Saimaa University of Applied Sciences
The Faculty of Technology, Lappeenranta
Double Degree Programme in Civil and Construction Engineering

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Hollow core slab seam reinforcement

Bachelor’s Thesis 2019

Abstract

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Hollow core slab seam reinforcement, 59 pages, 2 appendix
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Bachelor’s Thesis 2019
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The main objective of the study was to produce excel sheets for determining the correct amount of reinforcement within seams of the hollow core slabs. The secondary aim was to reach the information about the hollow core slabs and to research the process of progressive collapse. In theoretical part of the research, the principal issue was to reach the main idea and to reveal the term of the progressive collapse. The purpose of the sheets was to make designing of hollow-core slab far easier and faster excluding the human factor. In this way, Pöyry Finland Oy commissioned the work.

Data for this study were gathered from European norms and recommendations for designing from RIL publications, examples of calculation were also used. All the sources and guidelines in this thesis are based on Eurocode. The data for this thesis were collected from colleagues and their experiences in the fields of designing hollow-core structures.

The results can be applied to design engineers working with designing the hollow core slabs, which are commonly used in office buildings, apartment houses and parking buildings. The recommendations and studies made as a result of this study have been implemented in a few projects already. The feedback of that experience was taken into account and the related comments have been corrected.

Keywords: progressive collapse, peripheral ties, internal ties, hollow-core slab, accident limit state, ultimate limit state.

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1 Introduction

The client of the thesis is Pöyry Finland Oy. The calculation process and studying the phenomenon of progressive collapse are the most important parts of the thesis, in which the company is mainly interested. In addition, the main problems lie on the calculation. The first is that the company have not developed excel sheets for designing the seam reinforcement of the hollow core slabs and consequently the calculations were made from scratch using necessary materials and similar researches. Moreover, the second problem is that the whole calculation process needs to be carried in accordance with up-to-date norms. Still there are no design norms and standards for designing the seam reinforcement for preventing the progressive collapse in Russia. In addition, eventually the third one is that Pöyry Finland is a fast-growing construction company and because of an endless flow of projects, it is necessary to have a convenient tool on hand.

The idea of preventing progressive collapse nowadays is quite old and there is no need for discussion about the reasons, appearance and effects from this event more widely in the thesis. Nevertheless, practice shows that the hollow-core slabs are commonly used in many of Pöyry projects and designers are forced to handle the calculation by hands using plenty of information from different sources. Therefore the main purposes of this project are:

1) To explore the basics of designing the hollow core slab;

2) To produce excel sheets and quick selection tables for determining the correct amount of reinforcement within seams of the hollow core slabs;

3) To introduce all the steps of calculation with drawings, notes and charts in order to make the tool far more easier in use and to exclude the human factor;

2 The phenomenon of progressive collapse

Progressive collapse is the spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as *disproportionate collapse* [Bruce R. Ellingwood, 2007, p. 1].

The safety of a structure is determined by its ability to withstand loads and other actions or influences of phenomena. Such actions may be present during the whole process from construction to the rest of the structure's lifetime. (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.78)

The sufficient strength and stiffness of structure elements alone does not yet guarantee a safe structure. The components need to be connected to each other forming a coherent entity, which is sufficiently strong, stiff and stable.

Construction practice has learned that the chance of complete failure of a structure is mostly determined by accidental loads, which include: conflagrations, explosions, nature disasters, local overloading, foundation settlements and errors in design and construction. That is why in every design the possibility that during its lifetime the structure can be damaged by accidental loadings should be considered. (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.78)

Nowadays the idea of '*progressive collapse'* is quite old. The term was first used in 1968 in the United Kingdom when the Ronan Point apartment building was damaged by the partial collapse see Figure 1.

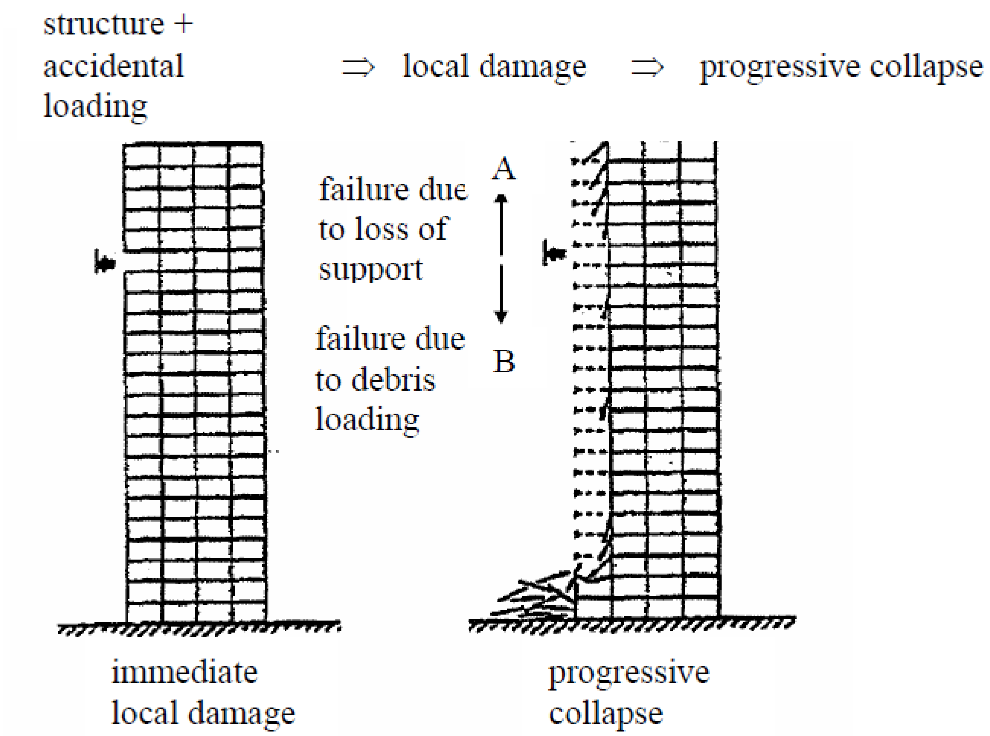


Figure 1. Example of progressive collapse (Ronan Point, England) (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.78)

The structure was a 22-storey precast concrete, bearing wall building. A gas explosion blew out the exterior wall panel and failure of the corner bay of the building propagated upward to the roof and downward almost to ground level. The investigation of the Ronan Point collapse showed other interesting aspects of the design. It was determined that strong winds and/or the effects of a fire could also have caused progressive collapse. In addition, it was found that the building had been built with very poor workmanship. Only half of the specified mortar had been placed in the connections and bolts and nuts were not tightened as specified.

2.1. Prevention of progressive collapse

Limiting the continuous collapse is to ensure that local damage does not spread to the wider area of the destroyed structure.

Frame stability is a crucial issue in multi-storey precast construction. To guarantee this frame stability an adequate stiffness, strength and the provisions to guard against the possibility of a collapse, which is disproportionate to its cause, must be provided. Tie forces between the precast elements must be mobilised in the event of accidental damage or abnormal loading and alternative means of transferring loads to the foundations must be sought if an element is rendered unserviceable. (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.79)

Structural integrity and interaction between elements are obtained by the use of horizontal floor and vertical column and wall ties, Figures 5-12. Peripheral ties (6,7,8,9) enclosing the floor fields and the internal ties (5) are essential for the diaphragm action of the floor. (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.80)

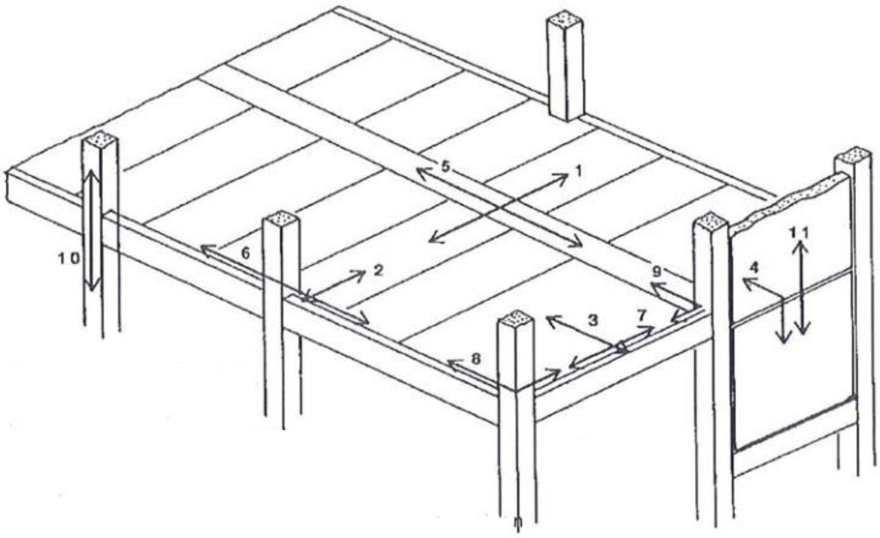


Figure 2. Example of fully tied precast concrete structure

The vertical ties (10,11) are connecting the separate wall and column elements together providing vertical continuity. The ties (1,2,3,4) are connecting the floor elements to their supports (beams or walls) to prevent loss of those supports in case of accident.

It is essential that the ties and connections are ductile (possess large plastic deformation capacity) to be able to dissipate the energy when damage occurs.

Tying the large panel structure together horizontally and vertically utilises the following structural mechanisms to bridge local failures, which are illustrated in Figure 3:

(a) vertical suspension of the wall panels
(b) cantilever action of the wall panels
(c) beam and arch action of the wall panel
(d) partial catenary and membrane action of successive spans of the floor planks
(e) diaphragm action of the floor planks

Additional design measures for multi-storey building frames:

Establishing of the internal connections in the horizontal floor in two mutually perpendicular directions provides the bearing capacity of the horizontal disks under tension and shear force as well as works on the entire length (Figure 2). (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.80)

It is necessary to establish peripheral ties at a distance ≤1.2 m by the edge in every single slab. These connections should be ensured the bearing capacity of the horizontal disks under tension and shear force. (SP 385.1325800.2018. Prevention of progressive collapse in buildings and structures. Design rules, p.16)

Horizontal ties are applied on the external columns or walls within the horizontal floor. These connections should ensure the perception of tensile forces of at least 20 kN per 1 linear meter of the facade of the building.

It is necessary to establish vertical ties that connect the columns of the frame building or structure along its entire height. These connections should be calculated on a tensile force equal to the value of the axial longitudinal force, which acts in the column of any of the floors with the main combinations of loads. Joint connections are not allowed to establish in the support unit and in the middle of the column height. The recommended value is 1/3 – 1/4 of the floor heights.

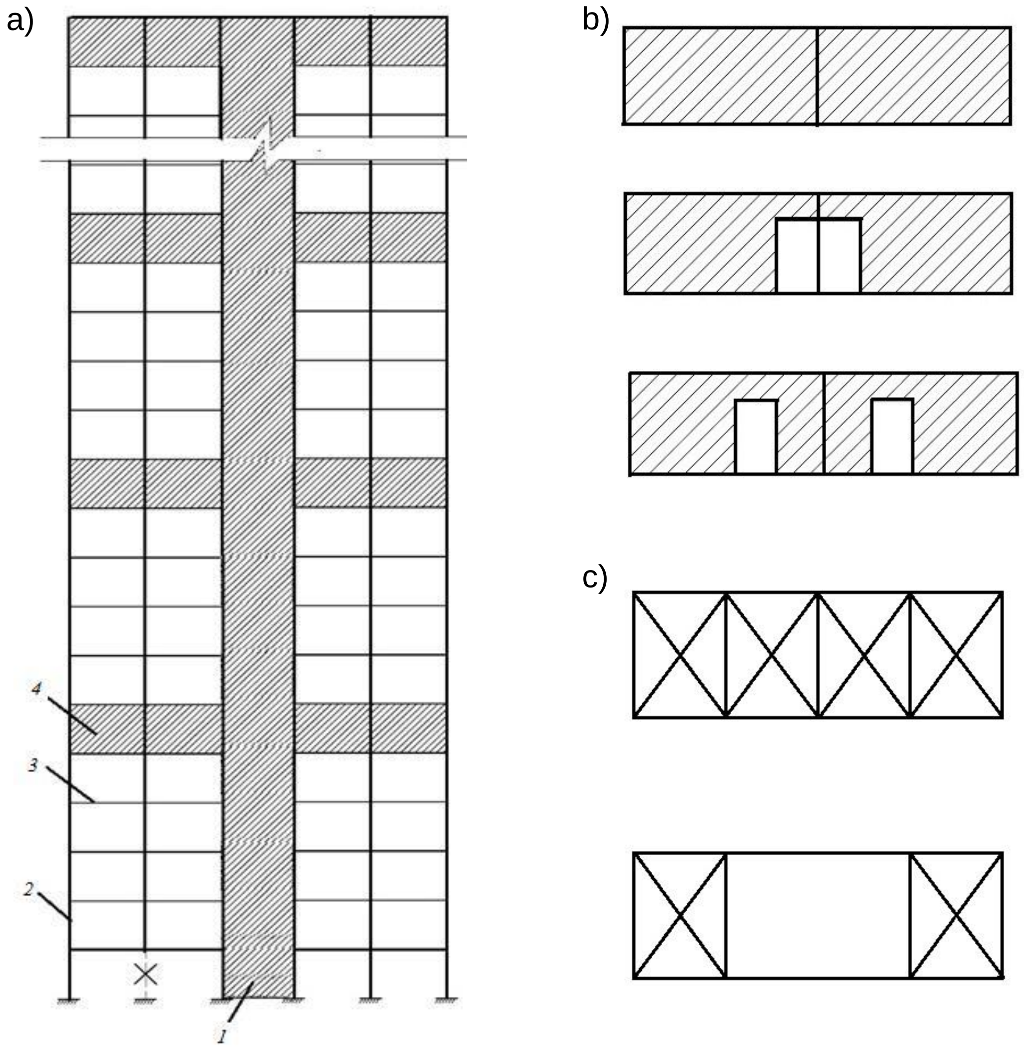
In order to ensure the joining of beams with floor slabs with calculated connections (for example, for a composite concrete slab), steel beams should be combined with monolithic floor slabs using stud bolts or special supports.

It is important to provide rigid connection between beams and columns of at least one direction.

Outrigger structures are placed into the carrier system of a multi-storey building (Figure 3 - a) in the form of systems of continuous or cross-sectional trusses

(Figure 3 - b, c), designed for the perception of efforts determined in accordance with the results calculations according to primary and secondary settlement schemes.

(SP 385.1325800.2018. Prevention of progressive collapse in buildings and structures. Design rules, p.17)



1 – stiffener core; 2 – columns; 3 – cross bar; 4 – outrigger structure

Figure 3. The layout of the outrigger structures (a) and the types of these structures which are made with solid (b) or through (c) section (SP 385.1325800.2018. Prevention of progressive collapse in buildings and structures. Design rules, p.18)

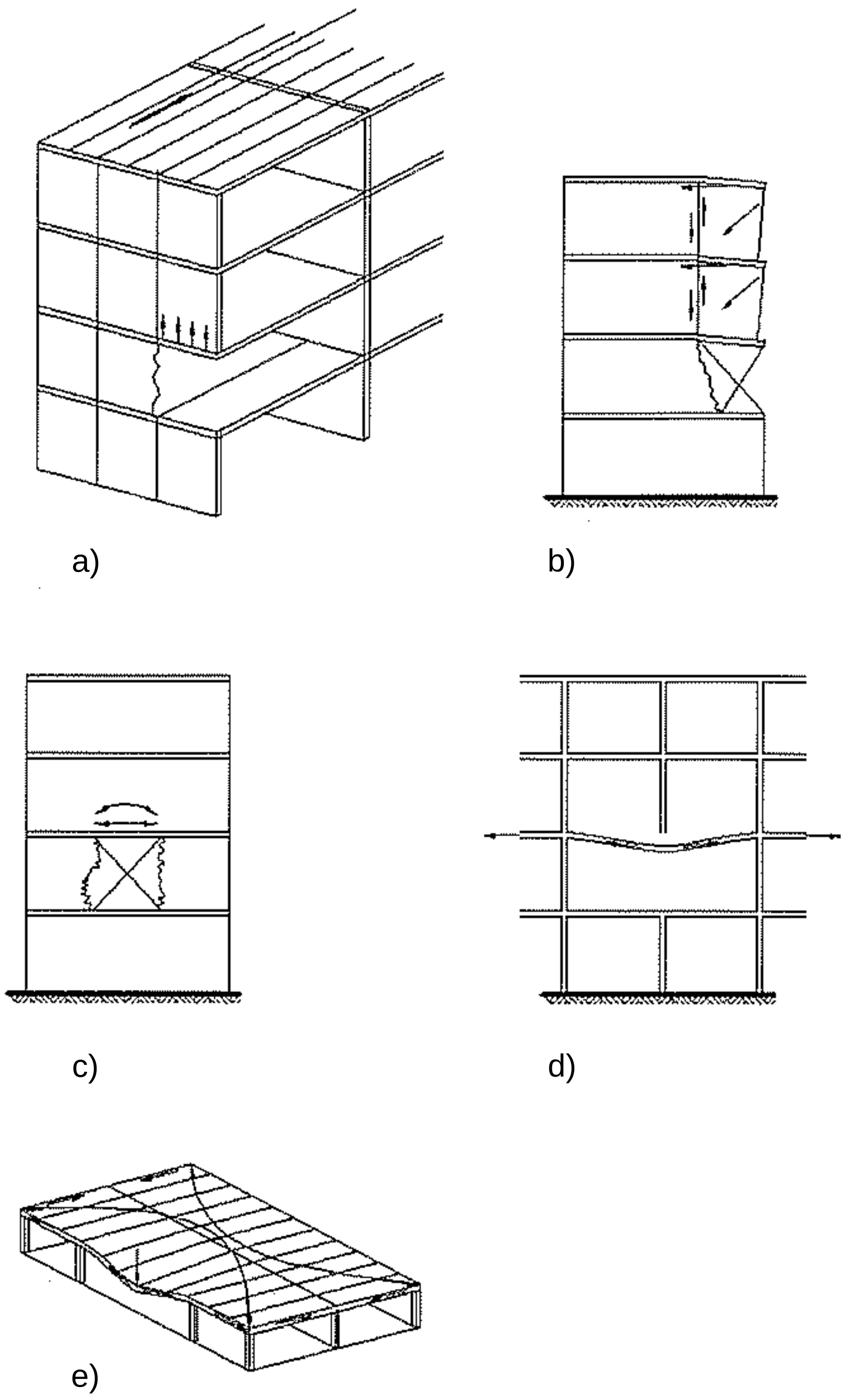


Figure 4. Some examples of alternative load paths

Ties in floors are primarily designed with regard to diaphragm action according to the needs identified in the structural analysis of the stabilising system. For instance, the peripheral tie in the floor has to act as tensile reinforcement in the floor diaphragm to resist the in-plane moments caused by wind and horizontal forces due to possible leaning of the building, columns out of plumb, second order effects etc. (Structural connections for Precast Concrete Buildings: Guide to Good Practise, p.81)

2.2. Control by norms

The SFS-EN 1991-1-7 requirement to reduce risk of a prior accident from an accident can be carried out in two ways:

1. Prevention of local damage
   (a) The elimination or reduction of the risk of collapse by structural measures;
   (b) Dimensioning as a key component of SFS-EN 1991-1-7 + the national accidents
2. Limiting the local damage
   (a) The substitution of the structural system
   (b) Dimensioning the joints for the forces

2.3. Prevention of local damage

Continuous collapse from the explosion can be prevented by designing the structures so that overpressure can be discharged without breaking the load-bearing structures. For example, in the event of gas explosion, the attachment of lightweight facade elements can be made so that the pressure can be discharged through them. The strength of the supporting elements should be greater than the lightweight facade elements. (Betoninormikortti 23 2012, 4.)

Local damage can be prevented either by removing the collapse risk by structural measures or by dimensioning the structures for the accident loads. Structural measures to prevent the fall of the species can include, for example, designing structures so that any overpressure can break out without breaking structures, or protecting structures against collisions, for example. (Betonininormikortti 23 2012, 4.)

The structure is designed so that loads from the damaged structural part can be carried by a replacement (alternative) structural system of the remaining constituents. The structural elements are joined to each other by joints with sufficient durability and toughness, so that a replacement structural system can be created and the loads of damaged parts can be transferred to an undamaged building. (Betonininormikortti 23 2012, 4.)

2.3.1 Collision

Collision-susceptible structures are dimensioned for impact forces in accordance with Chapter 4 of the SFS-EN 1991-1-7, which is considered to have been prevented because of a continuous collapse in accordance with paragraph 1b. When the structure is dimensioned for a collision load, collision damage is not considered as a collision

Places exposed to collision include, for example, the building pillars, the parking spaces, and the premises with service vehicle traffic, forklifts or similar non-road vehicles along the transport routes.

The horizontal load 25 kN can be used as a collision load in the yard areas for vehicles with a maximum weight of 3,5 t. for persons and service vehicles when balancing parquet floors. (Betonininormikortti 23 2012, 13.)

2.3.2 Conflagration

Fire-exposed structures are dimensioned according to the fire protection guidelines for concrete structures SFS-EN 1991-1-2. When the structure is dimensioned for a particular fire retardant, this structure is assumed to carry loads occurring during the fire during this time. As a result of deformation of the exposed structure of the fire, other structures may create deflections that can endanger their carrying capacity. The joints between the elements must be designed so that the building can withstand these deformations without falling.

In the event of fire situation, the additional inclination of the bearing structure in the horizontal structure of the horizontal structure shall be taken into account in addition to the obstructions caused by the installation tolerances and the rotation of the foundations, in addition to the SFS-EN-1991-1-2 clause 5.2. The horizontal forces caused by the inclination are calculated from the vertical loads affecting the fire situation.

The risk of continuous collapse caused by a normal fire situation and accidental loads is considered. The collapse risk caused by the exceptionally severe fire caused by its consequences is also a separate load case in relation to other accident situations.

3 Hollow core slab

The hollow slab is the most common element type used in concrete framed buildings. They are used in the lower, intermediate and top floors of residential, commercial and industrial buildings. The product standard for hollow slabs is SFS-EN 1168. (Elementtisuunnittelu a.)

Hollow core slabs are prestressed tile elements, which are lightened by hollow cavities in the longitudinal plane of the slab. Concrete grade C40-C70 is used for the manufacture. The tiles are cast on rolling casting over long steel castings. The pulp used in casting is so rigid that the molded and compacted plate of the casting machine maintains its shape without any molds. (Elementtisuunnittelu a.)

The height, number and shape of the cavities vary depending on the height of the hollow slab. The thickness of the hollow slab is 150, 200, 265, 320, 370 400 and 500 mm. (Figure 5). Hollow slabs have a standard width of 1200 mm, Using hollow slabs, it is possible to reach up to the span of 20 m.

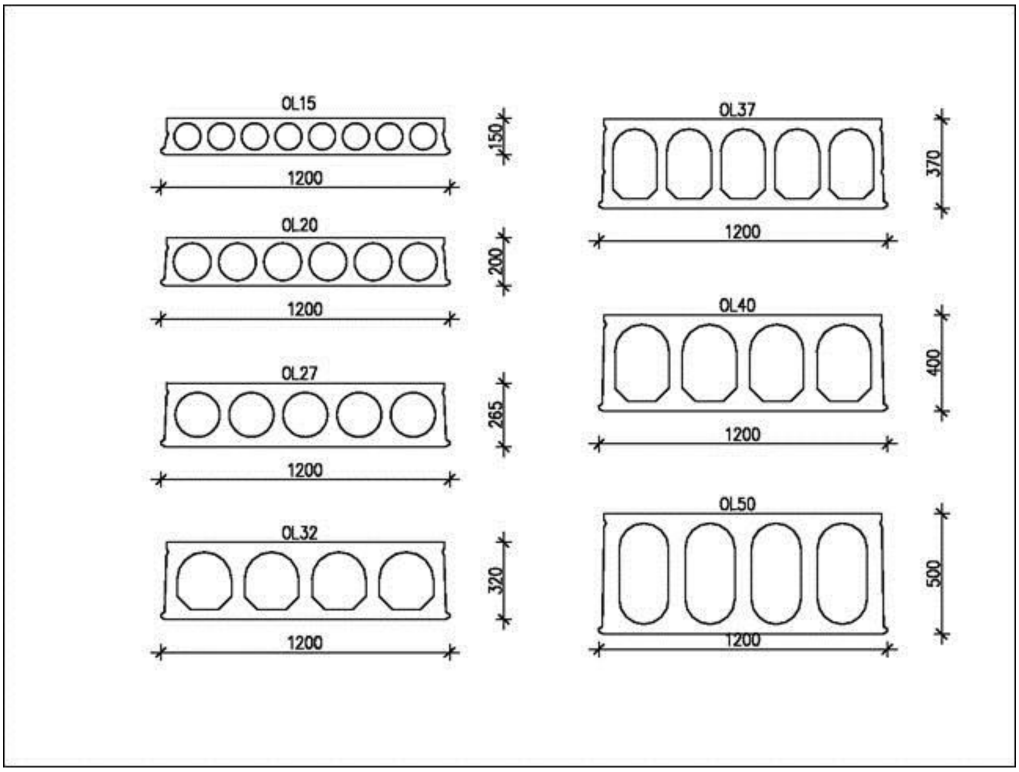


Figure 5. Dimensions of hollow core slab according to norms

(Elementtisuunnittelu a.)

3.1 The design process of hollow-core slab

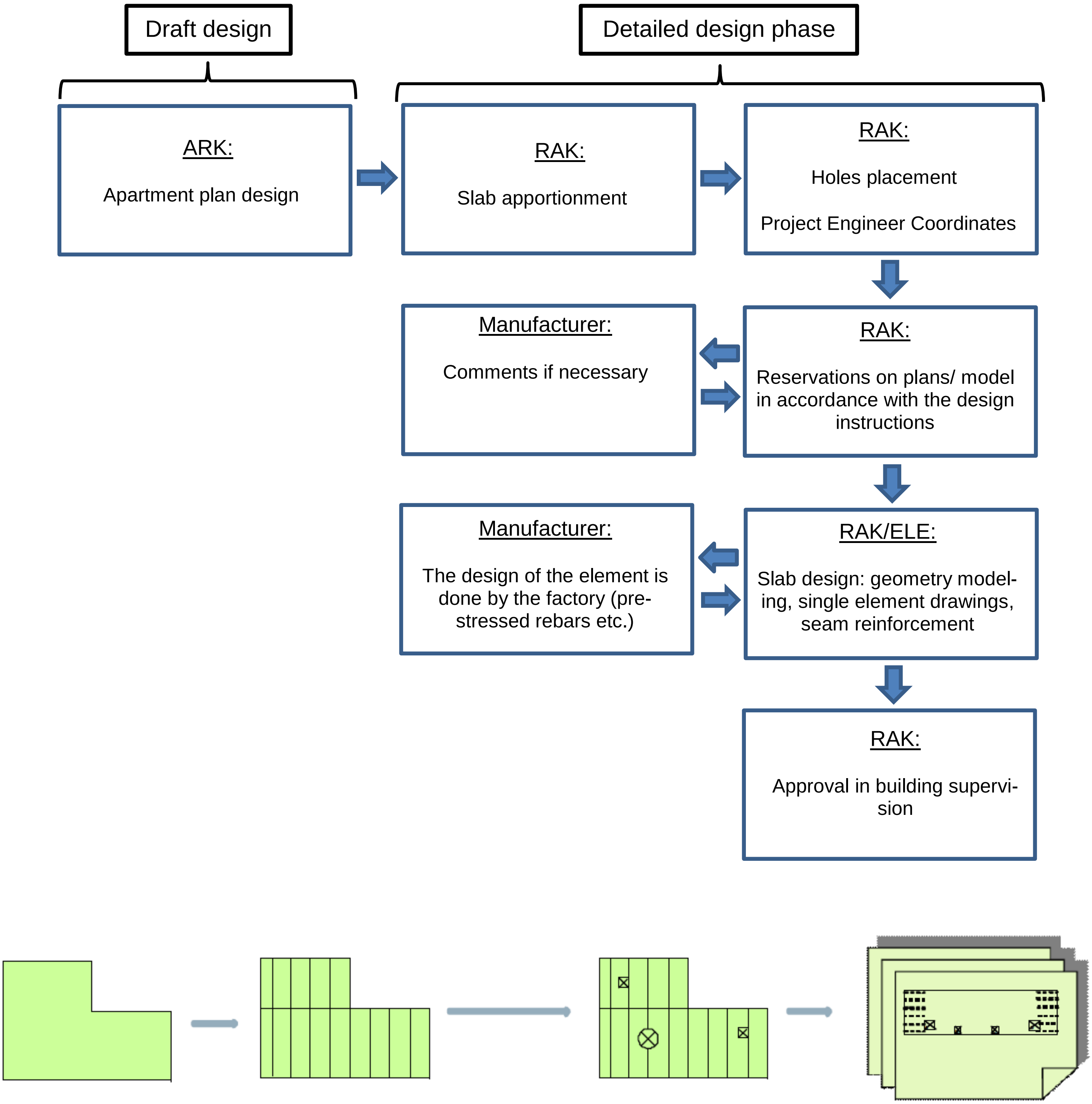


Figure 6. Design process of hollow core slab

(BT Ontelolaatastojen yhteiset suunnitteluohjeet PDF, p. 3)

1. Architectural Design

• The architect must take into account the bearing directions of the tiles as well as the placement and orientation of the flues. It should also be noted that the distance between the toilet seat and the element hinge is not too high.

2. Authoring

• When designing the tiles, consider flanges and changes in the wall lines, as well as other issues affecting the tiles. If necessary, use a narrowed slab on the edge of the drawer to place the hollow hole in the box, for example, in accordance with the hole punching instructions.

3. Hole placement

• For building technology designers, the holes in the hollow core holes must be observed in the holes placement.

4. Structural design

• It is the job of the structural designer to make sure that the holes are in accordance with the design guidelines and are feasible. If necessary, he can ask the tile manufacturers to comment on the draft of the hole.

5. Structural design / Element Design

• Types of tiles by project. All similar elements with the same symbol per project. The hollow-core manufacturer measures the elements and returns the completed plans for approval.

6. Structural designer

• The structural designer checks the holes, approves the plans for building control and delivers the necessary plans to the site.

(BT Ontelolaatastojen yhteiset suunnitteluohjeet PDF, p. 3)

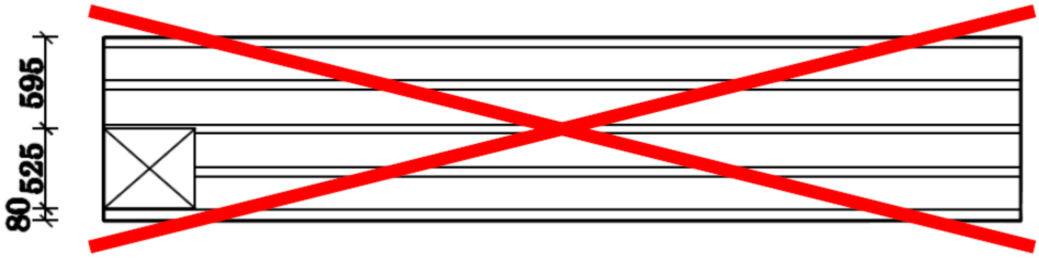


Figure 7. Example of slab with opening on the edge

The tiling of the slab as shown above is forbidden.

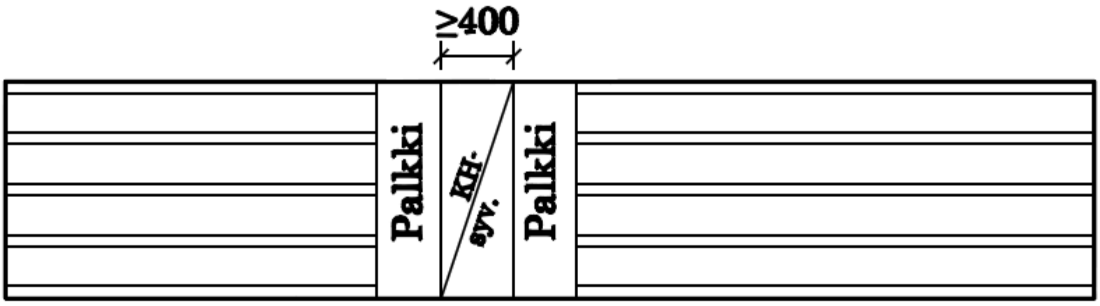


Figure 8. Restrictions of the size (BT Ontelolaatastojen yhteiset suunnitteluohjeet PDF, p.6)

It is strictly forbidden to make cross-section openings in the hollow core slabs. This causes a complete loss of the compression zone of the hollow core slab. The cross-section openings of the slab may only be made assuming a hole in the recess and thus complying with the limit values according to this hole punching instruction. The minimum length of the recess is 400 mm and the depth is the same as the depth of the slab.

The other common restrictions may be found from the source: (BT Ontelolaatastojen yhteiset suunnitteluohjeet PDF)

3.1.1 The initial data for design

The drawings supplied to the draw designer must include at least the following:

• Tile support length

• Identification of tiles

• Holes, reservations and their locations

• Eurocode load class

• Consequences class according to Eurocode

• All loading information for slabs (Hinges, bathroom elements, partitions, etc.) Permanent and payloads in accordance with the Eurocode.

• Slab Fire Resistance Requirement

• Label load class

• Design lifetime

If hollow slabs are supported by flexible support, the braid designer must have:

• The level drawing is marked with the type of beam as well as the beams as far as possible

• Beam dimensioning information as far as possible. (Steel beams also include plate thicknesses)

• Structural types from areas where hollow slabs are supported by flexible support.

• If the reinforcement grid has been designed beforehand, the size of the beam strip must be stated.

(BT Ontelolaatastojen yhteiset suunnitteluohjeet PDF, p. 4)

<u>Data Required for Slab Design:</u>

• Design drawings (.dwg)

• ARK drawings (.dwg)

• Cross-sections and Details (.pdf)

• Structural Types (.pdf)

• Hole Drawings (.dwg)

• Load class (A, B, C…)

• Consequences class (CC3… CC1)

• Element report

• Contact information

(Parma ontelolaatastot suunnitteluohje 2018-1, p.47)

Manufacturer Information Required:

A. Name, location, exact address, block and plot number.

B. Details of the Hollow-Drawer Plan Drawing, showing:

• Tile dimensions, seam sizes and support lengths

• tile symbols

• locations for holes and charges

• load class (A, B, C…)

• Consequences class (CC3… CC1)

• all load data (including partitions, concrete cover, grooves, etc.) required for structural design of the tiles

• the fire resistance requirements of the slab

• The stress class of the slab

• Design life

• Information about the structure to which the tile is to be supported

(Parma ontelolaatastot suunnitteluohje 2018-1, p.47)

The designer of the tile obtains the above-mentioned information from the corresponding structural designer and/or supplier. Based on this information, the slab manufacturer is able to calculate the stiffness of the combined cross-section and other required values to determine the stresses resulting from the interaction with the plate. The slab manufacturer also determines the need for cavity sealing (deep spotlights). (Parma ontelolaatastot suunnitteluohje 2018-1, p.47)

4 Horizontal load distribution

Wind actions fluctuate with time and act directly as pressures on the external surfaces of enclosed structures act indirectly on the internal surfaces because of porosity of the external surface. They may also act directly on the internal surface of open structures. Pressures act on areas of the surface resulting in forces normal to the surface of the structure or of individual cladding components. Additionally, when large areas of structures are swept by the wind, friction forces acting tangentially to the surface may be significant. (EN 1991-1-4)

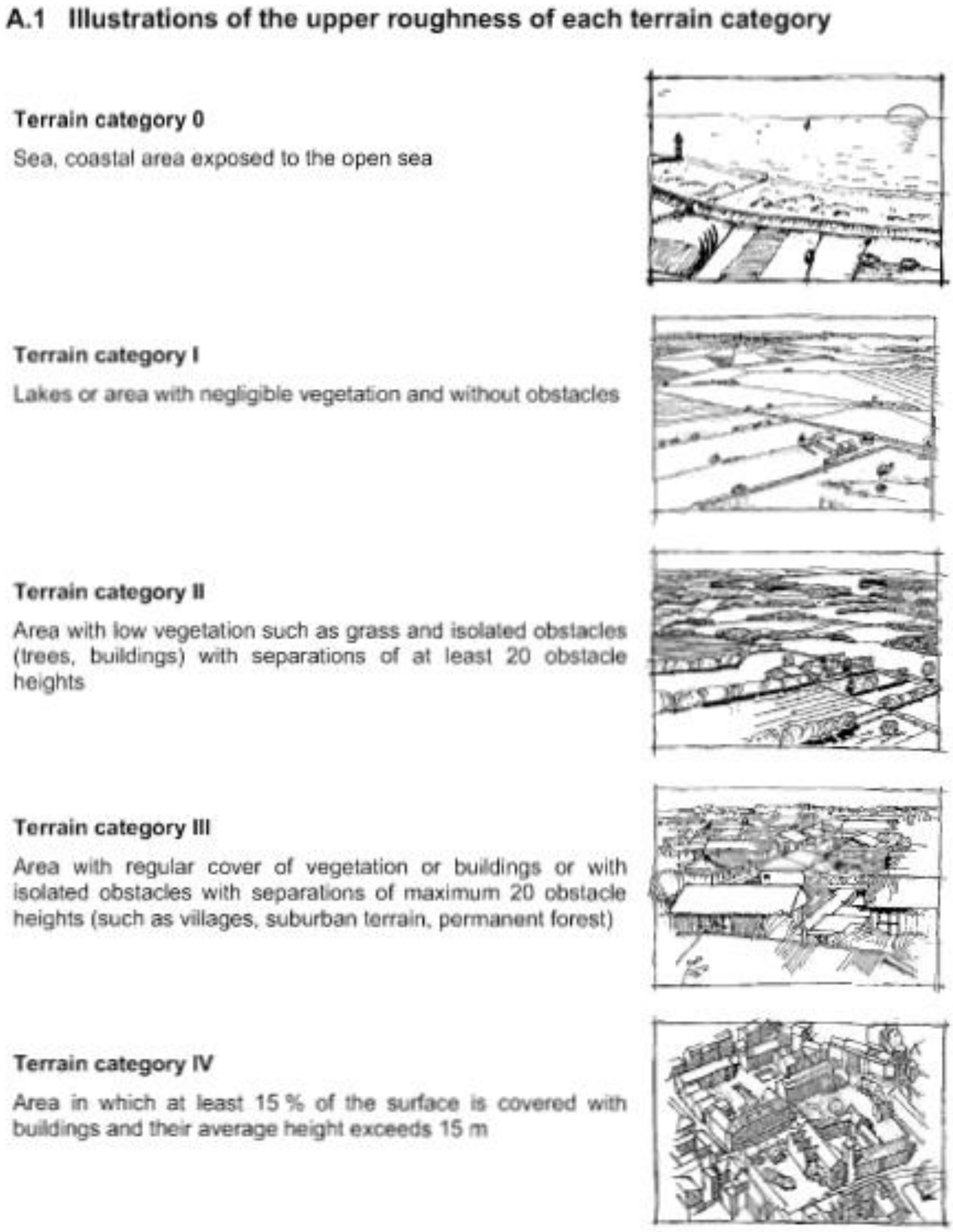
4.1 Wind loads

The source data for determining the wind load are the terrain class of the location of the building and the effect of the surface shape. Both affect the wind pressure of the wind, from which final wind power is calculated using either the power factor or the surface pressures and the pressure coefficients. In this thesis, wind loads are calculated using force factors. (RIL 201-1-2008, 124, 135)

The effect of the wind on the structure (i.e. the response of the structure), depends on the size, shape and dynamic properties of the structure. This part covers dynamic response due to along-wind turbulence in resonance with the along-wind vibrations of a fundamental flexural mode shape with constant sign. (EN 1991-1-4)

The response of structures should be calculated according to Section 5 from the peak velocity pressure, q_p at the reference height in the undisturbed wind field, the force and pressure coefficients and the structural factor GsGd (see Section 6). q_p depends on the wind climate, the terrain roughness and orography, and the reference height. q_p is equal to the mean velocity pressure plus a contribution from short-term pressure fluctuations. (EN 1991-1-4)

In Eurocode, the environmental conditions of a building are divided into five different categories 0, I, II, III, and IV. Descriptions of terrain classes are shown in Figure 9.


A.1 Illustrations of the upper roughness of each terrain category

Terrain category 0
Sea, coastal area exposed to the open sea

Terrain category I
Lakes or area with negligible vegetation and without obstacles

Terrain category II
Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights

Terrain category 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)

Terrain category IV
Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m

Figure 9. Terrain categories (EN 1991-1-4)

In coastal area where the terrain class changes directly from class, 0 to class IV, there may be a need for a more detailed examination. In these cases, the effective terrain class of the building can be determined with the help of Figure 10 and Figure 11, when the height *z* of the building and the distance from the coast are known.

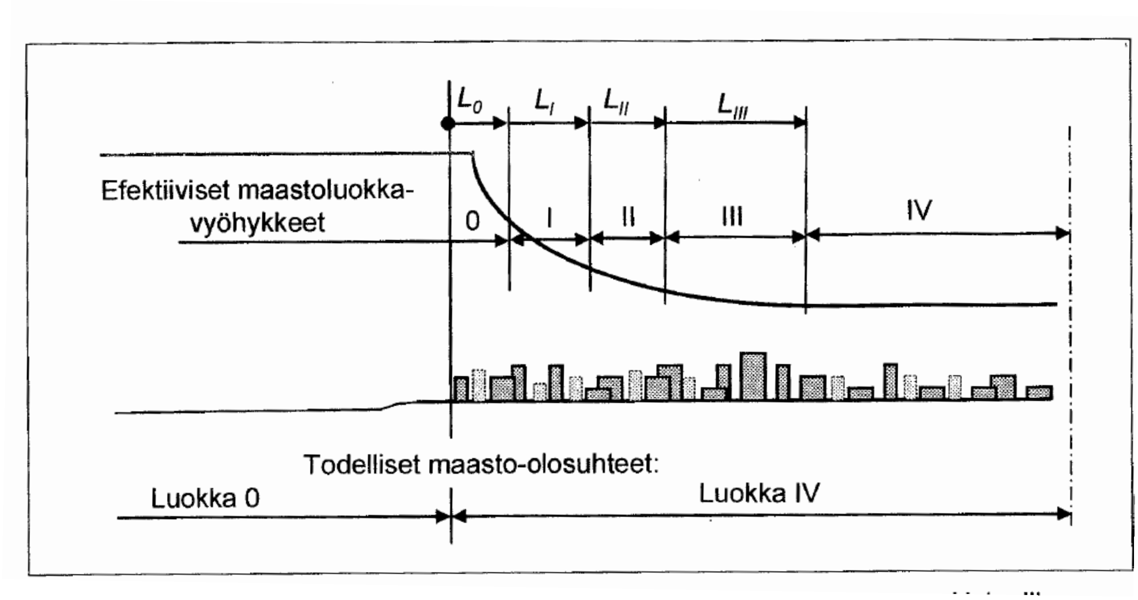


Figure 10. Effective zones in coastal cities (RIL 201-1-2008, 128)

	Efektiivisten maastoluokkien rajat			
z (m)	L_0 (km)	L_I (km)	L_{II} (km)	L_{III} (km)
60	2,0	4		
40	1,0	2,0	10	
≤ 20	0,5	1,0	2	10

Figure 11. Effective zone boundaries according to building height (z) in coastal cities (RIL 201-1-2008, 128)

Based on the height *z* measured from the ground, the specific value q_{p0} (z) of the wind speed, pressure can be calculated for each terrain class in the following expressions:

$$q_{p0}(z) = \begin{Bmatrix} 0{,}00893 \cdot \left[ln\left(\frac{\max(1,z)}{0{,}003}\right)\right]^2 + 0{,}0625 \cdot ln\left(\frac{\max(1,z)}{0{,}003}\right), maastoluokka\ 0 \\ 0{,}00794 \cdot \left[ln\left(\frac{\max(1,z)}{0{,}01}\right)\right]^2 + 0{,}0556 \cdot ln\left(\frac{\max(1,z)}{0{,}01}\right), maastoluokka\ I \\ 0{,}00995 \cdot \left[ln\left(\frac{\max(2,z)}{0{,}05}\right)\right]^2 + 0{,}0697 \cdot ln\left(\frac{\max(2,z)}{0{,}05}\right), maastoluokka\ II \\ 0{,}01279 \cdot \left[ln\left(\frac{\max(5,z)}{0{,}3}\right)\right]^2 + 0{,}0895 \cdot ln\left(\frac{\max(5,z)}{0{,}3}\right), maastoluokka\ III \\ 0{,}01513 \cdot \left[ln\left(\frac{\max(10,z)}{1{,}0}\right)\right]^2 + 0{,}1059 \cdot ln\left(\frac{\max(10,z)}{1{,}0}\right), maastoluokka\ IV \end{Bmatrix} \quad (1)$$

If the building is located on a single hill or ridge above the slope value of 0.05, then the wind velocity pressure adds an additional factor γD due to the terrain shape, which is not considered by the terrain. The modified velocity pressure taking into account the effect of surface deformation is calculated in the formula:

$$q_p(z) = \gamma_D \cdot q_{p0}(z) \qquad (2)$$ (RIL 201-1-2008, 130.)

The magnitude of the coefficient is influenced by the shape of the terrain, the slope and the location of the structure relative to the surface shape. Terrain erosion is considered as either one-sided or two-sided terrain, as shown in Figure 12. (EN 1991-1-4)

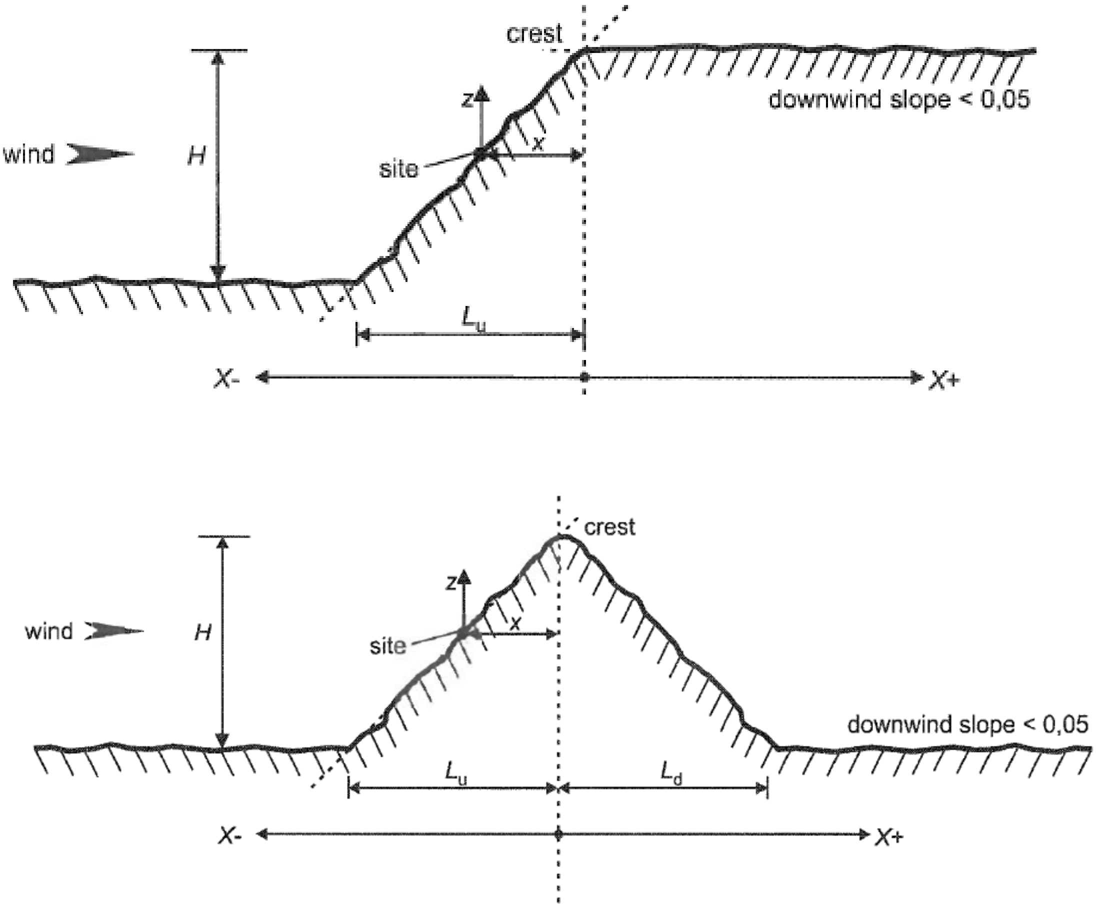


Figure 12. One-sided and Two-sided Terrain with Parameters

Determination of the increasing coefficient			
One-sided terrain		Two-sided terrain	
$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 + x/Lu)$	when x<0	$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 + x/Lu)$	when x<0
$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 - 0{,}33 \cdot x/Lu)$	when x≥0	$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 - 0{,}47 \cdot x/Ld)$	when x≥0

Figure 13. Determination of the increasing coefficient (RIL 201-1-2008, 130.)

where

Φ – upwind/downwind slope

x – horizontal distance of the site from the top of the crest

L_u – actual length of the upwind slope in the wind direction

L_d – actual length of the downwind slope in the wind direction

Φ refers to the slope, i.e. the ratio of the height and length of the terrain (H/Lu). If the slope exceeds the value of 0.3, the previous formulas are used as a value of 0.3. (RIL 201-1-2008, 130)

The total wind power *Fw* on the building can be calculated using the following formula:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z) \cdot A_{ref} \quad (3)$$

where

$c_s c_d$ - structural factor

c_f - force coefficient

$q_p(z)$ - peak velocity pressure

A_{ref} - the impact area of the wind load b·h, where b is the width of the building, i.e. the direction perpendicular to the wind flow and the height of the building

For low-rise buildings, i.e. the width is greater than the height, a value of 1.0 on the safe side can be used as a structural coefficient. When the height of the building is greater than the width, the structural coefficient can be determined by the height and the width of the building under consideration in Figure 14. The solution to the high side of high-rise buildings is also to choose the value of the structural factor directly at 1.0. (RIL 201-1-2008, 136, 138)

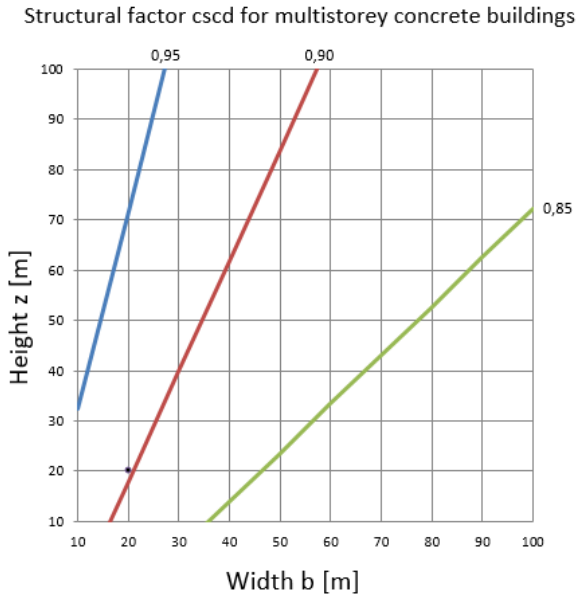


Figure 14. Determination of the structural factor cscd (RIL 201-1-2008, 138.)

To determine the power factor, you first need to calculate the effective slope of the building - λ, obtained from the following formulas:

$\lambda = 2 \cdot \frac{h}{b}$ kun h < 15 m (4)

$\lambda = 1{,}4 \cdot \frac{h}{b}$ kun h ≥ 50 m (5)

When the height of the building is between 15 and 50 meters, the value of the effective slope is interpolated between the two previous formulas. The final force coefficient is obtained from Table 1. When the effective slope and the building *d/b* are known, where *d* is the wind direction of the building. (RIL 201-1-2008, 136)

	Sivusuhde *d/b*								
λ	0,1	0,2	0,5	0,7	1	2	5	10	50
≤ 1	1,2	1,2	1,37	1,44	1,28	0,99	0,60	0,54	0,54
3	1,29	1,29	1,48	1,55	1,38	1,07	0,65	0,58	0,58
10	1,40	1,40	1,60	1,68	1,49	1,15	0,70	0,63	0,63

Table 1. Force coefficent values based on effective slope and dimentions ratio (RIL-201-1-2008, 137)

By replacing Formula 7 with the wind load impact area, the floor height H_{floor} which is obtained by applying a single layer intermediate floor load $q_{k,wind}$, a wind that can be used to dimension the cavity wall joints.

4.2 The additional moment arm caused by inclination

The additional moment arm caused by the inclination of the structures is obtained by first calculating the value of the inclination θi from the formula 6:

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \qquad (6)$$

where

θ_0 – basic value (the recommended value is 1/200)

α_h – the reduction factor for height is calculated from the formula 7:

$$\alpha_h = \frac{2}{\sqrt{L}}; \frac{2}{3} \leq \alpha_h \leq 1 \qquad (7)$$

α_m – the reduction factor considering the number of structure members is calculated from the formula 8:

$$\alpha_m = \sqrt{0{,}5\left(1 + \frac{1}{m}\right)} \qquad (8)$$

L – the total height of the building
m – the number of vertical members contributing to the total effect when considering the effect of the inclination on the stiffening system. (SFS-EN 1992-1-1, 55)

Effect H_i on floor diaphragm is calculated by formula:

$$H_i = \theta_i \frac{(N_b + N_a)}{2} \qquad (9)$$

where *Na* and *Nb* are longitudinal forces contributing to *Hi*

(SFS-EN 1992-1-1, 56.)

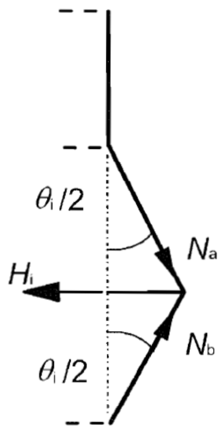


Figure 15. The effect of geometric imperfections (SFS-EN 1992-1-1, 56)

On practice, one layer of permanent and variable loads can be calculated on the intermediate floor, and multiplying their sum by the inclination value obtained from Formula 6 what gives an additional horizontal force acting on one intermediate floor as both permanent and variable loads. When these loads are still divided by the width of the building perpendicular to the load direction under consideration, the amount of intermediate load per meter, i.e. $g_{k,add}$ and $q_{k,add}$ is obtained (Asuinkerrostalon esimerkkilaskelmat, 8, 9, 20).

4.3 Total unit load

Once the wind load and both the permanent and the variable additional gravitational force have been determined, the total horizontal load on the unit can be determined by a load combination in which the wind acts as a dominant variable load and additional horizontal force causes both permanent load and secondary variable load. The total horizontal load p_d is calculated from formula 10:

$$p_d = 1{,}15 \cdot K_{Fi} \cdot g_{k,add.} + 1{,}5 \cdot K_{Fi} \cdot q_{k,wind} + 1{,}5 \cdot K_{Fi} \cdot \Psi_0 \cdot q_{k,add.} \quad (10)$$

where

K_{Fi} – coefficient determined by the penalty category

Ψ_0 – combination factor

(RIL 201-1-2008, 38.)

For the purpose of reliability differentiation, consequences classes (CC) may be established by considering the consequences of failure or malfunction of the structure as given in Table 2. (SFS-EN 1990.2002, 87)

Consequences Class	Description	Examples of buildings and civil engineering works
CC3	**High** consequence for loss of human life, *or* economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	**Medium** consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	**Low** consequence for loss of human life, *and* economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Table 2. Definition of consequences classes (SFS-EN 1990.2002, 87)

The criterion for classification of consequences is the importance, in terms of consequences of failure, of the structure or structural member concerned. (SFS-EN 1990.2002, 87)

Depending on the structural form and decisions made during design, particular members of the structure may be designated in the same, higher or lower consequence class than for the entire structure. (SFS-EN 1990.2002, 87)

One way of achieving reliability differentiation is by distinguishing classes of K_{FI} factors to be used in fundamental combinations for persistent design situations. For example, for the same design supervision and execution inspection levels, a multiplication factor K_{FI}, see Table 3, may be applied to the partial factors.

K_{FI} factor for actions	Reliability class		
	RC1	RC2	RC3
K_{FI}	0,9	1,0	1,1

Table 3. K_{FI} factors for actions (SFS-EN 1990.2002, 87)

There can be cases (e.g. lighting poles, masts, etc.) where, for reasons of economy, the structure might be in RC 1, but be subjected to higher corresponding design supervision and inspection levels. (SFS-EN 1990.2002, 88)

Changing the load combination factor Ψ_0 is determined by either changing the load type or mode, in which the changing load occurs, table 4.

Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

Table 4. Recommended values of Ψ factors for buildings (RIL-201-1-2008, 137)

5 Seam reinforcement

The location of floor ties are shown in Figure 5. The ties will usually have to be detailed specifically for robustness using the design forces given above and can be either reinforcement (e.g. H12) or helical prestressing strand (design strength 1580 N/mm_2). The bars or strand should be adequately lapped and embedded in in-situ concrete which has a minimum dimension of at least f+2H_{agg} +10 mm (i.e.

usually at least 50 mm) where f is the bar diameter and H_{agg} is the maximum aggregate size. 10 mm aggregate is often used to minimize the size of the concrete infill.

Typical horizontal tie details are shown in Figure 16. The following points should be noted:

- The opening up of adjacent cores at the end supports of hollow-core units should be avoided.
- The recommended maximum length of an open core in a hollow-core unit is 600 mm.

Vertical ties for Class 2B buildings and above should be designed to resist the accidental load combination, E_d

(O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 4)

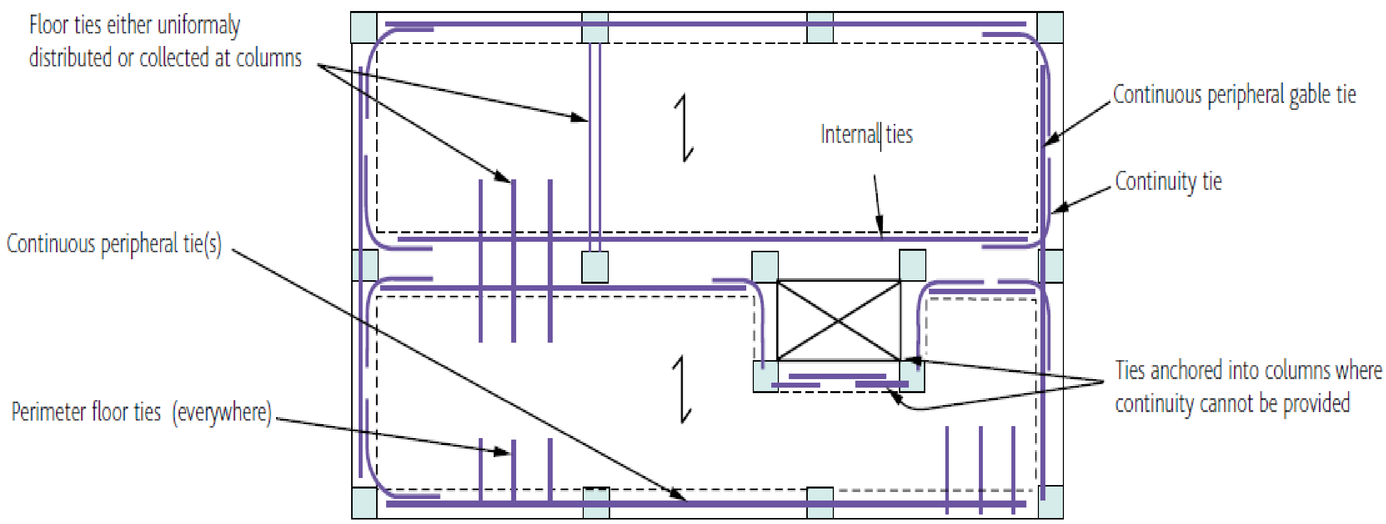


Figure 16. Floor ties for a concrete frame

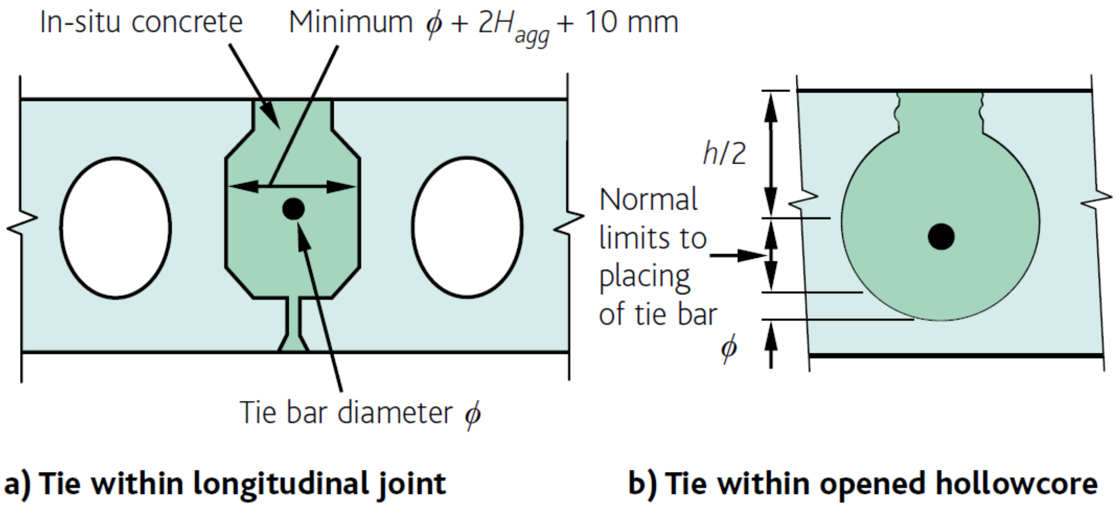


Figure 17. Position of floor tie for hollow-core units

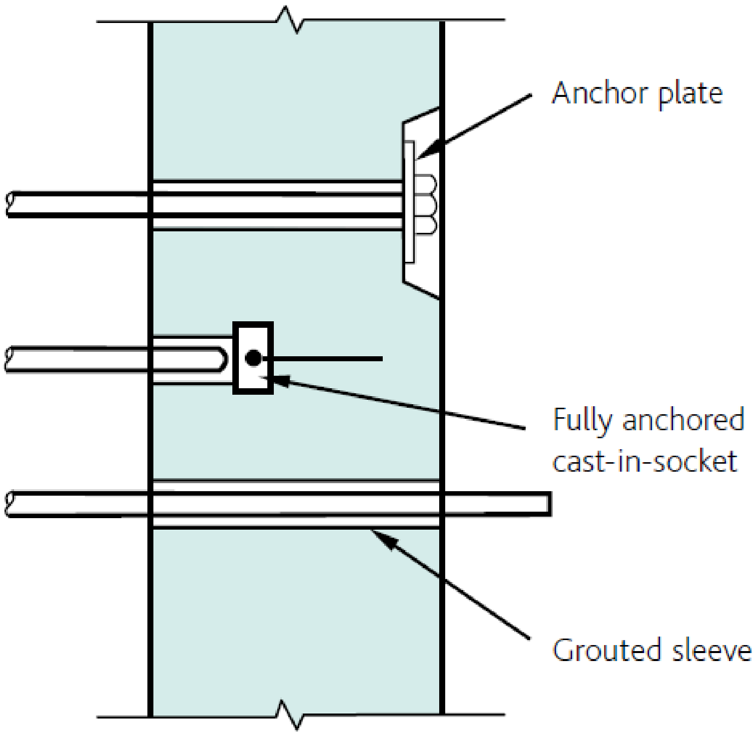


Figure 18. Alternative positions for internal ties at precast column position (O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 6)

5.1 Internal ties

There are two kinds of internal ties: lengthwise and transverse reinforcement of the floor slab as shown in Figure 19. These connections play an important role in the formation of the disk effect, i.e. when the slabs function as a single hanging membrane, no single slab falls in the event of an accident in which the supporting structure has collapsed under the slab. Seam reinforcement should transfer the load on the slab over the damaged structure. (Betoninormikortti 23 2012, 23.)

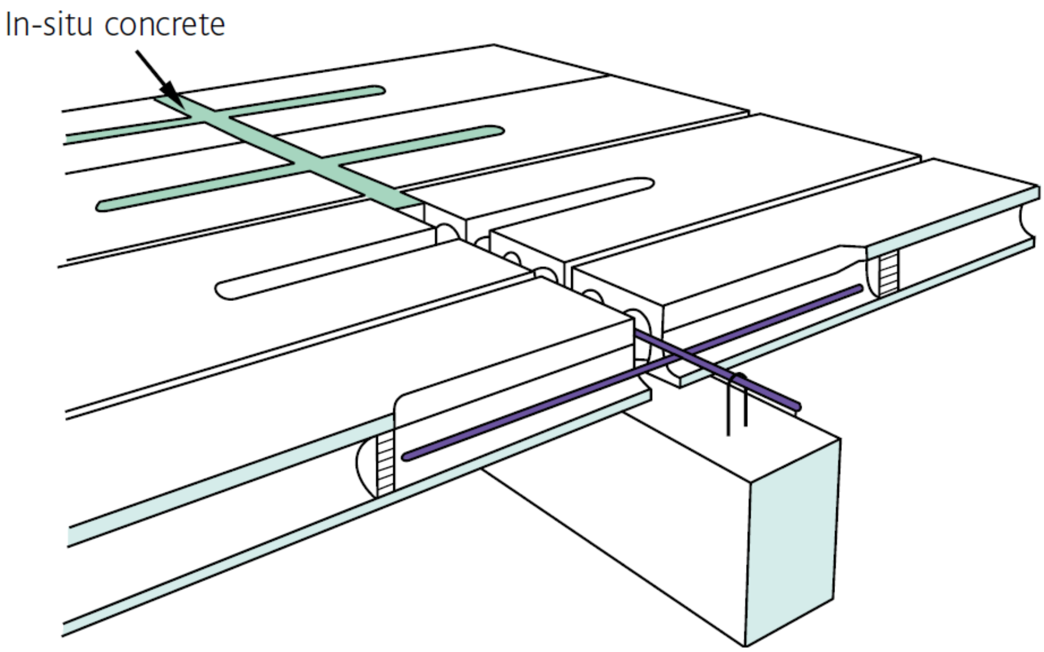


Figure 19. Internal floor ties within hollow-core units (O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 5)

At each floor and roof level, internal ties should be provided in two directions approximately at right angles. The internal ties may be spread evenly in slabs or may be grouped at walls or other positions. If located in walls, the reinforcement should be within 0.5 m of the top or bottom of the floor slabs.

(O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 3)

In the event of an accident, the purpose of the internal ties of the slab is to prevent the slab element from falling in a situation where the support at the other end of the slab has lost either it is full or partial bearing capacity because of the damage to the vertical structure. The purpose of the joint reinforcement is to provide a disk structure carrying the slab over the damage area. At the same time, the seam reinforcement also prevents the slab element from falling from the heat and moisture movements. The formation of diaphragm force is shown in Figure 20 and 21 (Betoninormikortti 23 2012, 25)

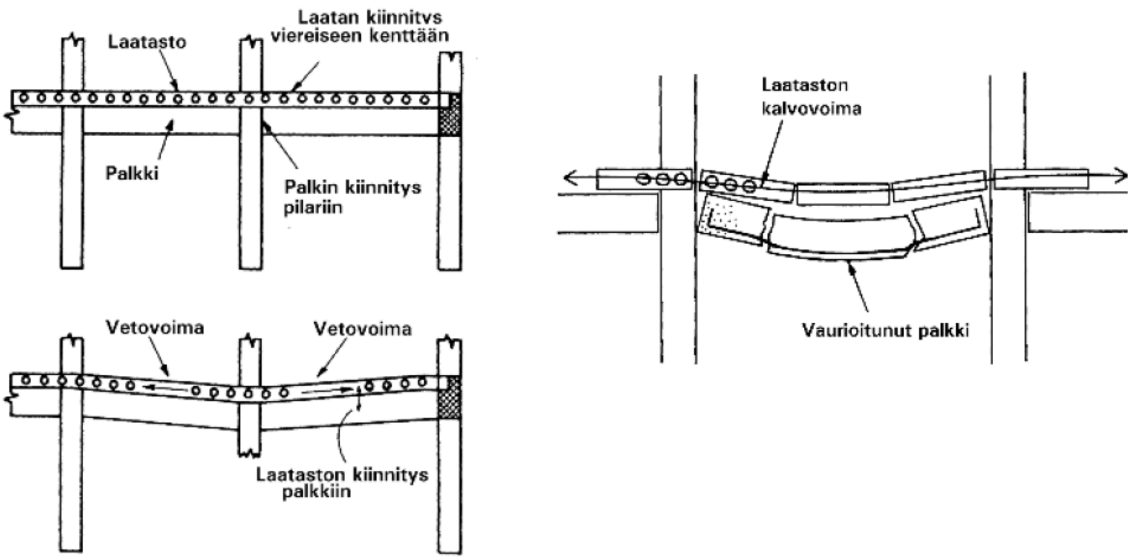


Figure 20. When the pillar or beam is damaged, the load on the slab is transferred over the damaged area by the tensile strength and seam materials of the tile / Elliot 1996 /

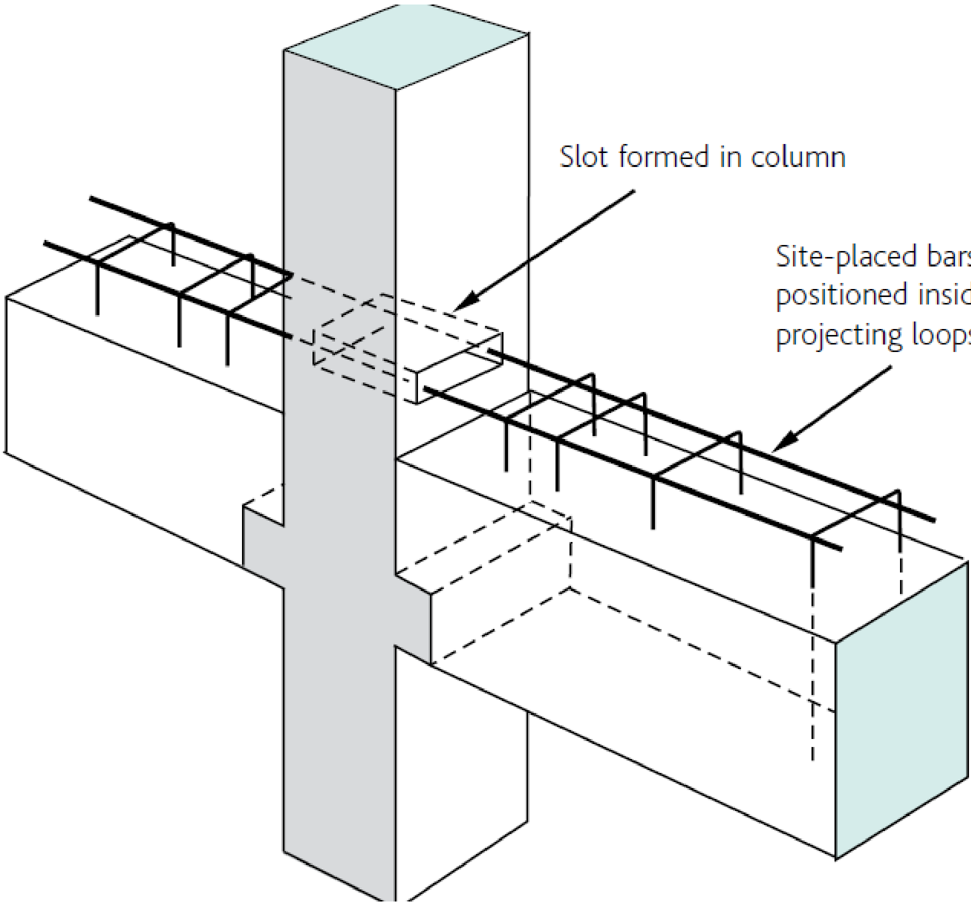


Figure 21. Internal ties taken through precast column

(O. Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 6)

The internal seam reinforcement is placed at the slab bearing structure, i.e. the wall or beam. The longitudinal seam reinforcement of the slabs extend over the load-bearing structure from one slab to the other and the transverse bearing over the bearing structure in the end joint of the slab. In the height direction, it is important to place the reinforcement in the middle of the seam to prevent moment at the end of the slab and to ensure the required concrete covering. The examples of inappropriate placement are shown in Figure 22 (Betoninormikortti 23 2012, 27)

The diameter of rebars is limited (f ≤ 16 mm) and a large amount of steel in the seam should be avoided. Suitable diameter of the internal rabars is 12 mm. Rebars steel - T10 k1200 sufficient for the required minimum value of 20 kN/m of joint rebars. (Betoninormikortti 23 2012, 27)

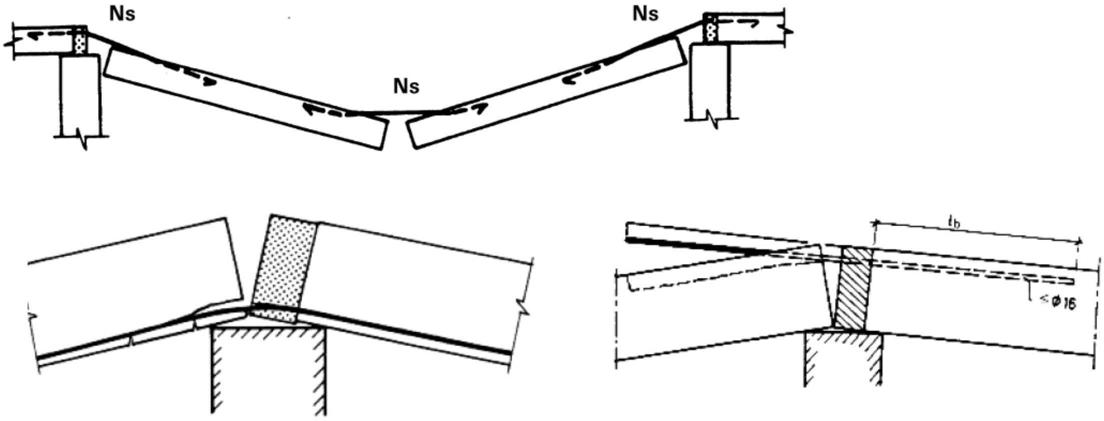


Figure 22. Inappropriate placement of rebars / FIP1982, Engström /

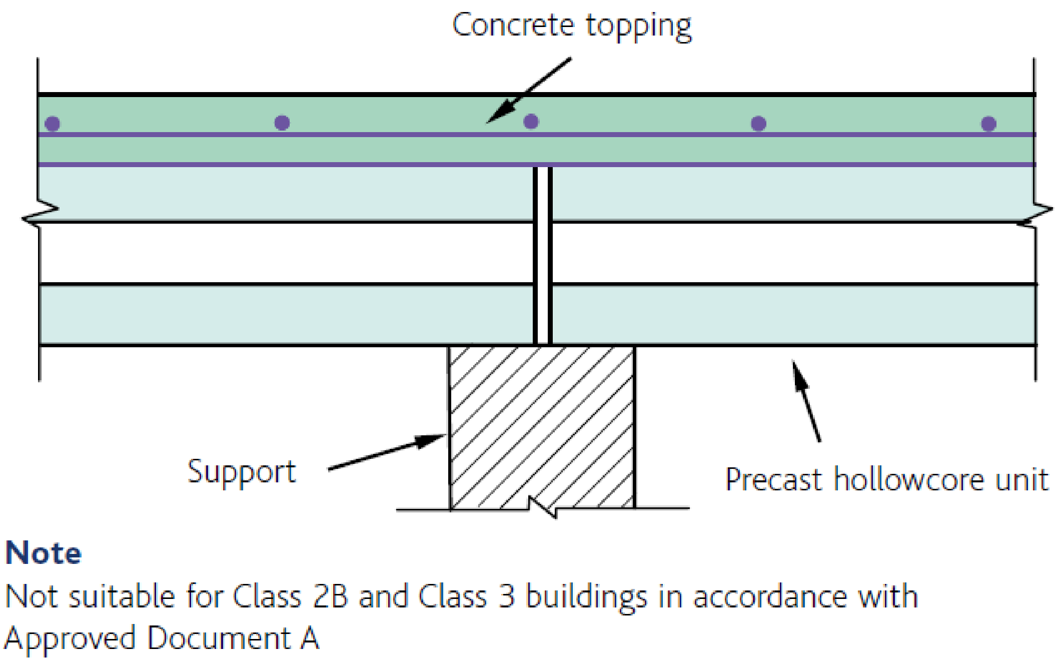


Figure 23. Internal floor ties within bonded concrete topping (O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 5)

The anchoring length of the longitudinal seam reinforcement of the tile depends not only on the adhesive properties of the seam reinforcement, but also on the adhesion properties of the braid and seam casting in the slab.

The adhesion of steel in a narrow, difficult-to-pour and sealable seam is not always the best. Therefore, the length of the anchoring of the joint reinforcement should be calculated according to the "bad" adhesion condition. At the edge of the building, the length and transverse seams of the slab are anchored around the ring reinforcement by means of a hook. (Betoninormikortti 23 2012, 27)

5.1.1 Dimensioning of the internal ties in the accident limit state.

In the SFS-EN 1991-1-7+AC there are four building Classes: 1, 2A, 2B and 3. The building type and occupancies for each class are given in Table 5, together with a summary of the requirements of SFS-EN 1991-1-7+AC.

Class	Building type and occupancy	Summary requirements
1	• House not exceeding 4 storeys. • Agricultural buildings. • Buildings into which people rarely go.	• No additional measures are likely to be necessary.
2A	• 5 storey single-occupancy houses. • Hotels, apartments and other residential buildings not exceeding 4 storeys. • Offices not exceeding 4 storeys. • Industrial buildings not exceeding 3 storeys. • Retailing premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey. • Single-storey educational buildings. • All buildings not exceeding 2 storeys to which members of the pubic are admitted and which contain floor areas exceeding 2000 m^2 at each storey.	• Horizontal ties, OR • Effective anchorage of floors to walls, as described in the codes of practice.
2B	• Hotels, apartments and other residential buildings exceeding 4 storeys, but not exceeding 15 storeys. • Educational buildings greater than 1 storey, but not exceeding 15 storeys. • Retail premises greater than 3 storeys but not exceeding 15 storeys. • Hospitals not exceeding 3 storeys. • Offices greater than 4 storeys but not exceeding 15 storeys. • All buildings to which members of the pubic are admitted and which contain floor areas exceeding 2000 m^2 but less than 5000 m^2 at each storey. • Car parking not exceeding 6 storeys.	• Horizontal ties and vertical ties as described in the codes of practice, OR • Show that the removal of a wall or column will cause only limited damage, OR • Design as 'key elements'.
3	• All buildings defined above as Class 2A and 2B that exceed the limits on area and/or number of storeys. • All buildings, containing hazardous substances and/or processes. • Grandstands accommodating more than 5000 spectators.	• Systematic risk assessment.
Note Basement storeys may be excluded provided they meet Class 2B criteria		

Table.5 Building classes and corresponding tying requirements

(O Brooker “How to design concrete buildings to satisfy disproportionate collapse requirements” p. 2)

Depending on the consequences class, the ***longitudinal seam reinforcement (A_{s3})*** of the slab must withstand the following forces:

Consequences class 1 and 2 in the accidental limit state

$$T = \begin{cases} \geq 20 \ \dfrac{kN}{m} \cdot s \\ \geq 70\,kN \\ \leq 150\,kN \end{cases} \qquad (11)$$

Consequences class 3a in the accidental limit state

$$T = \begin{cases} \geq F_t \cdot s \ \cdot \dfrac{(g_k + \Sigma\psi_i \cdot q_k) \cdot z}{37{,}5 \dfrac{kN}{m}} \\ \geq F_t \cdot s \\ \geq 70\,kN \end{cases} \qquad (12)$$

where

$$F_t \leq \begin{cases} 48 \dfrac{kN}{m} & \text{If the building has more than 15 floors} \\ (16 + 2{,}1 \cdot n_s) \cdot \dfrac{kN}{m} \end{cases} \qquad (13)$$

z – (x_3 in the calculating tool) – the greatest distance between vertical bearing structures (columns and walls) in the direction along the slabs

s – the distance between two longitudinal seams

n_s – number of floors

g_k – the characteristic value of the permanent actions

q_k – the characteristic value of the variable actions

Ψ_i – the variable load factor in the accident limit state

(Betoninormikortti 23 2012, 26)

The formulas 11 and 12 are almost identical to the formulas for the peripheral ties. In the case of longitudinal seam reinforcements, a minimum pulling force of 70 kN applies only to centralized ties with a spacing of more than 3,5 meters.

Thus, when looking at the floor slab, the bonding force will always be 20kN/m · s = 24kN in building classes 1 and 2, since the bandwidth *s* is always up to the width of the hollow core, i.e. 1,2 meters. Only in the building class 3a, the spacing of the hollow slabs begins to affect the dimensioning. (Betoninormikortti 23 2012, 27)

Depending on the consequences class, the ***transverse seam reinforcement (As₁)*** must withstand the following forces:

Consequences class 1 or 2 in the accidental limit state

$$T = \begin{cases} \geq k \cdot V_k \\ \geq 20kN/m \cdot s \\ \geq 70kN \\ \leq 150kN \end{cases} \quad (14)$$

Consequences class 3 in the accidental limit state

$$T = \begin{cases} \geq k \cdot V_k \\ \geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \sum \Psi_i q_k)}{6kN/m^2} \cdot \frac{x}{5m} \cdot s \\ \geq F_t \cdot s \end{cases} \quad (15)$$

where

$$F_t \leq \begin{cases} 48\,\frac{\mathrm{kN}}{\mathrm{m}} & \text{If the building has more than 15 floors} \\ (16 + 2{,}1 \cdot n_s) \cdot \frac{\mathrm{kN}}{\mathrm{m}} \end{cases}$$

V_k – the maximum beam support reactions to the column

k – the difference between the friction coefficients of column and beam

$$s = \frac{L_1 + L_2}{2}$$

L_1 , L_2 – the span between bearing structures

n_s – number of floors

x – depends on the case

- in case of bearing wall-slab structure - min (Lv/2 or 2,25H/2)
- in case of bearing beam-column structure - max (L3,L4,..)

g_k – the characteristic value of the permanent actions

q_k – the characteristic value of the variable actions

Ψ_i – the variable load factor in the accident limit state

(Betoninormikortti 23 2012, 20-21)

The minimum tensile value $k \cdot V_k$ in formulas 14, 15 applies only in case of column-beam frame where the transverse seams of the floor slab must also prevent the beam from falling over the column due to non-symmetrical heat and moisture movements. The difference *k* between the frictional forces of the joining surfaces depends precisely on the joints between the column and the beam and it is determined as:

k = 0,2 – if the joint has a rubber equalizer plate, a rubber bearing sheet

k = 0,3 – if both of joint surfaces made of steel

k = 0,4 – if the joint has the steel and concrete surfaces facing each other

k = 0,5 – in other cases

Attaching the element may allow for heat and moisture movements, but ultimately limits the displacement so that the element does not fall off the support.

However, the element does not need to be attached to a force, which is more than 150 kN or what the element supporting structure will last.

The connection of the element may be such that the fall of the element is prevented automatically or the heat and humidity movements cannot affect on the event. If the frictional force caused by the compression stress acting on the joint is greater than the joint stress capacity required preventing the element from falling (in accordance with calculations), no separate attachment is required in the joint. When the load resulting from the structure above the joint is greater than the support reaction of the element, the collapse of the element as a result of asymmetric heat and moisture movements is prevented.

(Betoninormikortti 23 2012, 17-18)

5.2 Peripheral ties

At each floor and roof level, an effectively continuous tie should be provided within 1,2 m of the floor edge. Structures with internal edges (e.g. atria and courtyards) should also have similar peripheral ties.

(O Brooker “How to design concrete buildings to satisfy disproportionate collapse requirements” p. 3)

In accordance with the term 'peripheral ties', the binding reinforcement is placed around the entire box. The purpose of the bond reinforcement is to connect the tile elements in such a way that the plate effect can develop. Therefore, the correct implementation of the link reinforcement is essential and the extensions and anchoring of the link reinforcement are carried out taking into account the requirements of the pulling reinforcement.

In the inner corners of the box form, anchoring is carried out as shown in Figure 24. According to the picture, the vertical steel can be easily placed in the tile, and the reinforcement perpendicular to the tile is anchored. If there is a surface pain in the drawer, the steels can be raised before the corner and anchored to the surface, which, when reinforced, also serves to improve the plate effect. Otherwise, the rebars are introduced into the slab from the openings made for the web and casting occurs through openings in the cavities of the top shell of the slab (Figure 25).

(Rakennusrungon vakavuustarkastelut, p. 55)

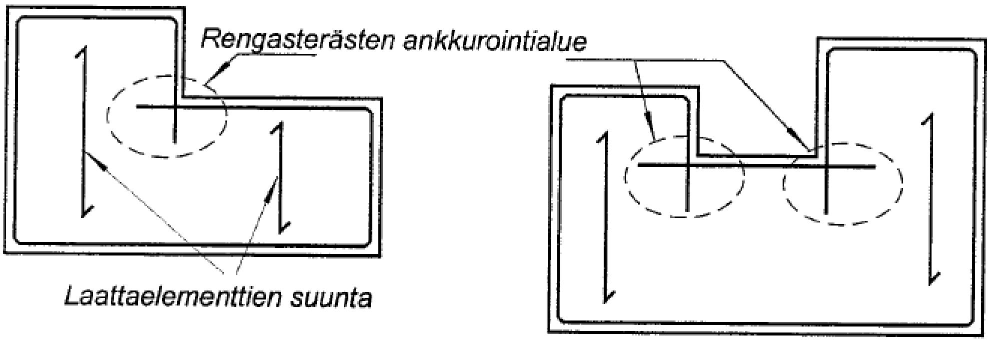


Figure 24. Arrangement of a link reinforcement to ensure plate effect

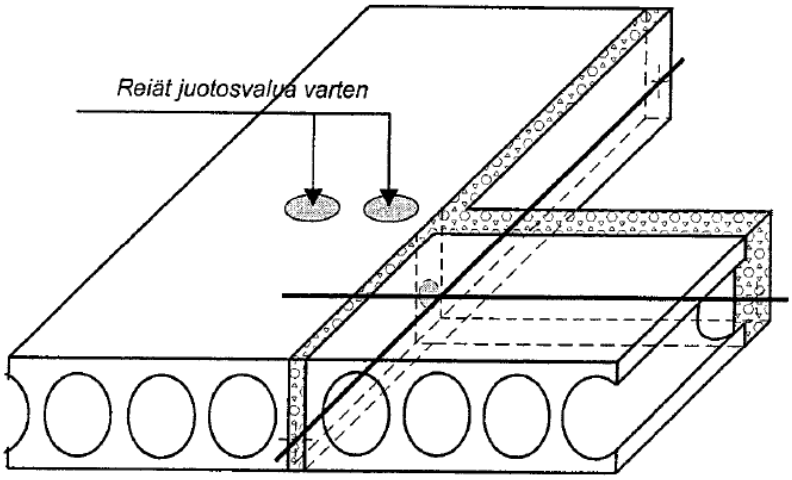


Figure 25. Placement of binding reinforcement in the inner corner of the slab when there is no structural surface concrete.

(Rakennusrungon vakavuustarkastelut, p. 56)

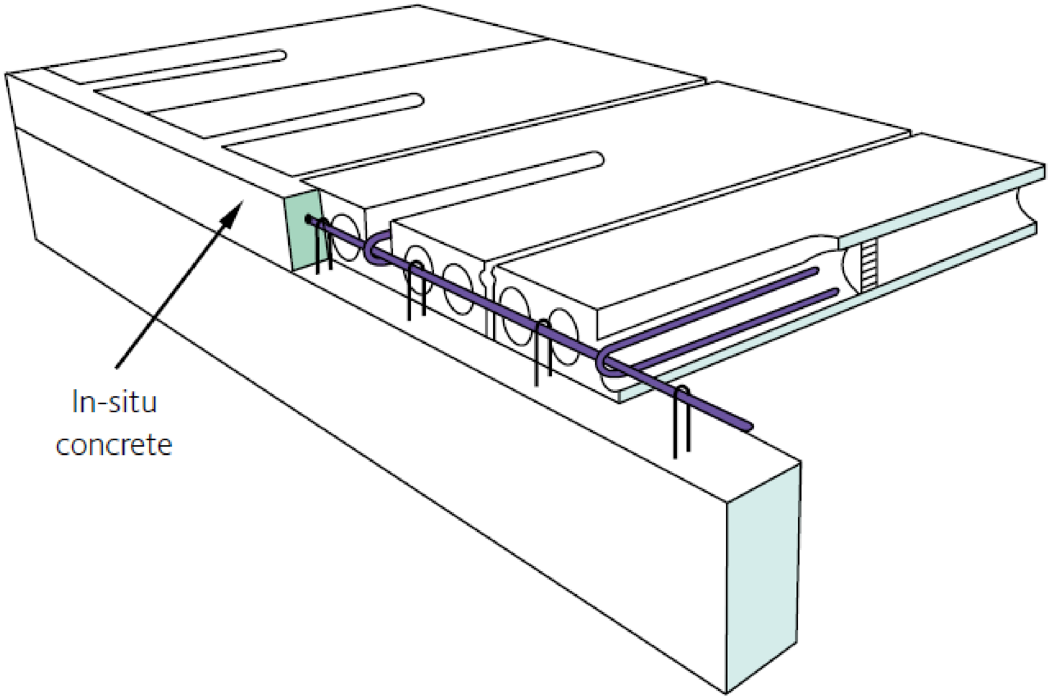


Figure 26. Perimeter floor ties within hollow-core units

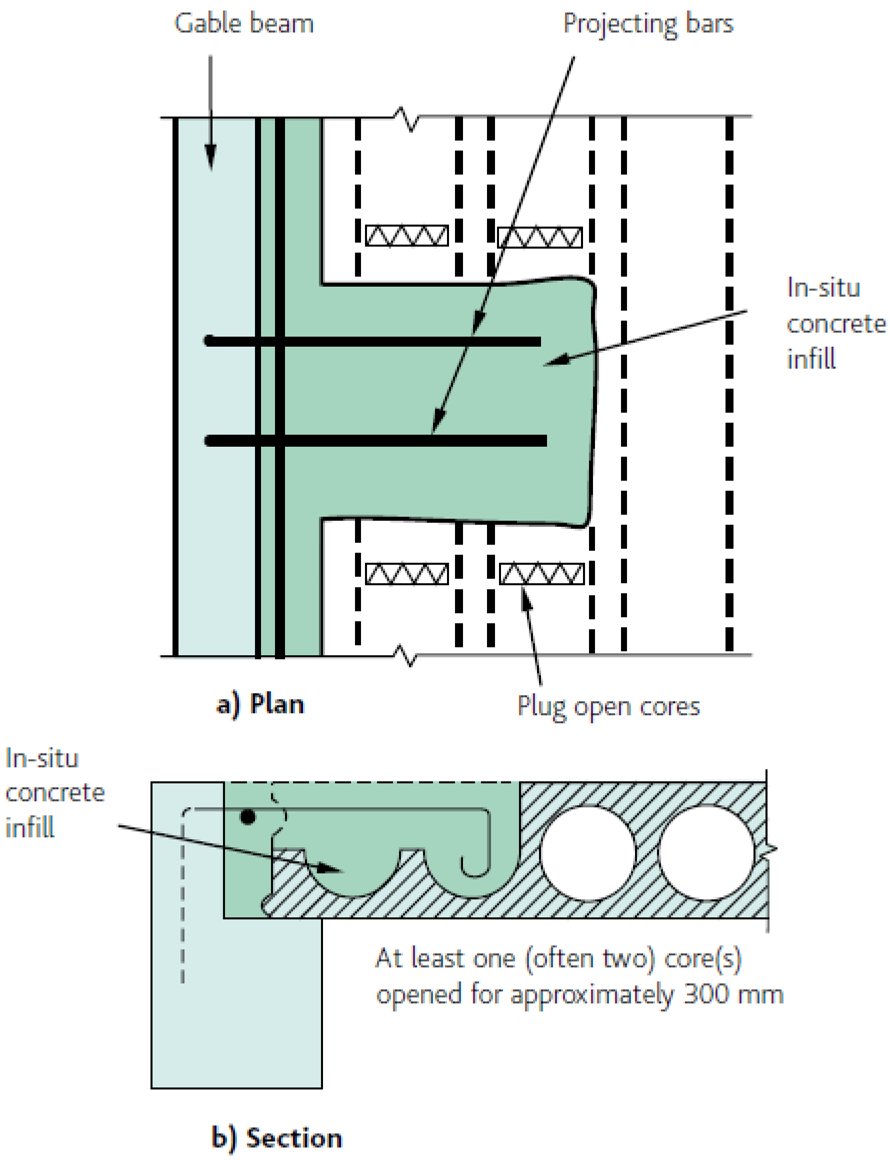


Figure 27. Perimeter ties where hollow-core units span parallel to edge beam

(O Brooker "How to design concrete buildings to satisfy disproportionate collapse requirements" p. 6)

5.2.1 Dimensioning of the peripheral ties in the ultimate limit state

The box is considered as a horizontal support $q_{w.Ed}$ for the horizontal loads, in which a compression arc and a corresponding draw bar are formed. The shear walls and stiffening cores are viewed either as projections or as simply supported structures. The walls or beams perpendicular to the direction of force are not considered to have side stiffness or impact on the operation of the drawbar.

(Rakennusrungon vakavuustarkastelut, p. 54)

The pulling force of the tow bar is taken completely by the peripheral sealing

$A_{sh} \geq F_{t.Ed.i} / f_{sd}$, where the force $F_{t.Ed.i} = \frac{M_{Ed.i}}{z_i}$ (i = 1 or 2) is calculated from the moments $M_{Ed.1}$ or $M_{Ed.2}$ due to load $q_{w.Ed}$, respectively:

- Case 2, when the support is oriented along to the wind direction,
  $M_{Ed.1} = q_{w.Ed} L_{h1}^2/2$, (16)
- Case 1, when the support is perpendicularly oriented to the wind direction,
  $M_{Ed.2} = q_{w.Ed} L_{h2}^2/8$. (17)

where

$q_{w.Ed}$ – is the total load on the box due to wind and addition horizontal force

L_h – the width of the box

The force exerted on the torsion is obtained by dividing the moment M_{Ed} by the moment arm z of the carrier. You can also calculate the amount of traction in the cross-sectional area directly by the formula:

$$A_s = \frac{M_{Ed}}{z f_{sd}} \qquad (18)$$

(Leskelä 2006, 545.)

f_{sd} is the design strength of steel, i.e. the specific strength divided by the coefficient of material strength of the reinforcement, which is 1,15 in the normal limiting dimensioning situation. (SFS-EN 1992-1-1, 26.)

The moment arm z_i (i = 1 or 2) is defined by the geometry of the slab (Figure 28):

- Single span structure

- $1 < L_{h2}/L_v < 2$ $\quad z_2 = 0,15L_v(3 + L_{h2}/L_v),\ z_2 \leq 0,75L_v$
- $L_{h2}/L_v \leq 1$ $\quad z_2 = 0,6L_{h2}$ (19)

- Structure with the stiffening core

- $0,5 < L_{h1}/L_v < 1$ $\quad z_1 = 0,8L_{v1}$ (20)
- $L_v \geq 2L_{h1}$ $\quad z_1 = 1,2L_{h1}$

, where L_v – the height of the box

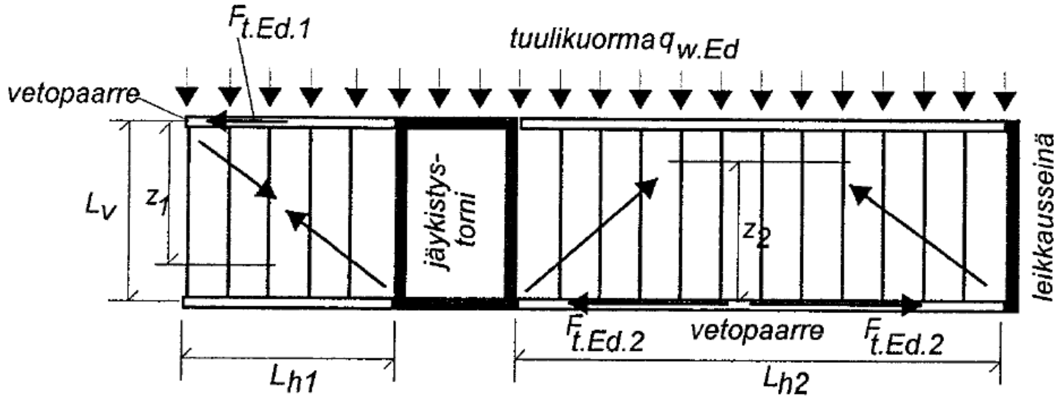


Figure 28. The structure of the slab floor under the influence of horizontal forces (Rakennusrungon vakavuustarkastelut, p. 54)

In the case of Figure 5.13 (a), for calculating the tire sealing, the tile dimensions of the slabs between the slabs first divide the load $q_{w.Ed}$ so that the tile field ´k´ (k=1, 2... n) receives load:

$$q_{w.Ed.k} = q_{w.Ed} \frac{L_k^{\,3}}{\sum_{j=1}^{n} L_j^{\,3}} \qquad (21)$$

Where L_k is the pitch of the tiles in the field ´k´ and n is the total number of fields. In each field, the ´k´ moments are calculated for the load $q_{w.Ed.k}$ and the moment arms for calculating the pulling forces as shown in Figure 29.

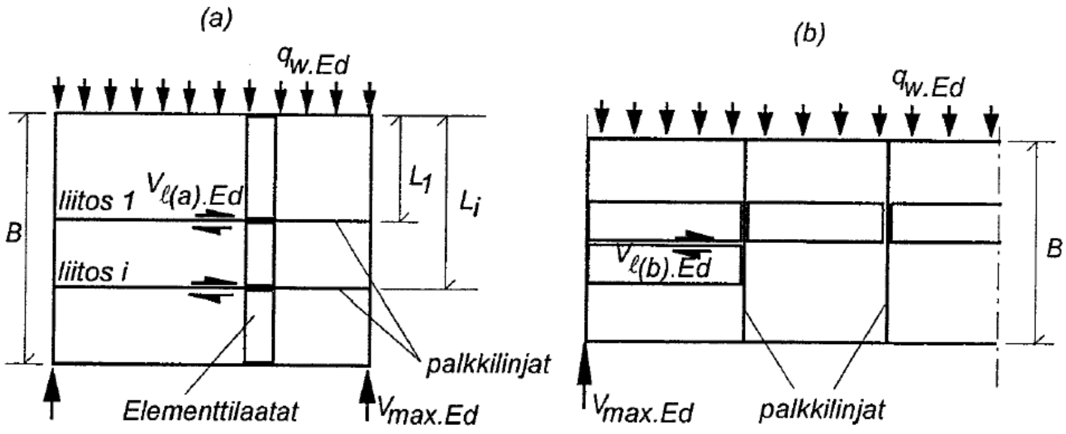


Figure 29. Calculation of the shear force according to the geometry and load direction of the slab. (Rakennusrungon vakavuustarkastelut, p. 53)

5.2.2 Dimensioning of the peripheral ties in the accidental limit state

The accident limit state determines the minimum number of tire seals, i.e. the steel must be at least equal to the dimension according to the accident limit state, unless further calculations are required for the normal breakdown condition.

The tensioning strength of the peripheral ties should withstand the forces of the normal fracture load combinations as well as the following forces:

Consequences class 1 or 2 in the accidental limit state

$$T = \begin{cases} \geq 20\ \dfrac{\mathrm{kN}}{\mathrm{m}} \cdot (s + a) \\ \geq 70\,\mathrm{kN} \\ \leq 150\,\mathrm{kN} \end{cases} \qquad (22)$$

Consequences class 3a in the accidental limit state

$$T = \begin{cases} \geq F_t \cdot (s+a) \cdot \dfrac{(g_k + \Sigma\psi_i \cdot q_k) \cdot z}{37{,}5\,\dfrac{kN}{m}} \\ \geq F_t \cdot (s+a) \\ \geq 70\,kN \end{cases} \qquad (23)$$

where

$$F_t \leq \begin{cases} 48\,\dfrac{kN}{m} & \text{If the building has more than 15 floors} \\ (16 + 2{,}1 \cdot n_s) \cdot \dfrac{kN}{m} \end{cases}$$

s – The distance from the edge of the building

a – is the distance of the ring rail from the edge of the building

x – The horizontal distance of the site from the top of the crest

n_s – Number of floors

g_k – the characteristic value of the permanent actions

q_k – the characteristic value of the variable actions

Ψ_i – the variable load factor in the accident limit state

(Betoninormikortti 23 2012, 25)

5.3 The shear resistance of the seams

The purpose of seam reinforcement is e.g. to prevent the free opening of cracks due to deformation of the plate and the joints of the slabs, which would reduce the shear strength of the seams.

The shear force $V_{\ell.Ed}$ is taken by the shear strength of the concrete alone and the design shear stresses are calculated assuming the effective height of the seam h_j (Figure 30). The concrete may be cracked in the seam, which is taken into account in the calculation of the shear strength. (Rakennusrungon vakavuustarkastelut, p. 53)

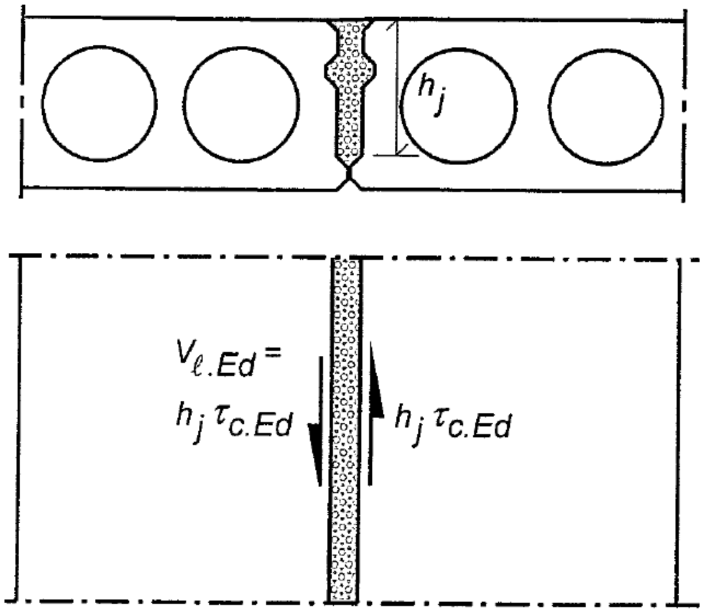


Figure 30. Determination of the effective height h_j between the slabs.

$$\tau_{c.Ed} = V_{\ell.Ed}/h_j \leq \tau_{Rdi} \qquad (24)$$

(Rakennusrungon vakavuustarkastelut, p. 53)

In addition to researching the accident limit state, the joints of the floor slab will have shear loads in the collapse state. The seams should withstand these shear stresses in order for the plate effect to occur. Seam shear stresses are only taken by a seam concrete with a shear resistance of τ_{Rdi} = 0,15 MPa, regardless of concrete physical properties, in the hollow core joints according to the EN 1992-1-1. If the calculated shear stress of the seam in the fracture state exceeds the shear strength of the seam (0,15 MPa), seam reinforcement should be placed in the longitudinal and transverse seams of the tile to increase the shear resistance of the seams. (Betoninormikortti 23 2012, 23)

6 Calculation part

The purpose of the calculation sheets was to simplify the calculation for young employees in a team and to reduce errors associated with the human factor when calculating the amount of reinforcement. That is the reason why no macros were applied. What a powerful tools they are, but the greater will be the challenges for people to understand and use them. As a result of the interconnected work with the team, the tool became convenient and visible in use. The second important aim was to settle on the quality and reliability of the calculations. As far as a calculation would be trusted and the purpose of it explained, no need for later editing should turn out. The Excel calculator was developed and piloted in the daily work in order to get empirical aspects on the subject. The feedback from colleagues is wished to produce new opinions and ideas of the preferred features for the tool.

In this thesis, sheets for calculating the required amount of peripheral ties and internal ties were completed. For the calculation, the user will need to enter the initial data, according to which reinforcement will be selected in one case or another. Also in the tool, there is an instruction sheet in which there is information from trusted sources, such as Eurocode and RIL directories.

Both parts of the calculation are presented in such a way that at the beginning, the user is required to enter the initial data for the design, and the result of all operations is the final selection of the number of rods and their total cross-sectional area.

6.1 Peripheral ties calculation

The first step in calculating is entering of the initial data. The designer needs to select consequences classes, basic dimensions of the building, geometry of the structure and characteristics of material. All the highlighted cells need to be filled by the designer and every entered value in cell is strictly limited to avoid errors in calculation.

In the geometry part there are 4 situations for calculating. The structure has either shear wall or a stiffening core structure and the arrangement of slabs affects on the selection. The designer selects the width of the box and the height in accordance with the project. Moment arm *z* calculates automatically. It is also important to check the distance from the peripheral ties to the edge (a_2, a_4).

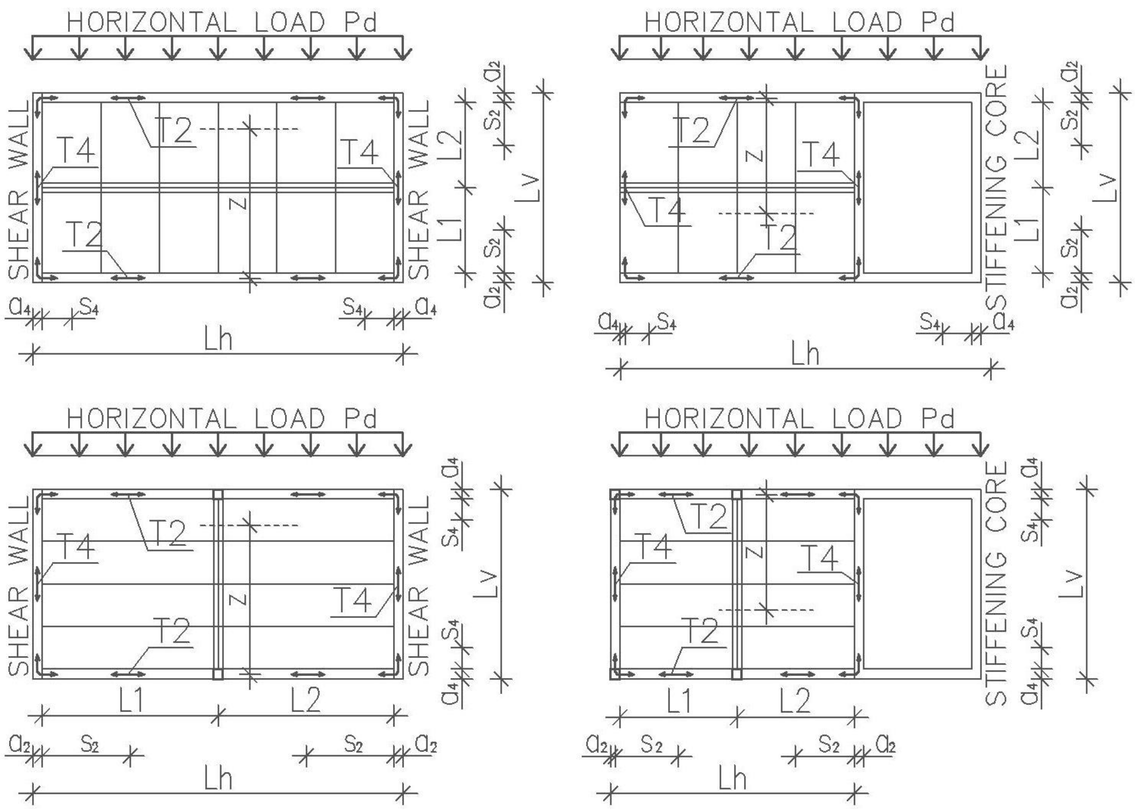


Figure 31. *Variants of the slab arrangements*

In choosing the characteristics of the material there are few most commonly used steel grades to select in the drop-down menu. The design strength of steel is determined automatically taking into account the coefficient of material and characteristic strength.

The tensile forces of the peripheral reinforcement are divided into forces T2 and T4, respectively, the lengthwise and the transverse direction. The value of s4 is always 0,6 meters in the hollow-core slab, as the next internal tie is located right in the nearest longitudinal seam.

In addition, the user must specify the value x_4, which is the space between the columns or walls in the tie direction. The calculation basis for x_2 automatically calculates with the dimensions already given, but for simplified calculating the peripheral reinforcements, it is considered only on the load-bearing walls, so x_2 is determined according to the nominal length of the bearing wall.

Right after the first part comes loads calculation. First - wind actions. Only 2 options for selection which are one-sided and two-sided terrain. Terrain category determines by the geographical features of the area and can be checked in the instruction sheet. Characteristic peak velocity pressure calculates automatically by the formula 1 stated in the beginning.

Then the designer enters the effective height, the actual length (Lu, Ld) and the horizontal distance of the site from the top of the crest as shown on the figure. Peak velocity pressure $q_{p(h)}$ determines automatically in accordance with the geometry.

The structural factor is also needed to be determined by the designer. When choosing the vertical distance *z* and width of the building *b*, the point automatically changes its position on the graph illustrated below so that it will be easy to check the value of $c_s c_d$ factor.

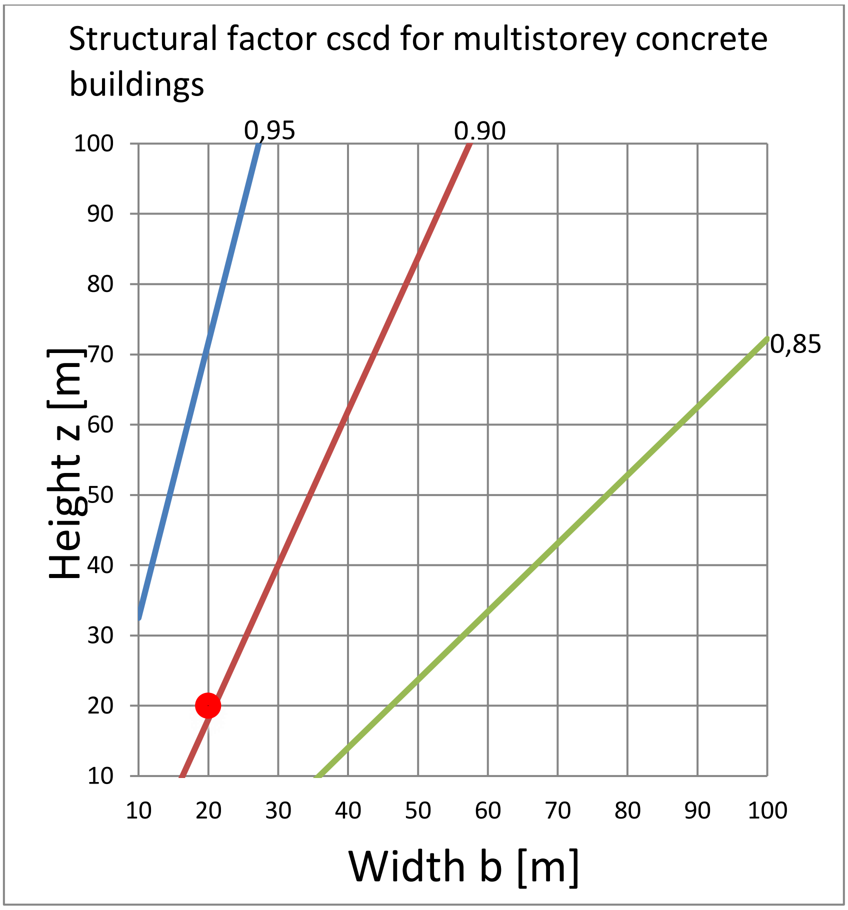


Figure 32. Determination of the structural factor cscd

The next step is the force coefficient determination. It is determined systematically in the instruction sheet as the value depends on the three components. Step 1 - $C_{f,0}$ is the force coefficient of rectangular sections with sharp corners and without free-end flow (d/b in affect) . Step 2 - Ψr is the reduction factor for square sections with rounded corners. Ψr depends on Reynolds number. Step 3 - Ψ_λ is the end-effect factor for elements with free-end flow. It depends on the effective slenderness value and $\phi = A/Ac$.

After the entire wind load value is given by multiplying all the values.

Other horizontal loads such as effect of inclination is also considered in this thesis. Initial data is the number of vertical members and loads acting on the floor. Then the additional horizontal force calculates automatically. Finally, overall load p_d can be calculated with Ψ_0 factor.

The characteristic values of the permanent and variable actions are given in the table on the instruction list. It is necessary to take into account the variable load factor in the accident limit state as it was mentioned already.

The rest of the operations automatically calculates the minimum amount of reinforcement according to both the ultimate and accident limit states. In the accident limit state, both forces T2 and T4 are determined, which will eventually be selected the greatest value, since the same amount of steel should circulate around the entire floor slab. In order to select the required amount of reinforcement the designer needs to play with the diameter and quantity of the rebars. The indicator shows an adequate value of the coefficient of utilization.

6.2 Internal ties calculation

The initial data for calculations are general information, geometry, characteristics and loads on the structure. The idea is quite similar to the peripheral ties calculation.

There are only two cases for determining in this part: bearing wall-slab and bearing beam-column structure. The selection affects on the calculations so that some of the unused sells are disappearing when choosing an exact case. Ties T3 are presented on the edge and between seam of slabs. It is important to choose the side (left/right) when calculating T3 ties on the edges. S3 is the spacing between slabs (i.e. distance from one lengthwise joint to another). The designer should enter dimensions of the bearing elements. The difference between the friction coefficients of column and beam is considered when calculating binding force T1.

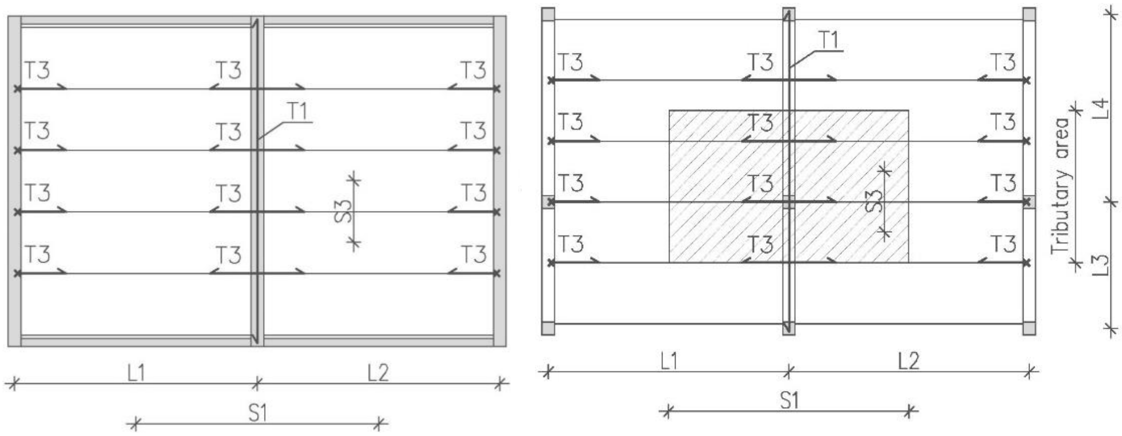


Figure 33. Variations of the slab arrangements

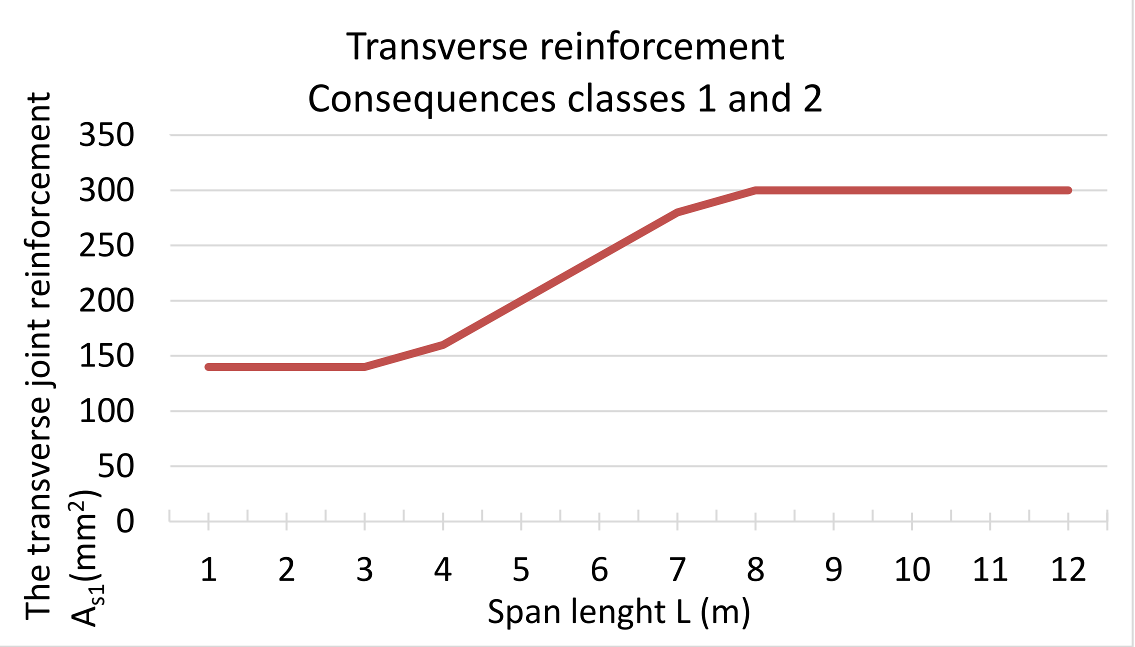
Finally, for the selection of the required amount of reinforcement (T1, T3 ties) the designer plays with the numbers and gets the utilization coefficient he/she needs.

7 Tables for quick selection of the reinforcement

The quick selection table was created for internal reinforcement only. The reason for this was the great demand for this type of calculation, as well as the simplicity of processing so much data. The amount of reinforcement was selected for the case of wall-slab structure, since in case of beam-column structure, it was also necessary to take into account the largest span in the direction of length Lv, which is established by a particular geometry in a particular case.

The initial and immutable data were the values of g_k, q_k and the height of the floor (3 meters). Thus, the required amount of reinforcement is selected taking into account the span length (L1, L2) and the number of floors. The tables were developed for classes 1, 2 and 3a. For classes 1 and 2 the value of As_3 remains constant because it depends only on the distance between the joints of the plates (formula 11).

The only variable value in formula (14) is the distance between the transverse bearing elements s_1, so the value of *Lv* does not affect the required amount of steel.



Graph 1. The influence of the span on the required amount of steel for the reinforcement of transverse joints.

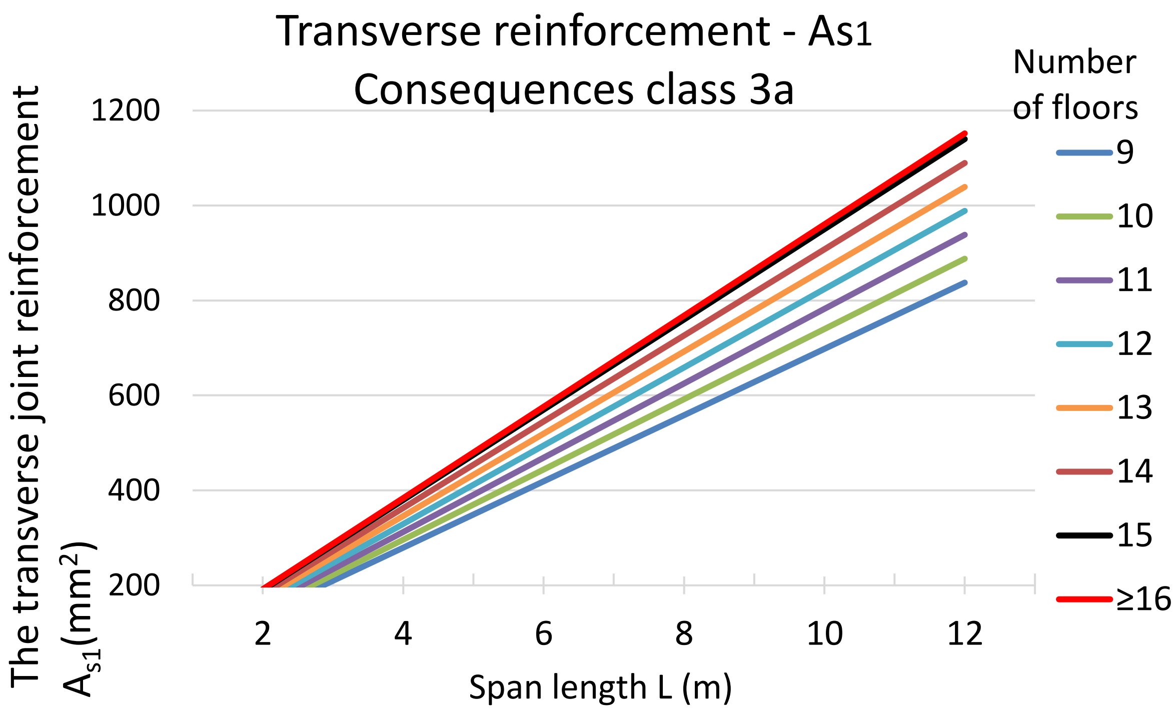
Graph 1 indicates that the amount of reinforcement 140 mm^2 does not change until reaching 3 meters. The reason for this is the minimum value for bending force - 70 kN. Further, there is an increase up to 8 meters where the required amount of reinforcement reaches 300 mm^2 with a maximum bending force of 150 kN.

The condition $T \geq F_t \cdot s$ in formula (15) is the maximum and determining factor in the selection of the required amount of reinforcement As_1.

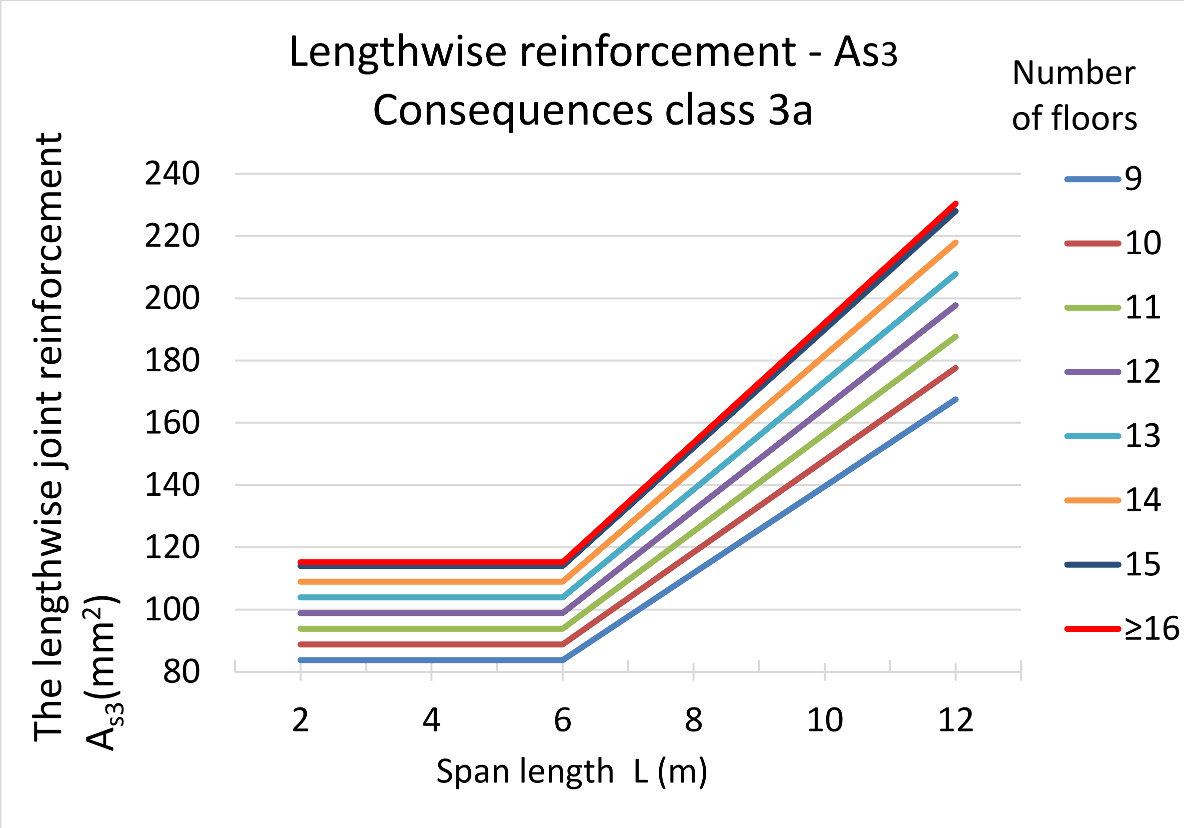
The formula $T \geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \sum \Psi_i q_k)}{6kN/m^2} \cdot \frac{x}{5m} \cdot s$ can be written as $T \geq F_t \cdot s \cdot \frac{x \cdot (g_k + \sum \Psi_i q_k)}{37{,}5kN/m}$

so that it is easier to understand that the value of *x*, along with the loads acting on the floor, must be greater than 37,5 kN/m to be taken into account in the calculation. For a standard floor height of 3, the value of *x* is taken as minimum between *Lv/2* and $2{,}25 \cdot H_{fl}/2$ and equal to 2,25 · 3/2 = 3,375 meters. This explains the fact that the value of *Lv* does not affect the selection of the required amount of reinforcement in the consequences class 3a.

It became possible to trace the ratio between the required amount of reinforcement and the number of floors in the consequences class 3a. Thus, the horizontal axis is the length of span *L* in the table, and the vertical axis is the number of floors from 9 to 16. In a building with more than 16 floors, the value of *Ft* remains constant. In graphs 2 and 3, you can follow all of the above-written.



Graph 2. Influence of the span length and the number of floors on the required amount of steel for the reinforcement of transverse joints.



Graph 3. Influence of the span length and the number of floors on the required amount of steel for the reinforcement of lengthwise joints.

Graphs show that an increase in the number of floors leads to an increase in the amount of steel required. In Graph 2, the number of reinforcement A_{s1} increases linearly since the span of two meters. Graph 3 presents the longitudinal reinforcement, which has a similar tendency, but at the beginning the number of reinforcement does not change. The span of more than 6 meters increases the amount of reinforcement required in a linear ratio. The reason for this is the condition $T \geq F_t \cdot s$ in the formula 12, which is taken into account until reaching a span of 6 meters.

8 Conclusions

Overall, the work has helped to understand the importance and the operation of the reinforcement on hollow-core structures. There are many different situations, which may arise when discussing about accidents. But due to the simplification and acceleration of design, the conditions for the durability of the seams have been determined. In the most demanding areas, structures should be explored in more detail, but, for example, when designing normal apartment buildings, there is no time for a more detailed analysis of structures in exceptional situations caused by accidents. In such projects a calculation tool may be the key for solving cost issues and saving much more extra time. Planning and accelerating the design was the goal of this thesis, and the result reached the goal.

I have reached more information about the phenomenon of the progressive collapse. I gained the knowledge of the main difference between ordinary calculation and calculation against progressive collapse.

Design engineers working on designing the hollow core slabs can use the results of this thesis. These structures are commonly used in office buildings, apartment houses and parking buildings and so on. The recommendations and studies made as a result of this study have been tested in a few projects already. The feedbacks of the experience was taken into consideration and related comments have been corrected. The tool is getting more useful and smarter with each feedback I get from the team.

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FIGURES

GRAPHS

TABLES

DESIGNER	THESIS CALCULATION TOOL	PÖYRY The connected company
Egor Vasin		
DATE :	*INTERNAL TIES CALCULATION*	PÖYRY FINLAND OY
XX.XX.2019		P.O. Box 4, 01621 Vantaa.,p. +358-10-3311

1. Output data

1.1. General information

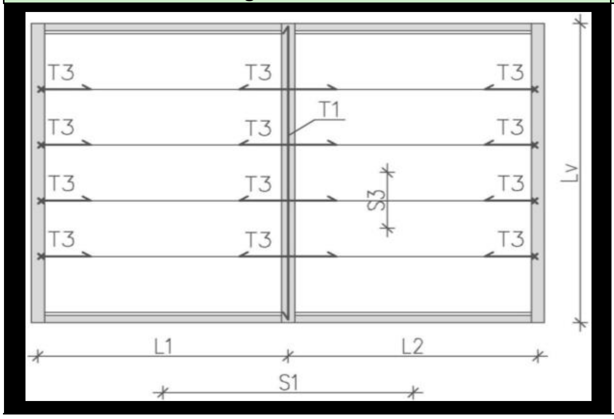
Consequences class in the accidental limit state		3	
Height of floor	H =	3	m
Number of floors	ns =	9	

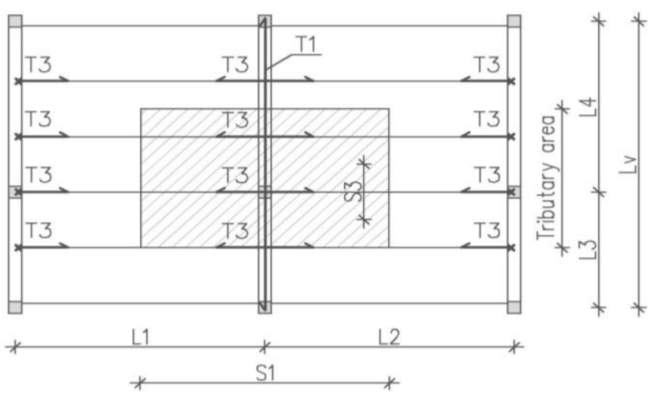
1.2. Geometry determination

Structure type		Bearing wall-slab structure	
Where to determine T3 by the edges? (L1/L2 span)		Left	
	L1 =	3,6	m
	L2 =	3,6	m
	Lv =	7,2	m
	S1 =	3,6	m
	S3 =	1,2	m

X1 (min (Lv/2; 2,25H/2))=	3,375	m	**X1 max(L3; L4,...) =**		m
$X3_{edge}$ =	3,6	m	**$X3_{edge}$ =**	3,6	m
X3 (max (L1; L2))=	3,6	m	**X3 max(L1; L2) =**	3,6	m

Bearing wall-slab structure	**Bearing beam-column structure**





1.3. Characteristics of material

Steel grade		A500HW
Characteristic strength of steel	fsk=	500 N/mm^2
The difference betweeen the friction coefficients of column and beam	μ=	0,2

1.4. Loads

The characteristic value of the permanent actions	gk=	5,5 kN/m^2
The characteristic value of the variable actions	qk=	2,5 kN/m^2
The variable load factor in the accident limit state	Ψ1=	0
	Ψ2=	0,3

Ψi =	Ψ1, snow-,ice- or wind load
	Ψ2, when the other loads

Dimensions of the bearing element

Lenght	L=	7,6 m
Width	b=	0,24 m
Height (If applied)	h=	0,34 m
Specific weight	γ=	25 kN/m^3
Unit weight of beam (in case of beam-column structure)	$g_{k,beam}$=	2,0 kN/m

2. Transverse joint reinforcement of the floor slabs

2.1 Determination of the binding force T1

Consequences classes 1 and 2:

T1=	≥ k · Vk	(in case of column-beam frame)	=	22,2	kN
	≥ 20 kN/m · s_1		=	72	kN
	≥ 70kN		=	70	kN
	≤150kN				
	The maximum beam support reactions to the column		Vk=	110,9	kN

Consequences class 3a:

T1=	≥ k · Vk	(in case of column-beam frame)	=	22,2	kN
	$\geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \sum \Psi i \cdot q_k)}{6\ kN/m^2} \cdot \frac{x_1}{5m} \cdot s_1$		=	70,67	kN
	≥ Ft · s_1		=	125,64	kN
	≥ 70kN		=	70	kN
	Ft is 48 kN/m or (16 + 2,1 ns) kN/m whichever is smaller		**Ft=**	34,9	kN/m
	Side force **T1** according to the consequence class		**T1=**	125,64	kN

2.2 Selection of the internal reinforcement

diameter of the rebars	(The recommended diameter is ≤ 16 mm)	**Ø=**	12	mm
quantity of the rebars		**n=**	3	pcs
The required steel quantity as a cross-sectional area		**As,req.=**	251,28	mm^2
Usable steel quantity as a cross-sectional area		**As=**	339,3	mm^2
Utilization		**KA=**	74,1 %	

3. Lenghtwise joint reinforcement of the floor slabs

3.1 Determination of the binding force T3

Consequences classes 1 and 2:

T3=	≥ 20kN/m · s_3	=	24	kN
	≥ 70kN	=	70	kN
	≤150kN			

Consequences class 3a:

T3=	$\geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \sum \Psi i \cdot q_k)}{6\ kN/m^2} \cdot \frac{x_3}{5m} \cdot s_3$	=	25,13	kN
	≥ 70kN	=	70	kN
	≥ Ft · s_3	=	41,88	kN
	Side force **T3** according to the consequence class	**T3=**	70	kN

3.2. Selection of the internal reinforcement

Diameter of the rebars	(The recommended diameter is ≤16 mm)	**Ø=**	10	mm
Quantity of the rebars		**n=**	2	pcs
The required steel amount as a cross-sectional area		**As,req.=**	140,00	mm^2
Usable steel amount as a cross-sectional area		**As=**	157,1	mm^2
Utilization		**KA=**	89,1 %	

3.3 Determination of the binding force T3 by the edges

Consequences classes 1 and 2:

T3=	$\geq 20kN/m \cdot s_3$	=	24	kN
	$\geq 70kN$	=	70	kN
	$\leq 150kN$			

Consequences class 3a:

T3=	$\geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \sum \Psi i \cdot q_k)}{6\,kN/m^2} \cdot \frac{x_{3\,edge}}{5m} \cdot s_3$	=	25,13	kN
	$\geq 70kN$	=	70	kN
	$\geq Ft \cdot s_3$	=	41,88	kN
	Side force **T3** according to the consequence class	**T3=**	70	kN

3.4. Selection of the internal reinforcement

Diameter of the rebars	**Ø=**	10	mm
Quantity of the rebars	**n=**	2	pcs
The required steel amount as a cross-sectional area	**As,req.=**	140,00	mm^2
Usable steel amount as a cross-sectional area	**As=**	157,1	mm^2
Utilization	**KA=**	89,1 %	

DESIGNER Egor Vasin	THESIS CALCULATION TOOL	PÖYRY The connected company
DATE : XX.XX.2019	***PERIPHERAL TIES CALCULATION***	**PÖYRY FINLAND OY** P.O. Box 4, 01621 Vantaa., p. +358-10-3311

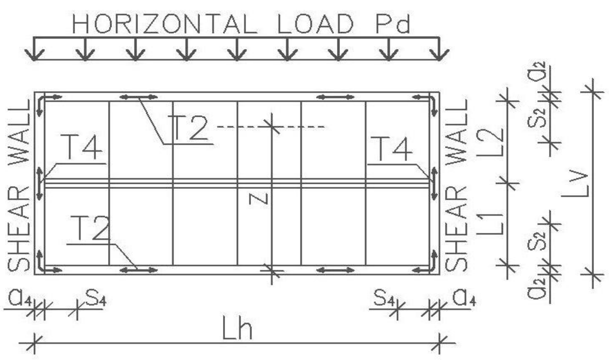
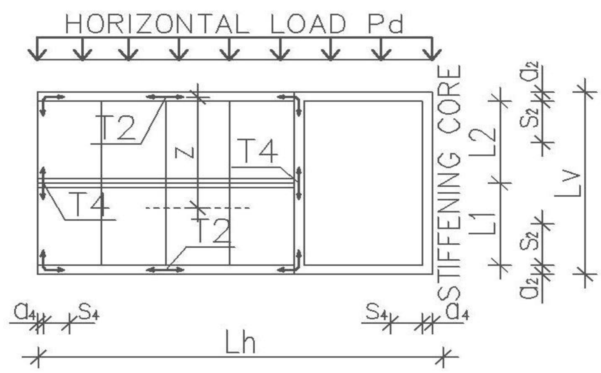
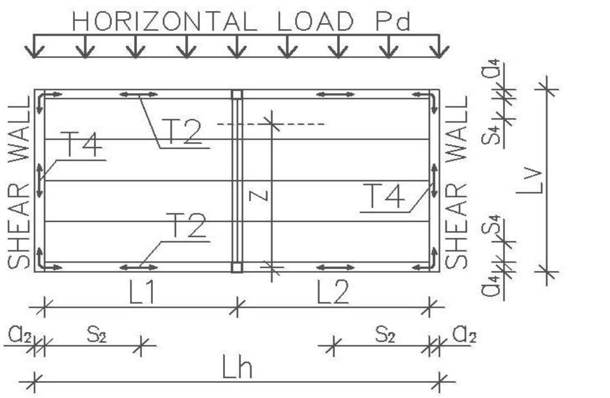
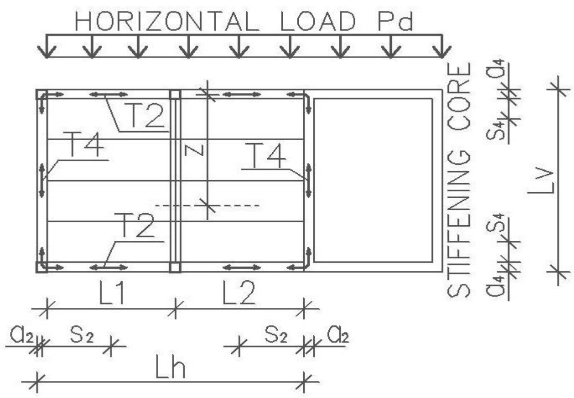
1. Output data

1.1. General information

Consequences class		CC2	
Consequences class in the accidental limit state		3	
The height of the building from the ground level	**h=**	20	m
The width of the building (transverse to the wind direction)	**b=**	20	m
Length of the building (wind direction)	**d=**	20	m
	if h<b ---> low building		
The height of floor	**H=**	3	m
Number of floors	**ns=**	15	

1.2. Geometry

Case			1	
Arrangement of slabs			SINGLE SPAN	
The width of the box		**Lh=**	20	m
The height of the box		**Lv=**	20	m
		Lh/Lv=	1,00	
Moment arm	z=0,15 · Lv · (3 + Lh/Lv), z ≤ 0,75Lv	**z=**	12	m
		s2=	2	m
		s4=	0,6	m
		a2=	0,2	m
		a4=	0,2	m
x2=	min (Lh/2; 2,25Hfloor/2)	**x2=**	3,375	m
x4=	max (L1,L2)	**x4=**	10	m

	SINGLE SPAN	STIFFENING CORE
1		
2		

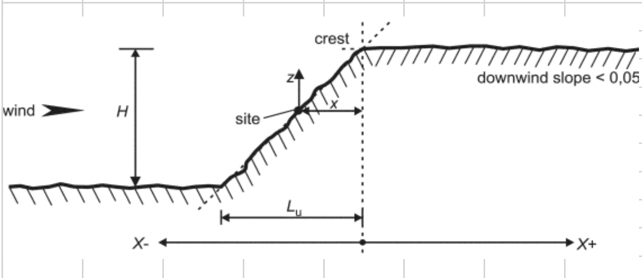
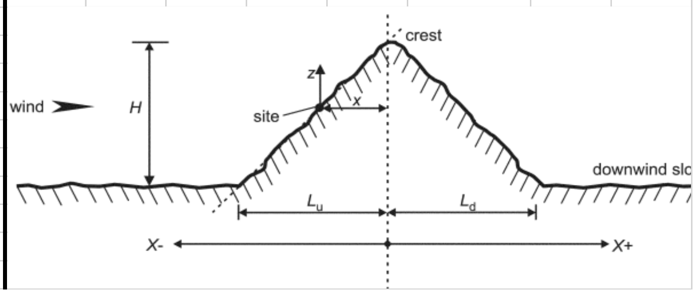
1.3. Characteristics of material

Steel		A500HW	
Characteristic strength of steel	fsk=	500	N/mm^2
Coefficient of material	γs=	1,15	
Design strength of steel	fsd=	434,78	N/mm^2

1.4. Loads

1.4.1. Wind actions

Case		1	
Terrain category		3	
		III	
Characteristic peak velocity pressure	$q_{p0}(h)$=	0,60	kN/m^2

One-sided terrain	Two-sided terrain
	

The effective height of the feature	H=	50	m
The actual length of the upwind slope in the wind direction	Lu=	200	m
The actual length of the downwind slope in the wind direction	Ld=	200	m
The horizontal distance of the site from the top of the crest	x=	-80	m
The upwind slope H/Lu in the wind direction (max=0,3)	Φu=	0,250	
The downwind slope H/Ld in the wind direction (max=0,3)	Φd=	0,250	
Increasing coefficient	γ_D=	1,98	
Peak velocity pressure $q_p(h) = \gamma_D \cdot q_{p0}(h)$	$q_p(h)$=	1,19	kN/m^2
The vertical distance from the lowest ground level of the site	z=	20	m
Structural factor (applied for low buildings only)	$c_s c_d$ =	0,9	
Effective slenderness λ = 1,4...2 h/b, when 15m < h < 50m	λ=	1,91	
Aspect ratio	d/b=	1	
Force coefficient	c_f=	1,35	
Wind load on the slab $q_{k,wind} = c_s c_d \cdot c_f \cdot q_p(h) \cdot H_{floor}$	$q_{k,wind}$=	4,34	kN/m

1.4.2 Other horizontal loads

Inclination $\theta i = \theta_0 \cdot \alpha_h \cdot \alpha_m$	θi=	0,0025	
Basic value θ_0 (the recommended value is 1/200)	θ_0=	0,005	
The reduction factor for height $\alpha_h = 2 / h^{1/2}$; where $2/3 \leq \alpha_h \leq 1$	α_h=	0,67	
The height of the building from the ground level	h=	20	m
The reduction factor for number of members $\alpha_m = (0{,}5 \cdot (1+1/m))^{1/2}$	α_m=	0,7416198	
The number of vertical members contributing to the total effect	m=	10	pcs
Floor loads			
Permanent actions	G_k=	4000	kN
Variable actions	Q_k=	800	kN
Additional horizontal force			
Permanent $g_{k,add.} = \theta i \cdot G_k / b$	$g_{k,add.}$=	0,49	kN/m
Variable $q_{k,add.} = \theta i \cdot Q_k / b$	$q_{k,add.}$=	0,10	kN/m
Overall load on the slabs			
$p_d = 1{,}15 \cdot K_{Fi} \cdot g_{k,add.} + 1{,}5 \cdot K_{Fi} \cdot q_{k,wind} + 1{,}5 \cdot K_{Fi} \cdot \Psi_0 \cdot q_{k,add.}$	p_d=	7,18	kN/m
	K_{FI}=	1	
	Ψ_0=	0,7	

1.4.3 Vertical loads

The characteristic value of the permanent actions	gk=	5,5	kN/m^2
The characteristic value of the variable actions	qk=	2,5	kN/m^2
The variable load factor in the accident limit state	$\Psi 1$=		(if applied)
	$\Psi 2$=	0,3	(if applied)

Ψi =	$\Psi 1$, snow-,ice- or wind load
	$\Psi 2$, when the other loads

2. Dimensioning of the peripheral reinforcement

Moment stress	M_{Ed}=	359,18	kNm
Tension force caused by moment M_{Ed} / z	F_d=	29,93	kN
The required amount of steel $A_{s1} = M_{Ed} / (z \cdot f_{sd})$	A_{s1}=	68,8	mm^2

3. Minimum quantity of the peripheral ties in the accident limit state

Consequences classes 1 and 2:

T2, T4=	≥ 20kN/m · (s + a)	T2=	44 kN
		T4=	16 kN
	≥ 70kN	=	70 kN
	≤150kN		

Consequences class 3a:

T2,T4=	$\geq \frac{F_t \cdot 0{,}8 \cdot (g_k + \Sigma \Psi i \cdot q_k)}{6\,kN/m^2} \cdot \frac{x}{5m} \cdot (s+a)$	T2=	58,78 kN
		T4=	63,33 kN
	≥ Ft · (s + a)	T2=	104,50 kN
		T4=	38,00 kN
	≥ 70kN	=	70 kN

Ft is 48 kN/m or (16 + 2,1 ns) kN/m whichever is smaller — **Ft=** 47,5 kN/m

Tensile strength according to the consequences classes

	T2=	104,5 kN
	T4=	70 kN
	max(T2,T4)=	104,5 kN
The amount of reinforcement working againt gravity forces A_{s2} = T / f_{sk}	$\mathbf{A_{s2}}$**=**	209 mm^2

4. Selection of the peripheral reinforcement

Diameter of the rebars	**Ø=**	10	mm
Quantity of the rebars	**n=**	3	pcs
The required steel amount as a cross-sectional area = max(A_{s1}, A_{s2})	**As,req.=**	209,00	mm^2
Usable steel amount as a cross-sectional area	**As=**	235,6	mm^2
Utilization	**KA=**	88,7 %	

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INSTRUCTIONS

Determination of consequences classes for building and structures

(In accordance with the National Building Code of Finland and Ministry of the Environment)

CC3 K_{Fi} = 1,1	The load-bearing system* with its bracing parts in buildings which are often occupied by a large number of people, such as: - residential, office and business buildings with more than 8 storeys** - concert halls, theatres, sports and exhibitions halls and spectator galleries - heavily loaded buildings or buildings with long spans. Special structures, such as high towers. Ramps as well as embankments and other structures that are located in environments sensitive to adverse effects of displacements, especially in areas of fine-grained soils.
CC2	Buildings and structures not belonging to classes CC3 or CC1. (K_{Fi} = 1)
CC1 K_{Fi} = 0,9	Buildings with 1 or 2 storeys** where people are only staying temporarily***, such as small warehouses and agricultural production buildings with a maximum floor area of 300 m2 or a maximum span of 6 metres. Structures which, when damaged, do not pose a major risk, such as: - low terraces and basement floors without cellar rooms - roof with crawl space, when roofing deck is the actual loadbearing structure - walls, windows, doors etc., which are mainly loaded horizontally by air pressure difference and which dont have a load-bearing or stabilising function in the load-bearing system.
*	however, small roofs and floors that are separate from the load-bearing system are in class CC2 if they do not form a part of the stiffening system of the whole structure.
**	underground floors included
***	Visiting the building daily is considered temporary staying but not staying for a longer time.

Categorisation of building classes (In accordance with SFS-EN_1991-1-7+AC)

1	● Single occupancy houses not exceeding 4 storeys. ● Agricultural buildings. ● Buildings into which people rarely go, provided no part of the building is closer to another building, or area where people do go, than a distance of 11/2 times the building height
2a **Lower Risk Group**	● 5 storey single occupancy houses. ● Hotels not exceeding 4 storeys. ● Flats, apartments and other residential buildings not exceeding 4 storeys. ● Offices not exceeding 4 storeys. ● Industrial buildings not exceeding 3 storeys. ● Retailing premises not exceeding 3 storeys of less than 1 000 m2 floor area in each storey. ● Single storey educational buildings. ● All buildings not exceeding two storeys to which the public are admitted and which contain floor areas not exceeding 2 000 m2 at each storey.
2b **Upper Risk Group**	● Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys. ● Educational buildings greater than single storey but not exceeding 15 storeys. ● Retailing premises greater than 3 storeys but not exceeding 15 storeys. ● Hospitals not exceeding 3 storeys. ● Offices greater than 4 storeys but not exceeding 15 storeys. ● All buildings to which the public are admitted and which contain floor areas exceeding 2 000 m2 but not exceeding 5 000 m2 at each storey. ● Car parking not exceeding 6 storeys.
3	● All buildings defined above as Class 2 Lower and Upper Consequences Class that exceed the limits on area and number of storeys. ● All buildings to which members of the public are admitted in significant numbers. ● Stadia accommodating more than 5 000 spectators. ● Buildings containing hazardous substances and/or processes.

The difference betweeen the friction coefficients of column and beam

μ=	0,2	→	if the joint has a rubber equalizer plate, a rubber bearing sheet
μ=	0,3	→	if both of joint surfaces made of steel
μ=	0,4	→	if the joint has the steel and concrete surfaces facing each other
μ=	0,5	→	in other cases

Characteristic values of the permanent and variable actions

Table 6.2 - Imposed loads on floors, balconies and stairs in buildings

Categories of loaded areas	q_k [kN/m^2]	Q_k [kN]
Category A		
- Floors	1,5 to2,0	2,0 to 3,0
- Stairs	2,0 to4,0	2,0 to 4,0
- Balconies	2,5 to 4,0	2,0 to 3,0
Category B	2,0 to 3,0	1,5 to 4,5
Category C		
- C1	2,0 to 3,0	3,0 to 4,0
- C2	3,0 to 4,0	2,5 to 7,0 (4,0)
- C3	3,0 to 5,0	4,0 to 7,0
- C4	4,5 to 5,0	3,5 to 7,0
- C5	5,0 to 7,5	3,5 to 4,5
category D		
- D1	4,0 to 5,0	3,5 to 7,0 (4,0)
- D2	4,0 to 5,0	3,5 to 7,0

Terrain category

0	Sea or coastal area exposed to the open sea
I	Lakes or flat and horizontal area with negligible vegetation and without abstacles
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights
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)
IV	Area in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m

Determination of the characteristic peak velocity pressure

$$q_{p0}(z)=\begin{cases} 0{,}00893 \cdot [ln(\frac{\max(1;z)}{0{,}003})]^2 + 0{,}0625 \cdot \ln(\frac{\max(1;z)}{0{,}003}), \textit{ terrain category } \mathbf{0} & = 1{,}24 \text{ kN/m}^2 \\ 0{,}00794 \cdot [ln(\frac{\max(1;z)}{0{,}01})]^2 + 0{,}0556 \cdot \ln(\frac{\max(1;z)}{0{,}01}), \textit{ terrain category } \mathbf{I} & = 0{,}88 \text{ kN/m}^2 \\ 0{,}00995 \cdot [ln(\frac{\max(2;z)}{0{,}05})]^2 + 0{,}0697 \cdot \ln(\frac{\max(2;z)}{0{,}05}), \textit{ terrain category } \mathbf{II} & = 0{,}77 \text{ kN/m}^2 \\ 0{,}01279 \cdot [ln(\frac{\max(5;z)}{0{,}3})]^2 + 0{,}0895 \cdot \ln(\frac{\max(5;z)}{0{,}3}), \textit{ terrain category } \mathbf{III} & = 0{,}60 \text{ kN/m}^2 \\ 0{,}01513 \cdot [ln(\frac{\max(10;z)}{1{,}0})]^2 + 0{,}1059 \cdot \ln(\frac{\max(10;z)}{1{,}0}), \textit{ terrain category } \mathbf{IV} & = 0{,}45 \text{ kN/m}^2 \end{cases}$$

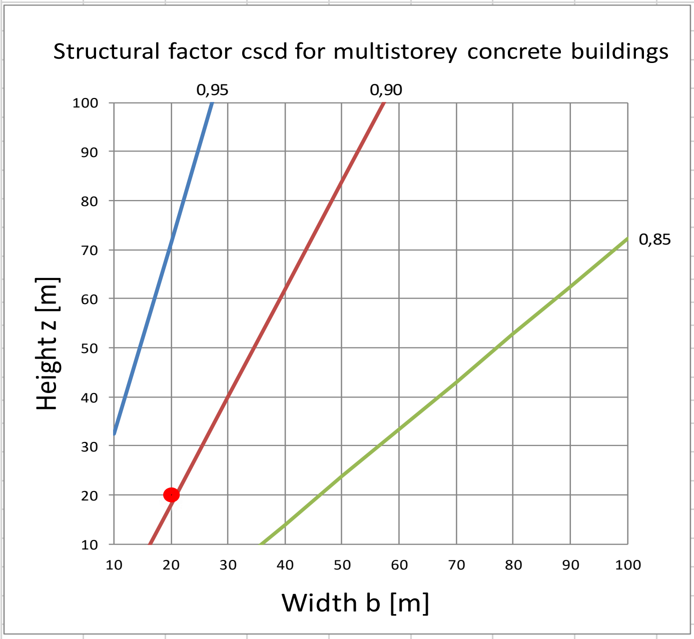
NOTE to self! **z** is the height of the building above the ground level

Determination of the increasing coefficient

One-sided terrain		Two-sided terrain	
$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 + x/Lu)$	when x<0	$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 + x/Lu)$	when x<0
$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 - 0{,}33 \cdot x/Lu)$	when x≥0	$\gamma_D = 1 + 2{,}8 \cdot \Phi \cdot (1 - 0{,}47 \cdot x/Ld)$	when x≥0

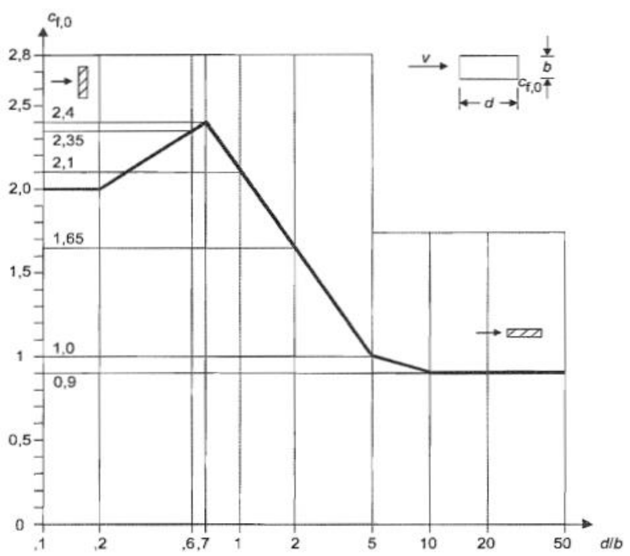
Determination of CsCd coefficient

(1) **CsCd** may be determined as follows:
a) For buildings with a height less than **15 m** the value of **CsCd** may be taken as **1.**
b) For facade and roof elements having a natural frequency greater than **5 Hz**, the value of **CsCd** may be taken as **1.**
c) For framed buildings which have structural walls and which are less than **100 m** high and whose height is less than 4 times the in-wind depth, the value of **CsCd** may be taken as **1.**
d) For chimneys with circular cross-sections whose height is less than **60 m** and 6,5 times the diameter, the value of **CsCd** may be taken as **1.**
e) Alternatively, for cases a), b), c) and d) above, values of **CsCd** may be derived from 6.3.1.
f) For civil engineering works (other than bridges, which are considered in Section 8), and chimneys and buildings outside the limitations given in c) and d) above, **CsCd** should be derived either from 6.3 or taken from Annex D.
NOTE 1 Natural frequencies of facade and roof elements may be calculated using Annex F (glazing spans smaller than 3 m usually lead to natural frequencies greater than 5 Hz)
NOTE 2 The figures in Annex D give values of **CsCd** for various types of structures. The figures give envelopes of safe values calculated from models complying with the requirements in 6.3.1.



Determination of Cf coefficient

Step 1 - $c_{f,0}$



$$c_f = c_{f,0} \cdot \Psi_r \cdot \Psi_\lambda$$

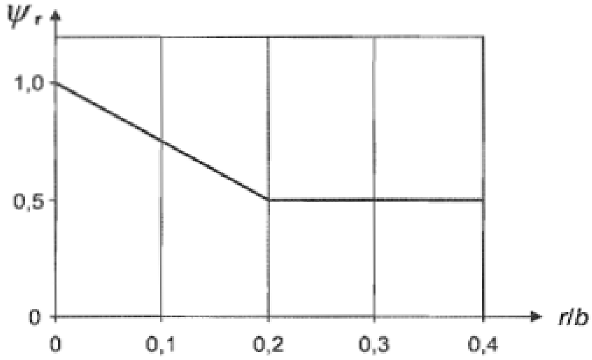
where:

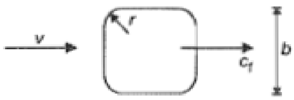
$C_{f,0}$ is the force coefficient of rectangular sections with sharp corners and without free-end flow as given by Figure 7.23.

Ψr is the reduction factor for square sections with rounded corners. **Ψr** depends on Reynolds number.

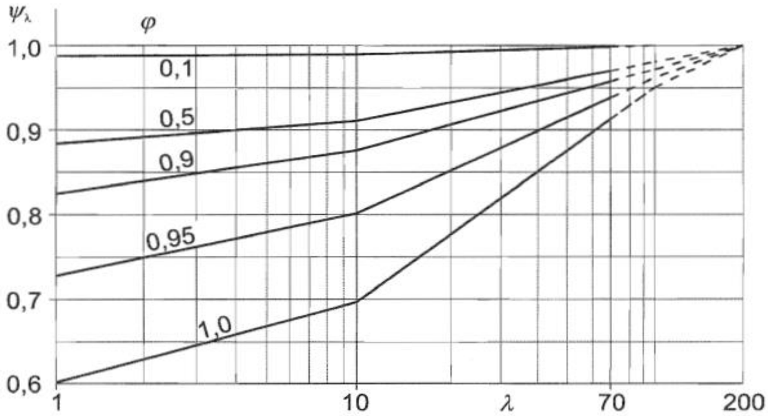
Ψλ is the end-effect factor for elements with free-end flow

Step 2 - Ψ_r





Step 3 - Ψ_λ



$\varphi = A/Ac$

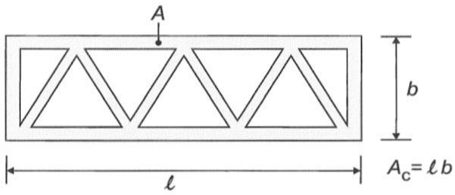


Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0

NOTE The ψ values may be set by the National annex.
* For countries not mentioned below, see relevant local conditions.

Determination of the effective slenderness

For polygonal, rectangular and sharp edged sections and lattice structures:

for $\ell \geq 50$ m, $\lambda = 1,4\ \ell/b$ or $\lambda = 70$, whichever is smaller

for $\ell < 15$ m, $\lambda = 2\ \ell/b$ or $\lambda = 70$, whichever is smaller

λ=	2	1,4	h/b	where h	15	50	m

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Bearing wall-slab structure

The required amount of the joint reinforcement in the cons. class 3a

L(m) / n_s	3,6			4,8			6			7,2			8,4			9,6			10,8		
	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$	A_{s1}	A_{s3}	$A_{s3'}$
9																					
10																					
11																					
12																					
13																					
14																					
15																					
16																					

A_s - The required amount of steel
A_{s1} - The transverse joint reinforcement
A_{s3} - The lengthwise joint reinforcement
$A_{s3'}$ - The lengthwise joint reinforcement by the edges
L - The maximum span length [m]
n_s - Number of floors

INITIAL DATA FOR DESIGN

H = 3 m (the height of floors)
S_3 = 1,2 m (the width of slabs)
fsk= 500 N/mm^2 (characteristic strength of steel)

Loads acting on the floor

gk = 5,5 kN/m^2
qk = 2,5 kN/m^2

The required amount of the transverse reinforcement in the cons. class 1 and 2

L(m)	**3,6**	**4,8**	**6**	**7,2**	**8,4**	**9,6**	**10,8**
A_{s1}							

A_{s3} - the value is constant in the consequences class 3a