{v.Rk.outerwall}\right) = 5.232\cdot kN$$

Partial factor for timber C24 (Table 2.3) $\gamma_{m.timb} := 1.3$

Partial factor for OSB (Table 2.3) $\gamma_{m.OSB} := 1.2$

Modification factor taking into account the effect of the duration of load and moisture content (Table 3.1)

$k_{mod.OSB.short} := 0.9$

$k_{mod.timb.short} := 0.9$

$$F_{v.Rd.outerwall} := F_{v.Rk.outerwall}\cdot\frac{\sqrt{k_{mod.timb.short}\cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb},\gamma_{m.OSB}\right)} = 3.622\cdot kN$$

$$F_{p.MAX} := \max\left(\frac{F_{p.wall.1}}{2},\frac{F_{p.wall.3}}{2}\right) = 3.6\cdot kN$$

$$UR_{ow.fl.0\_3} := \frac{3.6kN}{F_{v.Rd.outerwall}} = 0.994$$

Floor 4-7

Thickness of OSB $t_{OSB.fl.4\_7} := 20mm$

Nail diameter $d_{nail.outerwall.fl.4\_7} := 8mm$

The timber should be pre-drilled whwn the diameter of the nail exceeds 8mm

The timber does not need to be predrilled

Minimum centre distance between nails
(Table 8.2)

$$a_{wall.1.fl4\_7} := (4 + \cos(90))\cdot d_{nail.outerwall.fl.4\_7} = 0.028\,m$$

$$s_{s.wall.1.fl1} = 0.05\,m \qquad OK$$

The pointside penetration length should be at least 8d

$$l_{nail.outerwall.fl.4\_7.min} := 8d_{nail.outerwall.fl.4\_7} + t_{OSB} = 0.084\,m$$

Length of the nail $l_{nail.outerwall.fl.4\_7} := 90mm$

According to Figure 8.4 EC5

$$t_{timb.outerwall.fl.4\_7} := l_{nail.outerwall.fl.4\_7} - t_{OSB.fl.4\_7} = 70\cdot mm$$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.outerwall.fl.4.7} := 0.3\cdot f_u\cdot d_{nail.outerwall.fl.4\_7}{}^{2.6}\cdot my = 0.053\cdot kN\cdot m$$

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

$$d_{nail.ow.fl.4\_7} := \frac{d_{nail.outerwall.fl.4\_7}}{m}\cdot 1000 = 8 \qquad t_{OSB.ow.fl.4\_7} := \frac{t_{OSB.fl.4\_7}}{m}\cdot 1000 = 20$$

Caracteristic embedment strength for OSB (Equation 8.22)

$$f_{h.OBS.k.fl.4\_7} := \left(65\cdot d_{nail.ow.fl.4\_7}{}^{-0.7}\cdot t_{OSB.ow.fl.4\_7}{}^{0.1}\right)\frac{N}{(mm)^2} = 20.458\cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.15)

$$f_{h.timb.k.outerwall.fl.4\_7} := \left(0.082d_{nail.ow.fl.4\_7}{}^{-0.3}\cdot\rho_{k.C24.ow}\right)\cdot\frac{N}{(mm)^2} = 15.38\cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{OSB.fl.4\_7} := \frac{f_{h.timb.k.outerwall.fl.4\_7}}{f_{h.OBS.k.fl.4\_7}} = 0.752$$

$F_{ax.Rk} := 0$ According to 8.2.2 (2)

Characteristic load-carrying capacity per shear plane per fastner (Equation 8.6)

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7}_0} := f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot t_{\mathrm{OSB.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7}_1} := f_{\mathrm{h.timb.k.outerwall.fl.4\_7}} \cdot t_{\mathrm{timb.outerwall.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.a}} := \frac{f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot t_{\mathrm{OSB.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}}{1 + \beta_{\mathrm{OSB.fl.4\_7}}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.b}} := \left[\sqrt{\begin{aligned} &\beta_{\mathrm{OSB.fl.4\_7}} + 2 \cdot \beta_{\mathrm{OSB.fl.4\_7}}{}^2 \left[\begin{aligned} &1 + \frac{t_{\mathrm{timb.outerwall.fl.4\_7}}}{t_{\mathrm{OSB.fl.4\_7}}} \cdots \\ &+ \left(\frac{t_{\mathrm{timb.outerwall.fl.4\_7}}}{t_{\mathrm{OSB.fl.4\_7}}} \right)^2 \end{aligned} \right] \cdots \cdots \\ &+ \beta_{\mathrm{OSB.fl.4\_7}}{}^3 \cdot \left(\frac{t_{\mathrm{timb.outerwall.fl.4\_7}}}{t_{\mathrm{OSB.fl.4\_7}}} \right)^2 \end{aligned}} + -\beta_{\mathrm{OSB.fl.4\_7}} \cdot \left(1 + \frac{t_{\mathrm{timb.outerwall.fl.4\_7}}}{t_{\mathrm{OSB.fl.4\_7}}} \right) \right]$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.c}} := \frac{F_{\mathrm{ax.Rk}}}{4}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7}_2} := F_{\mathrm{v.Rk.outerwall.fl.4\_7.a}} \cdot F_{\mathrm{v.Rk.outerwall.fl.4\_7.b}} + F_{\mathrm{v.Rk.outerwall.fl.4\_7.c}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.d}} := 1.05 \cdot \frac{f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot t_{\mathrm{OSB.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}}{2 + \beta_{\mathrm{OSB.fl.4\_7}}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.e}} := \left[\sqrt{\begin{aligned} &2 \cdot \beta_{\mathrm{OSB.fl.4\_7}} \cdot \left(1 + \beta_{\mathrm{OSB.fl.4\_7}} \right) \cdots \\ &+ \frac{4 \cdot \beta_{\mathrm{OSB.fl.4\_7}} \cdot \left(2 + \beta_{\mathrm{OSB.fl.4\_7}} \right) \cdot M_{\mathrm{y.Rk.outerwall.fl.4.7}}}{f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}} \cdot t_{\mathrm{OSB.fl.4\_7}}{}^2} \end{aligned}} - \beta_{\mathrm{OSB.fl.4\_7}} \right]$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7}_3} := F_{\mathrm{v.Rk.outerwall.fl.4\_7.d}} \cdot F_{\mathrm{v.Rk.outerwall.fl.4\_7.e}} + F_{\mathrm{v.Rk.outerwall.fl.4\_7.c}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.f}} := 1.05 \cdot \frac{f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot t_{\mathrm{timb.outerwall.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}}{2 + \beta_{\mathrm{OSB.fl.4\_7}}}$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7.g}} := \left[\sqrt{\begin{aligned} &2 \cdot \beta_{\mathrm{OSB.fl.4\_7}}{}^2 \cdot \left(1 + \beta_{\mathrm{OSB.fl.4\_7}} \right) \cdots \\ &+ \frac{4 \cdot \beta_{\mathrm{OSB.fl.4\_7}} \cdot \left(1 + 2\beta_{\mathrm{OSB.fl.4\_7}} \right) \cdot M_{\mathrm{y.Rk.outerwall.fl.4.7}}}{f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}} \cdot t_{\mathrm{timb.outerwall.fl.4\_7}}{}^2} \end{aligned}} - \beta_{\mathrm{OSB.fl.4\_7}} \right]$$

$$F_{\mathrm{v.Rk.outerwall.fl.4\_7}_4} := F_{\mathrm{v.Rk.outerwall.fl.4\_7.f}} \cdot F_{\mathrm{v.Rk.outerwall.fl.4\_7.g}} + F_{\mathrm{v.Rk.outerwall.fl.4\_7.c}}$$

$$\begin{aligned} F_{\mathrm{v.Rk.outerwall.fl.4\_7}_5} := 1.15 \cdot &\sqrt{\frac{2 \cdot \beta_{\mathrm{OSB.fl.4\_7}}}{1 + \beta_{\mathrm{OSB.fl.4\_7}}}} \cdot \sqrt{2 \cdot M_{\mathrm{y.Rk.outerwall.fl.4.7}} \cdot f_{\mathrm{h.OBS.k.fl.4\_7}} \cdot d_{\mathrm{nail.outerwall.fl.4\_7}}} \cdots \\ &+ \frac{F_{\mathrm{ax.Rk}}}{4} \end{aligned}$$

$$F_{v.Rk.outerwall.fl.4\_7} = \begin{pmatrix} 3.273 \\ 8.613 \\ 3.001 \\ 2.889 \\ 3.601 \\ 4.458 \end{pmatrix} \cdot kN$$

$$F_{v.Rk.outerwall.fl.4\_7} := \min\left(F_{v.Rk.outerwall.fl.4\_7}\right) = 2.889 \cdot kN$$

$$F_{v.Rd.outerwall.fl.4\_7} := F_{v.Rk.outerwall.fl.4\_7} \cdot \frac{\sqrt{k_{mod.timb.short} \cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb}, \gamma_{m.OSB}\right)} = 2 \cdot kN$$

$$UR_{ow.fl.4\_7} := \frac{1.95kN}{F_{v.Rd.outerwall.fl.4\_7}} = 0.975$$

Floor 8-11

Thickness of OSB $t_{OSB.fl.8\_11} := 20mm$

Nail diameter $d_{nail.outerwall.fl.8\_11} := 5mm$

The timber should be pre-drilled whwn the diameter of the nail exceeds 8mm

The timber does not need to be predrilled

Minimum centre distance between nails (Table 8.2)

$$a_{wall.1.fl.8\_11} := (4 + \cos(90)) \cdot d_{nail.outerwall.fl.8\_11} = 0.018\,m$$

$$s_{s.wall.1.fl1} = 0.05\,m \qquad \text{OK}$$

The pointside penetration length should be at least 8d

$$l_{nail.outerwall.fl.8\_11.min} := 8d_{nail.outerwall.fl.8\_11} + t_{OSB} = 0.06\,m$$

Length of the nail $l_{nail.outerwall.fl.8\_11} := 60mm$

Minimum thickness of timber member to avoid pre-drilling (Equation)

$$t := \max\left[7 \cdot d_{nail.outerwall}, \left[(13 \cdot d_{nail.outerwall} - 30mm) \cdot \frac{\rho_{k.C24}}{400} \cdot \frac{m^3}{kg}\right]\right] = 0.133 \cdot m$$

According to Figure 8.4 EC5

$$t_{timb.outerwall.fl.8\_11} := l_{nail.outerwall.fl.8\_11} - t_{OSB.fl.8\_11} = 40 \cdot mm$$

Characteristic yield moment of fasteners, round nails (Equation 8.14)

$$M_{y.Rk.outerwall.fl.8\_11} := 0.3 \cdot f_u \cdot d_{nail.outerwall.fl.8\_11}^{\ 2.6} \cdot my = 0.016 \cdot kN \cdot m$$

For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

$$d_{nail.ow.fl.8\_11} := \frac{d_{nail.outerwall.fl.8\_11}}{m} \cdot 1000 = 5 \qquad t_{OSB.ow.fl.8\_11} := \frac{t_{OSB.fl.8\_11}}{m} \cdot 1000 = 20$$

Caracteristic embedment strength for OSB (Equation 8.22)

$$f_{h.OBS.k.fl.8\_11} := \left(65 \cdot d_{nail.ow.fl.8\_11}^{\ -0.7} \cdot t_{OSB.ow.fl.8\_11}^{\ 0.1}\right) \frac{N}{(mm)^2} = 28.427 \cdot MPa$$

Caracteristic embedment strength for C24 (Equation 8.15)

$$f_{h.timb.k.outerwall.fl.8\_11} := \left(0.082 d_{nail.ow.fl.8\_11}^{\ -0.3} \cdot \rho_{k.C24.ow}\right) \cdot \frac{N}{(mm)^2} = 17.709 \cdot MPa$$

Ratio between embedment strength of the members (equation 8.8)

$$\beta_{\text{OSB.fl.8\_11}} := \frac{f_{\text{h.timb.k.outerwall.fl.8\_11}}}{f_{\text{h.OBS.k.fl.8\_11}}} = 0.623$$

$F_{\text{ax.Rk}} := 0$ According to 8.2.2 (2)

Characteristic load-carrying capacity per shear plane per fastner (equation 8.8)

$$F_{\text{v.Rk.outerwall.fl.8\_11}_0} := f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11}_1} := f_{\text{h.timb.k.outerwall.fl.8\_11}} \cdot t_{\text{timb.outerwall.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.a}} := \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}}}{1 + \beta_{\text{OSB.fl.8\_11}}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.b}} := \left[\sqrt{\begin{array}{l}\beta_{\text{OSB.fl.8\_11}} + 2 \cdot \beta_{\text{OSB.fl.8\_11}}{}^2 \left[\begin{array}{l} 1 + \dfrac{t_{\text{timb.outerwall.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}} \ldots \\ + \left(\dfrac{t_{\text{timb.outerwall.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}}\right)^2 \end{array}\right] \ldots \ldots \\ + \beta_{\text{OSB.fl.8\_11}}{}^3 \cdot \left(\dfrac{t_{\text{timb.outerwall.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}}\right)^2 \end{array}} \ldots \atop + -\beta_{\text{OSB.fl.8\_11}} \cdot \left(1 + \dfrac{t_{\text{timb.outerwall.fl.8\_11}}}{t_{\text{OSB.fl.8\_11}}}\right)\right]$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.c}} := \frac{F_{\text{ax.Rk}}}{4}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11}_2} := F_{\text{v.Rk.outerwall.fl.8\_11.a}} \cdot F_{\text{v.Rk.outerwall.fl.8\_11.b}} + F_{\text{v.Rk.outerwall.fl.8\_11.c}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.d}} := 1.05 \cdot \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}}}{2 + \beta_{\text{OSB.fl.8\_11}}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.e}} := \left[\sqrt{\begin{array}{l} 2 \cdot \beta_{\text{OSB.fl.8\_11}} \cdot \left(1 + \beta_{\text{OSB.fl.8\_11}}\right) \ldots \\ + \dfrac{4 \cdot \beta_{\text{OSB.fl.8\_11}} \cdot \left(2 + \beta_{\text{OSB.fl.8\_11}}\right) \cdot M_{\text{y.Rk.outerwall.fl.8\_11}}}{f_{\text{h.OBS.k.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}} \cdot t_{\text{OSB.fl.8\_11}}{}^2} \end{array}} - \beta_{\text{OSB.fl.8\_11}}\right]$$

$$F_{\text{v.Rk.outerwall.fl.8\_11}_3} := F_{\text{v.Rk.outerwall.fl.8\_11.d}} \cdot F_{\text{v.Rk.outerwall.fl.8\_11.e}} + F_{\text{v.Rk.outerwall.fl.8\_11.c}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.f}} := 1.05 \cdot \frac{f_{\text{h.OBS.k.fl.8\_11}} \cdot t_{\text{timb.outerwall.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}}}{2 + \beta_{\text{OSB.fl.8\_11}}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11.g}} := \left[\sqrt{\begin{array}{l} 2 \cdot \beta_{\text{OSB.fl.8\_11}}{}^2 \cdot \left(1 + \beta_{\text{OSB.fl.8\_11}}\right) \ldots \\ + \dfrac{4 \cdot \beta_{\text{OSB.fl.8\_11}} \cdot \left(1 + 2\beta_{\text{OSB.fl.8\_11}}\right) \cdot M_{\text{y.Rk.outerwall.fl.8\_11}}}{f_{\text{h.OBS.k.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_11}} \cdot t_{\text{timb.outerwall.fl.8\_11}}{}^2} \end{array}} - \beta_{\text{OSB.fl.8\_11}}\right]$$

$$F_{\text{v.Rk.outerwall.fl.8\_11}_4} := F_{\text{v.Rk.outerwall.fl.8\_11.f}} \cdot F_{\text{v.Rk.outerwall.fl.8\_11.g}} + F_{\text{v.Rk.outerwall.fl.8\_11.c}}$$

$$F_{\text{v.Rk.outerwall.fl.8\_11}_5} := 1.15 \cdot \sqrt{\frac{2 \cdot \beta_{\text{OSB.fl.8\_11}}}{1 + \beta_{\text{OSB.fl.8\_11}}}} \cdot \sqrt{2 \cdot M_{\text{y.Rk.outerwall.fl.8\_11}} \cdot f_{\text{h.OBS.k.fl.8\_11}} \cdot d_{\text{nail.outerwall.fl.8\_1}}} \ldots \\ + \frac{F_{\text{ax.Rk}}}{4}$$

$$F_{v.Rk.outerwall.fl.8\_11} = \begin{pmatrix} 2.843 \\ 3.542 \\ 1.368 \\ 1.519 \\ 1.503 \\ 2.133 \end{pmatrix} \cdot kN$$

$$F_{v.Rk.outerwall.fl.8\_11} := \min\left(F_{v.Rk.outerwall.fl.8\_11}\right) = 1.368 \cdot kN$$

$$F_{v.Rd.outerwall.fl.8\_11} := F_{v.Rk.outerwall.fl.8\_11} \cdot \frac{\sqrt{k_{mod.timb.short} \cdot k_{mod.OSB.short}}}{\max\left(\gamma_{m.timb}, \gamma_{m.OSB}\right)} = 0.947 \cdot kN$$

$$UR_{ow.fl.8\_11} := \frac{0.9kN}{F_{v.Rd.outerwall.fl.8\_11}} = 0.95$$

I Control of boards

Outerwall - OSB

Material Parameters

Thickness of OSB $t_{OSB} = 0.02\,\mathrm{m}$

Area of OSB $A_{OSB.ef} := t_{OSB} \cdot h = 0.06\,\mathrm{m}^2$

Characteristic strength

Characteristic strength - Compression $f_{c.90.k.OSB} := 12.7\mathrm{MPa}$

Characteristic strength - Tension $f_{t.90.k.OSB} := 7\mathrm{MPa}$

Characteristic strength - Panel shear $f_{v.k.OSB} := 6.8\mathrm{MPa}$

Design strength

Design value of compression strength

$$f_{c.90.d.OSB} := \frac{k_{mod.OSB.short} f_{c.90.k.OSB}}{\gamma_{m.OSB}} = 9.525 \cdot \mathrm{MPa}$$

Design value of tensile strength

$$f_{t.90.d.OSB} := \frac{k_{mod.OSB.short} f_{t.90.k.OSB}}{\gamma_{m.OSB}} = 5.25 \cdot \mathrm{MPa}$$

Design value of panel shear

$$f_{v.d.OSB} := \frac{k_{mod.OSB.short} f_{v.k.OSB}}{\gamma_{m.OSB}} = 5.1 \cdot \mathrm{MPa}$$

Shear flow

$$f_{p.wall.1} := \frac{F_{p.wall.1}}{s_{s.wall.1}} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 9.333 \\ \hline 1 & 28 \\ \hline 2 & 36 \\ \hline 3 & 50 \\ \hline 4 & 64 \\ \hline 5 & 78 \\ \hline 6 & 92 \\ \hline 7 & 106 \\ \hline 8 & 118 \\ \hline 9 & 128 \\ \hline 10 & 138 \\ \hline 11 & 144 \\ \hline \end{array} \cdot \frac{\mathrm{kN}}{\mathrm{m}} \qquad f_{p.wall.3} := \frac{F_{p.wall.3}}{s_{s.wall.3}} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 7.778 \\ \hline 1 & 23.333 \\ \hline 2 & 36 \\ \hline 3 & 48 \\ \hline 4 & 60 \\ \hline 5 & 72 \\ \hline 6 & 84 \\ \hline 7 & 96 \\ \hline 8 & 108 \\ \hline 9 & 120 \\ \hline 10 & 132 \\ \hline 11 & 144 \\ \hline \end{array} \cdot \frac{\mathrm{kN}}{\mathrm{m}}$$

Shear flow force per unit length

$$f_{p.wall.1.fl.0} := \frac{\max\left(f_{p.wall.1}, f_{p.wall.1}\right)}{2} = 72 \cdot \frac{\mathrm{kN}}{\mathrm{m}}$$

Normal force $N_{d.wall.1} := f_{p.wall.1.fl.0} \cdot h = 214.2 \cdot \mathrm{kN}$

Stresses

Compression $\sigma_{c.90.d.wall.1} := \frac{N_{d.wall.1}}{A_{OSB.ef}} = 3.6 \cdot MPa$

Tension $\sigma_{t.90.d.wall.1} := \frac{N_{d.wall.1}}{A_{OSB.ef}} = 3.6 \cdot MPa$

Panel shear $\sigma_{v.90.d.wall.1} := \frac{N_{d.wall.1}}{A_{OSB.ef}} = 3.6 \cdot MPa$

Compression Resistance $\frac{\sigma_{c.90.d.wall.1}}{f_{c.90.d.OSB}} = 0.378$

Tension Resistance $\frac{\sigma_{t.90.d.wall.1}}{f_{t.90.d.OSB}} = 0.686$

Panel Resistance $\frac{\sigma_{v.90.d.wall.1}}{f_{v.d.OSB}} = 0.706$ OK

J Stiffness of wall segment

Floor 0-4

Thickness of OSB $t_{OSB} = 0.02\,m$

Caracteristic density for C24 $\rho_{k.C24.ow} := \rho_{k.C24} \cdot \frac{m^3}{kg} = 350$

Density for OSB $\rho_{k.OSB.ow} := \rho_{k.OSB} \cdot \frac{m^3}{kg} = 550$

Nail diameter $d_{nail.ow} := \frac{d_{nail.outerwall}}{m} \cdot 1000 = 14$

$\kappa_s := 1$

The slip modules (Table 7.1) $K_{ser.wall.1} := \left[\left(\sqrt{\rho_{k.C24.ow}\,\rho_{k.OSB.ow}} \right)^{1.5} \cdot \frac{d_{nail.ow}}{23} \right] \cdot \frac{N}{mm}$

$K_{ser.wall.1} = 5.594 \cdot \frac{kN}{mm}$

Displacement module $k_{wall.1} := \kappa_s \cdot K_{ser.wall.1} = 5.594 \cdot \frac{kN}{mm}$

Horisontal load $H_d := 1N$

Width of sheeting $b_b := 1200mm$

Characteristic shear modulus for OSB $G_k := 1080 \frac{N}{mm^2}$

Center distance between nails $s_{s.wall.1.fl1} = 0.05\,m$

Horizontal displacement of the header plate

$$u_{wall.1} := 4.5 \cdot \frac{s_{s.wall.1.fl1}}{b_b} \frac{H_d}{k_{wall.1} \cdot \frac{2}{3}} + \frac{H_d \cdot h}{G_k \cdot b_b \cdot t_{OSB}} = 1.651 \times 10^{-7}\,m$$

Stiffness of wall $k_{wall.1.fl.0\_4} := \frac{H_d}{u_{wall.1}} = 6.059 \times 10^{3} \cdot \frac{kN}{m}$

Floor 4-7

Thickness of OSB $t_{OSB.fl.4\_7} = 0.02\,m$

Nail diameter $d_{nail.ow.fl.4\_7} = 8$

The slip modules (Table 7.1) $K_{ser.wall.1.fl.4\_7} := \left[\left(\sqrt{\rho_{k.C24.ow}\rho_{k.OSB.ow}}\right)^{1.5} \cdot \frac{d_{nail.ow.fl.4\_7}}{23}\right] \cdot \frac{N}{mm}$

$K_{ser.wall.1.fl.4\_7} = 3.197 \cdot \frac{kN}{mm}$

Displacement module $k_{wall.1.fl.4\_7} := \kappa_s \cdot K_{ser.wall.1.fl.4\_7} = 3.197 \cdot \frac{kN}{mm}$

Horisontal load $H_d := 1N$

Center distance between nails $s_{s.wall.1.fl4} = 0.05\,m$

Horizontal displacement of the header plate

$$u_{wall.1.fl.4\_7} := 4.5 \cdot \frac{s_{s.wall.1.fl4}}{b_b} \frac{H_d}{k_{wall.1.fl.4\_7} \cdot \frac{2}{3}} + \frac{H_d \cdot h}{G_k \cdot b_b \cdot t_{OSB.fl.4\_7}} = 2.028 \times 10^{-7}\,m$$

Stiffness of wall $K_{wall.1.fl.4\_7} := \frac{H_d}{u_{wall.1.fl.4\_7}} = 4.932 \times 10^{3} \cdot \frac{kN}{m}$

Floor 8-11

Thickness of OSB $t_{OSB.fl.8\_11} = 0.02\,m$

Nail diameter $d_{nail.ow.fl.8\_11} = 5$

The slip modules (Table 7.1) $K_{ser.wall.1.fl.8\_11} := \left[\left(\sqrt{\rho_{k.C24.ow}\rho_{k.OSB.ow}}\right)^{1.5} \cdot \frac{d_{nail.ow.fl.8\_11}}{23}\right] \cdot \frac{N}{mm}$

$K_{ser.wall.1.fl.8\_11} = 1.998 \cdot \frac{kN}{mm}$

Displacement module $k_{wall.1.fl.8\_11} := \kappa_s \cdot K_{ser.wall.1.fl.8\_11} = 1.998 \cdot \frac{kN}{mm}$

Horisontal load $H_d := 1N$

Center distance between nails $s_{s.wall.1.fl10} = 0.05\,m$

Horizontal displacement of the header plate

$$u_{wall.1.fl.8\_11} := 4.5 \cdot \frac{s_{s.wall.1.fl8}}{b_b} \frac{H_d}{k_{wall.1.fl.8\_11} \cdot \frac{2}{3}} + \frac{H_d \cdot h}{G_k \cdot b_b \cdot t_{OSB.fl.8\_11}} = 2.556 \times 10^{-7}\,m$$

Stiffness of wall $K_{wall.1.fl.8\_11} := \frac{H_d}{u_{wall.1.fl.8\_11}} = 3.913 \times 10^{3} \cdot \frac{kN}{m}$

K Reactionforces in wall 1

Part 1

$$R_{12.wall.1.1.tot} := \begin{pmatrix} R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl11} \\ R_{N.wall.1.1.fl10} \\ R_{N.wall.1.1.fl9} \\ R_{N.wall.1.1.fl8} \\ R_{N.wall.1.1.fl7} \\ R_{N.wall.1.1.fl6} \\ R_{N.wall.1.1.fl5} \\ R_{N.wall.1.1.fl4} \\ R_{N.wall.1.1.fl3} \\ R_{N.wall.1.1.fl2} \\ R_{N.wall.1.1.fl1} \end{pmatrix} =$$

	0
0	1.575
1	7.843
2	-1.929
3	4.599
4	5.115
5	8.28
6	15.996
7	23.712
8	28.081
9	30.246
10	43.125
11	14.992

$\cdot kN$

$$\begin{pmatrix} R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} \cdots \\ + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl6} + R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} \cdots \\ + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl5} + R_{N.wall.1.1.fl6} + R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} \cdots \\ + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl4} + R_{N.wall.1.1.fl5} + R_{N.wall.1.1.fl6} + R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} \cdots \\ + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl3} + R_{N.wall.1.1.fl4} + R_{N.wall.1.1.fl5} + R_{N.wall.1.1.fl6} + R_{N.wall.1.1.fl7} \cdots \\ + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl2} + R_{N.wall.1.1.fl3} + R_{N.wall.1.1.fl4} + R_{N.wall.1.1.fl5} + R_{N.wall.1.1.fl6} \cdots \\ + R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} + R_{N.wall.1.1.fl11} \cdots \\ + R_{N.wall.1.1.fl12} \\ R_{N.wall.1.1.fl1} + R_{N.wall.1.1.fl2} + R_{N.wall.1.1.fl3} + R_{N.wall.1.1.fl4} + R_{N.wall.1.1.fl5} \cdots \\ + R_{N.wall.1.1.fl6} + R_{N.wall.1.1.fl7} + R_{N.wall.1.1.fl8} + R_{N.wall.1.1.fl9} + R_{N.wall.1.1.fl10} \cdots \\ + R_{N.wall.1.1.fl11} + R_{N.wall.1.1.fl12} \end{pmatrix} =$$

	0
0	1.575
1	9.418
2	7.488
3	12.087
4	17.202
5	25.482
6	41.478
7	65.19
8	93.271
9	123.517
10	166.641
11	181.633

$\cdot k$

$$R_{opening.wall.1.1} := \begin{pmatrix} R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl11} \\ R_{opening.wall.1.1.fl10} \\ R_{opening.wall.1.1.fl9} \\ R_{opening.wall.1.1.fl8} \\ R_{opening.wall.1.1.fl7} \\ R_{opening.wall.1.1.fl6} \\ R_{opening.wall.1.1.fl5} \\ R_{opening.wall.1.1.fl4} \\ R_{opening.wall.1.1.fl3} \\ R_{opening.wall.1.1.fl2} \\ R_{opening.wall.1.1.fl1} \end{pmatrix} =$$

	0
0	42.62
1	72.547
2	40.953
3	64.501
4	57.177
5	54.012
6	46.296
7	38.58
8	26.455
9	16.534
10	3.655
11	1.828

$\cdot kN$

$$\begin{pmatrix} R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} \cdots + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} \cdots + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl7} + R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} \cdots + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl6} + R_{opening.wall.1.1.fl7} + R_{opening.wall.1.1.fl8} \cdots + R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} \cdots + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl5} + R_{opening.wall.1.1.fl6} + R_{opening.wall.1.1.fl7} \cdots + R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} \cdots + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl4} + R_{opening.wall.1.1.fl5} + R_{opening.wall.1.1.fl6} \cdots + R_{opening.wall.1.1.fl7} + R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} \cdots + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl3} + R_{opening.wall.1.1.fl4} + R_{opening.wall.1.1.fl5} \cdots + R_{opening.wall.1.1.fl6} + R_{opening.wall.1.1.fl7} + R_{opening.wall.1.1.fl8} \cdots + R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} \cdots + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl2} + R_{opening.wall.1.1.fl3} + R_{opening.wall.1.1.fl4} \cdots + R_{opening.wall.1.1.fl5} + R_{opening.wall.1.1.fl6} + R_{opening.wall.1.1.fl7} \cdots + R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} + R_{opening.wall.1.1.fl10} \cdots + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \\ R_{opening.wall.1.1.fl1} + R_{opening.wall.1.1.fl2} + R_{opening.wall.1.1.fl3} \cdots + R_{opening.wall.1.1.fl4} + R_{opening.wall.1.1.fl5} + R_{opening.wall.1.1.fl6} \cdots + R_{opening.wall.1.1.fl7} + R_{opening.wall.1.1.fl8} + R_{opening.wall.1.1.fl9} \cdots + R_{opening.wall.1.1.fl10} + R_{opening.wall.1.1.fl11} + R_{opening.wall.1.1.fl12} \end{pmatrix} =$$

	0
0	42.62
1	115.166
2	156.12
3	220.621
4	277.798
5	331.81
6	378.106
7	416.686
8	443.141
9	459.675
10	463.331
11	465.158

$\cdot kN$

Part 2

$$R_{12.wall.1.2.tot} := \begin{pmatrix} R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl11} \\ R_{N.wall.1.2.fl10} \\ R_{N.wall.1.2.fl9} \\ R_{N.wall.1.2.fl8} \\ R_{N.wall.1.2.fl7} \\ R_{N.wall.1.2.fl6} \\ R_{N.wall.1.2.fl5} \\ R_{N.wall.1.2.fl4} \\ R_{N.wall.1.2.fl3} \\ R_{N.wall.1.2.fl2} \\ R_{N.wall.1.2.fl1} \end{pmatrix} =$$

	0
0	-54.735
1	86.812
2	-32.056
3	-22.341
4	-37.422
5	-35.092
6	-26.09
7	-17.088
8	-5.788
9	2.94
10	9.37
11	12.68

$\cdot kN$

$$\begin{pmatrix} R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} \cdots \\ + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} \cdots \\ + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl6} + R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} \cdots \\ + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl5} + R_{N.wall.1.2.fl6} + R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} \cdots \\ + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl4} + R_{N.wall.1.2.fl5} + R_{N.wall.1.2.fl6} + R_{N.wall.1.2.fl7} \cdots \\ + R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} \cdots \\ + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl3} + R_{N.wall.1.2.fl4} + R_{N.wall.1.2.fl5} + R_{N.wall.1.2.fl6} \cdots \\ + R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} \cdots \\ + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl2} + R_{N.wall.1.2.fl3} + R_{N.wall.1.2.fl4} + R_{N.wall.1.2.fl5} \cdots \\ + R_{N.wall.1.2.fl6} + R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} + R_{N.wall.1.2.fl9} \cdots \\ + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \\ R_{N.wall.1.2.fl1} + R_{N.wall.1.2.fl2} + R_{N.wall.1.2.fl3} + R_{N.wall.1.2.fl4} \cdots \\ + R_{N.wall.1.2.fl5} + R_{N.wall.1.2.fl6} + R_{N.wall.1.2.fl7} + R_{N.wall.1.2.fl8} \cdots \\ + R_{N.wall.1.2.fl9} + R_{N.wall.1.2.fl10} + R_{N.wall.1.2.fl11} + R_{N.wall.1.2.fl12} \end{pmatrix} =$$

	0
0	-54.735
1	32.078
2	0.022
3	-22.319
4	-59.741
5	-94.833
6	-120.923
7	-138.011
8	-143.799
9	-140.859
10	-131.489
11	-118.809

$\cdot kN$

$$R_{\mathrm{opening.wall.1.2}} := \begin{pmatrix} R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl11}} \\ R_{\mathrm{opening.wall.1.2.fl10}} \\ R_{\mathrm{opening.wall.1.2.fl9}} \\ R_{\mathrm{opening.wall.1.2.fl8}} \\ R_{\mathrm{opening.wall.1.2.fl7}} \\ R_{\mathrm{opening.wall.1.2.fl6}} \\ R_{\mathrm{opening.wall.1.2.fl5}} \\ R_{\mathrm{opening.wall.1.2.fl4}} \\ R_{\mathrm{opening.wall.1.2.fl3}} \\ R_{\mathrm{opening.wall.1.2.fl2}} \\ R_{\mathrm{opening.wall.1.2.fl1}} \end{pmatrix} =$$

	0
0	42.62
1	72.547
2	40.953
3	57.693
4	57.177
5	54.012
6	46.296
7	38.58
8	26.455
9	16.534
10	3.655
11	3.858

$\cdot \mathrm{kN}$

$$\begin{pmatrix} R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} \cdots + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} \cdots + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl7}} + R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} \cdots + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl6}} + R_{\mathrm{opening.wall.1.2.fl7}} + R_{\mathrm{opening.wall.1.2.fl8}} \cdots + R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} \cdots + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl5}} + R_{\mathrm{opening.wall.1.2.fl6}} + R_{\mathrm{opening.wall.1.2.fl7}} \cdots + R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} \cdots + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl4}} + R_{\mathrm{opening.wall.1.2.fl5}} + R_{\mathrm{opening.wall.1.2.fl6}} \cdots + R_{\mathrm{opening.wall.1.2.fl7}} + R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} \cdots + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl3}} + R_{\mathrm{opening.wall.1.2.fl4}} + R_{\mathrm{opening.wall.1.2.fl5}} \cdots + R_{\mathrm{opening.wall.1.2.fl6}} + R_{\mathrm{opening.wall.1.2.fl7}} + R_{\mathrm{opening.wall.1.2.fl8}} \cdots + R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} \cdots + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl2}} + R_{\mathrm{opening.wall.1.2.fl3}} + R_{\mathrm{opening.wall.1.2.fl4}} \cdots + R_{\mathrm{opening.wall.1.2.fl5}} + R_{\mathrm{opening.wall.1.2.fl6}} + R_{\mathrm{opening.wall.1.2.fl7}} \cdots + R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} + R_{\mathrm{opening.wall.1.2.fl10}} \cdots + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \\ R_{\mathrm{opening.wall.1.2.fl1}} + R_{\mathrm{opening.wall.1.2.fl2}} + R_{\mathrm{opening.wall.1.2.fl3}} \cdots + R_{\mathrm{opening.wall.1.2.fl4}} + R_{\mathrm{opening.wall.1.2.fl5}} + R_{\mathrm{opening.wall.1.2.fl6}} \cdots + R_{\mathrm{opening.wall.1.2.fl7}} + R_{\mathrm{opening.wall.1.2.fl8}} + R_{\mathrm{opening.wall.1.2.fl9}} \cdots + R_{\mathrm{opening.wall.1.2.fl10}} + R_{\mathrm{opening.wall.1.2.fl11}} + R_{\mathrm{opening.wall.1.2.fl12}} \end{pmatrix} =$$

	0
0	42.62
1	115.166
2	156.12
3	213.813
4	270.99
5	325.002
6	371.298
7	409.878
8	436.333
9	452.867
10	456.523
11	460.381

$\cdot \mathrm{kN}$

Part 3

$$R_{12.wall.3.3.tot} := \begin{pmatrix} R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl11} \\ R_{N.wall.1.3.fl10} \\ R_{N.wall.1.3.fl9} \\ R_{N.wall.1.3.fl8} \\ R_{N.wall.1.3.fl7} \\ R_{N.wall.1.3.fl6} \\ R_{N.wall.1.3.fl5} \\ R_{N.wall.1.3.fl4} \\ R_{N.wall.1.3.fl3} \\ R_{N.wall.1.3.fl2} \\ R_{N.wall.1.3.fl1} \end{pmatrix} =$$

	0
0	51.952
1	94.904
2	45.816
3	73.428
4	73.428
5	73.428
6	73.428
7	73.428
8	64.224
9	55.02
10	48.711
11	18.85

$\cdot \mathrm{kN}$

$$\begin{pmatrix} R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} \cdots \\ + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} \cdots \\ + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl6} + R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} \cdots \\ + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl5} + R_{N.wall.1.3.fl6} + R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} \cdots \\ + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl4} + R_{N.wall.1.3.fl5} + R_{N.wall.1.3.fl6} + R_{N.wall.1.3.fl7} \cdots \\ + R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} \cdots \\ + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl3} + R_{N.wall.1.3.fl4} + R_{N.wall.1.3.fl5} + R_{N.wall.1.3.fl6} \cdots \\ + R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} \cdots \\ + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl2} + R_{N.wall.1.3.fl3} + R_{N.wall.1.3.fl4} + R_{N.wall.1.3.fl5} \cdots \\ + R_{N.wall.1.3.fl6} + R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} + R_{N.wall.1.3.fl9} \cdots \\ + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \\ R_{N.wall.1.3.fl1} + R_{N.wall.1.3.fl2} + R_{N.wall.1.3.fl3} + R_{N.wall.1.3.fl4} \cdots \\ + R_{N.wall.1.3.fl5} + R_{N.wall.1.3.fl6} + R_{N.wall.1.3.fl7} + R_{N.wall.1.3.fl8} \cdots \\ + R_{N.wall.1.3.fl9} + R_{N.wall.1.3.fl10} + R_{N.wall.1.3.fl11} + R_{N.wall.1.3.fl12} \end{pmatrix} =$$

	0
0	51.952
1	146.856
2	192.672
3	266.1
4	339.528
5	412.956
6	486.384
7	559.812
8	624.036
9	679.056
10	727.767
11	746.617

$\cdot \mathrm{kN}$

L Reactionforces in wall 3

Part 1

$$R_{N.wall.3.1.tot} := \begin{pmatrix} R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl11} \\ R_{N.wall.3.1.fl10} \\ R_{N.wall.3.1.fl9} \\ R_{N.wall.3.1.fl8} \\ R_{N.wall.3.1.fl7} \\ R_{N.wall.3.1.fl6} \\ R_{N.wall.3.1.fl5} \\ R_{N.wall.3.1.fl4} \\ R_{N.wall.3.1.fl3} \\ R_{N.wall.3.1.fl2} \\ R_{N.wall.3.1.fl1} \end{pmatrix} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 9.71 \\ 1 & 29.752 \\ 2 & 27.282 \\ 3 & 25.632 \\ 4 & 25.092 \\ 5 & 24.768 \\ 6 & 24.552 \\ 7 & 24.398 \\ 8 & 24.282 \\ 9 & 24.192 \\ 10 & 24.12 \\ 11 & 36.72 \\ \hline \end{array} \cdot kN$$

$$\begin{pmatrix} R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} \cdots \\ + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} \cdots \\ + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl6} + R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} \cdots \\ + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl5} + R_{N.wall.3.1.fl6} + R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} \cdots \\ + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl4} + R_{N.wall.3.1.fl5} + R_{N.wall.3.1.fl6} + R_{N.wall.3.1.fl7} \cdots \\ + R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} \cdots \\ + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl3} + R_{N.wall.3.1.fl4} + R_{N.wall.3.1.fl5} + R_{N.wall.3.1.fl6} \cdots \\ + R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} \cdots \\ + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl2} + R_{N.wall.3.1.fl3} + R_{N.wall.3.1.fl4} + R_{N.wall.3.1.fl5} \cdots \\ + R_{N.wall.3.1.fl6} + R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} + R_{N.wall.3.1.fl9} \cdots \\ + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \\ R_{N.wall.3.1.fl1} + R_{N.wall.3.1.fl2} + R_{N.wall.3.1.fl3} + R_{N.wall.3.1.fl4} \cdots \\ + R_{N.wall.3.1.fl5} + R_{N.wall.3.1.fl6} + R_{N.wall.3.1.fl7} + R_{N.wall.3.1.fl8} \cdots \\ + R_{N.wall.3.1.fl9} + R_{N.wall.3.1.fl10} + R_{N.wall.3.1.fl11} + R_{N.wall.3.1.fl12} \end{pmatrix} = \begin{array}{|c|r|} \hline & 0 \\ \hline 0 & 9.71 \\ 1 & 39.462 \\ 2 & 66.744 \\ 3 & 92.376 \\ 4 & 117.468 \\ 5 & 142.236 \\ 6 & 166.788 \\ 7 & 191.186 \\ 8 & 215.468 \\ 9 & 239.66 \\ 10 & 263.78 \\ 11 & 300.5 \\ \hline \end{array} \cdot kN$$

Part 2

$$R_{N.wall.3.2.tot} := \begin{pmatrix} R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl11} \\ R_{N.wall.3.2.fl10} \\ R_{N.wall.3.2.fl9} \\ R_{N.wall.3.2.fl8} \\ R_{N.wall.3.2.fl7} \\ R_{N.wall.3.2.fl6} \\ R_{N.wall.3.2.fl5} \\ R_{N.wall.3.2.fl4} \\ R_{N.wall.3.2.fl3} \\ R_{N.wall.3.2.fl2} \\ R_{N.wall.3.2.fl1} \end{pmatrix} =$$

	0
0	289.907
1	510.076
2	382.317
3	322.32
4	286.62
5	250.92
6	215.22
7	179.52
8	143.82
9	108.12
10	72.42
11	37.74

$\cdot kN$

$$\begin{pmatrix} R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} \cdots \\ + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} \cdots \\ + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl6} + R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} \cdots \\ + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl5} + R_{N.wall.3.2.fl6} + R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} \cdots \\ + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl4} + R_{N.wall.3.2.fl5} + R_{N.wall.3.2.fl6} + R_{N.wall.3.2.fl7} \cdots \\ + R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} \cdots \\ + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl3} + R_{N.wall.3.2.fl4} + R_{N.wall.3.2.fl5} + R_{N.wall.3.2.fl6} \cdots \\ + R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} \cdots \\ + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl2} + R_{N.wall.3.2.fl3} + R_{N.wall.3.2.fl4} + R_{N.wall.3.2.fl5} \cdots \\ + R_{N.wall.3.2.fl6} + R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} + R_{N.wall.3.2.fl9} \cdots \\ + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \\ R_{N.wall.3.2.fl1} + R_{N.wall.3.2.fl2} + R_{N.wall.3.2.fl3} + R_{N.wall.3.2.fl4} \cdots \\ + R_{N.wall.3.2.fl5} + R_{N.wall.3.2.fl6} + R_{N.wall.3.2.fl7} + R_{N.wall.3.2.fl8} \cdots \\ + R_{N.wall.3.2.fl9} + R_{N.wall.3.2.fl10} + R_{N.wall.3.2.fl11} + R_{N.wall.3.2.fl12} \end{pmatrix} =$$

	0
0	289.907
1	799.982
2	$1.182 \cdot 10^{3}$
3	$1.505 \cdot 10^{3}$
4	$1.791 \cdot 10^{3}$
5	$2.042 \cdot 10^{3}$
6	$2.257 \cdot 10^{3}$
7	$2.437 \cdot 10^{3}$
8	$2.581 \cdot 10^{3}$
9	$2.689 \cdot 10^{3}$
10	$2.761 \cdot 10^{3}$
11	$2.799 \cdot 10^{3}$

$\cdot kN$

M Design in serviceability limit state

The acceleration in the top of the building

To determine the comfort requirements, the wind speed are calculated for a return period on average once every five years, according to ISO 6897 which is criteria for the "responses of people-to-horizontal motion of structures in the frequency range 0.063 to 1 Hz.

Reference wind speed (6.3.2(1)) $v_{b.5.years} := 0.855 \cdot v_b = 21.375 \frac{m}{s}$

Basic velocity pressure (Equation 4.8) $q_{b.HC} := \frac{1}{2} \cdot \rho \cdot v_{b.5.years}{}^2 = 0.286 \cdot kPa$

Mean velocity at height z (Equation 4.3) $v_{m.HC} := c_{r2} \cdot c_0 \cdot v_{b.5.years} = 26.72 \frac{m}{s}$

Standard deviation of the turbulent component of wind velocity (Equation 4.6)

$$\sigma_{v.HC} := k_r \cdot v_{b.5.years} \cdot k_l = 4.061 \frac{m}{s}$$

Turbulence intensity(Equation 4.7) $I_{v.HC} := \frac{\sigma_{v.HC}}{v_{m.HC}} = 0.152$

Peak velocity pressure (Formula 4.5(1)) $q_{p.HC} := \left[\left(1 + 6 \cdot I_{v.HC}\right) \left(k_r \cdot \ln\left(\frac{h_2}{z_0}\right)\right)^2 \right] \cdot q_{b.HC} = 0.853 \cdot \frac{kN}{m^2}$

According to old thesis (Equation 3-19, Kamal Handa)

The modfactor, which is set to 1 for the top of the housing $\phi_1 = 0.763$ $m_{s.rms} := 3.116 \cdot 10^4 \frac{kg}{m}$

The acceleration in the top of the building $a_{rms} := \frac{3 I_{v.HC} \cdot R_{cscd.2} \cdot q_{p.HC} \cdot d \cdot c_f \cdot \phi_1}{m_{s.rms}} = 0.083 \frac{m}{s^2}$

The maximum acceleration $a_{max} := k_{p.2} \cdot a_{rms} = 0.296 \frac{m}{s^2}$

According to EC - National annex

$$\phi_{1.x} := \left(\frac{h_2}{h_{tot}}\right)^{1.5} = 1$$

The acceleration standard deviation $\sigma_x := \frac{3 \cdot I_{v2} \cdot R_{cscd.2} \cdot q_{p2} \cdot d \cdot c_f \cdot \phi_{1.x}}{m_s} = 0.381 \frac{m}{s^2}$

The maximum acceleration $X_{max} := k_{p.2} \cdot \sigma_x = 1.355 \frac{m}{s^2}$

Time history of wind loading

Reference length scale

$$L_t := 300m$$

Reference height

$$z_t := 200m$$

Roughness length in meters

$$z_{0.z} := \frac{z_0}{m} = 0.05$$

$$\alpha_Z := 0.67 + 0.05\cdot\ln\left(z_0\right)_z = 0.52$$

The turbulent length scale (Equation B.1)

$$L_Z := L_t\cdot\left(\frac{z}{z_t}\right)^{\alpha_Z} = 122.943\,m$$

The non-dimensional frequency on reference height z_s (=0.6H)' (Equation)

$$f_{L.zn} := \frac{n_1\cdot L_Z}{v_m} = 39.34\,s$$

The spectral density function is in accordance (Equation B.2)

$$S_{L.fn} := \frac{6.8\cdot\frac{f_{L.zn}}{s}}{\left(1 + 10.2\cdot\frac{f_{L.zn}}{s}\right)^{\frac{5}{3}}} = 0.012$$

$$S_i := S_{L.fn}$$

The magnitude of the amplitudes

$$a_i = \sqrt{2\cdot S_i\cdot\Delta f}$$

the signal written as a sum of the sinusoidal waves

$$v_t = \sum\left(a_i\cdot\sin\left(f_t + \psi_i\right)\right)$$

The dynamic building factor

$$c_{d.2} := \frac{1 + 2\cdot k_{p.2}\cdot I_{v2}\cdot\sqrt{B^2 + R_{cscd.2}^{\;2}}}{1 + 7\cdot\sqrt{B^2}} = 0.262$$

The area of the long side of the facade

$$A_{LS} := 12\cdot h\cdot d = 980.286\,m^2$$

The fluctuated wind force on an area

$$F_t = \rho\cdot c_d\cdot A_{LS}\cdot v_m\cdot v_t$$

The wind forces caused by the mean part

$$F_m = \frac{\rho}{2}\cdot c_d\cdot A_{LS}\cdot v_m^{\;2}$$

The critical wind velocity for vortex shedding

$$v_{crit.i} = b\cdot\frac{n_{i.y}}{S_t}$$

$$v_{crit.i} \leq 1.25v_m$$

The starting wind velocity for galloping

$$v_{CG} = \frac{2S_c}{a_G}n_{i.y}\cdot b$$

The Scruton number

$$S_c := \frac{2\cdot\delta_s\cdot m_{e.2}}{\rho\cdot b^2} = 14.007$$

$$v_{CG} \geq 1.25 v_m$$

The mean part of the wind load

$$q_{p.mean} := \left[1\left(k_r \cdot \ln\left(\frac{h_2}{z_0}\right)\right)^2\right] \cdot q_b = 0.61 \cdot \frac{kN}{m^2}$$

The fluctuating part of the wind load

$$q_{p.fluc} := \left[\left(6 \cdot I_{v2}\right)\left(k_r \cdot \ln\left(\frac{h_2}{z_0}\right)\right)^2\right] \cdot q_b = 0.557 \cdot \frac{kN}{m^2}$$

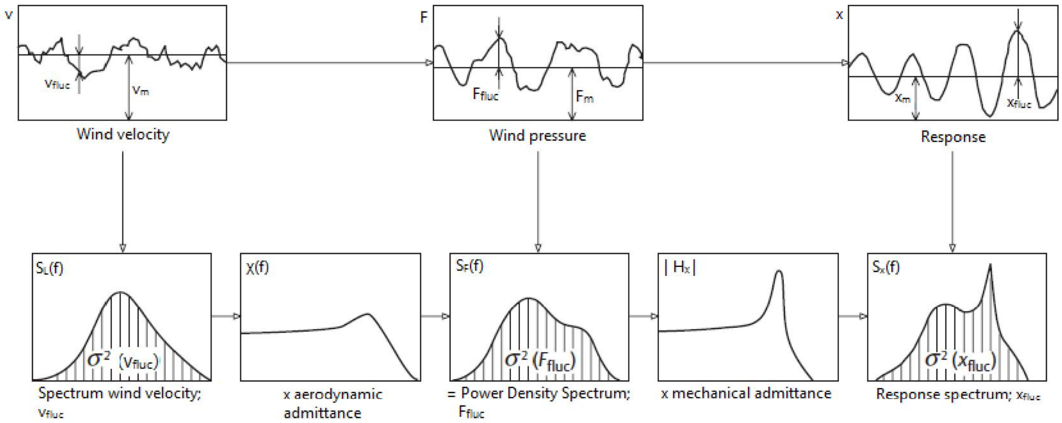
Turbulence intensity(Equation 4.7)

$$I_{v2} := \frac{\sigma_v}{v_{m2}} = 0.152$$

The result is a quasi-static wind loading, which does not represent the real dynamic behaviour of the wind and is thus unsuitable for dynamic analyses of structures subjected to wind.

Spectral analyses

A spectral analysis is a useful tool to determine the structural response on fluctuating wind loading.Besides it is possible to obtain a value for wind loading as a function of time, *q(t)*, which is required to perform dynamic response analyses.



A spectrum of the fluctuating wind velocity, $S_L(f_L)$, can be modelled by real data of wind. A spectrum portrays a variable from the time domain (wind statistics) into a frequency domain (spectrum). The function $S_L(f_L)$ is determined by the dimensionless frequency f_L. This frequency is introduced by EC1 and is non-dimensional because it is a function of both the fundamental frequency of the structure, n_1, and height z. As a consequence, the magnitude of the fluctuating wind is determined by the fundamental vibration mode of the structure and the location on the building facade, where fluctuations decrease upwards on the building.

The turbulent length scale

The non-dimensional frequency on reference height z_s (=0.6H)'
(Equation)

$$f_{L.zn} := \begin{pmatrix} 0.01 \\ 1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 11 \\ \dfrac{n_{1x} \cdot L_Z}{v_m} \end{pmatrix} =$$

	0
0	0.01
1	1
2	2
3	3
4	4
5	5
6	6
7	7
8	8
9	9
10	10
11	11
12	12.038

The spectral density function is in accordance (Equation B.2)

$$S_{L.fL} := \frac{6.8 \cdot f_{L.zn}}{\left(1 + 10.2 \cdot f_{L.zn}\right)^{\frac{5}{3}}} =$$

	0
0	0.058
1	0.121
2	0.082
3	0.065
4	0.054
5	0.047
6	0.042
7	0.038
8	0.035
9	0.032
10	0.03
11	0.028
12	0.027

$$\eta_h := \frac{4.6 \cdot h_{tot}}{L_Z} \cdot f_{L.zn} =$$

	0
0	0.013
1	1.347
2	2.694
3	4.041
4	5.388
5	6.735
6	8.082
7	9.429
8	10.776
9	12.123
10	13.47
11	14.817
12	16.215

$$\eta_b := \frac{4.6 \cdot d}{L_Z} \cdot f_{L.zn} =$$

	0
0	0.01
1	1.027
2	2.055
3	3.082
4	4.11
5	5.137
6	6.164
7	7.192
8	8.219
9	9.247
10	10.274
11	11.301
12	12.368

$$R_h := \frac{1}{\eta_h} - \frac{\left(1 - e^{-2\eta_h}\right)}{2\eta_h^{\ 2}} =$$

	0
0	0.991
1	0.485
2	0.303
3	0.217
4	0.168
5	0.137
6	0.116
7	0.1
8	0.088
9	0.079
10	0.071
11	0.065
12	0.06

The admittance function for the fundamental mode shape $\chi_{fL}^{2} = R_h \cdot R_b$

B - 8

$$R_b := \frac{1}{\eta_b} - \frac{\left(1 - e^{-2\eta_b}\right)}{2\eta_b^{2}} =$$

	0
0	0.993
1	0.56
2	0.37
3	0.272
4	0.214
5	0.176
6	0.149
7	0.129
8	0.114
9	0.102
10	0.093
11	0.085
12	0.078

The admittance function for the fundamental mode shape $\chi_{fL} := \sqrt{R_h \cdot R_b} = 1.246$

The variance of the fluctuating wind $\sigma_v := \sqrt{\left(I_{v2} \cdot v_m\right)^{2}} = 4.75 \frac{m}{s}$

The spectrum of the variance of the fluctuating wind velocity $S_{v.fL} := \frac{S_{L.fL} \cdot \sigma_v^{2}}{n_{1x}}$

The dynamic building factor $c_{d.2} := \frac{1 + 2 \cdot k_{p.2} \cdot I_{v2} \cdot \sqrt{B^{2} + R_{cscd.2}^{2}}}{1 + 7 \cdot \sqrt{B^{2}}} = 0.262$

The power density spectrum $S_{F.f.L} := \left(c_{d.2} \cdot A \cdot \rho \cdot v_m\right)^{2} \cdot S_{v.fL} \cdot \chi_{fL}^{2}$

The mechanical admittance function

$$H_x := \frac{1}{4\pi^{2} \cdot n_{1x}^{2} \cdot m \cdot \sqrt{\left[1 - \left(\frac{n}{n_1}\right)^{2}\right]^{2} + 4 \cdot \zeta \cdot \left(\frac{n}{n_1}\right)^{2}}}$$

The response spectrum $S_{x.fL} := \left(\left|H_x\right|\right)^{2} \cdot S_{F.f.L}$

The total response of the structure due to fluctuating wind actions

The quasi-static part $\sigma_{x.stat} = \frac{c_{d.2} \cdot A \cdot v_b \cdot \sigma_{v.DR}}{k}$

The resonance part

$$\sigma_{x.res} = \frac{1}{k}\cdot\sqrt{\frac{\pi\cdot n_{1x}\cdot S_{F.f.L}}{4\cdot\zeta}}$$

The dynamic response can be written as the resultant of a quasi-static- and a resonance component

$$\sigma_{x.dyn} = \sqrt{\sigma_{x.stat}^{\;2} + \sigma_{x.res}^{\;2}}$$

For a single degree-of-freedom system the structural response can be expressed as

$$\sigma_a = \sqrt{\frac{S_{F.f.L}\cdot\pi\cdot n_{1x}}{4\cdot\zeta\cdot m2}}$$

Reference length scale $L_t := 300m$

Reference height $z_t := 200m$

Roughness length in meters $z_{0.z} := \frac{z_0}{m} = 0.05$

$$\alpha_Z := 0.67 + 0.05\cdot\ln\left(z_{0.z}\right) = 0.52$$

The turbulent length scale (Equation B.1) $L_Z := L_t\cdot\left(\frac{z_d}{z_t}\right)^{\alpha_Z} = \begin{pmatrix} 44.026 \\ 122.943 \end{pmatrix} m$

The non-dimensional frequency on reference height *zs* (=0.6H)' (Equation)

$f_{L.zn} := \frac{n_{1x}\cdot L_Z}{v_m} = \begin{pmatrix} 4.311 \\ 12.038 \end{pmatrix}$ $\quad L_Z = \begin{pmatrix} 44.026 \\ 122.943 \end{pmatrix} m$

$N_f := 10$

$\Delta f_L := f_{L.zn_1} - f_{L.zn_0} = 7.727$

$\Delta f := \frac{\Delta f_L}{N_f} = 0.773$

$f_N := f_{L.zn_0} + \Delta f_L\cdot N_f = 81.583$

The spectral density function is in accordance (Equation B.2)

$$S_{L.N} := \frac{6.8 \cdot f_N}{\left(1 + 10.2 \cdot f_N\right)^{\frac{5}{3}}} = 7.52 \times 10^{-3}$$

The magnitude of the amplitudes

$$a_N := \sqrt{2 \cdot S_{L.N} \cdot \Delta f} = 0.108$$

$$t_i := \begin{pmatrix} 0 \\ 5 \\ 10 \\ 15 \\ 20 \\ 25 \\ 30 \\ 35 \\ 40 \\ 45 \\ 50 \\ 55 \\ 120 \end{pmatrix}$$

$$\psi_i := 0.025$$

The signal written as a sum of the sinusoidal waves

$$v_t := \sum_{i=1}^{N_f} \left(a_N \cdot \sin\left(f_N \cdot t_i + \psi_i\right)\right) =$$

	0
0	0.027
1	-0.486
2	-0.883
3	-1.07
4	-1.004
5	-0.699
6	-0.227
7	0.298
8	0.752
9	1.028
10	1.059
11	0.839
12	0.755

N Natural frequency

Simplifications:

Oppenings i walls has not been taken into consideration.

Hight of the building

$L := 36m$

$$n_{freq} := \frac{46(m)}{h_{tot}} = 1.278$$

$m_{tot} := 1121938kg$

$$m_{s.f} := \frac{m_{tot}}{L} = 3.116 \times 10^{4} \cdot \frac{kg}{m}$$

Ticknesses of walls

$t_i := 15mm + 245mm + 15mm = 0.275\,m$

$t_e := 11mm + 245mm + 11mm = 0.267\,m$

$$\frac{t_i}{2} = 137.5 \cdot mm$$

$$\frac{t_e}{2} = 133.5 \cdot mm$$

Distance from element TP to origo (origo is in the center of the building)

$l_{y.e1} := 11803mm$ $x_{e1} := -13522.5mm$ $y_{e1} := 0mm$

$l_{x.e2} := 26671mm$ $x_{e2} := 0mm$ $y_{e2} := 5667.5mm$

$l_{y.e6} := 11803mm$ $x_{e6} := 13522.5mm$ $y_{e6} := 0mm$

$l_{x.e7} := 26671mm$ $x_{e7} := 0mm$ $y_{e7} := -5667.5mm$

$l_{y.i1} := 5488mm$ $x_{i1} := -5560mm$ $y_{i1} := -2736mm$

$l_{y.i6} := 5488mm$ $x_{i6} := 6735mm$ $y_{i6} := -2736mm$

$l_{x.i2} := 12050mm$ $x_{i2} := 616mm$ $y_{i2} := -678mm$

$l_{y.i3} := 5919mm$ $x_{i3} := -1816mm$ $y_{i3} := 2527mm$

$l_{y.i4} := 5919mm$ $x_{i4} := 3654mm$ $y_{i4} := 2527mm$

$l_{x.i7} := 2003mm$ $x_{i7} := 2419mm$ $y_{i7} := 4070mm$

$l_{y.i8} := 2507mm$ $x_{i8} := 1651mm$ $y_{i8} := 2860mm$

Calculating the center of gravity

$$X_{TP} := \frac{\begin{array}{l}\left(l_{y.e1} \cdot t_e \cdot x_{e1}\right) + \left(l_{y.e6} \cdot t_e \cdot x_{e6}\right) + \left(l_{y.i1} \cdot t_i \cdot x_{i1}\right) + \left(l_{y.i6} \cdot t_i \cdot x_{i6}\right) + \left(l_{x.i2} \cdot t_i \cdot x_{i2}\right) \ldots \\ + \left(l_{y.i3} \cdot t_i \cdot x_{i3}\right) + \left(l_{y.i4} \cdot t_i \cdot x_{i4}\right) + \left(l_{x.i7} \cdot t_i \cdot x_{i7}\right) + \left(l_{y.i8} \cdot t_i \cdot x_{i8}\right)\end{array}}{\left(l_{y.e1} \cdot t_e + l_{y.e6} \cdot t_e + l_{y.i1} \cdot t_i + l_{y.i6} \cdot t_i + l_{x.i2} \cdot t_i + l_{y.i3} \cdot t_i + l_{y.i4} \cdot t_i + l_{x.i7} \cdot t_i + l_{y.i8} \cdot t_i\right)}$$

$X_{TP} = 541.545 \cdot mm$

$$Y_{TP} := \frac{\left(l_{x.e2}\cdot t_e\cdot y_{e2}\right) + \left(l_{x.e7}\cdot t_e\cdot y_{e7}\right) + \left(l_{y.i1}\cdot t_i\cdot y_{i1}\right) + \left(l_{y.i6}\cdot t_i\cdot y_{i6}\right) + \left(l_{x.i2}\cdot t_i\cdot y_{i2}\right) \cdots + \left(l_{y.i3}\cdot t_i\cdot y_{i3}\right) + \left(l_{y.i4}\cdot t_i\cdot y_{i4}\right) + \left(l_{x.i7}\cdot t_i\cdot y_{i7}\right) + \left(l_{y.i8}\cdot t_i\cdot y_{i8}\right)}{\left(l_{y.e1}\cdot t_e + l_{y.e6}\cdot t_e + l_{y.i1}\cdot t_i + l_{y.i6}\cdot t_i + l_{x.i2}\cdot t_i + l_{y.i3}\cdot t_i + l_{y.i4}\cdot t_i + l_{x.i7}\cdot t_i + l_{y.i8}\cdot t_i\right)}$$

$$Y_{TP} = 112.96\cdot mm$$

Moment of inertia

$$I = \frac{b\cdot h^3}{12} + A\cdot x_{TP}^2$$

$$I_{x.e1} := \frac{t_e\cdot l_{y.e1}^3}{12} + t_e\cdot l_{y.e1}\cdot\left(-x_{e1} + X_{TP}\right)^2 \qquad I_{x.e1} = 659.924\,m^4$$

$$I_{y.e1} := \frac{l_{y.e1}\cdot t_e^3}{12} + l_{y.e1}\cdot t_e\cdot\left(y_{e1} + Y_{TP}\right)^2 \qquad I_{y.e1} = 0.059\,m^4$$

$$I_{x.e2} := \frac{l_{x.e2}\cdot t_e^3}{12} + l_{x.e2}\cdot t_e\cdot\left(x_{e2} + X_{TP}\right)^2 \qquad I_{x.e2} = 2.131\,m^4$$

$$I_{y.e2} := \frac{t_e\cdot l_{x.e2}^3}{12} + t_e\cdot l_{x.e2}\cdot\left(y_{e2} - Y_{TP}\right)^2 \qquad I_{y.e2} = 641.84\,m^4$$

$$I_{x.e6} := \frac{t_e\cdot l_{y.e6}^3}{12} + t_e\cdot l_{y.e6}\cdot\left(x_{e6} - X_{TP}\right)^2 \qquad I_{x.e6} = 567.613\,m^4$$

$$I_{y.e6} := \frac{l_{y.e6}\cdot t_e^3}{12} + l_{y.e6}\cdot t_e\cdot\left(y_{e6} + Y_{TP}\right)^2 \qquad I_{y.e6} = 0.059\,m^4$$

$$I_{x.e7} := \frac{l_{x.e7}\cdot t_e^3}{12} + l_{x.e7}\cdot t_e\cdot\left(x_{e7} + X_{TP}\right)^2 \qquad I_{x.e7} = 2.131\,m^4$$

$$I_{y.e7} := \frac{t_e\cdot l_{x.e7}^3}{12} + t_e\cdot l_{x.e7}\cdot\left(-y_{e7} + Y_{TP}\right)^2 \qquad I_{y.e7} = 660.076\,m^4$$

$$I_{x.i1} := \frac{t_i\cdot l_{y.i1}^3}{12} + t_i\cdot l_{y.i1}\cdot\left(-x_{i1} + X_{TP}\right)^2 \qquad I_{x.i1} = 59.974\,m^4$$

$$I_{y.i1} := \frac{l_{y.i1}\cdot t_i^3}{12} + l_{y.i1}\cdot t_i\cdot\left(-y_{i1} + Y_{TP}\right)^2 \qquad I_{y.i1} = 12.259\,m^4$$

$$I_{x.i6} := \frac{t_i\cdot l_{y.i6}^3}{12} + t_i\cdot l_{y.i6}\cdot\left(x_{i6} - X_{TP}\right)^2 \qquad I_{x.i6} = 61.679\,m^4$$

$$I_{y.i6} := \frac{l_{y.i6}\cdot t_i^3}{12} + l_{y.i6}\cdot t_i\cdot\left(-y_{i6} + Y_{TP}\right)^2 \qquad I_{y.i6} = 12.259\,m^4$$

$$I_{x.i2} := \frac{l_{x.i2}\cdot t_i^3}{12} + l_{x.i2}\cdot t_i\cdot\left(x_{i2} - X_{TP}\right)^2 \qquad I_{x.i2} = 0.039\,m^4$$

$$I_{y.i2} := \frac{t_i\cdot l_{x.i2}^3}{12} + t_i\cdot l_{x.i2}\cdot\left(-y_{i2} + Y_{TP}\right)^2 \qquad I_{y.i2} = 42.17\,m^4$$

$$I_{x.i3} := \frac{t_i\cdot l_{y.i3}^3}{12} + t_i\cdot l_{y.i3}\cdot\left(-x_{i3} + X_{TP}\right)^2 \qquad I_{x.i3} = 13.799\,m^4$$

$I_{y.i3} := \frac{l_{y.i3} \cdot t_i^3}{12} + l_{y.i3} \cdot t_i \cdot \left(y_{i3} - Y_{TP}\right)^2$ $\qquad I_{y.i3} = 9.496\,m^4$

$I_{x.i4} := \frac{t_i \cdot l_{y.i4}^3}{12} + t_i \cdot l_{y.i4} \cdot \left(x_{i4} - X_{TP}\right)^2$ $\qquad I_{x.i4} = 20.521\,m^4$

$I_{y.i4} := \frac{l_{y.i4} \cdot t_i^3}{12} + l_{y.i4} \cdot t_i \cdot \left(y_{i4} - Y_{TP}\right)^2$ $\qquad I_{y.i4} = 9.496\,m^4$

$I_{x.i7} := \frac{l_{x.i7} \cdot t_i^3}{12} + l_{x.i7} \cdot t_i \cdot \left(x_{i7} - X_{TP}\right)^2$ $\qquad I_{x.i7} = 1.945\,m^4$

$I_{y.i7} := \frac{t_i \cdot l_{x.i7}^3}{12} + t_i \cdot l_{x.i7} \cdot \left(y_{i7} - Y_{TP}\right)^2$ $\qquad I_{y.i7} = 8.809\,m^4$

$I_{x.i8} := \frac{t_i \cdot l_{y.i8}^3}{12} + t_i \cdot l_{y.i8} \cdot \left(x_{i8} - X_{TP}\right)^2$ $\qquad I_{x.i8} = 1.21\,m^4$

$I_{y.i8} := \frac{l_{y.i8} \cdot t_i^3}{12} + l_{y.i8} \cdot t_i \cdot \left(y_{i8} - Y_{TP}\right)^2$ $\qquad I_{y.i8} = 5.207\,m^4$

$I_{x.tot} := I_{x.e1} + I_{x.e2} + I_{x.e6} + I_{x.e7} + I_{x.i1} + I_{x.i6} + I_{x.i2} + I_{x.i3} + I_{x.i4} + I_{x.i7} + I_{x.i8}$

$I_{x.tot} = 1.391 \times 10^3\,m^4$

$I_{y.tot} := I_{y.e1} + I_{y.e2} + I_{y.e6} + I_{y.e7} + I_{y.i1} + I_{y.i6} + I_{y.i2} + I_{y.i3} + I_{y.i4} + I_{y.i7} + I_{y.i8}$

$I_{y.tot} = 1.402 \times 10^3\,m^4$

Esitmated E-modulus for the building

$E_{tot} := 4530 \frac{N}{mm^2}$

Natural frequency

Since the building is under 50 m the equation from Eurocode 1 due not apply.

$f_{0.x} := \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{3E_{tot} \cdot I_{x.tot}}{m_{s.f} \cdot L^4}}$ $\qquad f_{0.x} = 3.024 \cdot Hz$

$f_{0.y} := \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{3E_{tot} \cdot I_{y.tot}}{m_{s.f} \cdot L^4}}$ $\qquad f_{0.y} = 3.036 \cdot Hz$

O Aero elastic effects

Vortex shedding

The effects from vortex shedding does not needs to be taken into consideration if the relationship between the maximum and minimum dimension in a plane perpendicular to the wind direction is less than six.

$$\frac{d}{b} = 2.326$$

$$\frac{b}{d} = 0.43$$

Galloping

Eurocode 1

The Structon number $S_c = 14.007$

The natural frequency $n_{i.y} := n_{1x}$

The factor for Galloping instability $a_G := 0.574$

The onset wind velocity of galloping (Equation E.18) $v_{CG} := \frac{2 \cdot S_c}{a_G} \cdot n_{i.y} \cdot b = 1.763 \times 10^3 \frac{m}{s}$

$$1.25 \cdot v_{m2} = 39.064 \frac{m}{s}$$

The following conditions should be fulfilled
(Equation E.19)

$$v_{CG} > 1.25 \cdot v_{m2}$$