STRUCTURAL DESIGN BASIS FOR HIGH-RISE RESIDENTIAL BUILDINGS

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ABSTRACT

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ABSTRACT

The purpose of this Bachelor's thesis was to study the structural design for high-rise residential buildings. The thesis was commissioned by Pöyry Oy Infrastructure Real Estate Design unit. The main aim was to make basic stability calculations based on the main loads of the structure and wind load for both directions of a 25-floor height building, which the company can compare with their own calculations. The main part of information for the solutions was taken from the Eurocodes, supervisor's and company's materials. Firstly, the history, different frame systems and main loads were discussed. The second part was completely dedicated to the calculations.

Based on the calculations it was discovered that the most difficult cases of the wall calculations were not in the top part of the building. The complicated section appears when the wall does not have enough essential normal force. Consequently, tensile reinforcement for the walls needs to be done.

Keywords High-rise, building, load, calculation.

Pages 41 pages

CONTENTS

1 GENERAL INFORMATION ABOUT HIGH-RISE BUILDINGS 1

1.1 Needs for high-rise buildings 1
1.2 Structural design of high rise buildings nowadays 2

2 FRAME SYSTEMS 4

2.1 Shear walls system 4
2.2 Rigid frames 5
2.3 Core and outrigger systems 6
2.4 Braced frames and shear trusses 7
2.5 Staggered system 8
2.6 Tubular system 8

3 LOADS 10

3.1 Vertical loads 10
3.2 Wind velocity and pressure 11
3.3 Special requirements for the wind load 12

4 GENERAL INFORMATION ABOUT THE PERSONAL PROJECT 13

4.1 A structural system of residential buildings 13
4.2 Cast-in-situ floor slabs 13
4.3 Walls 13
4.4 Example building 14

5 STABILITY CALCULATIONS 16

5.1 Vertical loads 16
5.2 Horizontal loads (wind) 17
5.3 Minimization of vertical loads 20
5.4 Additional horizontal load 20
5.5 Calculation of wall stiffness in the direction Y 22
5.6 Calculation of the force, which acting on the walls in the direction Y 25
5.7 Calculation and comparison of M(fall) and M(stab) in the direction Y 26
5.9 Calculation of wall stiffness in the direction X 34
5.10 Calculation of the forces, which acting on the walls in the direction X 36
5.11 Calculation and comparison of M(fall) and M(stab) in the direction X 37

6 CONCLUSION 40

REFERENCES 41

1 GENERAL INFORMATION ABOUT HIGH-RISE BUILDINGS

1.1 Needs for high-rise buildings

The construction of high-rise buildings was begun in the late 19^{th} century. The first high-rise buildings appeared in the United States, and within few decades they started to be built in Western Europe and Asia.

In Europe, in the beginning high-rise buildings were not built to solve any practical specific needs, but as a tribute to the technological progress and as an expression of force in society. A new approach to the placing of high-rise buildings in European cities was required with medieval centers and dominance of historical buildings. Then model of concentrated placing of multi-story houses in the center, as it is in USA, could not be used. That is why this problem was solved differently in various European countries.

The main reasons for the widespread construction of high-rise housing is the high price of land designated for the development and the lack of space in cities. That is why high-rise building construction solves both problems at once. This construction approach allows us to place on small land sites significant amount of indoor area to use, as for flats, offices, shops, hotel apartments, educational places etc. However, not only the high land price stimulated the rapid growth of that construction segment. The construction of multi-story buildings would have been impossible without the rapid development of building technologies, improvement of building materials and constant searching for new construction technologies, which allow to create high-performance designs at a minimal cost. The search for new engineering and composite solutions of the last 50 years has led to the creation of new types of buildings, which are more comfortable and meet all safety requirements.

It should be noted that high-rise buildings are complex technical constructions, which require a serious approach in all construction phases. Despite architectural simplicity multi-story buildings have complex technological and space planning solutions. A high-rise building has not only just walls, floors and highly placed roof, first of all it is a set of engineering equipment. Even the naming of all systems found in modern high-rise building can be daunting: heating, ventilation, fire protection, water supply, sewerage, waste disposal, automation, lifting equipment etc. Moreover, these systems have their own characteristics compared with those used in low-rise buildings. In most cases, it is required to put these systems in a building and establish their work, as every system can have an impact on the other. Even a qualitatively constructed high-rise building requires constant attention to the structure itself and all engineering systems. Therefore, multiple sensors are installed, which constantly deliver information about the technical situation.

The construction of multi-story buildings requires special attention to be paid to the problem of changing thermal regime of air, smoke protection in case of fire, heat supply, air conditioning, heating, ventilation, automation and control systems, security issues and psychological discomfort. It requires an unusual approach and original engineering solutions to manage all the problems in the most cases.

There are few examples to understand better all the complexity of the high-rise building. Traditional heating and water supply systems under the gravity in buildings with a height of 100 meters have a pressure more than 10 atm on the first floor. Even in the hundred-meter building water pressure will break any normal system. But gravity affects equally on other building's life-support systems. Gravity affects the garbage disposal and sewage. Speed dampers should be installed for those systems to reduce the noise. The gravitational force should also be considered in the fire protection system. Water used to extinguish the fire should flow to the drainage network. Also modern apartments should be waterproofing. Therefore, floors should have a small slope, through which water can flow into the gutter, in order not to fill the room and bring down the ceilings.

The design and installation of enclosing structures, engineering systems and safety systems of high-rise buildings require special attention. All of these systems cannot be installed without specific information support, detailed analysis of international experience and help of experts in various fields.

1.2 Structural design of high rise buildings nowadays

Buildings are called high-rise today if their height is more than 75 meters. Those buildings can have different spheres of use: hotels (W Marriott Marquis Dubai), office buildings (Metropolitan Life Insurance Company Tower), residential buildings (Pentominium), universities (Moscow State University). Usually high-rise buildings are multifunctional, in addition to the main purpose of living, shops, parking spots, cinemas and offices etc. are located there.

There are many countries including USA, which has a significant experience in structural design, construction process and exploitation of high-rise buildings. One of the first high-rise building was Woolworth Building in New-York (241m/ 57 floors), which was built more than 100 years ago, in 1913. For a long time the highest high-rise building was Empire State Building that has 102 floors and the total height is 381m

(448 m with antenna). Later, World Trade Center in New-York (415m, 417m) and Sears Tower in Chicago (442m) became the highest buildings. However, nowadays the construction of the highest skyscrapers was moved to the East, to the United Arab Emirates, Malasia, Taiwan and China.

Over the last years in big Russian cities such as Moscow and Saint Petersburg plenty of High-Rise Residential buildings were built such as “Scarlet Sails” (48 floors) and North valley (28 floors).

High-rise buildings, especially skyscrapers, have their own frame specifics, which distinguishes them from ordinary buildings. Firstly, if the height increases sharply, the load on the loadbearing systems increases, too. Thus, some structural systems were developed to manage this problem: rigid frame, framed tube, shear frame, braced frame, core structure, BES open element system and others.

The choice of the structural system depends on many factors. The main factors are the height of the building and geographic conditions, such as seismicity, soil characteristics, wind and snow loads. It should be noted that according to German researchers, the wind load in many cases is more important than seismic effects. One of the highest buildings of today, John Hancock Center in Chicago and the International Financial Center in Taipei are made on the trussed tube system, in which the outer perimeter walls are rigidly connected to the center tube and further strengthened by powerful diagonal connections. All the building, in this case, works as a rigid, stiff console.

To reduce vibrations of high-rise buildings under the influence of wind load, nowadays suspended inert mass is widely used on the top of the building.

The construction practice shows that rigid systems have a limited hardness. It is useful to use it until 40 floors. Framed tube systems can be used up to 50-60 floors. A bundled tube can handle 80-90 floors, and if it is needed to construct a building with more height, a truss-tube scheme should be used.

One of the basic requirements for high-rise buildings must be a complex security requirement, which should ensure useful evacuation ways in crisis situations, fire and anti-terrorist attacks. Also, reliable monitoring and control of all technical equipment systems, duplication of all life-support systems should be used in this type of building.

2 FRAME SYSTEMS

Various loadbearing frames can be found for high-story buildings. The best system is chosen according to different factors, such as the height of the building, architectural design and location. The higher a house is, the more different constructions are needed to manage all the horizontal loads. If there is a rigid frame building with a height of 50 floors, materials for the resisting horizontal loads are more than for the vertical. Thus, an appropriate frame system required to resist all loads and have an economic construction process. The most common structural systems for the high-rise building are discussed below.

2.1 Shear walls system

Shear walls system allow having a high resistance in their own plane to the horizontal loads.
Shear walls can be used in various ways. The first variant is to use system columns with slabs and shear walls. It has a twice more effective height than just columns and slabs. For example, a building with columns, slabs and shear walls has an effective height up to about 20 stories, whereas a system with columns and slabs has its effective height up to about 10 levels. A shear wall frame system is shown below in Figure 1. Shear walls have to be coupled by setting beams between the shear walls, as is shown in Figure 2. It is a very efficient way to enlarge the resistance of horizontal loads of a building. Also, there are a lot of spots to place windows and doors. To sum up, it is effective to construct a building with that type of frame up to 40 floors.

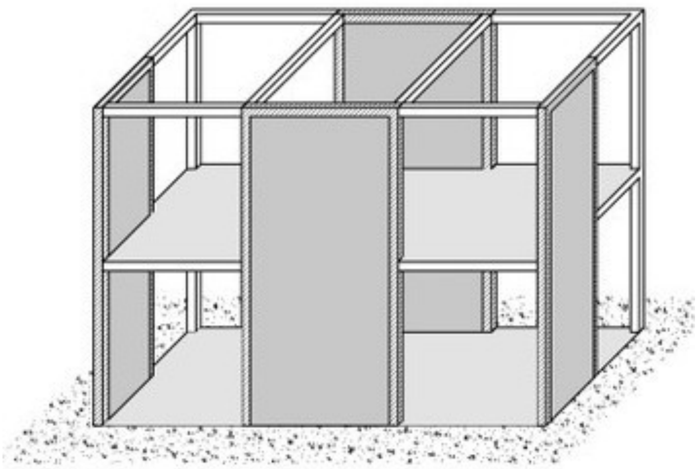

Figure 1. Illustration of shear walls frame system (Rist,V.C.& Swensson S. 2016)

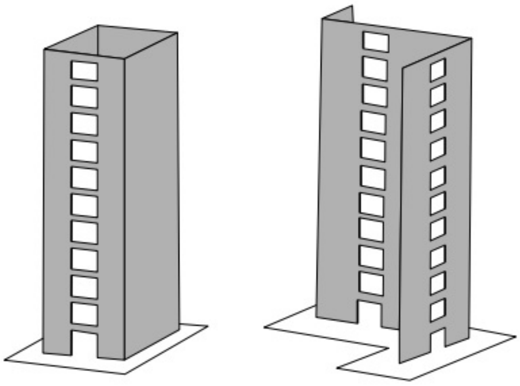

Figure 2. Illustration of coupled shear walls (Rist,V.C.& Swensson S. 2016)

2.2 Rigid frames

Rigid frames suit mostly for low-height buildings. A rigid frame is a structure of columns and beams with a connection, which resist the moments. A rigid frame system is shown in Figure 3. It manages horizontal loads by bending resistance of the whole structure (beams and columns). The sizes of columns are mostly controlled by the bending resistance, not by loading capacity, during designing phase with moment resisting frames. To reduce the horizontal load deflection a component with a high bending resistance is needed. Also, all connections must be designed properly as inadequate connections can cause major problems. Some deviation in the connection angle can contribute to a big incline of the whole construction.

The best materials to those types of frames are steel and concrete. The maximum height for steel is almost 30 floors, while concrete structures can be just 20 floors. If there is a building with more than 30 stories, there can be a risk of a big horizontal deflection, because of wind load and seismic activity. Also, connection structures in this case are too complex and expensive in order to resist big moments.

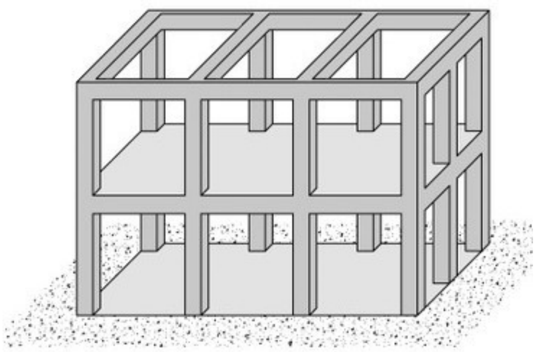

Figure 3. Illustration of the rigid frame system (Rist,V.C.& Swensson S. 2016)

2.3 Core and outrigger systems

Shear columns inside the building form the core around an elevator shaft and stairways. This core can resist all the loads: horizontal, vertical and torsional. The core system is effective up to 45 stories. Columns take all additional vertical forces.

A mix of rigid frames and shear walls can be used to construct a house up to 60 floors. The core of shear walls is around the elevator and stairways, whereas rigid frames are on exterior of the house. Core and rigid frame systems decrease the overturning moment and hazard for a core uplift.

An outrigger system can be added to the core to raise its bending resistance. The outrigger system consists of high stiff floors in the construction. Outriggers are connected to columns, which are placed along the perimeter to the ground. When the wind load acts on the structure, exterior columns reduce the moment for the core. Also, there are belt walls which resist the rotation of outriggers. Belt trusses or walls are placed on the perimeter of the outrigger floor. The core resists the shear and torsional forces, while the outrigger reduces the horizontal displacement, because of the bending. This type of system can be used in buildings of up to 150 stories.

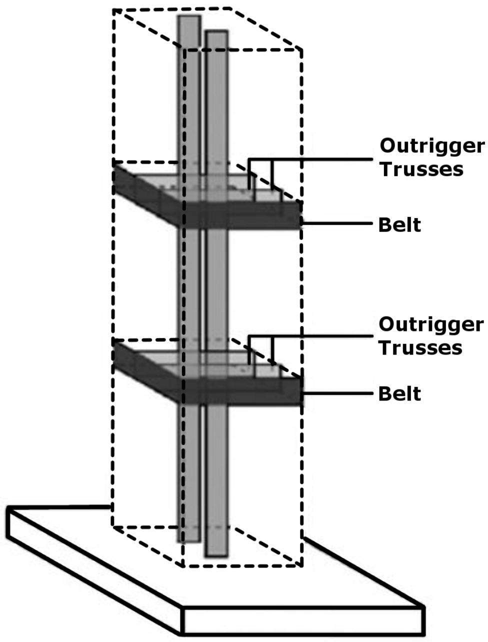

Figure 4. Illustration of the core and outrigger system (Rist,V.C.& Swensson S. 2016)

2.4 **Braced frames and shear trusses**

Sometimes diagonal braces in a rigid structure are settled to make this rigid frame stronger. Bracing systems are placed inside the frame system to manage horizontal forces; namely, it decreases the bending of columns and girders. This system is much more economical than frames with the moment resistance. Braced frames are often placed in the building core. The braced frame system is efficient up to 40 story buildings. Two types of bracing system frames are defined: eccentric braced frames and concentric braced frames. The concentric brace frame includes bracing elements crossing in the same point. As the concentric brace frame is stiff and has a very poor inelastic behavior, it is not appropriate for seismic zones. Whereas in seismic zones it is better to have an eccentric bracing system frame, as it is stiff, strong and has high inelastic characteristics of a resisting moment. This frame type is appropriate for 25-30 floors. Concentric and eccentric braced frames are illustrated in Figures 5 and 6 below.

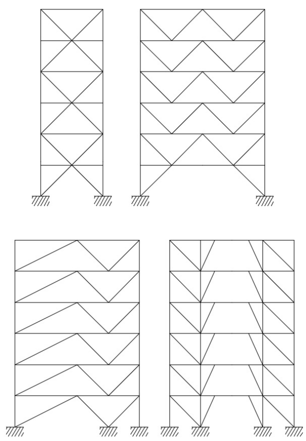

Figure 5. Illustration of concentric braced frames (Rist,V.C.& Swensson S. 2016)

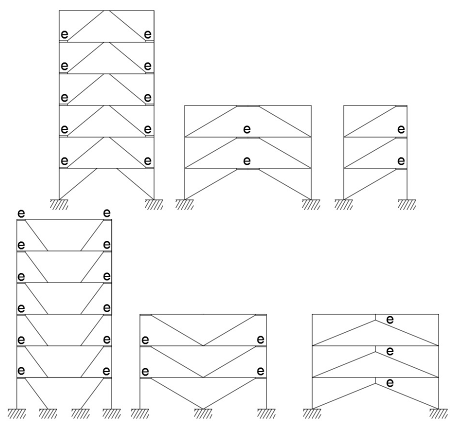

Figure 6. Illustration of eccentric braced frames (Rist,V.C.& Swensson S. 2016)

2.5 **Staggered system**

A staggered system is a variation of the truss system. Usually this system can be used up to 25 floors for narrow, long residential buildings. There are truss systems which are placed in a special sequence on each floor. This type of frame is illustrated in Figure 7. Those trusses resist the horizontal loads, whereas columns do not have any bending. Trusses should also have openings for the usage of a building, by removing some diagonal member. The stiff moment frame should be placed around the opening, to resist loads.

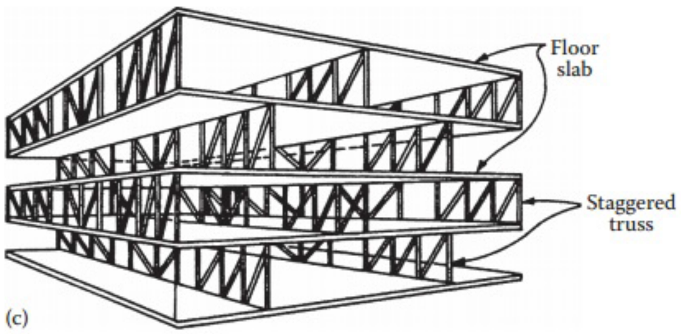

Figure 7. Illustration of a staggered truss system (Rist,V.C.& Swensson S. 2016)

2.6 **Tubular system**

A tubular system is used for buildings of up to 60 floors in height. This structure has tubes around the perimeter of construction, which take all horizontal and vertical loads of a structure. The tube frame is done by columns placed next to each other around the edge of the building. This type of structure is shown in Figure 8.

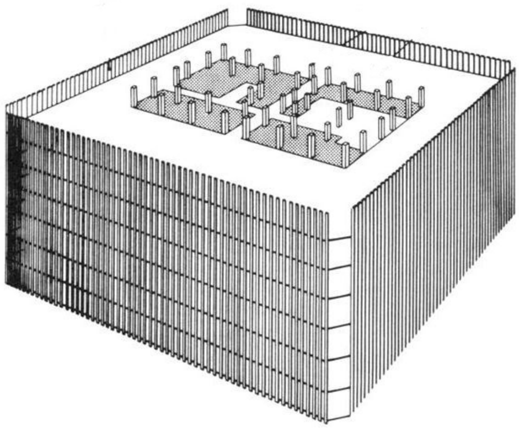

Figure 8. Illustration of the tubular system (Rist,V.C.& Swensson S. 2016)

To make a structure stiffer, external bracings can be added. This type of structure will be called an exterior diagonal tube system. It is shown in Figure 9. This type of frame is effective up to 100 floors.

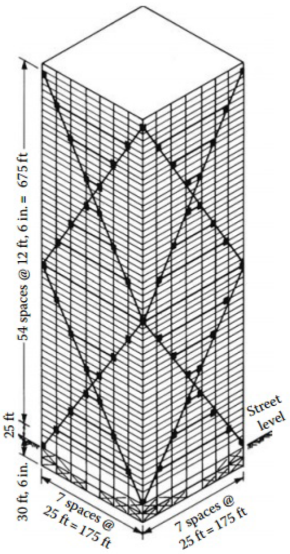

Figure 9. Illustration of an exterior diagonal tube system (Rist,V.C.& Swensson S. 2016)

A bundled tube system consists of connected tubes. This type of system is shown in Figure 10.This system resists big loads, as tubes are connected together, there are wide spacings for windows.

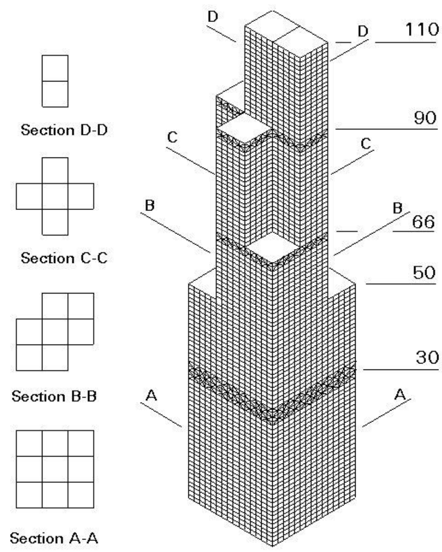

Figure 10. Illustration of a bundled tube system (Rist,V.C.& Swensson S. 2016)

A tube-in-tube system is obtained, if the core is placed inside the tube frame structure. There are advantages of both systems, whereas the frame resists great loads.

3 LOADS

The structure of any building is exposed to various types of loads. Thus, every construction has to manage with the vertical and horizontal loads that are mentioned below.

- Dead load. It includes the self weight of the whole structure (load bearing members, non-structural members and whole installation structures)
- Live load. It includes the whole load from occupants and furniture.
- Snow load.
- Lateral load. It includes the wind load.

All loads should be calculated according to the instructions in EN 1991-1-1, EN 1991-1-3, EN 1991-1-4. High-story houses should also be designed regarding the special requirements in Eurocodes, which are discussed below.

3.1 Vertical loads

In accordance with EC 1991-1-1, 6.2.2(2) the reduction factor a_n for the live load in the multiple story building should be determined within the formula below.

$$\alpha_n = \frac{2+(n-2)\psi_0}{n} \tag{1}$$

where:

n is the number of floors (more than 2) above the loaded structural elements from the same category.

Ψ is taken from EN 1990, Annex A1, Table A1.1.

Reduction factor can also be calculated with the use of the area according to EN 1991-1-1 chapter 6.3.1.2(10). a_b can be calculated within the formula below.

$$\alpha_A = \frac{5}{7} \cdot \psi_0 + \frac{A_0}{A} \leq 1.0 \tag{2}$$

A_0 – the constant, equals to 10 m^2.

A – loaded area

3.2 Wind velocity and pressure

EN 1991-1-4 gives instructions for the calculation of high-rise buildings up to 200 meters. The wind load concerning Eurocodes acts on the structure statically and dynamically. While tall buildings have to consider both parts of it, as dynamic part is in charge of the dynamic nature of wind, and the static affects just as distributed horizontal load of the wind. The reference peak velocity q_p the dynamic and static parts with the help of I_v *(*turbulence intensity*).* c_d is considered a dynamic response during the calculation process of statics loads.

q_b -basic velocity pressure
v_b - basic wind velocity
ρ – air density (1.25 kg/m^3)

$$q_b = \frac{1}{2} \cdot \rho_{air} \cdot v_b^2 \tag{3}$$

q_p – peak velocity pressure
I_v - turbulence intensity
c_0 – orography factor. The value varies if the ground levels are different in that area, for example hills of cliffs. It can be not considered if the slope of upwind less than 3^0 . The wind velocity rises in that case. EC 1991-1-4 A.3.
z_0 – roughness length. It can be found in appendix A.1.

$$q_p(z) = [1 + 6 \cdot I_v(z)] \cdot \left[k_r \cdot \ln\left(\frac{z}{z_0}\right) \cdot c_0(z)\right]^2 \cdot q_b \tag{4}$$

A simplified calculation version of q_p when orography factor is 1.0.

$$q_p(z) = c_e(z) \cdot q_b \tag{5}$$

$c_e(z)$ - exposure factor. It can be found in Figure A.7 EC 1991-1-4 A.1

$v_m(z)$ – mean wind velocity
v_b – basic wind velocity

$$v_m(z) = c_r(z) \cdot c_0(z) \cdot v_b \tag{6}$$

$c_r(z)$ – roughness factor

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \tag{7}$$

$$k_r = 0.19 \cdot \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{8}$$

z, $z_{o,II}$ – it can be found from the EC 1991-1-4 A.1.

3.3 Special requirements for the wind load

The wind force can be found from the equation below.

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \tag{9}$$

$$q_p(z) = c_e(z) \cdot q_b \tag{10}$$

$q_p(z)$ – is the peak velocity pressure.
c_f – force coefficient. It can be found in EN 1991-1-4 section 7,8.
A_{ref} – reference area.
c_s - non-simultaneous occurrence of peak wind pressure on the surface.
c_{d} – vibration effect to the structure due to turbulence.
$c_s c_d$ – should be calculated together from the following equation.

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s) \cdot \sqrt{B^2 + R^2}}{1 + 6 \cdot I_v(z_s)} \tag{11}$$

The value of $c_s c_d$ should be 1 for the buildings up to 15 m. In the case of high-story buildings, the EC 1991-1-4, Annex D should be used.

4 GENERAL INFORMATION ABOUT THE PERSONAL PROJECT

The main part of the project consists of calculation of basic loads and wind load for the 25 story high-rise building. Calculations are based on Euro-Codes.

4.1 A structural system of residential buildings

Residential buildings in Finland usually consist of reinforced concrete walls (shear wall system).

Cast-in-situ slabs, bearing partition wall elements, bearing external wall elements and light external wall elements are the basic elements of this system. The length and thickness of slabs can be chosen within the limits of the loading capacity. This system gives the architect freedom to design interior of the building.

4.2 Cast-in-situ floor slabs

The height of a cast-in-situ slab in residential buildings is usually 300 mm with a 20 mm concrete topping. Also, this height frequently comes from the requirements of sound insulation.

Cast-in-situ slab floors are stabilized by means of peripheral reinforcement around the whole floor. The resulting plates transmit horizontal forces (wind, stabilizing, seismic and other loads) to the vertical stabilizing structures in proportion to their rigidities.

4.3 Walls

The thickness of the bearing partition wall in a shear wall system is 180 mm, it provides a supporting surface for the slabs, sound insulation and vertical load bearing capacity for buildings up to 8 stories. In our case 250mm, 275mm, 300mm, 400mm wall thickness was selected for the load-bearing capacity for all the building from the 3rd to 25th floor.

Usually, the minimum internal wall thicknesses are 200 mm for a loadbearing structure and 80 mm for non-loadbearing walls.

This system is vertically stabilized by walls, which work as fixed-based cantilever columns.

If necessary the wall elements can be combined by means of vertical joints strengthened with steel and concrete dowels to form larger and more rigid units. As much as possible of the total vertical loading should be transmitted to the bracing walls. The best results are obtained if the wall sections are completely compressed throughout. If tensile stresses arise in the walls due to the action of horizontal forces, vertical reinforcement extending from one wall element to the next must be installed, usually in the form of ordinary deformed steel reinforcement bars. If the tensile forces are high, the elements can be connected together by means of prestressing tendons.

Bearing walls can be erected either parallel or transverse to the longitudinal axis of the building.

4.4 Example building

Drawings of the example building can be found in Figure 11 and Figure 12.
The following contains basic technical information and material aspects of the example building:

Number of floors:		25
Consequence class:		CC3
Distance between the floors:		3000 mm
Floors:		
	Concrete slab + concrete topping	300 mm + 20mm
Bearing walls:		250 mm, 275mm, 300mm, 400 mm
Non-bearing external walls:		160mm
Material properties:		
Concrete:		
	C23/37	Loadbearing walls, columns
	C18/30	Slabs
Reinforcement:		
	A500HW	Concrete reinforcement
	B500K	Reinforcing nets
Corrosivity categories for concrete structures:		
	XC1	Loadbearing structures
	XC2	Foundation walls and the basement
	XC1	External walls
Concrete tolerance:		10mm for cast-in-situ structures
Exploitation period:		100 years

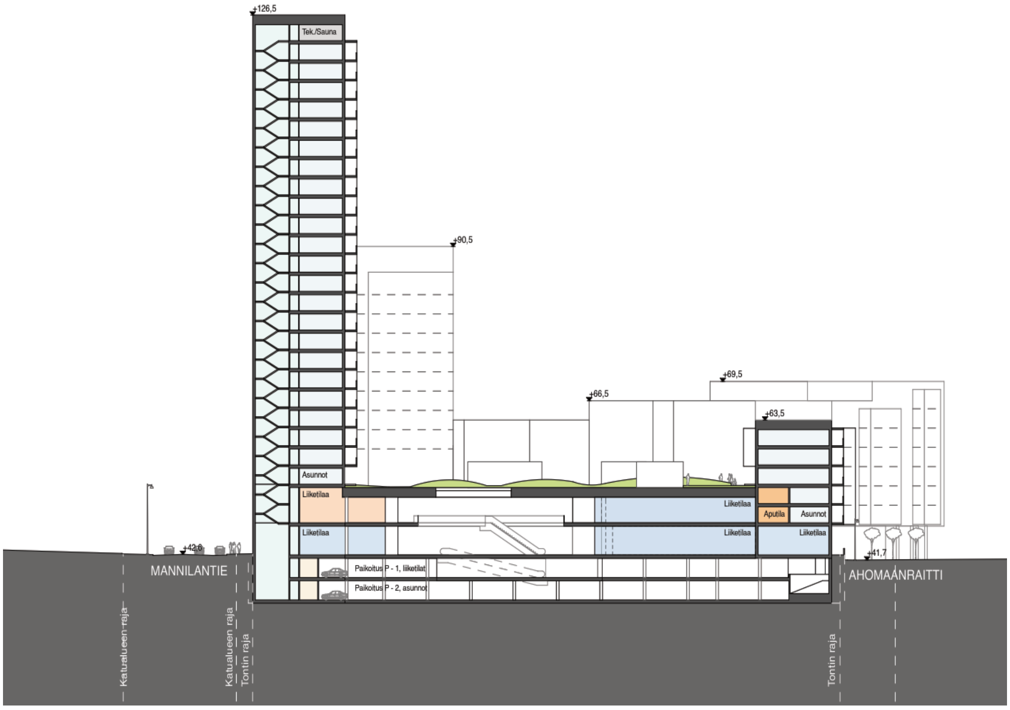

Figure 11. Side architectural view of the building

Figure 12. 3D side view of the whole construction complex

5 STABILITY CALCULATIONS

The stability calculations of the building will be discussed in the following chapters.

5.1 Vertical loads

The basic vertical loads are discussed below.

A. Cast-in-situ slab

g_1= 0.5 kN/m^2 concrete topping 20 mm
g_2 = 7.5 kN/m^2 Concrete slab 300 mm

q1 = 2.5 kN/m^2 live load
*personal-load 2kN/m^2 + lightweight walls 0.5 kN/m^2

In accordance with EC 1991-1-1, 6.2.2(2) reduction factor equals to

a_n = (2+(25-2)*0,7)/25=0.724

⇨ q_1= 0,724*2 + 0,5= 1.948 kN/m^2

B. A bearing walls

g_3= 14.4 kN/m * 200 mm concrete
g_4= 18 kN/m * 250 mm concrete
g_5= 19.8 kN/m * 275 mm concrete
g_6= 21.6 kN/m * 300 mm concrete
g_7= 28.8 kN/m * 400 mm concrete

C. A non-bearing external wall

g_8=12 kN/m * internal layer 80 mm concrete + external layer 80 mm concrete

D. Roof

q_2 = 7.5 kN/m^2 * Concrete slab 300mm
q_3 = 1 kN/m^2 * Layer above the concrete slab

5.2 **Horizontal loads (wind)**

The basic horizontal load calculation process will be discussed below.

The wind load is calculated according to SFS-EN 1991-1-4:2005.

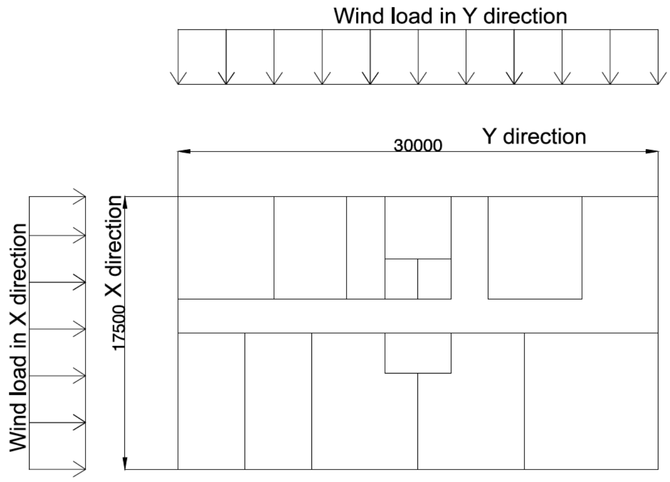

Figure 13. Schematic plan view

A. Direction Y, Floors 3-9

$\mathbf{F_w}$= c_s*c_d*c_f*q_p(n)*A_{ref} (12)

$\mathbf{F_w}$ = 75,513 kN

$\mathbf{c_s}$***c_d** = (1+2*k_p*$I_v(z_s)$*(B^2+R^2)^0.5)/(1+7* $I_v(z_s)$) (13)

$\mathbf{c_s}$***c_d** = 0.8875, (Figure 14)

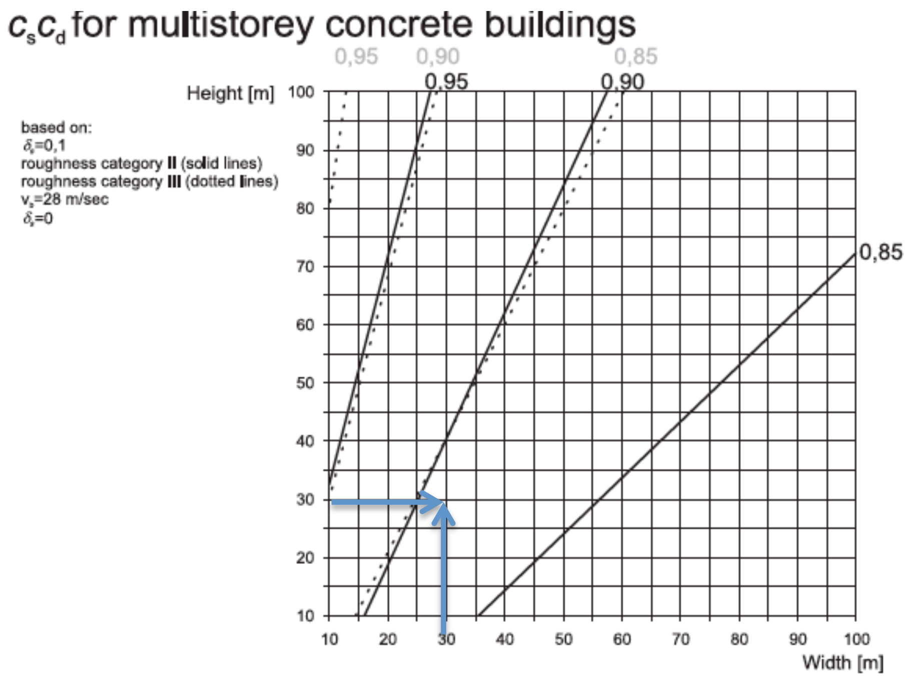

Figure 14. c_sc_d for multistory concrete buildings (SFS-EN 1991-1-4:2005)

$\boldsymbol{\lambda}$ = 1.82*h/b = 1.94, (14)

Using interpolation, as λ=2*h/b, when h <15 and λ=1.4*h/b, when h>=50

d/b = 0.58

c_f = 1.4 from the Table 5.2S, 5.3.1S, EN 1991-1-4

$\mathbf{q_p(32)}$= (1 + 7 * $I_v(z_s)$) * 0.5 * ρ * $v_m^2(z)$= $c_e(z)$ * q_b (15)

$\mathbf{q_p(32)}$= 675,28 N/m^2

$\mathbf{I_v(z)}$ - turbulence intensity
ρ – air density 1,225 kg/m^3
$\mathbf{v_m}$ – basic wind speed 21 m/s
$\mathbf{c_e}$ = 2,5 , exposure factor at 32 m (figure 15)

$\mathbf{I_v(z)}$ = $k_1/(c_0(z)*\ln(z/z_0))$ (16)

$\mathbf{k_1}$ – turbulence factor
$\mathbf{c_0}$ – the surface shape factor
z= 32m, height of the building from the soil level
$\mathbf{z_0}$ – roughness measurement

$\mathbf{q_b}$= 0.5* ρ*v_b^2 (17)

$\mathbf{q_b}$= 270,11 N/m^2

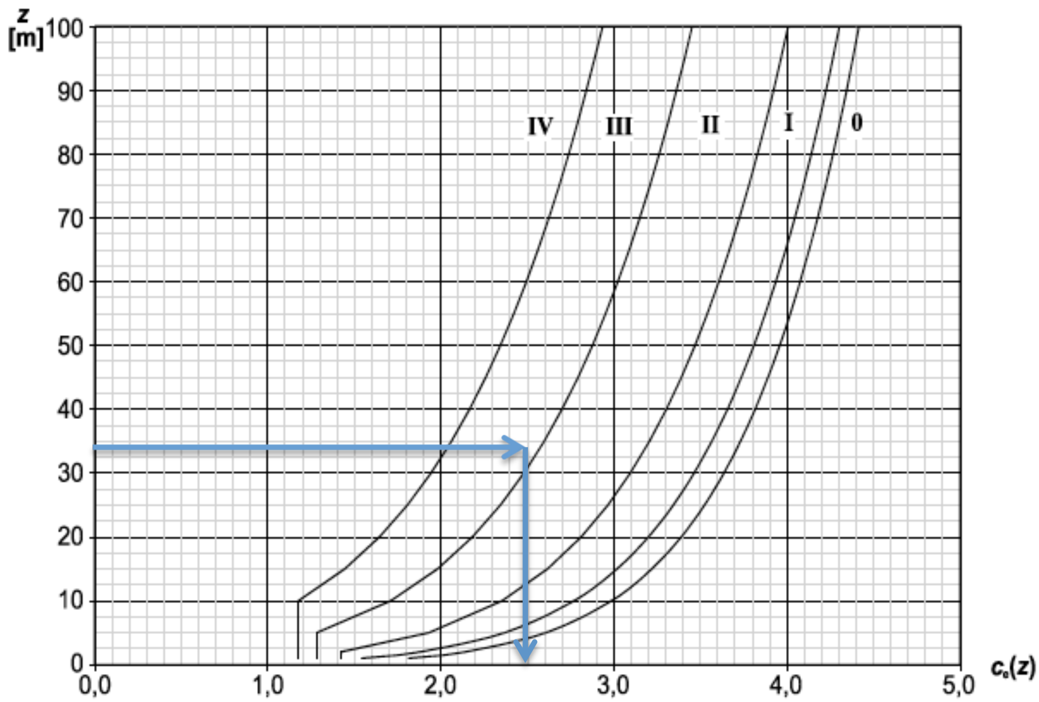

Figure 15. Illustration of the exposure factor $c_e(z)$ for c_0=1, k_r=1 (SFS-EN 1991-1-4:2005)

$\mathbf{A_{ref}} = b*h$ (18)

$\mathbf{A_{ref}} = 30*3 = 90\ (m^2)$

$\mathbf{F_w}= c_s*c_d*c*q_p(n)*A_{ref}$, wind load for the 1 story height (19)

$\mathbf{F_w}= 75{,}513\ (kN)$

B.

Direction Y, Floor 10-25		
H	81,00	m
c_s*c_d	0,9357	
λ	3,78	
d/b	0,58	
c_f	1,53	
c_e	3,27	
q_b	270,11	N/m^2
q_p	883,27	N/m^2
A_{ref}	90,00	m^2
F_w	113,805	kN

C.

Direction X, Floor 3-9		
H	32,00	m
c_s*c_d	0,935	
λ	3,34	
d/b	1,71	
c_f	1,2	
c_e	2,50	
q_b	270,11	N/m^2
q_p	675,28	N/m^2
A_{ref}	52,50	m^2
F_w	39,777	kN

D.

Direction X, Floor 10-25		
H	81,00	m
c_s*c_d	0,9673	
λ	6,48	
d/b	1,71	
c_f	1,25	
c_e	3,27	
q_b	270,11	N/m^2
q_p	883,27	N/m^2
A_{ref}	52,50	m^2
F_w	56,069	kN

5.3 Minimization of vertical loads

The basic minimization load calculation process will be discussed below.

$\mathbf{N_d(roof)}$ = 30*17,5*(0.9(7.5+1)) = 4016.25 kN (20)

$\mathbf{N_d}$**(loadbearing walls level 3-25)** = 8*4.25*0.9*18 + 6.325*0.9*18 +
+ 6.575*2*0.9*28.8 + 6.575*2*0.9*18 +6.325*0.9*18 +
+ 8.35*2*0.9*21.6+ 6.05*0.9*18 + 3.05*0.9*18 + 10.030*0.9*18 +
+ 6.3*0.9*18 + +6.03*0.9*19.8 + 3.475*0.9*18 + 8.3*0.9*18 +
+ 12.3*0.9*28.8 + 9.05*0.9*18= 2810 kN (21)

$\mathbf{N_d}$**(non loadbearing external wall) =** 30*2*0.9*11,52 = 622.08 kN (22)

$\mathbf{N_d}$**(walls level 3-9)** = N_d(loadbearing walls level 3-25) + N_d(non loadbearing external wall) = 3432 kN (23)

$\mathbf{N_d}$**(walls level 10-25)** = N_d(loadbearing walls level 3-25) + N_d(non loadbearing external wall) = 3432 kN (24)

$\mathbf{N_d}$**(floor)** = 0.9*8*17.5*30= 3780 kN (25)

$\mathbf{N_d}$**(stairwell)** = 6*2.94*0.9 *0.3*25= 114.31 kN (26)

5.4 Additional horizontal load

The values of addition of horizontal loads are based on the inclination of the vertical loadbearing structures. Additional horizontal loads act on each floor at the points of horizontal load action.

$\boldsymbol{\Theta i} = \alpha_n * \alpha_m * \Theta_0$ (27)

$\boldsymbol{\Theta_0} = 1/200$ (28)

$\boldsymbol{\alpha_n} = 2/h^{0.5}$ (29)

$\boldsymbol{\alpha_n} = 0.23$

$\boldsymbol{\alpha_n} = 2/3$, as $2/3 < \boldsymbol{\alpha_n} < 1.0$

$\boldsymbol{\alpha_m} = (0.5*(1+1/m))^{0.5}$ (30)

$\boldsymbol{\alpha_m} = 0.7416$

m = 10, number of stiffener walls

Ɵi = 0.002472

Hd (roof)= Ɵi*(N_d(walls)+N_d(roof)) (31)

Hd (roof) = 0.002472*(3432 + 4016.2) = 18.4 kN

Hd (floor 3-25) = 0.002472 * (3780 + 3432 + 114.3) = 18.11 kN (32)

Hd (roof direction Y) = 1.5* Fw + Hd(roof) (33)

Hd (roof direction Y) =1.5* 113,805+ 18.4 =189.1 kN/floor

Hd (roof direction X) = 1.5* Fw + Hd(roof) (34)

Hd (roof direction X) = 1.5* 56,069 + 18.4 = 102.5 kN/floor

Hd (floor 3-9 direction Y)= 1.5 * Fw + Hd (floor 3-9) (35)

Hd (floor 3-9 direction Y)= 1.5 * 75,513+ 18.11 = 131.4 kN/floor

Hd (floor 10-25 direction Y) = 1.5*Fw+ Hd (floor 10-25) (36)

Hd (floor 10-25 direction Y) = 1.5*113.805 + 18.11 = 188.8 kN/floor

Hd (floor 3-9 direction X) = 1.5 * Fw + Hd (floor 3-9) (37)

Hd (floor 3-9 direction X = 1.5 * 39,777+ 18.11 = 77.8 kN/floor

Hd (floor 10-25 direction X) =1.5*Fw + Hd (floor 10-25) (38)

Hd (floor 10-25 direction X) =1.5 * 56,069 +18.11= 102.2 kN/floor

5.5 Calculation of wall stiffness in the direction Y

A construction going from 3-25 floors and roof was taken into consideration. The basic wall stiffness calculation process in the direction Y will be discussed below.

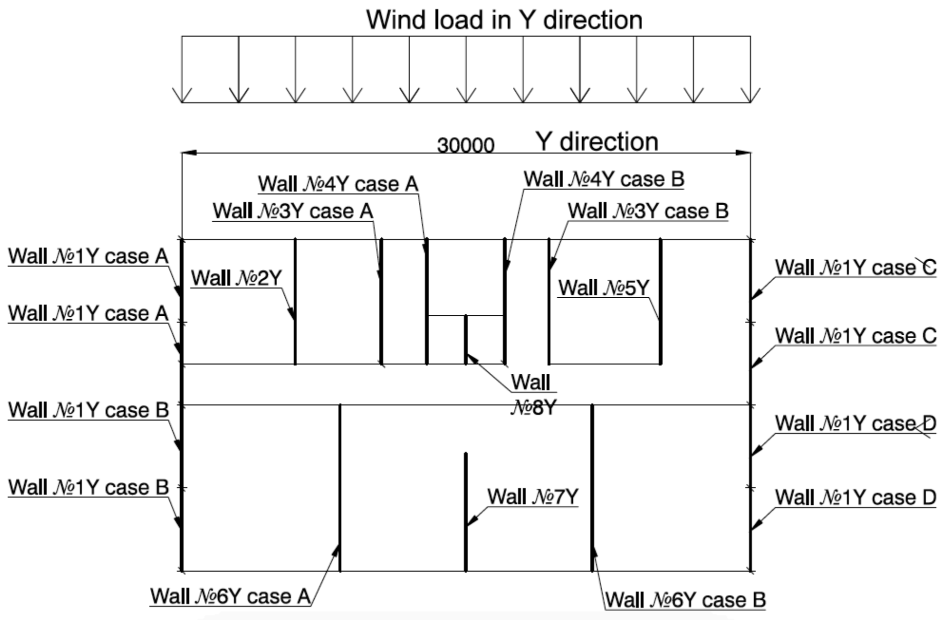

Figure 16. Loadbearing walls against wind load in the direction Y

a) Wall №1Y

n = 8, number of walls
L = 80m, height from the ground level to the top
h = 4,25 length of the wall
b = 0,25m, width of the wall
E = 30000 MN/m^2
k$_y$ = 1,2
v = 0,3

I = $b*h^3/12$ (39)

I = 1,6 m^4

A = b*h = 1,0625 m^2 (40)

k$_1$y = $E/((L^3/(3I_y) + (2*(1+ v)* k_y*L)/A))$ = 0,2805 MN/m (41)

∑k$_1$y = n* k_1x = 2,244 MN/m (42)

b) Wall №2Y

n	1	
L	80	m
h	6,325	m
b	0,25	m
E	30000	MN/mm^2
I	5,27	m^4
A	1,58125	m^2
k_y	1,2	
v	0,3	
k_{2y}	0,92214786	MN/m
$\sum k_{2y}$	0,92214786	MN/m

c) Wall №3Y

n	2	
L	80	m
h	6,575	m
b	0,4	m
E	30000	MN/mm^2
I	9,47	m^4
A	2,63	m^2
k_y	1,2	
v	0,3	
k_{3y}	1,656747754	MN/m
$\sum k_{3y}$	3,313495509	MN/m

d) Wall №4Y

n	2	
L	80	m
h	6,575	m
b	0,25	m
E	30000	MN/mm^2
I	5,92	m^4
A	1,64375	m^2
k_y	1,2	
v	0,3	
k_{4y}	1,035467347	MN/m
$\sum k_{4y}$	2,070934693	MN/m

e) Wall №5Y

n	1	
L	80	m
h	6,325	m
b	0,25	m
E	30000	MN/mm^2
I	5,27	m^4
A	1,58125	m^2
k_y	1,2	
v	0,3	
k_{5y}	0,92214786	MN/m
$\sum k_{5y}$	0,92214786	MN/m

f) Wall №6Y

n	2	
L	80	m
h	8,35	m
b	0,35	m
E	30000	MN/mm^2
I	16,98	m^4
A	2,9225	m^2
k_y	1,2	
v	0,3	
k_{6y}	2,959674683	MN/m
$\sum k_{6y}$	5,919349366	MN/m

g) Wall №7Y

n	1	
L	80	m
h	6,05	m
b	0,25	m
E	30000	MN/mm^2
I	4,61	m^4
A	1,5125	m^2
k_y	1,2	
v	0,3	
k_{7y}	0,807354716	MN/m
$\sum k_{7y}$	0,807354716	MN/m

h) Wall №8Y

n	1	
L	80	m
h	3,05	m
b	0,25	m
E	30000	MN/mm^2
I	0,59	m^4
A	0,7625	m^2
k_y	1,2	
v	0,3	
k_{8y}	0,103785989	MN/m
$\sum k_{8y}$	0,103785989	MN/m

5.6 Calculation of the force, which acting on the walls in the direction Y

The basic horizontal force acting on each wall calculation process in the direction Y will be discussed below.

$\boldsymbol{\sum k_i y}$– sum of the whole wall stiffness
$\boldsymbol{v_y}$ - deflection
$\boldsymbol{\sum k_i y} = \sum k_1 y + \sum k_2 y + \sum k_3 y + \sum k_4 y + \sum k_5 y + \sum k_6 y + \sum k_7 y + \sum k_8 y$ (43)

$\boldsymbol{\sum k_i y}$ = 16,92 MN/m

The maximum force was taken in that case, which acts on the highest wall and roof.

$\boldsymbol{F_y}$ = Hd(roof direction Y) = 189,1 kN = 0,1891 MN

$\boldsymbol{v_y}$ = Hd(roof direction Y) / $\sum k_i y$ (44)

$\boldsymbol{v_y}$ = 0,0115989 m

$\boldsymbol{Q_i y} = k_i y * F_y$ (45)

Q1y	0,003253568	MN
Q2y	0,010695902	MN
Q3y	0,019216453	MN
Q4y	0,012010283	MN
Q5y	0,010695902	MN
Q6y	0,034328973	MN
Q7y	0,009364427	MN
Q8y	0,001203803	MN

5.7 Calculation and comparison of M(fall) and M(stab) in the direction Y

The basic calculation and comparison process of M(fall) and M(stab) in the Y direction will be discussed below.

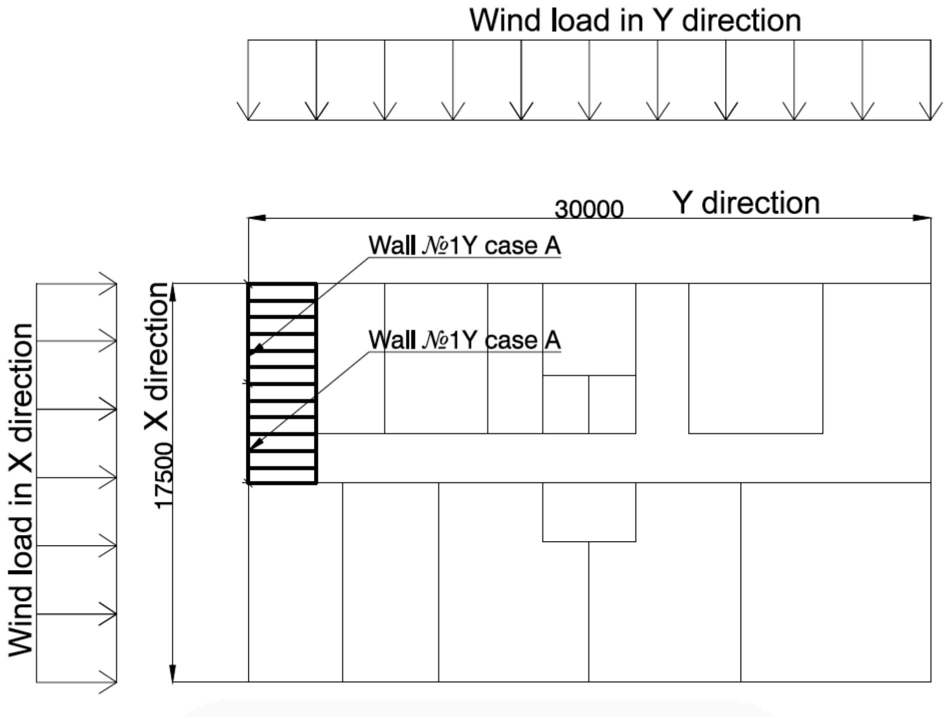

Figure 17. Schematic plan view of the building. Calculation of the forces, which are acting on the wall №1Y case A

Y1) Wall №1Y case A

The wall with the total length of 17,5m was taken into consideration. Floors, which were taken in those calculations, are 3-25. However it was divided into 4 pieces, so the length is 4,25 m from the structural drawings. Also, the span of the spread load, which goes from the floor, in that case is 5,93*0,5.

a_i – the height from the soil level to the point, where the load acts (Figure 18).

H_1d = Q_1y = 20,59 kN (Figure 18)

a_3 =11m, **a_4**= 14m … **a_{roof}**= 81m (Figure 18)

M(fall) – moment, which tries to collapse and overturn the structure.

M(fall) = Q_1y (roof direction Y) * (a_3 + … + a_{roof}) (46)

M(fall) = 3556,15 kNm

Nd(wall 4,25m, height 11-81m) = g_4(wall 250mm) * L (4,25m) (47)

Nd(wall 4,25m, height 11-81m) = 18 * 4,25 = 76,5 kN

Nd(roof 4,25m, height 11-81m) = 0,5*L(wall)*L(loading area) * (0,9 *(q(concrete slab)+ q(topping)) (48)

Nd(roof 4,25m, height 11-81m) = 0,5*4,25*5,93*(0,9*(7,5+1)) = 96,4 kN

Nd(floor 8,75m,height 11-81m) =0,5*4,25*5,93*(0,9*(7,5+0,5))=90,73kN

∑Nd = Nd(roof 4,25m, height 11-81m) + 17*Nd(wall 4,25m, height 11-81m) + 17* Nd(floor 8,75m,height 11-81m) (49)

∑Nd = 3775,43 kN

a – distance from the edge of wall side to middle point of compression area (Figure 18).
e – distance from the middle point of compression area to the vertical load (Figure 18).

∑Nd = 0,25 * 2a * f_{cd} (50)

f_{cd}= 17000 kN/m^2

a = ∑Nd/(0,25 * 2* f_{cd}) (51)

a = 0,444m

e = L/2 – a (52)

e = 4,25/2 – 0,44 = 1,68 m

M(stab) = e * ∑Nd (53)

M(stab) = 6345,87 kNm

M(stab) > **M(fall)**, those walls do not need tensile rebars.

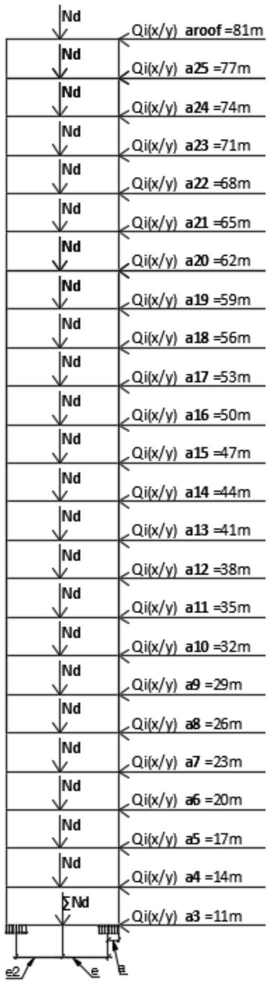

Figure 18. Schematic side view of the checked wall

Y2) Wall №1Y case B

H1d = Q1y	3,25	kN
M (falling)	3556,15	kNm
Nd(wall 4,25m)	76,50	kN
Nda (roof)	134,93	kN
Nd(floor 3-25 direction Y)	126,99	kN
∑Nd	4611,71	kN
fcd	17000,00	kN
a	0,54	m
e	1,58	m
M(stab)	7297,78	kNm
M(stab)>M(falling)		

Y3) Wall №1Y case C

H1d = Q1y	3,25	kN
M (falling)	3556,15	kNm
Nd(wall 4,25m)	76,50	kN
Nda (roof)	77,62	kN
Nd(floor 3-25 direction Y)	73,06	kN
∑Nd	3367,89	kN
fcd	17000,00	kN
a	0,40	m
e	1,73	m
M(stab)	5822,33	kNm
M(stab)>M(falling)		

Y4) Wall №1Y case D

H1d = Q1x	3,25	kN
M (falling)	3556,15	kNm
Nd(wall 4,25m)	76,50	kN
Nda (roof)	142,65	kN
Nd(floor 3-25 direction Y)	134,26	kN
∑Nd	4779,31	kN
fcd	17000,00	kN
a	0,56	m
e	1,56	m
M(stab)	7468,77	kNm
M(stab)>M(falling)		

Y5) Wall №2Y

H1d = Q2y	10,70	kN
M (falling)	11690,62	kNm
Nd(wall 6,325m)	113,85	kN
Nda (roof)	239,03	kN
Nd(floor 3-25 direction Y)	224,97	kN
∑Nd	7693,02	kN
fcd	17000	kN
a	0,905	m
e	2,257	m
M(stab)	17366,56	kNm
M(stab)>M(falling)		

Y6) Wall №3Y case A

H1d = Q3y	19,22	kN
M (falling)	21003,58	kNm
Nd(wall 6,575m)	189,36	kN
Nda (roof)	173,53	kN
Nd(floor 3-25 direction Y)	163,32	kN
∑Nd	7932,56	kN
fcd	17000,00	kN
a	0,93	m
e	2,35	m
M(stab)	18675,29	kNm
M(stab)<M(falling)		

M(stab) **< M(fall)**. Tensile reinforcement is needed.

Nd(wall 6,575)	189,36	kN
e2	5,14	m
Ns	452,82	kN
fyd	435,00	N/mm2
As	1040,97	mm2

As – area of tensile needed to resist M(fall)
Ns= $(1/e_2)*(M_{fall} - \sum Nd*e)$ – force needed to resist M(fall)
e_2= l - 0,5m - a – distance from the middle of compression area to the middle of the tensile area

Y7) Wall №3Y case B

H1d = Q3y	19,22	kN
M (falling)	21003,58	kNm
Nd(wall 6,575m)	189,36	kN
Nda (roof)	168,50	kN
Nd(floor 3-25 direction Y)	158,59	kN
∑Nd	7823,38	kN
fcd	17000,00	kN
a	0,92	m
e	2,37	m
M(stab)	18518,74	kNm
M(stab)<M(falling)		
Nd(wall 6,575m)	189,36	kN
e_2	4,50	m
Ns	552,03	kN
fyd	435,00	N/mm2
As	1269,03	mm2

Y8) Wall №4Ycase A

H1d = Q4y	12,01	kN
M (falling)	13127,24	kNm
Nd(wall 6,575m)	118,35	kN
Nda (roof)	139,66	kN
Nd(floor 3-25 direction Y)	131,45	kN
∑Nd	5635,20	kN
fcd	17000,00	kN
a	0,66	m
e	2,62	m
M(stab)	14789,79	kNm
M(stab)>M(falling)		

Y9) Wall №4Y case B

H1d = Q4y	12,01	kN
M (falling)	13127,24	kNm
Nd(wall 6,575m)	118,35	kN
Nda (roof)	90,91	kN
Nd(floor 3-25 direction Y)	85,56	kN
∑Nd	4577,03	kN
fcd	17000,00	kN
a	0,54	m
e	2,75	m
M(stab)	12582,38	kNm
M(stab)>M(falling)		
Nd(wall 6,575)	118,35	kN
e2	5,54	m
Ns	98,41	kN
fyd	435,00	N/mm2
As	226,23	mm2

Y10) Wall №5Y

H1d = Q5y	10,70	kN
M (falling)	11690,62	kNm
Nd(wall 6,325m)	113,85	kN
Nda (roof)	254,15	kN
Nd(floor 3-25 direction Y)	239,20	kN
∑Nd	8021,22	kN
fcd	17000,00	kN
a	0,94	m
e	2,22	m
M(stab)	17797,70	kNm
M(stab)>M(falling)		

Y11) Wall №6Y case A

H1d = Q6y	34,33	kN
M (falling)	37521,57	kNm
Nd(wall 8,35m)	180,36	kN
Nda (roof)	464,71	kN
Nd(floor 3-25 direction Y)	437,37	kN
∑Nd	14054,83	kN
fcd	17000,00	kN
a	1,65	m
e	2,52	m
M(stab)	35439,13	kNm
M(stab) < M(falling)		
Nd(wall 8,35m)	180,36	kN
e2	6,20	m
Ns	336,07	kN
fyd	435,00	N/mm2
As	772,57	mm2

Y12) Wall №6Y case B

H1d = Q6y	34,33	kN
M (falling)	37521,57	kNm
Nd(wall 8,35m)	180,36	kN
Nda (roof)	470,30	kN
Nd(floor 3-25 direction Y)	442,63	kN
∑Nd	14176,16	kN
fcd	17000,00	kN
a	1,67	m
e	2,51	m
M(stab)	35542,70	kNm
M(stab < M(falling)		
Nd(wall 8,35m)	180,36	kN
e2	6,18	m
Ns	320,09	kN
fyd	435,00	N/mm2
As	735,84	mm2

Y13) Wall №7Y

H1d = Q7y	9,36	kN
M (falling)	10235,32	kNm
Nd(wall 6,05m)	108,90	kN
Nda (roof)	278,85	kN
Nd(floor 3-25 direction Y)	262,45	kN
∑Nd	8448,53	kN
fcd	17000,00	kN
a	0,99	m
e	2,03	m
M(stab)	17159,43	kNm
M(stab)>M(falling)		

Y14) Wall №8Y

H1d = Q8y	1,20	kN
M (falling)	1 315,76	kNm
Nd(wall 3,05m)	54,90	kN
Nda (roof)	43,17	kN
Nd(floor 3-25 direction Y)	40,63	kN
∑Nd	2 144,74	kN
fcd	17 000,00	kN
a	0,25	m
e	2,77	m
M(stab)	5 946,67	kNm
M(stab)>M(falling)		

5.9 Calculation of wall stiffness in the direction X

A construction going from 3-25 floors and roof was taken into consideration. The basic wall stiffness calculation process in the direction X will be discussed below.

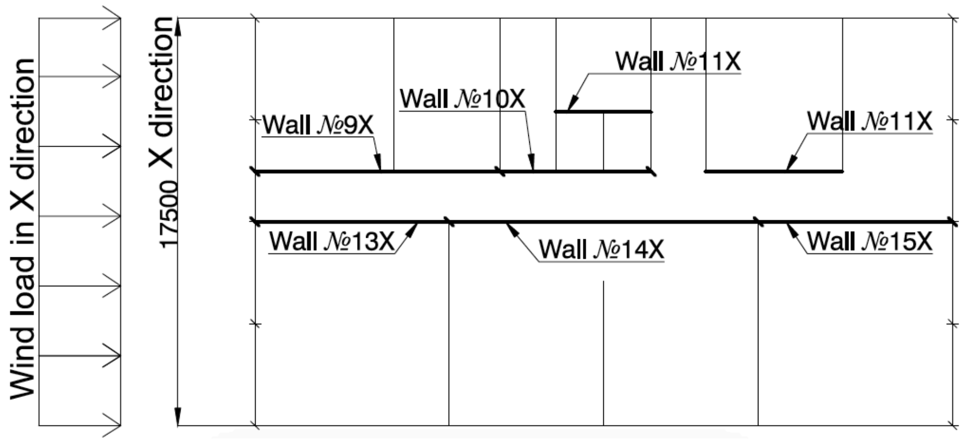

Figure 19. Loadbearing walls against wind load in the direction X

a) Wall №9X

n	1,00	
L	80,00	m
h	10,03	m
b	0,25	m
E	30 000,00	MN/mm^2
I	21,02	m^4
A	2,51	m^2
ky	1,20	
v	0,30	
k_{9x}	3,65	MN/m
$\sum k_{9x}$	3,65	MN/m

b) Wall №10X

n	1,00	
L	80,00	m
h	6,30	m
b	0,25	m
E	30 000,00	MN/mm^2
I	5,21	m^4
A	1,58	m^2
ky	1,20	
v	0,30	
k_{10x}	0,91	MN/m
$\sum k_{10x}$	0,91	MN/m

c) Wall №11X

n	1,00	
L	80,00	m
h	6,03	m
b	0,28	m
E	30 000,00	MN/mm^2
I	5,02	m^4
A	1,66	m^2
ky	1,20	
v	0,30	
k_{11x}	0,88	MN/m
$\sum k_{11x}$	0,88	MN/m

d) Wall №12X

n	1,00	
L	80,00	m
h	3,48	m
b	0,25	m
E	30 000,00	MN/mm^2
I	0,87	m^4
A	0,87	m^2
ky	1,20	
v	0,30	
k_{12x}	0,15	MN/m
$\sum k_{12x}$	0,15	MN/m

e) Wall №13X

n	1,00	
L	80,00	m
h	8,30	m
b	0,25	m
E	30 000,00	MN/mm^2
I	11,91	m^4
A	2,08	m^2
ky	1,20	
v	0,30	
k_{13x}	2,08	MN/m
$\sum k_{13x}$	2,08	MN/m

f) Wall №14X

n	1,00	
L	80,00	m
h	12,30	m
b	0,40	m
E	30 000,00	MN/mm^2
I	62,03	m^4
A	4,92	m^2
ky	1,20	
v	0,30	
k_{14x}	10,71	MN/m
$\sum k_{14x}$	10,71	MN/m

g) Wall №15X

n	1,00	
L	80,00	m
h	9,05	m
b	0,25	m
E	30 000,00	MN/mm^2
I	15,44	m^4
A	2,26	m^2
ky	1,20	
v	0,30	
k_{15x}	2,69	MN/m
$\sum k_{15x}$	2,69	MN/m

5.10 Calculation of the forces, which acting on the walls in the direction X

The basic horizontal force acting on each wall calculation process in the direction X will be discussed below.

$\sum k_i x$	21,06470429	MN/m
F_y = Hd(roof side B)	0,1025	MN
v_x	0,00486596	m

Q9x	0,01776275	MN
Q10x	0,00443431	MN
Q11x	0,00427882	MN
Q12x	0,00074666	MN
Q13x	0,01010422	MN
Q14x	0,05209551	MN
Q15x	0,01307772	MN

5.11 Calculation and comparison of M(fall) and M(stab) in the direction X

The basic calculation and comparison process of M(fall) and M(stab) in the X direction will be discussed below.

X1) Wall №9X

H1d = Q9x	17,76	kN
M(falling)	19 414,69	kNm
Nd(wall 10,03m)	180,54	kN
Nda (roof)	317,47	kN
Nd(floor 3-25 direction X)	298,79	kN
∑Nd	10 862,81	kN
fcd	17 000,00	kN
a	1,28	m
e	3,74	m
M(stab)	40 594,56	kNm
M(stab)>M(falling)		

X2) Wall №10X

H1d = Q10x	4,43	kN
M (falling)	4 846,70	kNm
Nd(wall 6,3m)	113,40	kN
Nda (roof)	151,34	kN
Nd(floor 3-25 direction X)	144,77	kN
∑Nd	5 831,02	kN
fcd	17 000,00	kN
a	0,69	m
e	2,46	m
M(stab)	14 367,62	kNm
M(stab)>M(falling)		

X3) Wall №11X

H1d = Q11x	4,28	kN
M (falling)	4 676,75	kNm
Nd(wall 6,03m)	119,39	kN
Nda (roof)	192,59	kN
Nd(floor 3-25 direction X)	181,26	kN
∑Nd	6 807,02	kN
fcd	17 000,00	kN
a	0,80	m
e	2,21	m
M(stab)	15 071,93	kNm
M(stab)>M(falling)		

X4) Wall №12X

H1d = Q12x	0,75	kN
M (falling)	816,10	kNm
Nd(wall 3,475m)	108,54	kN
Nda (roof)	85,80	kN
Nd(floor 3-25 direction X)	80,75	kN
∑Nd	4 250,22	kN
fcd	17 000,00	kN
a	0,50	m
e	1,24	m
M(stab)	5 259,54	kNm
M(stab)>M(falling)		

X5) Wall №13X

H1d = Q13x	10,10	kN
M (falling)	11 043,92	kNm
Nd(wall 8,3m)	149,40	kN
Nda (roof)	331,60	kN
Nd(floor 3-25 direction X)	312,10	kN
∑Nd	10 484,53	kN
fcd	17 000,00	kN
a	1,23	m
e	2,92	m
M(stab)	30 578,40	kNm
M(stab)>M(falling)		

X6) Wall №14X

H1d = Q14x	52,10	kN
M (falling)	56 940,40	kNm
Nd(wall 12,3m)	354,24	kN
Nda (roof)	491,41	kN
Nd(floor 3-25 direction X)	462,50	kN
∑Nd	18 459,79	kN
fcd	17 000,00	kN
a	2,17	m
e	3,98	m
M(stab)	73 437,85	kNm
M(stab)>M(falling)		

X7) Wall №15X

H1d = Q15x	13,08	kN
M (falling)	14 293,95	kNm
Nd(wall 9,05m)	162,90	kN
Nda (roof)	364,16	kN
Nd(floor 3-25 direction X)	342,74	kN
∑Nd	11 488,28	kN
fcd	17 000,00	kN
a	1,35	m
e	3,17	m
M(stab)	36 457,34	kNm
M(stab)>M(falling)		

6 CONCLUSION

The aim of the thesis was to make basic stability calculations for a 25-story high-rise building for Pöyry Oy to be able to compare the results of this thesis with their own office calculations.

All of the calculation materials were taken from Eurocodes, the supervisor's project materials and the company's materials. The stability calculations were done by hand with the help of the Excel tables. Also, special construction programs for the calculation of the main loads were not used for this thesis. Formulas and coefficients were taken from the Finnish Eurocode and calculations were constantly monitored by HAMK supervisor (Tapio Korkeamäki) and Pöyry Oy supervisors (Aleksi Pöyhönen, Ville Virnes).

The calculation process was done four times. As the project was in the designing phase there were a lot of structural changes. Different wall sizes and the location of loadbearing walls required different stability calculations each time. The calculations were made manually. Something was simplified. Nowadays designing programs can calculate almost everything exactly, and then the results can be compared with this thesis. That is why the company also has their own calculations, which are more precise.

All of the calculations were done for the walls in X and Y directions. As the structural calculations are complicated for that type of building, all the loadbearing and nonloadbearing walls were taken without door and window openings to simplify the calculation process. To be on the safe side, the biggest horizontal load for each building side was taken into consideration. The wind load for the roof level was taken for the calculation for the whole structure. Also, the vertical load was minimized and taken without the live load. Thus, it was done to define the most complicated wall cases.

All in all, when all of the calculations were done, it was detected that the most difficult cases for the stability of the building were walls with a small acting normal force. Thus, those walls need tensile rebars to resist the falling moment, which is caused by horizontal forces.

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