Structural Analysis of an Office Building with Robot Structural Analysis and Manual Calculation

A case study of HAMK ‘N’ building

Bachelor’s thesis

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Construction Engineering

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ABSTRACT

The purpose of this Bachelor’s thesis was to conduct a structural analysis of structural elements of a concrete multi-storey building with Autodesk Robot Structural analysis and traditional manual calculation. The aim was to compare the results obtained from the structural analysis of both methods to discover if there is any alteration in the results. The thesis only concentrates on structural elements of the building but not on the design of Structural connections. The main aim was to examine the behaviour of structure when the most critical load combination is applied on the building.

The building is the property of HAMK University of Applied Sciences. The building was undergoing a process of connecting two buildings ‘C’ and ‘D’ with the new building ‘N’ by demolishing the existing passage connecting the two buildings. The building will be used for office, classroom and library purposes upon completion.

The results of the thesis show that for the given applied load on the structure the load combinations by traditional manual calculations is almost impossible to executed due to many possible combinations which leads to difficulties to find out the critical load. This also leads to the overestimation of critical load by introducing a high value for safety factors, resulting in oversizing of structural elements eventually increasing the cost of the structure. Moreover, it is tedious and time consuming to perform the calculation manually which makes using software for structural analysis more beneficial and efficient. However, there are certain drawbacks of a structural analysis by software as it is not completely perfect yet; there are some calculations which are obligated to calculate manually.

Keywords  Robot Structural Analysis, Eurocode 2, office building, manual calculation

Pages  87 pages including appendices 38 pages
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Full Form</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC</td>
<td>Eurocode</td>
</tr>
<tr>
<td>RSA</td>
<td>Robot Structural Analysis</td>
</tr>
<tr>
<td>SLS</td>
<td>Serviceability Limit State</td>
</tr>
<tr>
<td>SW</td>
<td>Self Weight</td>
</tr>
<tr>
<td>ULS</td>
<td>Ultimate Limit State</td>
</tr>
</tbody>
</table>
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Appendices
Appendix 1 Calculations
Appendix 2 Floor plans and Materials properties
1 INTRODUCTION

1.1 Introduction of Eurocodes

During history, humans have developed many sets of disciplines while designing the structure for the better performance of the structure during its intended period of life and for the safety of the occupant. Many countries around the world developed their own sets of design rules for simplicity of designing the structure within the country. However, none of the building codes are coherent in the European Union which leads to difficulties in free trade within the European Union. To solve the coherency, common technical rules for the design of the structure were required within Europe. It all started in 1975 when the European Committee for Standardisation developed certain sets of rules while designing the structure upon the request of the European Commission. It has been modified and developed in the course of time to form the present Eurocodes. The Eurocodes are intended to be compulsory for European public works. There are ten structural Eurocodes until this date covering basics of design, action on structure, design and detailing and geotechnical and earthquake design. The links between the Eurocodes are shown in Figure 1 below.

![Figure 1. Link between the Eurocodes (Bond et al. 2006)](image)

1.2 Eurocode 2

Eurocode 2 consists of detailed sets of rules to design concrete structures. It deals with all types of concrete like reinforced concrete, pre-stressed
concrete etc. It consists of seven chapters and four appendices. The four chapters are shown below.

Part 1-1: General rules and rules for buildings
Part 1-2: Structural fire design
Part 2: Reinforced and pre-stressed concrete bridges
Part 3: Liquid retaining and containing structures

This Thesis only deals with part 1-1 also called EN 1992-1-1 because it concentrates on a multi-story reinforced concrete office building.

2 PROJECT DESCRIPTION

HAMK University of Applied Sciences, Visamaki, Hämeenlinna was undergoing a process of connecting two buildings ‘C’ and ‘D’ with the new building ‘N’ by demolishing the existing passage connecting those two buildings. The building will be used for office and classroom purposes after completion.

The slab of the new building was supported with the existing building with the rectangular beam element embedded with 2T12 rebar with a spacing of 150mm and chemical anchor with a diameter of 20mm connected in the existing building on each floor. The exterior frame of the building was made up of prefabricated reinforced concrete walls. The floor system was different in different floors. The ground floor and the roof floor consisted of extra insulation in order to protect the structure from unnecessary moisture from the external environment along with hollow core slabs of 350mm, which was supported by Peikko’s delta beams on each floor. Circular reinforced columns were used as the vertical member of the structure whereas structural steel columns were used for the openings of the doors on the ground floor and the ceiling room. A Pad foundation of varying dimensions was used for the columns whereas strip foundations were used as a supporting element for the external walls. The load of the structure was transferred via columns to foundation piles embedded in the bedrock.

The building was designed using Eurocode SFS EN-1990, EN-1991, EN-1992 standards as well as the Finnish National Annexes.

The 3D model of the ‘N’ building is shown in Figure 2.
The following Table 1 lists the properties of the building

**Table 1. Building Description**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building type</td>
<td>Multi storey office building</td>
</tr>
<tr>
<td>Building location</td>
<td>Visamäentie 35, Hämeenlinna</td>
</tr>
<tr>
<td>No. of Storey</td>
<td>2+1(celling room)</td>
</tr>
<tr>
<td>Clear floor height</td>
<td>4.15m; 3.95m; 3m</td>
</tr>
<tr>
<td>Total height of building</td>
<td>9.5m+3.2m</td>
</tr>
<tr>
<td>Gross rectangular area of building</td>
<td>22m*22.7m</td>
</tr>
<tr>
<td>Gross area of the first floor of building</td>
<td>$450m^2$</td>
</tr>
<tr>
<td>Consequence class</td>
<td>CC2</td>
</tr>
<tr>
<td>Reliability Class</td>
<td>RC2</td>
</tr>
<tr>
<td>Category of use</td>
<td>B</td>
</tr>
<tr>
<td>Design working life</td>
<td>50 years; Foundation 100 years</td>
</tr>
<tr>
<td>Terrain Category</td>
<td>3</td>
</tr>
<tr>
<td>Imposed load</td>
<td>7.5 kN/m$^2$ (upon request)</td>
</tr>
<tr>
<td>Snow load on the ground</td>
<td>2.5 kN/m$^2$</td>
</tr>
<tr>
<td>Seismic load</td>
<td>None</td>
</tr>
<tr>
<td>Basic wind velocity</td>
<td>21 m/s</td>
</tr>
<tr>
<td>Structural floor element</td>
<td>Hollow core slab</td>
</tr>
<tr>
<td>Wall thickness</td>
<td>435mm</td>
</tr>
<tr>
<td>Floor thickness</td>
<td>670mm; 450mm; 930mm</td>
</tr>
<tr>
<td>Type of foundation</td>
<td>Pad foundation+ strip foundation+ Piles</td>
</tr>
</tbody>
</table>
2.1 **Materials used in building with exposer class and concrete cover**

The concrete class, exposer class and concrete covers used in the building were different at different places, which is given in Appendices.

2.2 **Concrete cover**

The procedure of choosing concrete cover in the structure is described in this section. First, the structural class was chosen which is 4 as default. Alteration of the structural class is done using Table 2.

<table>
<thead>
<tr>
<th>Table 2. Structural class and exposure class(SFS - EN1992 Eurocode 2004)</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Design Working Life of 100 years</th>
<th>Increase</th>
<th>Increase</th>
<th>Increase</th>
<th>Increase</th>
<th>Increase</th>
<th>Increase</th>
<th>Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
<td>Class by 2</td>
</tr>
<tr>
<td>≥ C30/37</td>
<td>≥ C30/37</td>
<td>≥ C35/45</td>
<td>≥ C40/50</td>
<td>≥ C40/50</td>
<td>≥ C40/50</td>
<td>≥ C40/50</td>
<td>≥ C45/55</td>
</tr>
<tr>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
</tr>
<tr>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
</tr>
<tr>
<td>Special Quality Control of the concrete production ensured</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
<td>Reduce</td>
</tr>
<tr>
<td></td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
<td>Class by 1</td>
</tr>
</tbody>
</table>

Then according to the design life and special quality control of the concrete production of the structural element the structural class of the structural element can be altered. For example, this beam and column were designed for 50 years of life and no special quality control is ensured. The structural classes of the elements is shown in Table 3.

<table>
<thead>
<tr>
<th>Table 3. Structural class of beam and column</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Elements</th>
<th>Structural class</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>S4</td>
<td>No reduction</td>
</tr>
<tr>
<td>Column</td>
<td>S4</td>
<td>No reduction</td>
</tr>
</tbody>
</table>

Values for minimum concrete cover for the given structural class and exposure class can be determined by using Table 4 below.
Table 4. Environmental requirement for $C_{\text{min}, \text{dur}}$ (SFS - EN1992 Eurocode 2004)

<table>
<thead>
<tr>
<th>Structural Class</th>
<th>Exposure Class according to Table 4.1 (Eurocode 2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X0</td>
</tr>
<tr>
<td>S1</td>
<td>10</td>
</tr>
<tr>
<td>S2</td>
<td>10</td>
</tr>
<tr>
<td>S3</td>
<td>10</td>
</tr>
<tr>
<td>S4</td>
<td>10</td>
</tr>
<tr>
<td>S5</td>
<td>15</td>
</tr>
<tr>
<td>S6</td>
<td>20</td>
</tr>
</tbody>
</table>

$C_{\text{min}, \text{dur}}$ - Beam (XC1/S4) 15mm

$C_{\text{min}, \text{dur}}$ - Column (XC1/S4) 15mm

Hence, nominal cover can be calculated by using the given equation. It is shown in Table 4.

$$C_{\text{nom}} = \max[C_{\text{min}} + \Delta_{\text{c,dev}}, 20\text{mm}]$$

Where,

$\Delta_{\text{c,dev}} = 10\text{mm}$

Table 5. Nominal concrete cover

<table>
<thead>
<tr>
<th>Elements</th>
<th>Nominal cover(mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>25</td>
</tr>
<tr>
<td>column</td>
<td>25</td>
</tr>
</tbody>
</table>

As shown in Table 5 above, the nominal concrete cover can be computed.

2.3 Steel Reinforcement

Steel reinforcement of medium ductility A500HW with varying diameters was chosen in this project.

3 BUILDING STRUCTURAL COMPONENTS
3.1 Exterior wall

The outer framework of the building was made of prefabricated non-bearing walls on north and south side whereas the remaining sides of the building were directly connected with the existing building’s wall. The total thickness of the wall is 435mm. The wall was made of different layers as shown in Figure 3 below. The external wall was supported by the strip foundation.

![Layers of exterior wall](image)

3.2 Columns

There were ten prefabricated circular reinforced columns in both the first floor and second floor with a diameter of 380mm whereas there were four reinforced concrete columns of dimension 280mm*280 mm on the roof. (C30/37) concrete class was used in the building with the concrete cover of 25mm. Peikko’s precast hidden corbel was used in columns to support delta beams. It consisted of a corbel plate bolted to a machined steel column plate integrated into the column.

3.3 Beams

Delta beams were employed in the building to carry the loads from the floor. Hollow core slabs were put on the flange of delta beams. Delta beams of size D24-400 were used on third floor while D32-400 beams were used on the rest of the structure. The slab was supported by the rectangular beam element of 200mm*280mm embedded with 2T12 rebar with a spacing of 150mm and chemical anchor with a diameter of 20mm connected in the existing building side on each floor. The shape of the delta beam is shown in Figure 4.
3.4 Floor

Imposed loads on the building were transferred to the delta beams via floor slabs. Hollow core slabs were used as the structural floor element in the building. Three types of floors with a different layer were used in this structure. The dimensions and materials of the floor are shown in Table 6.

Table 6. Dimensions and materials of floor

<table>
<thead>
<tr>
<th>Floor</th>
<th>Dimensions and Materials</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>10mm bitumen 150mm Insulation 150mm Insulation 150mm Concrete 320mm Hollow core slab</td>
</tr>
<tr>
<td>Intermediate floor</td>
<td>100mm Concrete 30mm Insulation 320mm Hollow core slab</td>
</tr>
<tr>
<td>Ground floor</td>
<td>100mm Concrete 30mm Sealing product 320mm Hollow core slab 220mm Insulation</td>
</tr>
</tbody>
</table>

A picture showing the cross section of the intermediate floor is shown in Figure 5.

Figure 5. Cross section of intermediate floor
3.5 Foundation

Loads from the building were transferred to the ground via foundation. A Strip foundation with the dimension of 1000mm*700mm was used for exterior walls. The loads from the columns were first transferred to the plinth of dimension 500mm*800mm then, it was transferred to the pad foundation with variable dimensions from 1650mm*1900mm*700mm to 1900mm*1900mm*700mm. The load bearing capacity of the soil was not enough to support the structure load. Therefore, piles were used to transfer the loads from the foundation to the bedrock. The column was connected to the plinth by using a column shoe whereas the plinth was connected to the pad foundation by anchor rods and lapping reinforcement from the footing and plinth.

4 LOAD DESCRIPTION

4.1 Dead load

Dead load is defined as Action that is likely to act throughout the given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value (SFS-EN 1990 2002, 2005)

In this project, the self-weight of the building was considered as the dead load. The partial safety factors used for permanent actions according to the Finnish National Annex is shown in Table 7 below.

<table>
<thead>
<tr>
<th>Partial safety factors</th>
<th>EQU</th>
<th>STR6.10 ‘a’</th>
<th>STR 6.10 ‘b’</th>
</tr>
</thead>
<tbody>
<tr>
<td>( Y_{Gk_{sup}} )</td>
<td>1.10</td>
<td>1.35</td>
<td>( \xi * 1.35 )</td>
</tr>
<tr>
<td>( Y_{Gk_{inf}} )</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Where,
\[ \xi = 0.85 \]

The dead loads in the building are highlighted in Table 8.
Table 8. Roof and other elements

<table>
<thead>
<tr>
<th>Elements</th>
<th>Dead load ($G_{k,\text{roof}}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hollow core slab 320mm</td>
<td>4.05 $kN/m^2$</td>
</tr>
<tr>
<td>Concrete finish+ Lightweight aggregate</td>
<td>3.5 $kN/m^2$</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7.55 $kN/m^2$</strong></td>
</tr>
<tr>
<td>Elements (Other Floors)</td>
<td></td>
</tr>
<tr>
<td>Hollow core slab 320mm</td>
<td>4.05 $kN/m^2$</td>
</tr>
<tr>
<td>Concrete finish+ Lightweight aggregate</td>
<td>3 $kN/m^2$</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>7.05 $kN/m^2$</strong></td>
</tr>
<tr>
<td>Other dead load in the building</td>
<td></td>
</tr>
<tr>
<td>Self-weight of the Delta beam+ Concrete cover</td>
<td>2.7 $kN/m$</td>
</tr>
<tr>
<td>Self-weight of the column</td>
<td>3.14 $kN/m$</td>
</tr>
<tr>
<td>Self-weight of the wall</td>
<td>47.5 $kN/m$</td>
</tr>
</tbody>
</table>

4.2 **Live load**

Live load is defined as Action for which the variation in magnitude with time is neither negligible nor monotonic (SFS-EN 1990 2002, 2005). The partial safety factors used for variable actions in the Finnish National Annex is shown in Table 9 below.

<table>
<thead>
<tr>
<th>Partial safety factors</th>
<th>EQU</th>
<th>STR 6.10 ’a’</th>
<th>STR 6.10 ’b’</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{Q kufav}$</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>$\gamma_{Q kfav}$</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The imposed load on the building doesn’t act simultaneously all the time; hence, to decrease the cost of the building the applied load can be reduced using a reduction factor:

$$\alpha_A = \frac{5}{7} \psi_0 + \frac{A_0}{A}$$

Where,

$\psi_0 = 0.7$

$A_0 = 10m^2$

$A = \text{Influence area}$
The Finnish National Annex for imposed loads on the different part of the structure is shown in Table 10.

Table 10. Imposed load on building parts (SFS - EN1991-1-1 Eurocode 2002)

<table>
<thead>
<tr>
<th>Categories of loaded areas</th>
<th>$q_m$ [kN/m$^2$]</th>
<th>$Q_a$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Category A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Floors</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>- Stairs</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>- Balconies</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>Category B</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>Category C</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- C1</td>
<td>2.5</td>
<td>3.0</td>
</tr>
<tr>
<td>- C2</td>
<td>3.0</td>
<td>3.0</td>
</tr>
<tr>
<td>- C3</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- C4</td>
<td>5.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- C5</td>
<td>6.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Category D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- D1</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>- D2</td>
<td>5.0</td>
<td>7.0</td>
</tr>
</tbody>
</table>

4.2.1 Live load on the building

The building was used for office purposes and library. There might be a gathering of many people; hence, the imposed load on the building was taken as 7.5 $kN/m^2$ upon request on each floor while 4 $kN/m^2$ was taken at roof because the roof will be only used for maintenance purposes.

4.3 Snow load

The building is in Hämeenlinna; hence, according to Eurocode 1991-1-3 under Finnish National Annex, the characteristic value of snow load on the ground is 2.5 $kN/m^2$ as shown in Figure 6.
Figure 6. Snow load on ground in Finland (SFS - EN1991-1-3 Eurocode 2003)

The value of coefficient ‘ψ’ for building is given in the National Annex for SFS-EN 1990:2002. They are copied in the Table 11 below.

Table 11. The value of coefficient ‘ψ’ for building (SFS - EN1991-1-3 Eurocode 2003)

<table>
<thead>
<tr>
<th>Snow load</th>
<th>( ψ_0 )</th>
<th>( ψ_1 )</th>
<th>( ψ_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_k &lt; 2,75 \text{ kN/m}^2 )</td>
<td>0.7</td>
<td>0.4</td>
<td>0.2</td>
</tr>
<tr>
<td>( s_k \geq 2,75 \text{ kN/m}^2 )</td>
<td>0.7</td>
<td>0.5</td>
<td>0.2</td>
</tr>
</tbody>
</table>

*) Outdoor locations and balconies \( ψ_0 = 0 \) combined with load classes A, B, F and G.

Snow load on the roof can be calculated as follows:

\[
s = \mu_1 \times C_e \times C_t \times s_k
\]

Where,

- \( C_e \) = Exposure coefficient
- \( C_t \) = Thermal coefficient

The value of thermal coefficient is recommended as ‘1’ by the Finnish National Annex whereas the value of exposure coefficient can be calculated using Table 12 below.

Table 12. Exposure coefficient (SFS - EN1991-1-3 Eurocode 2003)
The snow load shape coefficient can be calculated using Figure 7 below.

**Figure 7. Snow load shape coefficient (SFS-EN1991-1-3 Eurocode 2003)**

In case of this project, $\alpha = 0$

Hence from Figure 7 $\mu_1 = 0.8$

The roof is a flat roof with a projection of 3.3m and there were existing higher buildings nearby. Therefore, the load arrangement should be used for both the undrifted and drifted load arrangements. The snow load shape coefficients that should be used for roofs abutting to taller construction works are given in the following expressions in Figure 8.

**Figure 8. Snow load coefficient (SFS-EN1991-1-3 Eurocode 2003)**
Drifted load arrangement for structure near taller constructions is shown in Figure 9.

\[ \mu_1 = 0.8 \text{ (Flat roof)} \]
\[ \mu_2 = \mu_s + \mu_w \]

Where,
\[ \mu_s = \text{Snow load shape coefficient due to sliding of snow from the upper roof} \]
For \( \alpha \leq 15^\circ \), \( \mu_s = 0 \)
\[ \mu_w = \text{Snow load shape coefficient due to wind} \]
\[ \mu_w = \frac{b_1 + b_2}{2n} \leq \frac{\gamma h}{s_k} \]

Where,
\[ \gamma = \text{Weight density of snow, which for this calculation may be taken as 2 kN/m}^3 \]
\( s_k = \text{is the characteristic value of snow load on tile ground} \)

Drift length,
\[ l_s = 2 \ast h \]
4.4 Wind load


4.4.1 Basic values

The basic velocity of wind can be calculated by using the following equation

\[ V_b = C_{dir} \times C_{season} \times V_{b,0} \]

Where,

\( V_{b,0} \) = Fundamental value of basic wind velocity
\( C_{dir} \) = Direction factor
\( C_{season} \) = Season factor

According to the National Annex the recommended values of \( C_{dir} \) and \( C_{season} \) is taken as 1. While \( V_{b,0} \) = is taken as 21m/s²

4.4.2 Mean wind

The Mean wind velocity at height \( z \) above the terrain can be calculated using following equation

\[ V_m(z) = C_r(z) \times C_0(z) \times V_{b,0} \]

Where,

\( C_0(z) \) = Orography factor
\( C_r(z) \) = Terrain roughness factor

4.4.3 Terrain roughness factor

The terrain roughness factor can be calculated by using the following expression:

\[ C_r(z) = K_r \times \ln \left( \frac{z}{z_0} \right) \]

Where,

\( z_0 \) = Roughness height
\( K_r \) = Terrain factor depending on the roughness length \( z_0 \)
\[ K_r = 0.19 \left( \frac{z_0}{z_{0,ii}} \right)^{0.07} \]

The values of \( z_0, z_{0,ii} \) can be found in Table 13 below.

<table>
<thead>
<tr>
<th>Terrain category</th>
<th>( z_0 ) m</th>
<th>( z_{0,ii} ) m</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Sea or coastal area exposed to the open sea</td>
<td>0.003</td>
<td>1</td>
</tr>
<tr>
<td>I Lakes or flat and horizontal area with negligible vegetation and without obstacles</td>
<td>0.01</td>
<td>1</td>
</tr>
<tr>
<td>II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights</td>
<td>0.05</td>
<td>2</td>
</tr>
<tr>
<td>III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)</td>
<td>0.3</td>
<td>5</td>
</tr>
<tr>
<td>IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m</td>
<td>1.0</td>
<td>10</td>
</tr>
</tbody>
</table>

**NOTE:** The terrain categories are illustrated in A.1.

### 4.4.4 Wind turbulence

The standard deviation of turbulence can be calculated by using the following expression

\[ \sigma_v = K_r \times v_b \times K_1 \]

Where,
- \( K_r \) = Terrain factor
- \( K_1 \) = Turbulence factor (Recommended value from national annex is 1)
- \( v_b \) = Basic wind velocity

Turbulence Intensity can be calculated by using the following expression

\[ I_v = \frac{\sigma_v}{V_m} \]

Where,
\[ V_m = \text{Mean wind velocity} \]
\[ \sigma_v = \text{Standard deviation of turbulence} \]

4.4.5 Peak velocity pressure

The peak velocity pressure can be calculated by using the equation below

\[ q_p = \left[ 1 + 7I_v \right] \frac{1}{2} * \rho * V_m^2 \]

Alternatively, peak velocity pressure for the structure of height up to 100m can be calculated by using the expression below

\[ q_p(z) = C_e(z) * q_b \]

Where,
\[ q_b = \text{basic velocity pressure}; \]
\[ q_b = \frac{1}{2} * \rho * v_b^2; \quad \rho = \text{Air density (1.25kg/m}^3) \]

The Exposer factor \((C_e(z))\) can be calculated by using Figure 10 below

![Figure 10. Illustration of Exposure factor for \(C_e=a, K_r=1\) (SFS-EN1991-1-4 Eurocode 2005)](image-url)
4.4.6 Net wind pressure

Net wind pressure on the surface can be calculated by computing values of external and internal wind pressure.

External wind pressure on the surface of the structure is given by:
\[ W_e = C_{pe} q_p \]

Internal wind pressure on the surface of the structure is given by:
\[ W_i = C_{pi} q_p \]

Where,
\( C_{pe} \) = Pressure coefficient for the external pressure
\( C_{pi} \) = Pressure coefficient for the internal pressure

The values of external pressure coefficient are given by EN 1991-1-4:2005(E) as shown in Table 14 and 15 below whereas for internal pressure where opening ratio is not considered justified should be taken as more onerous of +0.2 and -0.3.

<table>
<thead>
<tr>
<th>Zone</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>h/d</td>
<td>( C_{pe,10} )</td>
<td>( C_{pe,1} )</td>
<td>( C_{pe,10} )</td>
<td>( C_{pe,1} )</td>
<td>( C_{pe,10} )</td>
</tr>
<tr>
<td>5</td>
<td>-1.2</td>
<td>-1.4</td>
<td>-0.6</td>
<td>-1.1</td>
<td>-0.5</td>
</tr>
<tr>
<td>1</td>
<td>-1.2</td>
<td>-1.4</td>
<td>-0.8</td>
<td>-1.1</td>
<td>-0.5</td>
</tr>
<tr>
<td>≤ 0.25</td>
<td>-1.2</td>
<td>-1.4</td>
<td>-0.8</td>
<td>-1.1</td>
<td>-0.5</td>
</tr>
</tbody>
</table>

Table 14. External pressure coefficient (SFS-EN1991-1-4 Eurocode 2005)
Table 15. External pressure for flat roof (SFS-EN1991-1-4 Eurocode 2005)

In case of ratio of \(h/d\) is between the given values, linear interpolation must be used to calculate the value of \(h/d\).

Finally, net pressure can be calculated by using the following expression

\[
p = C_s C_d q_p C_{pe10} - q_p C_{pi}
\]

Where,
19

\[ C_s C_d = \text{Structural factors (Recommended value for building height less than 15m is 1.}} \]

5  LOAD COMBINATION

The structure shall be designed so that it doesn’t face any kind of structural damage during its intended period of life. There are different types of load acting on the structure and most of the time these loads act in simultaneously resulting in the overloading in the structure. Therefore, to overcome such problem in future the most critical combination of load must be considered while designing the structure. EN 1991 gives the load combinations depending on whether the overall structure is considered as a rigid body (EQU) or design of structural element (STR) need to be carried out.

The load combination was executed by using the Finnish National Annex as shown in table 16 and 17.


<table>
<thead>
<tr>
<th>Persistent and transient design situations</th>
<th>Permanent actions</th>
<th>Leading variable action (*)</th>
<th>Accompanying variable actions (*)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Unfavourable</td>
<td>Favourable</td>
<td>Main (if any)</td>
</tr>
<tr>
<td>(Eq. 6.10)</td>
<td>1.10 ( K_{n1} G_{H} )</td>
<td>0.90 ( G_{H,ref} )</td>
<td>1.50 ( K_{n1} Q_{H,3} )</td>
</tr>
</tbody>
</table>

(*) Variable actions are those considered in Table A.1.1

\( K_{n1} \) depends on the reliability class given in table B2 of Annex B as follows:
- In reliability class RC3 \( K_{n1} = 1.1 \)
- In reliability class RC2 \( K_{n1} = 1.0 \)
- In reliability class RC1 \( K_{n1} = 0.9 \).

The reliability classes are associated with the consequence classes CC3 … CC1 given in Annex B.

As shown in Table 17 above, the factor $K_{FI}$ was taken as 1 since the structure belongs to the reliability class 2.

6 ROBOT STRUCTURAL ANALYSIS

Robot structural analysis is an advanced analysis software launched by Autodesk. For the simplicity only, grid N3-N3 of the structure were modelled in the Robot structural analysis software as shown in Figure 11. After completion of the modelling loads were applied on the structure and a manual load combination was done according to the Finnish National Annex.

Finally, the analysis was executed to check the behaviour of the structure.
6.1 Structural model

All the members of the structure from grid N3-N3 were defined as shown in Figure 12 below.

Figure 12. RSA model of Grid 3-3
6.2 Load description

All the loads applied on the structure were defined in the robot structural analysis as shown in Table 18 below.

Table 18. Load description from RSA

<table>
<thead>
<tr>
<th>Case</th>
<th>Load type</th>
<th>Load type of Case</th>
<th>Load value (m) (kN) (Deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Permanent load on structure</td>
<td>self-weight</td>
<td>P2 Negative Factor x 1.00</td>
</tr>
<tr>
<td>1</td>
<td>Permanent load on structure</td>
<td>uniform load</td>
<td>P2 = 53.34 (kN)</td>
</tr>
<tr>
<td>1</td>
<td>Permanent load on structure</td>
<td>uniform load</td>
<td>P2 = 40.94 (kN)</td>
</tr>
<tr>
<td>2</td>
<td>Permanent load from wall x 1</td>
<td>radial force</td>
<td>Fx = 5.64 (kN)</td>
</tr>
<tr>
<td>2</td>
<td>Permanent load from wall x 2</td>
<td>radial force</td>
<td>Fx = 6.68 (kN)</td>
</tr>
<tr>
<td>2</td>
<td>Permanent load from wall x 3</td>
<td>radial force</td>
<td>Fx = 5.64 (kN)</td>
</tr>
<tr>
<td>4</td>
<td>All span loading</td>
<td>uniform load</td>
<td>P2 = 51.05 (kN)</td>
</tr>
<tr>
<td>5</td>
<td>All span loading</td>
<td>uniform load</td>
<td>P2 = 51.05 (kN)</td>
</tr>
<tr>
<td>6</td>
<td>All span loading</td>
<td>Case I uniform load</td>
<td>P2 = 27.25 (kN)</td>
</tr>
<tr>
<td>7</td>
<td>All span loading</td>
<td>Case II uniform load</td>
<td>P2 = 51.05 (kN)</td>
</tr>
<tr>
<td>8</td>
<td>All span loading</td>
<td>Case III uniform load</td>
<td>P2 = 27.25 (kN)</td>
</tr>
<tr>
<td>9</td>
<td>All span loading</td>
<td>Case IV uniform load</td>
<td>P2 = 51.05 (kN)</td>
</tr>
<tr>
<td>10</td>
<td>Wind x 1</td>
<td>radial force</td>
<td>Fx = 5.38 (kN)</td>
</tr>
<tr>
<td>11</td>
<td>Wind x 2</td>
<td>radial force</td>
<td>Fx = 5.38 (kN)</td>
</tr>
<tr>
<td>12</td>
<td>Wind x 3</td>
<td>radial force</td>
<td>Fx = 5.38 (kN)</td>
</tr>
<tr>
<td>13</td>
<td>Wind x 4</td>
<td>radial force</td>
<td>Fx = 5.38 (kN)</td>
</tr>
<tr>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

The self-weight of the elements modelled in the software was defined automatically by software itself and the self-weight of concrete finishes and fillings was manually introduced to the software, as it was not defined by the software.

6.3 Load combination

There are altogether 1528 load combinations for the defined loading on the structure. However, only 46 combinations are shown in Table 19.

Table 19. Load combination table from RSA
### 6.4 Design of column

After load combination of the action, the maximum compressive force on the column for all combinations was checked. It was found that element 14 got the maximum compressive force from load combination 24. The compression forces on the columns are shown in Figure 13.

<table>
<thead>
<tr>
<th>Combinations</th>
<th>Name</th>
<th>Analysis type</th>
<th>Combination</th>
<th>Case nature</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>16 (C)</td>
<td>ULS1+2*1.35</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>10 (C)</td>
<td>ULS2-2*1.35</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>17 (C)</td>
<td>ULS3+2*0.90</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>18 (C)</td>
<td>ULS4+2*0.90</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>19 (C)</td>
<td>ULS5+3*1.35</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>20 (C)</td>
<td>ULS6+3*1.35</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>21 (C)</td>
<td>ULS7+3*0.90</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>22 (C)</td>
<td>ULS8+3*0.90</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>23 (C)</td>
<td>ULS9+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>24 (C)</td>
<td>ULS10+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>25 (C)</td>
<td>ULS11+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>26 (C)</td>
<td>ULS12+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>27 (C)</td>
<td>ULS13+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>28 (C)</td>
<td>ULS14+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>29 (C)</td>
<td>ULS15+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>30 (C)</td>
<td>ULS16+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>31 (C)</td>
<td>ULS17+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>32 (C)</td>
<td>ULS18+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>33 (C)</td>
<td>ULS19+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>34 (C)</td>
<td>ULS20+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>35 (C)</td>
<td>ULS21+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>36 (C)</td>
<td>ULS22+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>37 (C)</td>
<td>ULS23+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>38 (C)</td>
<td>ULS24+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>39 (C)</td>
<td>ULS25+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>40 (C)</td>
<td>ULS26+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>41 (C)</td>
<td>ULS27+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>42 (C)</td>
<td>ULS28+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>43 (C)</td>
<td>ULS29+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>44 (C)</td>
<td>ULS30+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>45 (C)</td>
<td>ULS31+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>46 (C)</td>
<td>ULS32+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>47 (C)</td>
<td>ULS33+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>48 (C)</td>
<td>ULS34+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>49 (C)</td>
<td>ULS35+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>50 (C)</td>
<td>ULS36+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>51 (C)</td>
<td>ULS37+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>52 (C)</td>
<td>ULS38+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>53 (C)</td>
<td>ULS39+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>54 (C)</td>
<td>ULS40+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>55 (C)</td>
<td>ULS41+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>56 (C)</td>
<td>ULS42+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>57 (C)</td>
<td>ULS43+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>58 (C)</td>
<td>ULS44+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>59 (C)</td>
<td>ULS45+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
<tr>
<td>60 (C)</td>
<td>ULS46+2*1.15</td>
<td>Linear Combinati</td>
<td>Structural</td>
<td>DL2+DL11 1.55</td>
<td>(DL2+DL1) 1.15</td>
</tr>
</tbody>
</table>
Since load combination 24 gives the maximum compression on the element 2, the design of the columns was based on the force values from element 2, as the forces had maximum values at that element.

As shown in Figure 14 and 15 above, the moment in the element is very small so it can be neglected during manual calculation.
6.4.1 Forces and Moments on column

The forces and moment on the column were calculated by RSA for load combination 24. The values of normal forces and moments on the column 2 are shown in Figure 16.

![Figure 16. Forces and moments on element 2](image)

6.4.2 Column geometry

The column was designed in the RSA as shown in Figure 17 below, then the properties of column was defined.
6.4.3 Column result

The normal force versus moment graph from RSA is shown in Figure 18.

![Figure 18. Column result for combination number 2](image)

6.4.4 Provided reinforcement for column

After finding the maximum design moment on the column, the provided reinforcement to overcome the given moment was defined and the number of bars with diameters and spacing is shown in Table 20. It was found that the total area of the provided reinforcement was 1357mm².

<table>
<thead>
<tr>
<th>Steel Grade</th>
<th>Diameter (mm)</th>
<th>Shape Code</th>
<th>Number</th>
<th>Spacing (m)</th>
<th>(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A500HV 8</td>
<td>8</td>
<td>00</td>
<td>22</td>
<td>4<em>0,15 + 14</em>0,25 + 4*0,15</td>
<td>4,97</td>
</tr>
<tr>
<td>A500HV 12</td>
<td>12</td>
<td>00</td>
<td>13</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.4.5 Reinforcement Diagram

Thirteen pieces of rebar with a diameter of 12mm reinforcement were used as longitudinal reinforcement whereas 22 stirrups of diameter 8 were used. The stirrups were spaced 150mm near end whereas 250 in the
middle. The reinforcement diagram obtained from the robot analysis is shown in Figure 19.

![Reinforcement Diagram](image)

Figure 19. Reinforcement diagram for element 2 from RSA

### 6.5 Design of Beam

Load combination, which gives the maximum moment and shear force on the beam, is shown in Figure 20 below. It was found that the load combination number 24 gives the maximum moment and shear force on the beam number 16 with the moment value of 700.65kNm and shear force value of 452.03kN as shown in Figure 21 below.
Figure 20. Model showing moments on beam

Figure 21. Element 16 applied moment
Table 21. Element 16 shear force and moment

<table>
<thead>
<tr>
<th></th>
<th>FZ (kN)</th>
<th>MY (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAX</td>
<td>452.03</td>
<td>700.65</td>
</tr>
<tr>
<td>Bar</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>Point</td>
<td>12</td>
<td>0.50</td>
</tr>
<tr>
<td>Case</td>
<td>24 (C)</td>
<td>24 (C)</td>
</tr>
<tr>
<td>MIN</td>
<td>-455.89</td>
<td>-126.76</td>
</tr>
<tr>
<td>Bar</td>
<td>15</td>
<td>30</td>
</tr>
<tr>
<td>Point</td>
<td>12</td>
<td>22</td>
</tr>
<tr>
<td>Case</td>
<td>964 (C)</td>
<td>517 (C)</td>
</tr>
</tbody>
</table>

The result of shear force and moment on element 16 is shown in Table 21 above. Since the beam modeled in the RSA was not accurate due to the inability of finding deltabeam model to export directly into the model, the generalization model was modeled from section definition tab from RSA. The beam used in the structure was Peikko’s delta beam, as the beam’s resistance was already checked by the Peikko group the chart was available to check the resistance of the beam.

![Delta beam load bearing capacity](image)

Figure 22. Delta beam load bearing capacity (Peikko Oy 2018)

From the Figure 22, the load capacity for 6.2m D32-400 beam is approximately 140kN/m. The maximum bending moment and shear resistance is given by equation in Figure 23.
Maximum design shear force

\[ V_{Ed} = \frac{PL}{2} = \frac{140 \times 6.2}{2} = 434kN \]

Maximum design moment

\[ M_{Ed} = \frac{PL^2}{8} = \frac{140 \times 6.2^2}{8} = 672.7kNm \]

6.5.1 Extra moment resistance provided by concrete above delta beam

Since the delta beam will be cast inside the concrete which results in a substantial increase of moment resistance of beam. The extra moment capacity due to concrete is calculated as shown in Figure 24 below:
Height of concrete above delta beam:
\( (h) = 450\,mm - 300\,mm = 120\,mm \)

Area of concrete\( (A_c) = 660\,mm \times 120\,mm = 7.92 \times 10^4 mm^2 \)

Distance from centroid to axis of rotation
\( d = \frac{450\,mm}{2} - \frac{120\,mm}{2} = 165\,mm \)

Second moment of area of concrete
\( I_c = \frac{bh^3}{12} = \frac{660 \times 120^3}{12} = 9.5 \times 10^7 mm^4 \)

Second moment of area from axis of rotation
\( I = I_c + A \times d^2 = 2.25 \times 10^9 mm^4 \)

Moment capacity due to concrete
\[ M = \frac{\sigma \times I}{y} = \frac{30 \times \frac{N}{mm^2} \times 2.25 \times 10^9 mm^4}{225 mm} = 3.002 \times 10^8 N. mm \]
\[ M = 300.2kNm \]

Combined moment resistance due to concrete and delta beam
\( M_{Rd} = 672.2 + 300.2 = 972.4kNm \)

As applied moment on the beam \( (700.65kNm) \) is less than moment resistance \( (972.4kNm) \). Hence the beam is verified.

6.6 Design of Foundation

6.6.1 Pad footing

The structural load from the building was beyond the capacity of applicable pad footing dimension; due to that reason the pad footing was not used in the structure. To overcome that problem piles were introduced in the structure to carry the load from the structure. Due to a lack of features to design piles in RSA it cannot be executed.

7 MANUAL CALCULATION
Unlike using structural analysis software, in this section, traditional manual calculation was used to design the structure of the building according to EN 1992 along with Finnish National Annex. Each structural member like beams, columns, slabs, and foundation were designed according to ULS following the design of structural member (STR) combination.

7.1 **Load description and combination**

7.1.1 **Permanent action**

The characteristic permanent action on grid 3-3 of the building is highlighted in Table 22 and loading diagram on the structure is shown in Figure 25.

The total roof permanent line load= Self weight of floor*span+ Self-weight of beam
6.8m*(3.5+4.05) kN/m²+2.7kN/m=54.04kN/m

Total floor permanent line load= Self weight of floor+ Self-weight of beam
6.8m*(3.0+4.05) kN/m²+2.7kN/m=50.64kN/m

<table>
<thead>
<tr>
<th>Nature of load</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent load on roof (g_{kroo})</td>
<td>54.04kN/m</td>
</tr>
<tr>
<td>Permanent load on Second floor (g_{k2})</td>
<td>50.64kN/m</td>
</tr>
<tr>
<td>Permanent load on First floor (g_{k1})</td>
<td>50.64kN/m</td>
</tr>
<tr>
<td>Circular column self-weight (g_{kcir})</td>
<td>3.14 kN/m</td>
</tr>
<tr>
<td>Square column self-weight (g_{ksqcol})</td>
<td>1.56 kN/m</td>
</tr>
<tr>
<td>Plinth self-weight (g_{kplinth})</td>
<td>10 kN/m</td>
</tr>
</tbody>
</table>

Figure 25. Permanent load on building
Table 23. Reaction from permanent action from Figure 24

<table>
<thead>
<tr>
<th>Vertical support reaction</th>
<th>Total load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_A$</td>
<td>524.62</td>
</tr>
<tr>
<td>$R_B$</td>
<td>1175.56</td>
</tr>
<tr>
<td>$R_C$</td>
<td>1018.81</td>
</tr>
<tr>
<td>$R_D$</td>
<td>715.07</td>
</tr>
<tr>
<td>$R_E$</td>
<td>385.89</td>
</tr>
</tbody>
</table>

The reaction forces on the support from permanent action is shown in Table 23 above.

7.1.2 Permanent load due to imperfection of wall

During the erection of the wall due to imperfection, some of the load from the wall was transferred into the structure instead of direct transfer of wall through the footing of the wall. The transferred load from the wall must also be considered. The load from the wall is shown in Figure 26 below:

![Figure 26](image-url)

Figure 26. Cases for permanent load from wall due to imperfection

Table 24. Reaction from permanent load due to imperfection from Figure 26.

<table>
<thead>
<tr>
<th>Vertical support reaction</th>
<th>CASE 1</th>
<th>CASE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{AX}$</td>
<td>-0.36</td>
<td>0.36</td>
</tr>
<tr>
<td>$R_{BX}$</td>
<td>-0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>$R_{CX}$</td>
<td>-0.34</td>
<td>0.34</td>
</tr>
<tr>
<td>$R_{DX}$</td>
<td>-0.37</td>
<td>0.37</td>
</tr>
<tr>
<td>$R_{EX}$</td>
<td>-0.33</td>
<td>0.33</td>
</tr>
</tbody>
</table>
The vertical reactions on the supports due to imperfection are shown in Table 24 above.

7.1.3 Imposed load

The building is used for office purposes and library. There might be a gathering of many people; hence, the Imposed load on the building was taken as 7.5 kN/m² upon request on each floor while 4 kN/m² was taken at roof because the roof will be only used for maintenance purposes.

The Imposed line load on the grid 3-3 was calculated and different possible load cases were defined according to Eurocode as shown in Figure 27 below.

Imposed line load on the roof 6.8 m * 4 kN/m² = 27.2 kN/m

Imposed line load on the floor 6.8 m * 7.5 kN/m² = 51 kN/m

![Figure 27. Imposed load cases](image-url)
The load cases for imposed loads are shown in Figure 27 and the reaction obtained from those cases are noted in Table 25 below.

Table 25. Reaction from imposed load cases

<table>
<thead>
<tr>
<th>Vertical support reaction</th>
<th>CASE 1</th>
<th>CASE 2</th>
<th>CASE 3</th>
<th>CASE 4</th>
<th>CASE 5</th>
<th>CASE 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_A$</td>
<td>400.52</td>
<td>400.52</td>
<td>0</td>
<td>400.52</td>
<td>0</td>
<td>400.52</td>
</tr>
<tr>
<td>$R_B$</td>
<td>959.14</td>
<td>400.52</td>
<td>558.62</td>
<td>959.14</td>
<td>558.62</td>
<td>400.52</td>
</tr>
<tr>
<td>$R_C$</td>
<td>829.94</td>
<td>271.32</td>
<td>558.62</td>
<td>829.94</td>
<td>271.32</td>
<td></td>
</tr>
<tr>
<td>$R_D$</td>
<td>557.50</td>
<td>271.32</td>
<td>286.18</td>
<td>557.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$R_E$</td>
<td>286.18</td>
<td>0</td>
<td>286.18</td>
<td>0</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

7.1.4 Snow load

Snow load on the roof was calculated based on EN 1991-1-3 along with the Finnish National Annex. The step by step process of calculation is shown in Appendix 1. There is only one case of snow load that was taken into consideration which is shown below in Figure 28.

Figure 28. Snow load on structure

The line load on the structure from snow load is shown in Figure 28 and the reaction forces exerted by the support due to snow load 27 is shown in Table 26 below.
Table 26. Reaction from snow load

<table>
<thead>
<tr>
<th>Vertical support reaction</th>
<th>CASE 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rₐ</td>
<td>98.96</td>
</tr>
<tr>
<td>Rₐ</td>
<td>172.10</td>
</tr>
<tr>
<td>Rₐ</td>
<td>143.08</td>
</tr>
<tr>
<td>Rₐ</td>
<td>135.86</td>
</tr>
<tr>
<td>Rₐ</td>
<td>59.17</td>
</tr>
</tbody>
</table>

7.1.5 Wind load

The wind load was calculated by following EN 1991-1-4 with Finnish National Annex. The calculation of the wind load is shown in Appendix 1. However, the four load cases for the wind load are shown in Figure 29.

Figure 29. Wind load cases
The reaction forces on the support obtained from Figure 29 are shown in Table 27 below.

Table 27. Reaction from wind load cases

<table>
<thead>
<tr>
<th>Vertical support reaction</th>
<th>CASE 1</th>
<th>CASE 2</th>
<th>CASE 3</th>
<th>CASE 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R_{Ax}$</td>
<td>-7.92</td>
<td>19.15</td>
<td>-13.94</td>
<td>5.56</td>
</tr>
<tr>
<td>$R_{Bx}$</td>
<td>-7.94</td>
<td>9.08</td>
<td>-6.94</td>
<td>5.54</td>
</tr>
<tr>
<td>$R_{Cx}$</td>
<td>-8.41</td>
<td>7.27</td>
<td>-5.65</td>
<td>5.83</td>
</tr>
<tr>
<td>$R_{Dx}$</td>
<td>-10.64</td>
<td>7.13</td>
<td>-5.65</td>
<td>7.30</td>
</tr>
<tr>
<td>$R_{Ex}$</td>
<td>-20.16</td>
<td>6.90</td>
<td>-5.38</td>
<td>13.30</td>
</tr>
</tbody>
</table>

7.2 Load combination

Load combinations were done according to EN 1990 following the National Annex as shown in Table 28. Since wind load and permanent load from the imperfection don’t play a role in the maximum reaction on the base supports. They can be ignored in case of finding vertical reaction. However, it should be included to compute the horizontal force. There will be plenty of load combination which will be very tedious to calculate manually, so the most obvious critical combination is only considered. The load case which produces maximum base pressure on the supports is shown in Figure 30 and cases for loss of equilibrium is shown in Figure 31. The calculation notes of Table 28 are presented in Appendix 1.

Table 28. Reactions from load combination

<table>
<thead>
<tr>
<th>Load combination</th>
<th>$R_A$</th>
<th>$R_B$</th>
<th>$R_C$</th>
<th>$R_D$</th>
<th>$R_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>STR 6.10a</td>
<td>708.23</td>
<td>1587</td>
<td>1375</td>
<td>752.62</td>
<td>520.84</td>
</tr>
<tr>
<td>STR 6.10b-1</td>
<td>1308</td>
<td>2971</td>
<td>2567</td>
<td>1802</td>
<td>935</td>
</tr>
<tr>
<td>STR 6.10b-2</td>
<td>1172</td>
<td>2617</td>
<td>2258</td>
<td>1430</td>
<td>832.92</td>
</tr>
<tr>
<td>STR 6.10b-3</td>
<td>576.06</td>
<td>1839</td>
<td>1474</td>
<td>1051</td>
<td>409.35</td>
</tr>
<tr>
<td>STR 6.10b-4</td>
<td>620.59</td>
<td>1737</td>
<td>1416</td>
<td>990</td>
<td>435.98</td>
</tr>
</tbody>
</table>
Figure 30. Combination for maximum base pressure

Figure 31. Loss of equilibrium cases
7.3 Design of column

In this section, the method of column design is explained. There are two types of design approaches discussed in Eurocode 2 for the design of RC columns namely Nominal Stiffness method and Nominal Curvature method. For this project, Nominal curvature method was adopted to take into account of Second order effect in the column. The simple flow chart for the design approach of Nominal curvature method is shown in Figure 32.

![Flow chart for nominal curvature method (Frost 2011)](image)

Figure 32. Flow chart for nominal curvature method (Frost 2011)
7.3.1 Geometric Imperfection

Geometric imperfections may be considered by means of a parameter $e_0$ that contributes to the first order moment through the axial load.

$$\theta_i = \theta_0 \alpha_h \alpha_m$$

Where,

$$\theta_0 = \frac{1}{200}$$

$$\alpha_h = \frac{2}{\sqrt{l}}$$ ; $l$= Length of member

$$\alpha_m = \sqrt{0.5 \times \left(1 + \frac{1}{m}\right)}$$ ; $m$= no. of vertical member

Due to inclination, there is eccentricity generated in a column which is given by

$$e_0 = \max \left[\frac{h}{30}, 20, \theta_i \frac{l_0}{2}\right]$$

$l_0 = \text{Effective height}$

The buckling modes and corresponding effective length of the column is given by Eurocode 2 as shown in Figure 33

![Buckling modes and corresponding effective lengths for isolated member](SFS-EN1992 Eurocode 2004)

Figure 33. Buckling modes and corresponding effective lengths for isolated member (SFS-EN1992 Eurocode 2004)

Slenderness limit:
Slenderness limit can be calculated according to EC2 as shown below,
\[ \lambda_{lim} = \frac{20 \ast A \ast B \ast C}{\sqrt{n}} \]

Where
\[ A = \frac{1}{1+0.2h_{ef}} \quad \text{(if } h_{ef} \text{ is not known, } A = 0.7) \]
\[ B = \sqrt{1 + 2\omega} \quad \text{(if } \omega \text{ is not known } B=1.1) \]
\[ C = 1.7 - r_m \quad \text{(if } r_m \text{ is not known, } C = 0.7) \]

\[ n = \frac{N_{Ed}}{f_{cd}A_c} \]
\[ r_m = \frac{M_{01}}{M_{02}} \]

\( M_{01}, M_{02} \) = First order end moments

A detailed calculation of design of column for this building can be found in Appendix 1.

7.4 Design of Beam

Reinforced concrete beam must be designed in such a way that it should withstand the most unfavourable load applied on it. There are certain steps one should follow according to Eurocode to design the beam. The process is summarized below.

7.4.1 Flexure Design

For the given load applied on the beam. The Maximum design moment is calculated. Then the height of the beam is determined \((L=h/10)\). After calculating the height of the beam, Effective depth \((d)\) is calculated as:

\[ d = h - \frac{\varnothing_{\text{main}}}{2} - \varnothing_{\text{stirrup}} - c_{\text{nom}} \]

Where,
\[ \varnothing_{\text{stirrup}} = \text{Diameter of stirrup} \]
\[ \varnothing_{\text{main}} = \text{Diameter of main rebar} \]
\[ c_{\text{nom}} = \text{Nominal concrete cover} \]
After computing, effective depth relative moment is calculated as follows

\[
\mu = \frac{M_{Ed}}{b^*d^2f_{cd}}
\]

Then height ratio is calculated as shown below

\[
\beta = 1 - \sqrt{1 - 2\mu}
\]

After calculation of height ratio lever arm is calculated as follows

\[
z = d \left(1 - \frac{\beta}{2}\right)
\]

Finally, the area of required reinforcement on tension zone is calculated and the minimum requirement is checked according to Eurocode.

\[
A_s = \frac{M}{z*f_{yd}}
\]

Now,

\[
A_{s,min} = \max \left[ 0.26 * \frac{f_{ctm}}{f_{yk}} * b * d \right] \left[ 0.0013 * b * d \right]
\]

In this way, the required reinforcement for flexure design is calculated. The same sets of rules are applied to design this structure. The calculation for this building can be found in Appendix 1.

### 7.4.2 Shear Design

For the given load applied on the beam, the maximum design shear force is calculated. Before anything else, one should check if the beam requires any shear reinforcement or the capacity of beam without shear reinforcement is adequate. The capacity of beam without shear rebar, that is given by Eurocode is presented below.

\[
V_{Rd,c} = \min \left\{ C_{Rd,c} * k * \left(100 * \rho_1 * f_{ck}\right)^\frac{1}{3} + k_1 \sigma_{cp} \right\} * b_w * d
\]

Where,

\[
C_{Rd,c} = \frac{0.18}{\gamma_c}
\]

\[
k = 1 + \sqrt{\frac{200}{d}} \leq 2
\]

\[
\rho_1 = \frac{A_{sf}}{b_w*d}
\]
If the value of shear resistance is smaller than the applied shear force, then the beam requires the shear reinforcement. The shear capacity of the beam is smaller of tension capacity of shear reinforcement and compression capacity of concrete strut.

\[
k_1 = 0.15 \text{ (Recommended)}
\]

\[
\sigma_{cp} = \frac{N_{Ed}}{A_c} < 0.2 \times f_{cd}
\]

\[
v_{\text{min}} = 0.035 \sqrt{k^3} \times f_{ck}
\]

The maximum effective shear reinforcement for \( \cot \theta = 1 \)

\[
A_{sw,\text{max}} f_{yw,d} \leq \frac{1}{2} \alpha_{cw} v_1 f_{cd} \sin \alpha
\]

Where,

\( \alpha = \) Angle between shear reinforcement and the beam axis perpendicular to the shear force
\( \theta = \) Angle between the concrete compression strut and the beam axis perpendicular to the shear force
\( A_{sw} = \) Cross-sectional area of the shear reinforcement
\( s = \) Spacing of the stirrups
\( f_{yw,d} = \) Design yield strength of the shear reinforcement
\( v_1 = \) Strength reduction factor for concrete cracked in shear (0.6=Recommended)
\( \alpha_{cw} = \) Coefficient taking account of the state of the stress in the compression chord (1=Non-prestressed concrete)

If the shear reinforcement is arranged in a direction parallel to the shear force direction, \( \alpha = 0 \) then shear resistance of the beam can be calculated as follow:

Tension capacity of shear reinforcement

\[
V_{Rd,s} = \frac{A_{sw}}{s} \times z \times f_{yw,d} \times \cot \theta
\]
Compression capacity of compression strut

\[ V_{Rd,\text{max}} = \frac{\alpha_{cw} b_w v_1 f_{cd}}{\cot \theta + \tan \theta} \]

\[ V_{Rd} = \min \left[ \frac{V_{Rd,s}}{V_{Rd,\text{max}}} \right] \]

The maximum effective cross-sectional area of shear reinforcement for \( \cot \theta = 1 \)

\[ \frac{A_{sw,\text{max}} f_{yw} d}{b_w s} \leq \frac{1}{2} \alpha_{cw} v_1 f_{cd} \]

In this way, the required reinforcement for shear is calculated. The same sets of rules are applied to design this structure.

7.5 Design of slab

The slab can be designed according to the maximum applied load on the structure in per square meter, from the obtained maximum load the right dimension of the slab was chosen from the design chart shown in Figure 34 below. The maximum applied load on the building was 7kN/m² imposed load added with 7.55kN/m² permanent load which gives 14.55kN/m²

![Figure 34. Load carrying capacity of hollow core slab](image)
As can be seen in Figure 34 above, a hollow core slab of 320mm with the length of 7.8m can carry about 19kN/m². Hence, the hollow core slab of 320mm with the maximum length of 7.8m were used in the building.

7.6 Design of Foundation

All the applied load on the building is transferred via foundation to the ground. There are different types of foundation in existence, However, the design of pad foundation and strip foundation is described in this thesis. Every foundation must be designed in such a way that the bearing pressure from the foundation mustn’t exceed the bearing strength of soil and the foundation must be strong enough to overcome forces and moment applied in the foundation. There are certain steps one should follow according to Eurocode to design the foundation. The process is summarized below.

7.6.1 PAD footing design

At first, the total load from the foundation on the footing was calculated. The base area of the footing is calculated using the given formula

\[
\text{Base area} = \frac{\text{Total characteristics load}}{\text{Bearing pressure of the soil}}
\]

After finding the area of the base, the structural design of the base is executed for the maximum load to the base by the column.

Flexure Design
Maximum bending moment is on the face of the column on pad footing. The ultimate load is taken due to the ultimate load on one side of the footing section. The maximum design moment can be calculated in such a case using the given formula

\[
M_{Ed} = \frac{1}{2} \times p_d \times c^2
\]

After computing moment effective depth of the foundation is calculated from the equation below

\[
d = h - \frac{\phi_{\text{main}}}{2} - \phi_{\text{stirrup}} - c_{\text{nom}}
\]
Where,

\[ \phi_{\text{stirrup}} = \text{Diameter of stirrup} \]
\[ \phi_{\text{main}} = \text{Diameter of main rebar} \]
\[ c_{\text{nom}} = \text{Nominal concrete cover} \]

After computing, effective depth relative moment is calculated as follow

Relative moment (\(\mu\)) = \( \frac{M_{\text{Ed}}}{b \cdot d^2 \cdot f_{cd}} \)

Then height ratio is calculated as shown below:

\[ \beta = 1 - \sqrt{1 - 2\mu} \]

After calculation of height ratio lever arm is calculated as follows:

\[ z = d \left( 1 - \frac{\beta}{2} \right) \]

Finally, the area of required reinforcement on tension zone is calculated and the minimum requirement is checked according to Eurocode.

\[ A_s = \frac{M}{2 \cdot f_{yd}} \]

Now,

\[ A_s,_{\text{min}} = \max \left[ 0.26 \cdot \frac{f_{\text{ctm}}}{f_{yk}} \cdot b \cdot d, \frac{0.0013 \cdot b \cdot d}{f_{cd}} \right] \]

In this way, the required reinforcement for flexure design is calculated. The same sets of rules are applied to design this structure. The calculation for this building can be found in Appendix 1.

Punching Shear Design

The height of the foundation must be designed in such a way, that it should not fail via punching shear failure. The Finnish design method was adopted instead of Eurocode to check the punching shear resistance of the foundation. The resistance is given by

\[ V_{\text{cr}} := k \cdot \beta \cdot (1 + 50p) \cdot u \cdot d \cdot f_{\text{ctc}} \]

(FINLEX 2001)

After calculation the design shear force is calculated using given formula below for a round column

\[ V_{Ed} = p_{Ed} \left[ b f^2 - \frac{\pi}{4} \left( \frac{d_{\text{col}}}{2} + 2d \right)^2 \right] \]
If $V_{cr} > V_{Ed}$ no punching shear failure in the foundation.

8 CONCLUSION, COMPARISON AND RESULTS

It was found that load definition is similar when conducted manually or by using software calculation. However, the calculation of load combination manually was a very tedious and time-consuming job. In this project, there are altogether 1540 load combinations by software. If those combinations must be calculated manually, it would be difficult. Moving from combinations during design of column there are some alterations between manual calculation and software calculation for the same load combination because the software took inclination factor $\Theta_0$ as 1/100 whereas Eurocodes suggest it be 1/200. In manual calculation correlation factor was taken as $Kr$ 0.34 whereas in software obtained that value from a trial method. The difference in the values obtained from the software and manual calculation is shown in Table 29 below.

<table>
<thead>
<tr>
<th></th>
<th>Manual calculation</th>
<th>RSA calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Imperfection</td>
<td>11.18mm</td>
<td>11.2mm</td>
</tr>
<tr>
<td>Slenderness limit</td>
<td>10.81</td>
<td>13.39</td>
</tr>
<tr>
<td>Slenderness</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>First order moment</td>
<td>23.72kNm</td>
<td>23.76 kNm</td>
</tr>
<tr>
<td>Second order eccentricity</td>
<td>14.29mm</td>
<td>15.6mm</td>
</tr>
<tr>
<td>Nominal second order moment</td>
<td>30.31kNm</td>
<td>33.15 kNm</td>
</tr>
<tr>
<td>Maximum design moment</td>
<td>54 kNm</td>
<td>59.75 kNm</td>
</tr>
<tr>
<td>Provided reinforcement</td>
<td>1131 mm2</td>
<td>1357 mm2</td>
</tr>
</tbody>
</table>

During the design of the beam, there were some challenges as RSA was unable to design delta beam because of holes in the geometry. However, the cross section of delta beam was designed from the section definition tab in RSA. Instead of checking the capacity of beam via software the capacity was checked from the Peikko’s standard pre-defined capacity. Only the maximum moment and shear on the beam were obtained from the software while shear resistance and moment resistance were checked through a manual calculation.
For slab design, it was not required to check the calculation via software because the strength of a hollow core slab was known which was used to check if the applied critical load exceeds the capacity. For the foundation design, piles were used because of inadequate load bearing capacity of the soil. Since RSA doesn’t have the features to design piles, it was abandoned. However, punching capacity of the foundation was checked and due to sufficient height of the pad foundation, there was no need for punching reinforcement. RSA was unable to calculate the reinforcement for punching shear even if there was punching. It needs to be calculated manually.

To conclude using software saves time and money whereas manual calculation is a very lengthy process and requires plenty of work. Also, during manual calculation most of the time an iterative method must be used which consume a great deal of time. Furthermore, manual calculation is prone to human error which results in using a higher safety factor which eventually costs oversizing of the structure resulting an expense of extra budget. There are some problems which were not possible to calculate by software; in such cases manual calculation proved to be efficient.

The computation world is developing day by day. It always has something new to give for upcoming generation. It can be assumed that after decades manual calculation will be completely overtaken by computer calculation which is a good thing because it will save time and money for everybody.

REFERENCES


LOADS ON THE STRUCTURE

Appendix 1

Permanent line load on the roof (SW of floor + SW of beam)

\[
\frac{7.55 \text{kN}}{\text{m}^2} \times 6.8 \text{m} + 2.7 \text{kN/m} = 54.04 \text{kN/m}
\]

Permanent load on the floor (SW of floor + SW of beam)

\[
\frac{7.05 \text{kN}}{\text{m}^2} \times 6.8 \text{m} + 2.7 \text{kN/m} = 50.64 \text{kN/m}
\]

Load from imperfection of wall

Lower wall

\[
\frac{1}{300} \times 6.8 \text{m} \times 47.5 \text{kN/m} = 1.077 \text{kN}
\]

Division points=2

\[
\frac{1.077 \text{kN}}{2} = 0.538 \text{kN}
\]

Upper wall

\[
\frac{1}{300} \times 4.15 \text{m} \times 47.5 \text{kN/m} = 0.657 \text{kN}
\]

Imposed load on roof

\[
4 \text{kN/m}^2 \times 6.8 \text{m} = 27.2 \text{kN/m}
\]

Imposed load on floor

\[
7.5 \text{kN/m}^2 \times 6.8 \text{m} = 51 \text{kN/m}
\]

Wind load

Wind from +x on higher structure

\[
3.53 \text{kN}
\]

Wind from +x on lower structure

\[
10.16 \text{kN}
\]
Wind from \(-x\) on higher structure: 3.53kN
Wind from \(-x\) on lower structure: 10.16kN
Wind from \(+y\) on higher structure: -2.73kN
Wind from \(+y\) on lower structure: -14.69kN
Wind from \(-y\) on higher structure: -3.64kN
Wind from \(-y\) on lower structure: -15.93kN

snow load
Snow load calculation

Building location  
Hämeenlinna

Roof slope $\alpha$  
$\alpha := 0$

Roof structure  
Mono pitch with projection

Building length  
$l := 22 \text{m}$

Building breadth  
$b := 22.7 \text{m}$

Building height  
$h := 10.7 \text{m}$

\[ s_k = 2.5 \frac{\text{kN}}{\text{m}^2} \]

Characteristics snow load on the ground

<table>
<thead>
<tr>
<th>Topography</th>
<th>$C_e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Windswept*</td>
<td>0.8</td>
</tr>
<tr>
<td>Normal b</td>
<td>1.0</td>
</tr>
<tr>
<td>Sheltered c</td>
<td>1.0</td>
</tr>
</tbody>
</table>

* Windswept topography: flat unobstructed area exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees.

\[ C_e := \begin{cases} 1 & \text{Normal topography} \\ 1.0 & \text{Sheltered topography} \end{cases} \]

Exposure coefficient

\[ C_e := 1 \]

Thermal Coefficient

\[ C_t := 1 \]
Snow load shape coefficient

\[ \mu_1 = 0.8 \]

Snow load on the roof

\[ s = \mu_1 \cdot C_e \cdot C_t \cdot s_k = 2.4 \text{ kN/m}^2 \]

**Drifted load arrangement for projected structure**
Because $\alpha < 15$

**Snow load shape coefficient at zone A due to projected structure**

Weight density of snow

$$\gamma := 2 \text{ kN/m}^3$$

Coefficient $b_1$

$$b_1 := 7.06 \text{ m}$$

Coefficient $b_2$

$$b_2 := 9.04 \text{ m}$$

Height of projection

$$H := 3.2 \text{ m}$$

Snow load shape coefficient due to wind

$$\mu_{wA} := \min \left( \frac{b_1 + b_2}{2H}, \frac{\gamma \cdot H}{s_k} \right) = 2.516$$

**Snow load shape coefficient at zone B due to projected structure**

Weight density of snow

$$\gamma := 2 \text{ kN/m}^3$$

Coefficient $b_1$

$$b_1 := 7.06 \text{ m}$$

Coefficient $b_2$

$$b_2 := 6.6 \text{ m}$$

Height of projection

$$H := 3.2 \text{ m}$$
snow load shape coefficient due to wind

$$ \mu_{WB} \equiv \min \left( \frac{b_1 + b_2}{2H}, \frac{\gamma H}{s_k} \right) = 2.134 $$

Snow load coefficient at Zone C

$$ \mu_{C} \equiv 0.8 $$

Drifted length

$$ l_s := 2H = 6.4 \text{m} $$

Snow load on zone A

$$ s_A \equiv \mu_{WA} C_e C_t s_k = 6.289 \frac{\text{kN}}{\text{m}^2} $$

Snow load on zone B

$$ s_B \equiv \mu_{WB} C_e C_t s_k = 5.336 \frac{\text{kN}}{\text{m}^2} $$

Snow load on zone C

$$ s_C \equiv \mu_{W} C_e C_t s_k = 2 \frac{\text{kN}}{\text{m}^2} $$

**Snow load shape coefficient at zone F due to projected structure**
Weight density of snow
\[ \gamma = \frac{2 \text{kN}}{m^3} \]

Coefficient b1
\[ b_1 = 9.1 \text{m} \]

Coefficient b2
\[ b_2 = 4.6 \text{m} \]

Height of projection
\[ H = 3.2 \text{m} \]

Snow load shape coefficient due to wind
\[ \mu_{wF} := \min \left\{ \frac{b_1 + b_2}{2H}, \frac{\gamma \cdot H}{s_k} \right\} = 2.141 \]

Drifted length
\[ l_s = 2H = 6.4 \text{m} \]

Snow load on Zone F
\[ s_F := \mu_{wF} C_e C_t s_k = 5.352 \frac{\text{kN}}{m^2} \]
Drifted load arrangement due to taller Building near structure

Because $\alpha < 15$

**Snow load shape coefficient at zone D due to Tall structure**

Weight density of snow

\[ \gamma := \frac{2}{3} \text{kN/m}^3 \]

Coefficient $b_1$

\[ b_1 := 51 \text{m} \]

Coefficient $b_2$

\[ b_2 := 22 \text{m} \]

Height of projection

\[ H := 3.5 \text{m} \]

snow load shape coefficient due to wind

\[ \mu_{wD} := \min \left( \frac{b_1 + b_2}{2H}, \frac{\gamma \cdot H}{s_k} \right) = 2.8 \]

Snow load coefficient at Zone E
\[ \mu_1 = 0.8 \]

Drifted length

\[ l_\text{drift} = 2 \cdot H = 7 \text{m} \]

Snow load on zone D

\[ s_D = \mu_w D C_e C_t s_k = 7 \frac{\text{kN}}{\text{m}^2} \]

Snow load on zone E

\[ s_E = \mu_1 C_e C_t s_k = 2 \frac{\text{kN}}{\text{m}^2} \]

**Combined snow load**

Snow load on top of projected roof

\[ s_{\text{proof}} = 2 \frac{\text{kN}}{\text{m}^2} \]

**Total snow load due to tall Building**

\[
\begin{align*}
\text{Snow load on shaded region} & = 2.45 \text{m} \cdot 4.15 \frac{\text{kN}}{\text{m}^2} + 6.28 \text{m} \cdot 4.15 \frac{\text{kN}}{\text{m}^2} + 20.358 \frac{\text{kN}}{\text{m}} \\
\text{Total snow load due to projection} & = 5.33 \text{kN/m}^2 \\
\text{Line load} & = 5.33 \text{kN/m}^2 \cdot 4.15 \text{m} = 22.12 \frac{\text{kN}}{\text{m}} \]
\]

\[
\begin{align*}
\text{Line load} & = 6.28 \frac{\text{kN}}{\text{m}^2} \cdot 4.15 \text{m} = 26.062 \frac{\text{kN}}{\text{m}} \\
\end{align*}
\]
Total load on shaded region of zone F

\[
\frac{2 \text{ kN}}{\text{m}^2} \times 4.15 \text{ m} = \frac{8.3 \text{ kN}}{\text{m}}
\]

snow load on shaded region of zone F

\[
\frac{5.35 \text{ kN}}{\text{m}^2} \times 2.65 \text{ m} = 2.215 \frac{\text{kN}}{\text{m}^2}
\]

Combined load on Grid 3-3

\[
(8.3 + 20.35) \frac{\text{kN}}{\text{m}} = 28.65 \frac{\text{kN}}{\text{m}}
\]

\[
(22.12 + 20.35) \frac{\text{kN}}{\text{m}} = 42.47 \frac{\text{kN}}{\text{m}}
\]

\[
(26.06 + 20.35) \frac{\text{kN}}{\text{m}} = 46.41 \frac{\text{kN}}{\text{m}}
\]
Wind load calculation

Building data

Building location: Hämeenlinna
Pitch of the roof $\alpha$: $\alpha := 0$
Roof structure: Mono pitch with projection

Building length: $l := 21\text{m}$
Building breadth: $b := 22\text{m}$
Building height: $h := 9.5\text{m}$
Terrain category: 3

Reference height: $z := 9.5\text{m}$; $h < b$

<table>
<thead>
<tr>
<th>Terrain category</th>
<th>$z_0$</th>
<th>$z_{\text{min}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 Sea or coastal area exposed to the open sea</td>
<td>0.003</td>
<td>1</td>
</tr>
<tr>
<td>I Lakes or flat and horizontal area with negligible vegetation and without obstacles</td>
<td>0.01</td>
<td>1</td>
</tr>
<tr>
<td>II Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights</td>
<td>0.05</td>
<td>2</td>
</tr>
<tr>
<td>III Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)</td>
<td>0.3</td>
<td>5</td>
</tr>
<tr>
<td>IV Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15m</td>
<td>1.0</td>
<td>10</td>
</tr>
</tbody>
</table>

The terrain categories are illustrated in Annex A.1.
Fundamental value of basic wind velocity \( v_{b0} := 21 \frac{m}{s} \)

Direction factor \( C_{dir} := 1 \)
Season factor \( C_{season} := 1 \)

Basic velocity of wind \( v_{b} := v_{b0} \cdot C_{dir} \cdot C_{season} = 21 \frac{m}{s} \)

Roughness height \( z_{0} = 0.3 \)
Factor \( z_{0ii} = 0.05 \)

Terrain factor \( K_{r} := 0.19 \left( \frac{z_{0}}{z_{0ii}} \right)^{0.07} = 0.215 \)

Terrain roughness factor \( C_{r} := K_{r} \ln \left( \frac{z}{z_{0}m} \right) = 0.744 \)

Orography factor \( C_{0} := 1 \)

Mean wind velocity \( v_{m} := C_{r} C_{0} v_{b} = 15.629 \frac{m}{s} \)

Density of air \( \rho := 1.25 \frac{kg}{m^{3}} \)

Factor \( K_{1} := 1 \)

Standard deviation of turbulence \( \sigma_{v} := K_{r} v_{b} \cdot K_{1} = 4.523 \frac{m}{s} \)

Turbulence intensity \( I_{v} := \frac{\sigma_{v}}{v_{m}} = 0.289 \)

Characteristics peak velocity pressure \( q_{p} := \left( 1 + 7 \cdot I_{v} \right) \frac{1}{2} \cdot \rho \cdot v_{m}^{2} = 0.462 \frac{kN}{m^{2}} \)
Crosswind dimension

Parameter: $b = 22 \text{ m}$

Depth of structure

Parameter: $d = 21 \text{ m}$

Ratio of $h$ and $d$

Parameter: $\frac{h}{d} = 0.452$

Length parameter $e$

Parameter: $e := \min(b, 2h) = 19 \text{ m}$

Length of zone A

Parameter: $\frac{e}{5} = 3.8 \text{ m}$

Length of zone B

Parameter: $\frac{4}{5} e = 15.2 \text{ m}$

Length of zone C

Parameter: $d - e = 2 \text{ m}$

| Table 7.1 — Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings |
|---|---|---|---|---|---|---|---|
| Zone | $C_{pAge,10}$ | $C_{pAge,1}$ | $C_{pAge,10}$ | $C_{pAge,1}$ | $C_{pAge,10}$ | $C_{pAge,1}$ | $C_{pAge,10}$ | $C_{pAge,1}$ |
| $h/d$ | | | | | | | | |
| 5 | -1.2 | -1.4 | -0.8 | -1.1 | -0.5 | +0.8 | +1.0 | -0.7 |
| 1 | -1.2 | -1.4 | -0.8 | -1.1 | -0.5 | +0.8 | +1.0 | -0.5 |
| $< 0.25$ | -1.2 | -1.4 | -0.8 | -1.1 | -0.5 | +0.7 | +1.0 | -0.3 |
Since the ratio of $h/d$ is between 0.25 and 1 it must be interpolated

External pressure for upwind face D

\[ C_{pe10} \geq 0.8 \quad \text{For } h/d=1 \]
\[ C_{pe10} \geq 0.7 \quad \text{For } h/d=0.25 \]

For,
\[ \frac{h}{d} = 0.452 \]
\[ C_{pe10} = 0.7 + \left( \frac{h}{d} - 0.25 \right) \frac{0.8 - 0.7}{1 - 0.25} = 0.727 \]

External pressure for downwind face E

\[ C_{pe10} \geq -0.5 \quad \text{For } h/d=1 \]
\[ C_{pe10} \geq -0.3 \quad \text{For } h/d=0.25 \]

For,
\[ \frac{h}{d} = 0.452 \]
\[ C_{pe10} = -0.3 + \left( \frac{h}{d} - 0.25 \right) \frac{-0.5 + 0.3}{1 - 0.25} = -0.354 \]

<table>
<thead>
<tr>
<th>Roof type</th>
<th>Zone</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(G_{FL,10})</td>
<td>(G_{FL,10})</td>
<td>(G_{FL,1})</td>
<td>(G_{FL,10})</td>
</tr>
<tr>
<td>Sharp eaves</td>
<td>-1.8</td>
<td>-2.5</td>
<td>-1.2</td>
<td>-2.0</td>
<td>-0.7</td>
</tr>
<tr>
<td></td>
<td>-1.6</td>
<td>-2.2</td>
<td>-1.1</td>
<td>-1.8</td>
<td>-0.7</td>
</tr>
<tr>
<td>With Parapets</td>
<td>-1.4</td>
<td>-2.0</td>
<td>-0.9</td>
<td>-1.6</td>
<td>-0.7</td>
</tr>
<tr>
<td>(h/d=0.025)</td>
<td>-1.2</td>
<td>-1.8</td>
<td>-0.8</td>
<td>-1.4</td>
<td>-0.7</td>
</tr>
<tr>
<td>Curved Eaves</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(r/h = 0.05)</td>
<td>-1.0</td>
<td>-1.5</td>
<td>-1.2</td>
<td>-1.8</td>
<td>-0.4</td>
</tr>
<tr>
<td>(r/h = 0.10)</td>
<td>-0.7</td>
<td>-1.2</td>
<td>-0.8</td>
<td>-1.4</td>
<td>-0.3</td>
</tr>
<tr>
<td>(r/h = 0.20)</td>
<td>-0.5</td>
<td>-0.8</td>
<td>-0.5</td>
<td>-0.8</td>
<td>-0.3</td>
</tr>
<tr>
<td>Mansard Eaves</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(\alpha = 30^\circ)</td>
<td>-1.0</td>
<td>-1.5</td>
<td>-1.0</td>
<td>-1.5</td>
<td>-0.3</td>
</tr>
<tr>
<td>(\alpha = 45^\circ)</td>
<td>-1.2</td>
<td>-1.8</td>
<td>-1.3</td>
<td>-1.9</td>
<td>-0.4</td>
</tr>
<tr>
<td>(\alpha = 60^\circ)</td>
<td>-1.3</td>
<td>-1.9</td>
<td>-1.3</td>
<td>-1.9</td>
<td>-0.5</td>
</tr>
</tbody>
</table>
**Figure 7.6 — Key for flat roofs**

Length of Zone F on crosswind direction

\[ \frac{e}{4} = 4.75 \text{m} \]

Length of Zone F on depth direction

\[ \frac{e}{10} = 1.9 \text{m} \]

Length of Zone G on depth direction

\[ \frac{e}{2} = 9.5 \text{m} \]
Length of Zone F on crosswind direction

\[ \frac{2e}{5} = 7.6 \text{m} \]

Length of Zone H on depth direction

\[ d - \frac{e}{2} = 11.5 \text{m} \]

Length of Zone I on depth direction

Wind pressure on external surface

Zone A

\[ w_{eA} = q_p(1.2) = -0.554 \frac{\text{kN}}{\text{m}^2} \]

Zone B

\[ w_{eB} = q_p(-0.8) = -0.37 \frac{\text{kN}}{\text{m}^2} \]

Zone C

\[ w_{eC} = q_p(-0.5) = -0.231 \frac{\text{kN}}{\text{m}^2} \]

Zone D

\[ w_{eD} = q_p(0.727) = 0.336 \frac{\text{kN}}{\text{m}^2} \]

Zone E

\[ w_{eE} = q_p(-0.354) = -0.164 \frac{\text{kN}}{\text{m}^2} \]

Zone F

\[ w_{eF} = q_p(-1.8) = -0.831 \frac{\text{kN}}{\text{m}^2} \]

Zone G

\[ w_{eG} = q_p(-1.2) = -0.554 \frac{\text{kN}}{\text{m}^2} \]

Zone H

\[ w_{eH} = q_p(-0.7) = -0.323 \frac{\text{kN}}{\text{m}^2} \]

Zone I

\[ w_{eI} = q_p(0.2) = 0.092 \frac{\text{kN}}{\text{m}^2} \]
Internal pressure should be taken more onerous of +0.2 and -0.3

\[ C_{pi} = -0.3, 0.2 \]

**Structural factor**

- \( C_s = 1 \)
- \( C_d = 1 \)

**Net pressure**

\( P(C_{pe10}, C_{pi}) := C_s \cdot q \cdot q_{p} \cdot C_{pe10} - q_{p} \cdot C_{pi} \)

**Zone A**

\[ P(-1.2, 0.2) = -0.647 \frac{kN}{m^2} \]
\[ P(-1.2, -0.3) = -0.416 \frac{kN}{m^2} \]

**Zone B**

\[ P(-0.8, 0.2) = -0.462 \frac{kN}{m^2} \]
\[ P(-0.8, -0.3) = -0.231 \frac{kN}{m^2} \]

**Zone C**

\[ P(-0.5, 0.2) = -0.323 \frac{kN}{m^2} \]
\[ P(-0.5, -0.3) = -0.092 \frac{kN}{m^2} \]

**Zone D**

\[ P(0.727, 0.2) = 0.243 \frac{kN}{m^2} \]
\[ P(0.727, -0.3) = 0.474 \frac{kN}{m^2} \]

**Zone E**

\[ P(-0.354, 0.2) = -0.256 \frac{kN}{m^2} \]
\[ P(-0.354, -0.3) = -0.025 \text{ kN/m}^2 \]

Zone F

\[ P(-1.8, 0.2) = -0.924 \text{ kN/m}^2 \]

\[ P(-1.8, -0.3) = -0.693 \text{ kN/m}^2 \]

Zone G

\[ P(-1.2, 0.2) = -0.647 \text{ kN/m}^2 \]

\[ P(-1.2, -0.3) = -0.416 \text{ kN/m}^2 \]

Zone H

\[ P(-0.7, 0.2) = -0.416 \text{ kN/m}^2 \]

\[ P(-0.7, -0.3) = -0.185 \text{ kN/m}^2 \]

Zone I

\[ P(0.2, 0.2) = 0 \text{ kN/m}^2 \]

\[ P(0.2, -0.3) = 0.231 \text{ kN/m}^2 \]
Vertical support reaction from building
Calculation for maximum and minimum vertical support reaction

Reliability class 2

Maximum support reaction at A from permanent action

Maximum support reaction at A from snow load

Maximum support reaction at A from imposed load

Combination factor for snow and imposed load

Combination factor for imposed load

Common Combination factor $\psi$

Largest reaction at A

STR 6.10a

Leading action imposed load STR 6.10b-1

Leading action snow STR 6.10b-2

Largest reaction at B

Maximum support reaction at B from permanent action

Maximum support reaction at B from snow load

Maximum support reaction at B from imposed load

$K_{FI} = 1$

$G_{kA} = 524.62 \text{kN}$

$Q_{kAsnow} = 98.96 \text{kN}$

$Q_{kAimposed} = 400.52 \text{kN}$

$\psi_{0s} := 0.7$

$\psi_{0I} := 0.7$

$\psi := 0.7$

$1.35 K_{FI} G_{kA} = 708.237 \text{kN}$

$1.15 K_{FI} G_{kA} + 1.5 K_{FI} Q_{kAimposed} + 1.5 \psi Q_{kAsnow} = 1.308 \times 10^{3} \text{kN}$

$1.15 K_{FI} G_{kA} + 1.5 K_{FI} Q_{kAsnow} + 1.5 \psi Q_{kAimposed} = 1.172 \times 10^{3} \text{kN}$

$G_{kB} = 1175.56 \text{kN}$

$Q_{kBsnow} = 172.10 \text{kN}$

$Q_{kBimposed} = 959.14 \text{kN}$
Combination factor $\psi$

$\psi := 0.7$

STR 6.10a

$1.35 K_F I G_{kB} = 1.587 \times 10^3 \text{kN}$

Leading action imposed load STR 6.10b-1

$1.15 K_F I G_{kB} + 1.5 K_F I Q_{kBimposed} + 1.5 \psi \cdot Q_{kBsnow} = 2.971 \times 10^3 \text{kN}$

Leading action snow STR 6.10b-2

$1.15 K_F I G_{kB} + 1.5 K_F I Q_{kBimposed} + 1.5 \psi \cdot Q_{kBimposed} = 2.617 \times 10^3 \text{kN}$

**Largest reaction at C**

Maximum support reaction at C from permanent action

$G_{kC} := 1018.8 \text{kN}$

Maximum support reaction at C from snow load

$Q_{kCsnow} := 143.08 \text{kN}$

Maximum support reaction at C from imposed load

$Q_{kCimposed} := 829.94 \text{kN}$

Combination factor $\psi$

$\psi := 0.7$

STR 6.10a

$1.35 K_F I G_{kC} = 1.375 \times 10^3 \text{kN}$

Leading action imposed load STR 6.10b-1

$1.15 K_F I G_{kC} + 1.5 K_F I Q_{kCimposed} + 1.5 \psi \cdot Q_{kCimposed} = 2.567 \times 10^3 \text{kN}$

Leading action snow STR 6.10b-2

$1.15 K_F I G_{kC} + 1.5 K_F I Q_{kCimposed} + 1.5 \psi \cdot Q_{kCimposed} = 2.258 \times 10^3 \text{kN}$

**Largest reaction at D**
Maximum support reaction at D from permanent action

Maximum support reaction at D from snow load

Maximum support reaction at D from imposed load

Combination factor $\psi$

$\psi = 0.7$

STR 6.10a

Leading action imposed load STR 6.10b-1

$1.15 K_{FI} G_{KD} + 1.5 K_{FI} Q_{kD imposed} + 1.5 \psi \cdot Q_{kD snow} = 1.802 \times 10^3 \text{kN}$

Leading action snow STR 6.10b-2

$1.15 K_{FI} G_{KD} + 1.5 K_{FI} Q_{kD snow} + 1.5 \psi \cdot Q_{kD imposed} = 1.612 \times 10^3 \text{kN}$

Largest reaction at E

Maximum support reaction at E from permanent action

Maximum support reaction at E from snow load

Maximum support reaction at E from imposed load

Combination factor $\psi$

$\psi = 0.7$

STR 6.10a

Leading action imposed load STR 6.10b-1

$1.15 K_{FI} G_{KE} + 1.5 K_{FI} Q_{kE imposed} + 1.5 \psi \cdot Q_{kE snow} = 935.08 \text{kN}$
Leading action snow STR 6.10b-2

\[1.15 \cdot \text{K} \cdot G \cdot k_E + 1.5 \cdot \text{K} \cdot Q \cdot k_{\text{snow}} + 1.5 \cdot \psi \cdot Q \cdot k_{\text{imposed}} = 832.926 \text{kN}\]

**Minimum support reaction at A**

Support reaction at A from permanent action

\[Q_{kA} := 524.62 \text{kN}\]

Support reaction at A from snow load

\[Q_{kB} := 98.96 \text{kN}\]

Minimum support reaction at A from imposed load

\[Q_{kB} := 0 \text{kN}\]

Combination factor \(\psi\)

\[\psi := 0.7\]

Largest reaction at A

Leading action imposed load STR 6.10b-3

\[0.9 \cdot \text{K} \cdot G \cdot k_A + 1.5 \cdot \text{K} \cdot Q \cdot k_{\text{imposed}} + 1.5 \cdot \psi \cdot Q \cdot k_{\text{snow}} = 576.066 \text{kN}\]

Leading action snow STR 6.10b-4

\[0.9 \cdot \text{K} \cdot G \cdot k_A + 1.5 \cdot \text{K} \cdot Q \cdot k_{\text{imposed}} + 1.5 \cdot \psi \cdot Q \cdot k_{\text{snow}} + 576.066 = 620.598 \text{kN}\]

**Minimum reaction at B**

Support reaction at B from permanent action

\[Q_{kB} = 1175.56 \text{kN}\]

Support reaction at B from snow load

\[Q_{kB} := 172.10 \text{kN}\]

Minimum support reaction at B from imposed load

\[Q_{kB} := 400.52 \text{kN}\]

Combination factor \(\psi\)

\[\psi := 0.7\]
Leading action imposed load STR 6.10b-3

\[ 0.9 K_F I G_K + 1.5 K_F I Q_{K_{imposed}} + 1.5 \psi \cdot Q_{K_{snow}} = 1.839 \times 10^3 \text{kN} \]

Leading action snow STR 6.10b-4

\[ 0.9 K_F I G_K + 1.5 K_F I Q_{K_{snow}} + 1.5 \psi \cdot Q_{K_{imposed}} = 1.737 \times 10^3 \text{kN} \]

**Minimum reaction at C**

Support reaction at C from permanent action

\[ G_{kC} = 1018.8 \text{kN} \]

Support reaction at C from snow load

\[ Q_{kC_{snow}} = 143.08 \text{kN} \]

Minimum support reaction at C from imposed load

\[ Q_{kC_{imposed}} = 271.32 \text{kN} \]

Combination factor \( \psi \)

\[ \psi = 0.7 \]

Leading action imposed load STR 6.10b-3

\[ 0.9 K_F I G_K + 1.5 K_F I Q_{kC_{imposed}} + 1.5 \psi \cdot Q_{kC_{snow}} = 1.474 \times 10^3 \text{kN} \]

Leading action snow STR 6.10b-4

\[ 0.9 K_F I G_K + 1.5 K_F I Q_{kC_{snow}} + 1.5 \psi \cdot Q_{kC_{imposed}} = 1.416 \times 10^3 \text{kN} \]

**Minimum reaction at D**

Support reaction at D from permanent action

\[ G_{kD} = 557.50 \text{kN} \]

Support reaction at D from snow load

\[ Q_{kD_{snow}} = 135.86 \text{kN} \]

Minimum support reaction at D from imposed load

\[ Q_{kD_{imposed}} = 271.32 \text{kN} \]
Combination factor $\psi$  

$\psi := 0.7$

Leading action imposed load STR 6.10b-1

$$0.9K_{FI}G_kD + 1.5K_{FI}Q_{kDimposed} + 1.5\psi Q_{kDsnow} = 1.051 \times 10^3 \cdot kN$$

Leading action snow STR 6.10b-2

$$0.9K_{FI}G_kD + 1.5K_{FI}Q_{kDsnow} + 1.5\psi Q_{kDimposed} = 990.426 kN$$

Minimum reaction at $E$

Support reaction at $E$ from permanent action

$G_{kE} := 385.8 \text{ kN}$

Support reaction at $E$ from snow load

$Q_{kEsnow} := 59.17 \text{ kN}$

Minimum support reaction at $E$ from imposed load

$Q_{kEimposed} := 0 \text{ kN}$

Combination factor $\psi$

$\psi := 0.7$

Leading action imposed load STR 6.10b-3

$$0.9K_{FI}G_{kE} + 1.5K_{FI}Q_{kEimposed} + 1.5\psi Q_{kEsnow} = 409.358 kN$$

Leading action snow STR 6.10b-4

$$0.9K_{FI}G_{kE} + 1.5K_{FI}Q_{kEsnow} + 1.5\psi Q_{kEimposed} = 435.984 kN$$
**Column design hand calculation**

**Column Geometry**
Diameter of column
\[ d_c := 400 \text{mm} \]
Clear length of the column
\[ l_c := 5000 \text{mm} \]
Concrete details
Characteristics strength of concrete
\[ f_{ck} := 30 \frac{N}{\text{mm}^2} \]
Partial safety factor for concrete
\[ \gamma_c := 1.5 \]
Coefficient \( \alpha_{cc} \)
\[ \alpha_{cc} := 0.85 \]
Maximum aggregate size
\[ d_g := 20 \text{mm} \]
Design strength of concrete
\[ f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 17 \frac{N}{\text{mm}^2} \]
Column mean value
\[ f_{cm} := 8 \text{MPa} + f_{ck} = 38 \text{MPa} \]
Area of concrete
\[ A_c = \frac{\pi \cdot d_c^2}{4} = 1.257 \times 10^4 \text{mm}^2 \]

**Reinforcement details**
Characteristics yield strength of reinforcement
\[ f_{yk} := 500 \frac{N}{\text{mm}^2} \]
Partial safety factor for steel
\[ \gamma_s := 1.15 \]
Design yield strength of reinforcement
\[ f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.783 \frac{N}{\text{mm}^2} \]
Modulus of Elasticity
\[ E_s := 200 \frac{kN}{\text{mm}^2} \]
Longitudinal bar diameter
\[ \phi := 12 \text{mm} \]
Link diameter
\[ \phi_{link} := 8 \text{mm} \]

**Axial load and bending moments from frame analysis**
Design axial load
\[ N_{Ed} := 2121.20 \text{kN} \]
The moment can be ignored because of small values.

**Minimum concrete cover check**
Min cover for durability
\[ c_{\text{dur}} := 15\text{mm} \]

Min cover required for bond
\[ c_{\text{min}} := \max\left(\phi, c_{\text{dur}}, 10\cdot\text{mm}\right) = 15\cdot\text{mm} \]

Allowance for deviation
\[ \Delta c_{\text{dev}} := 10\text{mm} \]

Minimum nominal cover
\[ C_{\text{nom}} := c_{\text{min}} + \Delta c_{\text{dev}} = 25\cdot\text{mm} \]

Effective depth
\[ d := d_c - C_{\text{nom}} - \phi_{\text{link}} - \frac{\phi}{2} = 361\cdot\text{mm} \]

### Effective length of column

**Effective length** \( l_c \): **Conservative factors for braced columns**

<table>
<thead>
<tr>
<th>End condition at top</th>
<th>End condition at bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Condition 1</td>
<td>Condition 2</td>
</tr>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>0.75</td>
</tr>
<tr>
<td>2</td>
<td>0.80</td>
</tr>
<tr>
<td>3</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**Key**

- **Condition 1**: Column connected monolithically to beams on each side that are at least as deep as the overall depth of the column in the plane considered. Where the column is connected to a foundation this should be designed to carry moment in order to satisfy this condition.
- **Condition 2**: Column connected monolithically to beams on each side that are shallower than the overall depth of the column in the plane considered by generally not less than half the column depth.
- **Condition 3**: Column connected to members that do not provide more than nominal restraint to rotation.

**Note**

Table taken from *Manual for the design of concrete building structures to Eurocode 2* [21]. The values are those used in BS 8110: Part 1: 1987[14] for braced columns. These values are close to those values that would be derived if the contribution from adjacent columns were ignored.

Conservative factors
\[ f := 1 \]

Condition 3

Effective length
\[ l_0 := f \cdot l_c = 5\text{m} \]

**Column slenderness**

\[ I := \frac{\pi}{4} \left( \frac{d_c}{2} \right)^4 = 1.257 \times 10^9\text{mm}^4 \]

Second moment of area of column

Radius of gyration
\[ i_y := \sqrt{\frac{I}{A_c}} = 0.1\text{m} \]

Slenderness ratio
\[ \lambda := \frac{l_0}{i_y} = 50 \]

**Slenderness limit for buckling about y axis**
Factor A
\[ A := 0.7 \]

Factor B
\[ B := 1.1 \]

Factor C
\[ C := 0.7 \]

Relative normal force
\[ n := \frac{N_{Ed}}{A_c f_{cd}} = 0.993 \]

Slenderness Limit
\[ \lambda_{\text{lim}} := \frac{20 A \cdot B \cdot C}{\sqrt{n}} = 10.818 \]
\[ \lambda_{\text{lim}} < \lambda \quad \text{Second order must be considered} \]

Local second order bending moment about y-axis

Relative humidity
\[ \text{RH} = 50 \]
Column perimeter in contact with atmosphere
\[ u := \pi \cdot d = 1.134 \text{m} \]

Age of concrete at loading
\[ t_0 := 28 \text{ days} \]
Notional size of column
\[ h_0 := \frac{2 \cdot A_c}{u} = 0.222 \text{m} \]

Factor \( \alpha_1 \)
\[ \alpha_1 := \left( \frac{35 \text{MPa}}{f_{cm}} \right)^{0.7} = 0.944 \]

Factor \( \alpha_2 \)
\[ \alpha_2 := \left( \frac{35 \text{MPa}}{f_{cm}} \right)^{0.2} = 0.984 \]

Relative humidity factor
\[ \phi_{\text{RH}} := \left( 1 - \frac{\text{RH}}{100} \right) \cdot \alpha_1 + \frac{\alpha_2}{0.1 \cdot \sqrt{200}} = 1.778 \]

Concrete strength factor
\[ \beta_{f_{cm}} = 16.8 \left( \frac{1 \text{MPa}}{f_{cm}} \right) = 2.725 \]

Concrete age factor
\[ \beta_{t_0} := \frac{1}{0.1 + \frac{t_0}{0.2}} = 0.488 \]
Notional creep coefficient
\( \phi_0 := \phi_{RH} \beta_{fcm} \beta_{t0} = 2.366 \)

Final creep development factor
\( \beta_{c\infty} := 1 \)

Final creep coefficient
\( \phi_{\infty} := \phi_0 \beta_{c\infty} = 2.366 \)

Ratio of SLS to ULS moment
\( r_M := 0.65 \)

Factor \( \beta \)
\[
\beta := 0.35 + \frac{f_{ck}}{200\text{MPa}} - \frac{\lambda}{150} = 0.167
\]

Effective creep ratio
\( \phi_{ef} := \phi_{\infty} r_M = 1.538 \)

Axial load correction factor
\[
K_r = \frac{n_u - n}{n_u - n_{bal}} \leq 1
\]
\( K_r := 0.35 \)

Creep factor
\( K_{\phi} := \max(1, 1 + \beta \phi_{ef}) = 1.256 \)

Curvature distribution factor
\( c := 10 \)

Factor \( e_2 \)
\[
e_2 := K_r K_{\phi} \frac{f_{yd}}{E_y (0.45 d)} \frac{l_0^2}{c^3} = 14.29 \text{mm}
\]

Nominal second order moment
\( M_2 := N_{Ed} e_2 = 30.314 \text{kN·m} \)

**Geometric imperfection**

\( \theta_0 := \frac{1}{200} \)
\( \alpha_h := \frac{2}{\sqrt{5}} = 0.894 \)
\( m := 1 \)
\( \alpha_m := \sqrt{0.5 \left(1 + \frac{1}{m}\right)} = 1 \)
\( \theta_1 := \theta_0 \alpha_h \alpha_m = 4.472 \times 10^{-3} \)

The imperfection causes eccentricity
\( e_0 := \frac{\theta_1 l_0}{2} = 11.18 \text{mm} \)

The first order design moment
\( M_{0Ed} := N_{Ed} e_0 = 2.372 \times 10^4 \text{J} \)
Total design value of moment

\[ M_{Ed} = M_{0Ed} + M_2 = 5.403 \times 10^4 \text{J} \]

Required reinforcement on column

\[ \frac{N_{Ed}}{d^2 f_{cd}} = 0.957 \]

\[ \frac{M_{Ed}}{d^3 f_{cd}} = 0.068 \]

From figure above since \( d/h = 0.86 \)

\( \omega := 0.2 \)

\[ A_{sreq} := \frac{\omega A_c f_{cd}}{f_{yd}} = 982.69 \text{mm}^2 \]

Choose 10 T12

Longitudinal bar diameter

\( \phi := 12 \text{mm} \)

\[ A_{sprov} := 10 \cdot \frac{\pi \cdot \phi^2}{4} = 1.131 \times 10^3 \text{mm}^2 \]

\[ A_{smin} := \max \left( 0.1 \frac{N_{Ed}}{f_{yd}}, 0.002 A_c \right) = 487.876 \text{mm}^2 \]

\[ A_{smax} := 0.04 A_c = 5.027 \times 10^3 \text{mm}^2 \]

Selection of link

Minimum diameter

\( \phi \_\text{linkmin} := \max(6 \text{mm}, 0.25\phi) = 6 \text{mm} \)
Chose 8mm link

Link diameter
\( \phi_{\text{link}} = 8\text{mm} \)

Anchorage with a bend

Anchorage length
\( \max\{10 \cdot \phi_{\text{link}}, 70\text{mm}\} = 80\text{mm} \)

Anchorage with a hook
\( \max\{5 \cdot \phi_{\text{link}}, 50\text{mm}\} = 50\text{mm} \)

Spacing for the link
\( S_{\text{clmax}} := \min(20 \cdot \phi, d, 400\text{mm}) = 240\text{mm} \)

Reduction of link
\( S_{\text{clred}} := 0.6 S_{\text{clmax}} = 144\text{mm} \)

Choose

In between column \( 250\text{mm} \)
Near lapped joint and above beam and column \( 150\text{mm} \)
**Column design from RSA**

1 **Level:**
   - Name: Level +5,00
   - Reference level: 0,00 (m)
   - Concrete creep coefficient: \( \phi_p = 1,54 \)
   - Cement class: N
   - Environment class: XC1
   - Structure class: S4

2 **Column: Column2**

2.1 **Material properties:**
   - Concrete: C30/37
   - \( f_{ck} = 30,00 \) (MPa)
   - Unit weight: 2501,36 (kG/m3)
   - Aggregate size: 20,0 (mm)
   - Longitudinal reinforcement: A500HW
   - \( f_{yk} = 500,00 \) (MPa)
   - Ductility class: C
   - Transversal reinforcement: A500HW
   - \( f_{yk} = 500,00 \) (MPa)

2.2 **Geometry:**

   2.2.1 C  
   Diameter = 400,0 (mm)
   2.2.2 Height: L = 5,00 (m)
   2.2.3 Slab thickness = 0,00 (m)
   2.2.4 Beam height = 0,00 (m)
   2.2.5 Cover = 25,0 (mm)

2.3 **Calculation options:**
   - Calculations according to SFS-EN 1992-1-1
   - Seismic dispositions: No requirements
   - Precast column: no
   - Pre-design: no
   - Slenderness took into account: yes
• Compression: with bending
• Ties: to slab
• Fire resistance class: No requirements

2.4 Loads:

<table>
<thead>
<tr>
<th>Case</th>
<th>Nature</th>
<th>Group</th>
<th>$\gamma_i$</th>
<th>N</th>
<th>My(s)</th>
<th>My(i)</th>
<th>Mz(s)</th>
<th>Mz(i)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ULS/10=2<em>1.15 + 1</em>1.15 + 4<em>1.50 + 14</em>1.05 3.80</td>
<td>0.00</td>
<td>design(Structural)</td>
<td>2</td>
<td>1.00</td>
<td>2121.20</td>
<td>1,50</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$\gamma_i$ - load factor

2.5 Calculation results:

Safety factors $R_d/Ed = 1,03 > 1.0$

2.5.1 ULS/ALS Analysis

Design combination: ULS/10=2*1.15 + 1*1.15 + 4*1.50 + 14*1.05 (C)
Combination type: ULS
Internal forces:
  $N_{sd} = 2121.20$ (kN)  $M_{sd}y = 2,88$ (kN*m)  $M_{sd}z = 0,00$ (kN*m)
Design forces:
Cross-section in the middle of the column
  $N = 2121.20$ (kN)  $N*etotz = 59,75$ (kN*m)  $N*etoty = 42,42$ (kN*m)

Eccentricity:
  $e_z (My/N)$  $e_y (Mz/N)$
Static
  $e_{Ed}$: 1,4 (mm)  0,0 (mm)
Imperfection
  $e_i$: 11,2 (mm)  0,0 (mm)
II order
  $e_2$: 15,6 (mm)  0,0 (mm)
Minimal
  $emin$: 20,0 (mm)  20,0 (mm)
Total
  $etot$: 28,2 (mm)  20,0 (mm)

2.5.1.1 Detailed analysis-Direction Y:

2.5.1.1.1 Slenderness analysis

Non-sway structure

<table>
<thead>
<tr>
<th>L (m)</th>
<th>Lo (m)</th>
<th>$\lambda$</th>
<th>$\lambda_{lim}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>5,00</td>
<td>5,00</td>
<td>50,00</td>
<td>13,39</td>
</tr>
</tbody>
</table>

Slender column

2.5.1.1.2 Buckling analysis

$M_2 = 3,80$ (kN*m)  $M_1 = 1,50$ (kN*m)  $M_{mid} = 2,88$ (kN*m)
Case: Cross-section in the middle of the column, Slenderness taken into account
$M_0e = 0.6*M_02+0.4*M_01 = 2,88$ (kN*m)
$M_0emin = 0.4*M_02$
$M_0 = max(M_0e, M_0emin)$

$e_a = 0.1*Lo/2 = 11,2$ (mm)
$0.1 = 0.01 + 0.89 * 0.00$
$0.00 = 0.01$
$0.89 = 0.89$
$0.00 = (0.5(1+1/m))^0.5 = 1,00$
$m = 1,00$

Method based on nominal curvature
$M_2 = N * e_2 = 33,15$ (kN*m)
e_2 = 10*2 / c * (1/r) = 15,6$ (mm)
c = 10,00
$(1/r) = K_r*K_p*(1/r_0) = 0,01$
$K_r = 0,32$
$K_p = 1 + \beta * \psi = 1,26$
$\beta = 0.35 + fck/200$ * $\lambda = 0,17$
$\psi = 1,54$
1/r0 = (fyd/Es)/(0.45*d) = 0.02
\[ (5.35) \]
\[ d = 313.8 \text{ (mm)} \]
\[ Es = 200000.00 \text{ (MPa)} \]
\[ fyd = 434.78 \text{ (MPa)} \]

MEdmin = 42,42 (kN*m)
MEd = max(MEdmin,M0Ed + M2) = 59,75 (kN*m)

2.5.1.2. Detailed analysis-Direction Z:

M2 = 0,00 (kN*m)  M1 = 0,00 (kN*m)  Mmid = 0,00 (kN*m)
Case: Cross-section in the middle of the column, Slenderness not taken into account
M0e = 0.6*M02 + 0.4*M01 = 0,00 (kN*m)
M0emin = 0.4*M02
M0 = max(M0e, M0emin)
ea = 0.0 (mm)
Ma = N*ea = 0,00 (kN*m)
MEdmin = 42,42 (kN*m)
M0Ed = max(MEdmin, M0 + Ma) = 42,42 (kN*m)

2.5.2 Reinforcement:

Real (provided) area  Asr = 1357,17 (mm2)
Ratio:  \[ \rho = 1.08 \% \]

2.6 Reinforcement:

Main bars (A500HW):
- 12 \( \phi 12 \)  \( l = 4.98 \) (m)

Transversal reinforcement: (A500HW):
- stirrups:  30 \( \phi 8 \)  \( l = 1.21 \) (m)

3 Material survey:

- Concrete volume  = 0.63 (m3)
- Formwork = 6.28 (m2)

- Steel A500HW
  - Total weight  = 67,36 (kG)
  - Density  = 107,21 (kG/m3)
  - Average diameter  = 10,5 (mm)
  - Reinforcement survey:

<table>
<thead>
<tr>
<th>Diameter</th>
<th>Length</th>
<th>Weight</th>
<th>Number</th>
<th>Total weight</th>
</tr>
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<tbody>
<tr>
<td>(m)</td>
<td>(m)</td>
<td>(kG)</td>
<td>(No.)</td>
<td>(kG)</td>
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<tr>
<td>8</td>
<td>1.21</td>
<td>0.48</td>
<td>30</td>
<td>14.34</td>
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<tr>
<td>12</td>
<td>4.98</td>
<td>4.42</td>
<td>12</td>
<td>53.02</td>
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<tr>
<td>KOHDE (RAKENNEOSA)</td>
<td>LIJULUOKKA (vähinütün)</td>
<td>RASITUSLUOKKA</td>
<td>Nimetinä vähimmäis- betonipesä ulottuvuuden teräksen pintoa [mm] (RSL)</td>
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<td>---------------------------------------</td>
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<td>---------------------------------------------------------------------</td>
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<tr>
<td>Anturat (PV)</td>
<td>C30/37</td>
<td>XC2</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Anturakuulat (PV)</td>
<td>C30/37</td>
<td>XC2</td>
<td>35</td>
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</tr>
<tr>
<td>Väestönsojan betonirakenteet (PV)</td>
<td>C30/37</td>
<td>XC1</td>
<td>20</td>
<td></td>
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<tr>
<td>Maanvaroaita, Tä-teosta (PV)</td>
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<tr>
<td>Sisätiloissa</td>
<td>C35/45</td>
<td>XC1</td>
<td>20</td>
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<tr>
<td>Ulkotiloissa</td>
<td>C35/45</td>
<td>XC3,4; XF1; Säänkestävä</td>
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</tr>
<tr>
<td>Kantava, alojihdaalta</td>
<td>C30/37</td>
<td>XC1</td>
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<tr>
<td>Ulkotiloissa</td>
<td>C35/45</td>
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<td>Polkkalelementti</td>
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<td>XC1</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Ulkotiloissa</td>
<td>C35/45</td>
<td>XC3, XF1; Säänkestävä</td>
<td>35</td>
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<td>Panteke-elementti</td>
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<tr>
<td>Yläpinta</td>
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<tr>
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<td>C35/45</td>
<td>XC3, XF1; Säänkestävä</td>
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<td>C35/45</td>
<td>XC3,4; XF1; Säänkestävä</td>
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<tr>
<td>Elementtit ja maavaudut</td>
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<td>Sisätiloissa</td>
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<td>XC1</td>
<td>Saunottavan rakenneosan mukateali</td>
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<tr>
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<td>C32/40</td>
<td>XC4; XF3; Säänkestävä</td>
<td>50</td>
<td></td>
</tr>
</tbody>
</table>

Suojapakkauksien suurin soiltu mittapäikkeæ: ±10mm

Betonirakenteiden käyttöikäsuunnittelu

Toimistorakennus, normaalitaso

Koko rakennuksen suunnittelukäyttöikä: 50 v

Betonirakenteiden suunnittelukäyttöikä
Perustukset: 100 v
Kantava runko (sisärakenteet): 100 v
Ulkoseinät: 50 v

Figure 35. Nominal concrete cover and building design life (Joensuun Juva Oy 2016).