DESIGN PROCEDURES FOR A SMALL CONCRETE PARKING GARAGE

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Bachelor Degree in Construction Engineering		Abstract
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ABSTRACT

Concrete is the most popular material for many constructions projects due to its strength, durability, reflectivity and versatility. The structure designer must understand these properties of concrete and how they can affect the design procedures as well as designing decisions in order to get sustainable and efficient results.

This thesis report describes the design procedures of a small concrete parking garage by emphasizing on its load-bearing structures.

This thesis outlines basic design principles of concrete structures according to eurocode 2. This thesis also examines basic calculations to provide an understandable document which shows the design procedures of a small concrete parking garage.

The aim of this thesis was to gain a broad understanding of the design principles of massive structures, as well as its design challenges. The results of this study could be used to design concrete structures.

Keywords Strength, Durability, versability,and load-bearing structures

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1 Figures

Q_k :Characteristic value of imposed load

Q_{sk} :Characteristic value of snow load

G_k :Characteristic value of permanent load

Ψ:Coefficient factor for imposed loads

V_k:Shear load on the beam

W:Distributed load on beam

L_b:Length of beam

C_{nom}:Normal concrete cover

C_{min} : Minimum concrete cover

Δ_{Cdev}:Allowance in design for deviation

$C_{min,b}$:Minimum cover due to bond requirement

$C_{min,dur}$:Minimum cover due to environmental condition

$C_{min,\Upsilon}$: Additive safety element

$C_{min,st}$:Reduction of minimum cover for use of stainless steel

Δ_{Cdur} :Reduction of minimum cover for use of additional protection

μ:The relative moment, which can be given by this expression b:Width of beam cross section

M_{Ed}:Design value bending moment on the beam

d:Bending effective depth of beam, d can be calculated as:

f_{cd}:Design value of concrete strength

f_{ck}:Characteristic value of concrete strength

f_{ck}:Characteristic value of concrete strength

Υ_c:Concrete partial safety factor

f_{yk}:Characteristic value of steel strength

Υ_s:Steel partial safety factor

Φh:Diameter of tension reinforcement bar

Φ1:Diameter of shear reinforcement bar

μ:Coefficient of friction between the tendons and their ducts

X:Neutral axis depth

f_{ctm}:Mean value of axial tensile strength of concrete

b_t:Width of tension zone

f_{ctk}:characteristic value of axial tensile strength

S:Unit length of beam (m)

A_{sw}:Shear reinforcement cross-sectional area

$A_{sw,min}$:Minimum allowed shear reinforcement cross-sectional area

$S_{1,max}$:Maximum shear reinforcement spacing

b: Cross section width

h:Cross section depth or length

i:Moment of gyration

I_C:Section moment of area

l_0:Buckling length of beam

2 Introduction

Parking as part of overall transportation system is the one of the crucial issues of 21st century. As the number of cars increases in Finland the need of parking places closer to the the destination of cars owners create big challenge in designing. It is the duty of engineers to combine those complex challenges and offer a solution, which is suitable for all those challenges. This is the reason why the design of parking facility requires a combined design approach from an architect and a structure engineer to ensure the quality of the parking facility as well as safety of the users.

2.1 General introduction

This thesis studies the design procedures of a small concrete parking garage and the thesis also outlines the relationships between parking places' geometries and the dimensions of chosen structures that carries loads to the foundations The thesis examines various factors that need to be considered during the design of structures of parking garage as well as the factors affecting the insertion of parking spaces in building layout. In addition, the thesis report outlines different design procedures of concrete structures according to Eurocodes and Finnish national annexes´ guidelines.

3 Objective and limitations

The aim of this thesis was to examine the design of load-bearing structures of small concrete parking garage. In addition the objective was to provide a document that can be used to design small as well as massive concrete structures. The results obtained from this thesis were not implemented.

The thesis report emphasises the structures that carries loads. However, some of the structures such as the foundation require the information about subsoil where the parking garage is going to be build, without this information the design of foundation will not be accurate. Therefore,this thesis report analyses only the structural components, which do not require the information on subsoil to be analyzed.

3.1 Case study

This thesis consists of designing procedures of small concrete parking garage, which can receive up to 40 passenger cars at the same time. The parking garage has only two floors, first floor and the ground floor. The first floor can be used by the driver from the right road and the ground floor from the left road. It will not be possible for the driver from the ground floor to access the first floor. The figure 1 gives the general idea of how the parking garage will look like after construction.

Figure 1: Ground floor parking places can only be accessible by users of left road and first floor by the users of the right road.

4 Materials and methods

The designing procedures of small concrete parking garage will be done into the following procedures:

1. Design of parking places
2. Classification of loads
3. Loads combinations
4. Design of ground floor
5. Design of beam

6. Design of column
7. Design of intermediate floor

4.1 Design of parking spaces

Parking facilities are designed to be easy to use and accessible to the drivers. These are the main factors to consider while designing parking facilities, and the driving direction of the chosen parking design must be clear to the parking users and able to provide guiding informations (RT 98-12367 2016, 2)

The parking facility must be connected to the road in a way that make the accessibility of parking facility easy and safe for the users. According to the guidelines for design of the parking facilities there must be at least one entry and one exit gate for 300... 400 parking places, the entry and the exit gate capacity is is one car at a time. (RT 98-12367 2016, 2). For large and complex parking facilities,the design of parking facilities may require working together with transport enginer in order to get sustainable and effective results.

4.1.1 Parking place size

The capacity of parking garage and how it is intended for use are the main factors to consider while designing the parking place size. The parking facility should be designed so that every parking space can be driven into at once without reversing at whatever angle the parking places are inserted into the parking facility layout. Parking place width must be matched to the width of driveway suitable for driveability. (RT 98-12367 2016, 3)

Table 1:Dimensioning instructions of parking places (RT 98-12367 2016, 3)

Normal parking space width	2,5 m
Space for family	2,8 m
Parking place for short time	≥ 2,6 m
Parking space from the wall	≥ 2,8 m

Place for people with reduced mobility	3, m
The length of parking space	5,0 m
Estimated area of parking place	25……30 m2
Space for pedestrian	≥ 1,8 m
Parking place height	min 2. 2 m

4.1.2 Parking place location

While designing parking places different factors need to be considered such as the physical condition of the users. The parking places for people with physical disabilities should be closer to the elevator or the entry of the parking hall. It is recommended that every 50 parking places should have at least 2 places for people with physical disabilities and then for larger parking facilities it is recommended to put one parking place for people with physical disabilities after every 50 parking places. (RT 98-12367 2016, 5). The floor of parking facilities must be plane without any slope,and in case this is unavoidable the slope must be less than 2%.

The following figure shows the insertion of parking places into the building layout. The parking places have been inserted in the building layout at 60 degrees.

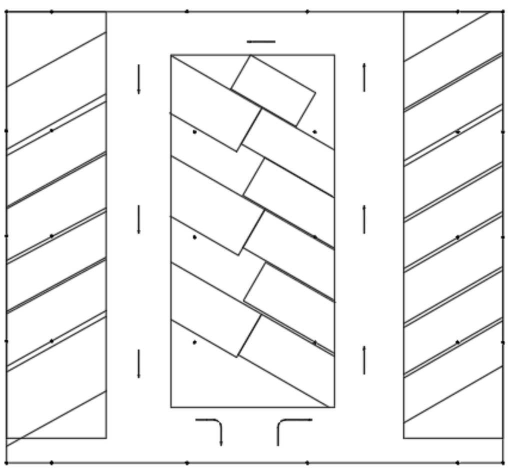


Figure 2: Parking facilities are designed so that every free space can be easily accessible.

4.1.3 Insertion of columns in parking places

For efficient use of parking places, the pillars are recommended to be placed between the rows of the parking places. For the perpendicular parking places, the span of the beam should be at least 17m, whereas for the diagonal parking places the span of the beam can be shorter than 17m. For long-term parking place facilities, the column must be placed at one meter from the wall when there is a column in every three parking places. The driving direction of diagonal parking places must be always in one-way whereas perpendicular parking places are always in two ways. (RT 98-12367 2016, 4).

4.2 Classification of loads

The loads can be classified into the following categories cccording to their directions

1. Vertical loads eg: dead loads, snow loads and imposed loads
2. Horizontal loads eg :wind loads
3. Longitudinal loads eg: Braking forces this is considered in special case design.

This thesis will only anlyse vertical loads, because the main part of parking garage is below the ground level. Therefore the effect of wind loads on structures can be neglected.

4.2.1 Permanent loads

The self-weight of structural and non-structural members should be taken into consideration in combinations of actions as single action.The self-weight of construction work includes weight of structure and non-structure of elements (SFS-EN 1991-1 2002, 13).

4.2.2 Imposed loads

In the situation when the imposed loads are acting simultaneously with other variable actions such as snow and machinery the total imposed load will be considered as single action. On the roofs the imposed loads, wind actions and snow loads cannot be applied simultaneously (SFS-EN 1991-1 2002, 15). The Table 2 shows the value of imposed loads to consider according to the categories of the vehicles.

Table 2:q_k maybe taken within range of 1.5kN/m2 to 2.5kN/m2 (SFS-EN 1990 2002, 6.4.3.2)

Categories of traffic areas	q_k [kN/m^2]	Q_k [kN]
Category F Gross vehicle weight: ≤ 30 kN	q_k	Q_k
Category G 30 kN < gross vehicle weight ≤ 160 kN	5,0	Q_k

4.2.3 Snow load

The value of snow load depends on the place where the building is located. For parking facility with open flat roof, the snow load calculations procedures can be the same as that of the monopitch roof. The snow load can be calculated using the following expression (SFS-EN 1991-1-3 2003, 5.2)

$$S_d = \mu_i * C_e * C_t * S_k \tag{1}$$

Where:

S_d: Is the design of value of snow loads

μ_i: Is the snow shape coefficient. For monopitch roof the values of μ_i are shown in figure3.

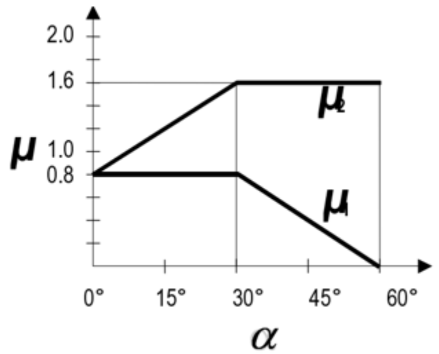


Figure 3: Snow load shape coefficients for monopith roof (SFS-EN 1991-1-3 2003, 5.3.2)

C_e: Is the exposure coefficient. The values of C_e for different topographies are shown in the table3.

Table 3: Recommended values of C_e for different topographies.

Topography	C_e
Windswept [a]	0,8
Normal [b]	1,0
Sheltered [c]	1,2

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

[b] *Normal topography*: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.

[c] *Sheltered topography*: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.

S_k:Is the characteristic values of snow load on ground. The values of S_k are given in the figure 4.

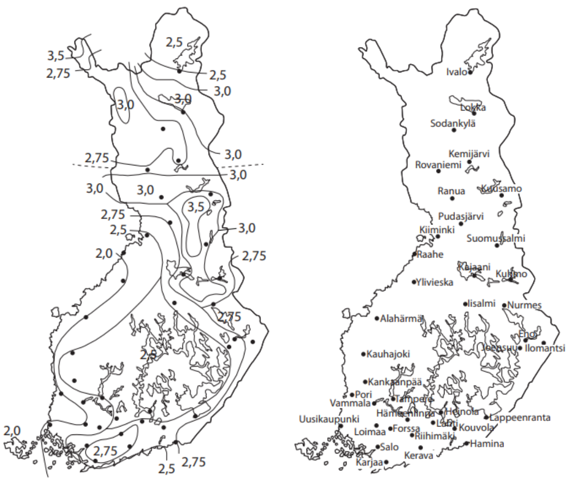


Figure 4: Snow load changes with place where the building is located (RT RakMK-21716 2016).

4.3 Loads combinations

The loads combination can be done in two ways:

1. Ultimate limit state (ULS)
2. Serviceability limit state (SLS)

4.3.1 Ultimate limit state (ULS)

In ultimate limit state the structure is analysed considering the safety of the people and the structure. In this thesis all structure analysis will be carried out in ultimate limit state (ULS).The structure analysis in ultimate limit state include:

1. Loss of equilibrium of structure (EQU)
2. Internal failure or excessive structural deformation (STR)
3. Failure or excessive deformation of ground (GEO)
4. Failure through time dependent effects (FAT) e.g fatigue (SFS-EN 1990 2002, 6.4.1).

In normal situation of structure analysis, the load combination is given by the following equations (SFS-EN 1990 2002, 6.4.32):

$$1{,}15\ KFI\ Gkj\ +\ 1{,}5\ KFI\ Qk,1\ +\ 1{,}5\ KFI\ \Sigma\ \psi 0,i\ Qk,i \qquad (2)$$

$$1{,}35\ KFI\ Gkj \qquad (3)$$

Where:

- G_{kj}:Is permanent load
- $Q_{k,1}$:Is main imposed load
- $Q_{k,i}$:Is other imposed loads
- Ψ: Is coefficient factor for imposed loads. The table4 shows the values of Ψ according to the categories of buildings.

Table 4: Recommended values of Ψ factors for buildings (SFS-EN 1990 2002, A.1.2.2)

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

4.3.2 Serviceability limit state (SLS)

In Serviceability limit state the design emphasizes the functionality of the structure or the structural members under normal use and normal condition. In addition the design examines deformation that can affect the appearance of structure,and the comfort of the users.

In serviceability limit state the design analyses the limit functional effectiveness of the structure and the damage that is likely to affect the durability of the structure (SFS-EN 1990 2002, 6.5.2). For serviceabiity limit state the load combination can be divided into 3 parts:

1. **Characteristic combination:**

$$G_{kj}+Q_{K,1}+\Sigma\ \Psi 0,I\ Q_{k,I} \quad \text{(SFS-EN 1990- 14b)} \qquad (4)$$

2. **Frequent combination**

$$G_{kj} + \psi 1,1\ Q_{k,1} + \Sigma\ \psi 2,i\ Q_{k,i} \quad \text{(SFS-EN 1990- 15a)} \qquad (5)$$

3. Quasi-permanent combination

$$G_{kj} + \Sigma\ \psi 2,i\ Q_{k,I} \qquad \text{(SFS-EN 1990- 15b)} \qquad (6)$$

Where:

G_{kj} :Permanent load

$Q_{k,1}$:Main imposed load

$Q_{k,i}$:Others imposed load applied on same time

$\psi 1,1$:Main imposed load combination factor

$\psi 2,i$:Other imposed load combination factor

4.4 Design of ground floor.

The ground floor can be designed as ground slab or pile slab.The efficient design between these two types of slab depend on the bearing capacity of subsoil on which the building is going to be build.

4.4.1 Ground slab and Pile slab

The ground floor structure of the parking garage can be implemented in the same way as other concrete floors. It can be either as ground slab such as hollow core slab or cast in place concrete slab.

The pile slab consist of reinforcing subsoil by piling and then the concrete slab is set on the piles. The numbers of piles depend on the subsoil properties as well as the magnitude of forces that come from the slab to the subsoil. The pile slab is used when the bearing capacity of subsoil is not enough to suport all loads that come from the slab.

Ground floor concrete slab is possible to use on the subsoil where the bearing capacity of ground is high enough to resist all imposed loads which come from the slab,because all loads that come from slab are transfered directly to the subsoil. For this reason,the concrete slab design should meet bearing capacity of the subsoil

For the ground slab, the concrete is poured against the ground, when to subsoil is frosty the insulation is installed against the ground and concrete slab is poured on it (Kailajärvi 2019, 6). When the soil is frosty it must be removed and replaced by a gravel which has a higher bearing capacity. The filter cloth is put between frosty subsoil and gravels to prevent the frosty soil to form a mixture with gravel. The frostyness of subsoil is usually mentioned in subsoil survey report done by geoengineer. The maximum diameter of gravels can be up to the 2/3 the thickness of ground floor structure. The thickness of the gravel layer depends on the desired bearing capity of subsoil (InfraRyl 2021/1 2021).

4.4.2 Designing procedure

The designing procedure of slab is done by considering one meter part of slab as a beam of one meter of width. All procedures in designing of the beam are also applied in the design of slab. However the calculation procedures of moment for slab may differ from that one of the beam depending on type of slab.

4.4.3 One way slab

One way slab is a slab which is supported by two beams in the opposite direction to carry load in one direction. It has a length and breadth ratio which is equal or greater than two. One way slab is likely to bend in one direction, which means that it requires main reinforcement only in bending direction.

4.4.4 Two ways slab

Two ways slab is a slab which is supported in four directions. It has a length and breadth ratio which is less than two and bending is likely to happen in both directions. Two ways slab requires main reinforcement in both directions.

4.5 Design of beam

In this construction of a small concrete parking garage, all beams are precast concrete of two different sizes.The first beam has a span of 9m and cross section of 480mmx880mm, whereas the second has a span of 3m and cross section of 480mmx680mm. All beams will be made in the factory and adjusted on construction site. The beam will be supported at both ends by a column of cross section dimensions of 480mmx480mm.

The design of beam will be done in the following steps:

1. Determining the working life of structure
2. Determining concrete class
3. Determining the loads combinations
4. Determining the shear load loads and bending moment on beam
5. Calculating concrete cover
6. Calculating beam resistance capacity to bending moment
7. Cracking possibilities of concrete and rebars spacing in beam.
8. Calculating beam resistance capacity to shear force
9. Determing the design shear stress
10. Determing the design shear stress
11. Shear reinforcement calculation

4.5.1 Determining the working life of parking garage

The parking garage and other civil engineering structures are in class 5, which means that their working life design is 100 years. The structures must be designed so that the deterioration over its design working life would not impair the performance of the structure, considering its environment and the anticipated level of maintenance (SFS-EN 1990 2002, 2.3). Classifications of working like of structures according to Eurocode are shown in the table 4.

Table 5: Classification of working life according to Eurocode (SFS-EN 1990 2002, 2.3)

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures (1)
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures

4.5.2 Determining concrete class.

Reinforced concrete structures are vulnerable to the corrosion initiated by carbonisation reaction or corrosion initiated by chlorides. The concrete with water/cement ratio below 0.4 will provide efficient resistance to the ingress of carbon dioxide that would cause carbonisation.The efficient water/cement ratio would improve the protection of the steel reinforcement from corrosion and this will increases the quality of the structure and its working life assuming that no other deterioration mechanism occurs. The table 6 shows the classification of concrete classes according to where the concrete is intened for use.

Table 6: Reinforced concrete classes and their using conditions (SFS-EN 206 2021, 4.1.1).

2	Corrosion induced by carbonation	
Where concrete containing reinforcement or other embedded metal is exposed to air and moisture, the exposure shall be classified as follows:		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity; Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact; Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity; External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2

The intermediaite floor of parking garage is the same time the roof building. The structure is susceptible to the rain, therefore the suitable concrete class to be used in this condition is XC3. Freezing and unfovarable weather can reduce the working life of a structure by increasing the risk of corrosion of the steel reinforcement. According to the table below the suitable concrete to use in this condition is XF3. The table 7 shows classification of concrete according to the environmental conditions.

Table 7: Freezing/thaw attach with or without de-icing agent (SFS-EN 206 +A2 2021, 4.5).

5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing

4.5.3 Determining the shear loads and bending moment on a beam

The shear force is the algebric sum of all vertical forces acting on either side of the point of the beam. The bending moment at any point of a loaded beam is the sum all of moments due to all verticals forces acting on either side of the point on the beam.

a. Shear force

The maximum shear force of simple supported and uniformly distributed load can be calculated as:

$$V_k = \frac{w * L_b}{2} \tag{7}$$

Where:

V_k:Shear force on the beam

W:Distributed load on the beam

L_b:Span of the beam

b. Bending moment

The maximum bending moment of a simple supported beam and uniformly distributed load occurs at the middle span of the beam, and it can be calculated using the expression 7:

$$M_{max} = \frac{\mathrm{W} * L_b^2}{8} \tag{8}$$

Where:

M_{max}: Maximum bending moment

L_b: Length of beam

4.5.4 Concrete cover

The concrete cover helps the transmission of bond forces safely, and it also protects steel against corrosion and provides to the structure an efficient fire resistance. Normal concrete cover can be calculated as follow (SFS-EN 1992-1-1 2002, 4.4.1.1)

$$C_{nom} = C_{min} + \Delta \mathrm{C}_{\mathrm{dev}} \tag{9}$$

Where:

C_{nom}: Normal concrete cover

$\mathrm{C}_{\mathrm{min}:}$ Minimum concrete cover

ΔC_{dev}: Allowance in design for deviation

According to Finnish national standard SFS the allowance in design for deviation will be calculated using the table 8:

Table 8: Deviation factor ΔCdev (SFS-EN 1992-1-1 2002, A.2.1)

h or *b* (mm)	Reduced deviations (mm)	
	Cross-section dimension $\pm\Delta h$, Δb (mm)	Position of reinforcement $+\Delta c$ (mm)
≤ 150	5	5
400	10	10
≥ 2500	30	20

Note 1: Linear interpolation may be used for intermediate values.
Note 2: $+\Delta c$ refers to the mean value of reinforcing bars or prestressing tendons in the cross-section or over a width of one metre (e.g. slabs and walls).

The Maximum value of C_{min} satisfying the requirement for both bond and environmental condition shall be used as described in the equation 10

$$C_{min} = max\{ C_{min,b};\ C_{min,dur} + C_{min,\Upsilon} - C_{min,st} - \Delta C_{dur}\ ;\ 10mm\} \qquad (10)$$

Where:

$C_{min,b}$: Minimum cover due to bond requirement

$C_{min,dur}$:: Minimum cover due to environmental condition

$C_{min,\Upsilon}$ :: Additive safety element

$C_{min,st}$: Reduction of minimum cover for use of stainless steel.

ΔC_{dur}:: Reduction of minimum cover for use of additional protection

The value of minimum cover $C_{min,b}$ depends on type of reinforcement used, the table 9 can be used in the calculation of $C_{min,b}$.

Table 9: Minimum required value of $C_{min,b}$. (SFS-EN 1992-1-1 2002, 4.4.1)

Bond Requirement	
Arrangement of bars	Minimum cover $c_{min,b}$*
Separated	Diameter of bar
Bundled	Equivalent diameter (ϕ_n)(see 8.9.1)
*: If the nominal maximum aggregate size is greater than 32 mm, $c_{min,b}$ should be increased by 5 mm.	

The minimum concrete cover $C_{min,dur}$ due to the environment can be found using this table, however for 100 years working life design the structural class should be increased by 2 (SFS-EN 1992-1-1 2002, 4.4.1, T4.5N).

Table 10:The minimum concrete cover Cmin, dur required for durability of reinforcement (SFS-EN 1992-1-1 2002, 4.4.1).

Environmental Requirement for $c_{min,dur}$ (mm)							
Structural Class	**Exposure Class according to Table 4.1**						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1	XD2 / XS2	XD3 / XS3
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S6	20	25	35	40	45	50	55

4.5.5 Design of concrete beam under bending moment

The figure 5 shows the stress and strain distribution into the beam, for a rectangular cross section beam. The information from in this picture is only valid for concrete class under C50/60 (www.elementtisuunnittelu.fi 2021).

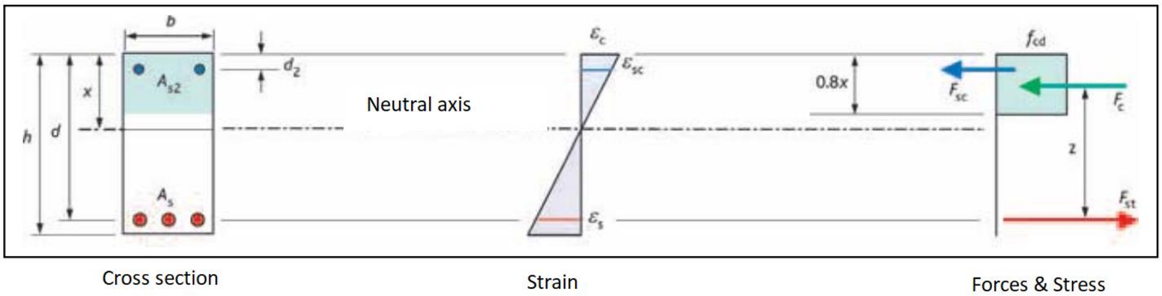


Figure 5: Effective stress distribution in accordance with Eurocode EN 1992-1-1 for concrete class less than C50/60 for rectangular cross section beam (www.elementtisuunnittelu.fi 2021).

The primary step in analysing beam is to check if the beam`s cross section is enough to carry all loads. This is done by comparing the coefficient of friction between the tendons and their ducts (μ) and then design coefficient of friction between the tendons and their ducts (μ_d).

μ: Is the coefficient of friction between the tendons and their ducts, can be given by the expression 11:

$$\mu = \frac{Med}{b * d^2 * f_{cd}} \tag{11}$$

μ_d:Design coefficient of friction between the tendons and their ducts can be given by solving the equation 12 or μ_d can be found from the table 11:

$$\mu_d = 0{,}960\delta - 0{,}264\delta^2 - 0{,}371 \; when \; \delta \leq 1, \; \delta = \frac{z}{d} \tag{12}$$

Table11:The table below shows the value of µd in function of sigma; however, this table can only be used for the rectangular cross section beam (www.elementtisuunntittelu.fi 2021)

µ	z/d	µ	z/d
0,07	0,964	0,15	0,919
0,08	0,958	0,16	0,913
0,09	0,953	0,17	0,906
0,10	0,947	0,18	0,900
0,11	0,942	0,19	0,894
0,12	0,932	0,196	0,890
0,13	0,930		
0,14	0,925		

Where:

δ: Increment/ Redistribution factor

b: Width of beam cross section

M_{Ed}:Design value bending moment on the beam

d: Effective depth of beam, d can be calculated as:

$$d = h - \frac{\Phi h}{2} - \frac{\Phi 1}{2} - C_{nom} \tag{13}$$

f_{cd}: Design value of concrete compressive strength,

$$f_{cd} = \frac{f_{ck}}{\Upsilon_c} \tag{14}$$

f_{cd}:Design value of concrete compressive strength

Υ_c: Concrete partial safety factor

Φh:Diameter of tension reinforcement bar

$\Phi 1$:Diameter shear reinforcement bar

When $\mu \leq \mu_d$ calculation proceed by checking the minimum permissible cross-section area of reinforcining bars, when the condition is not fullfilled the beam cross section should be increased. (SFS-EN 1992-1-1, 9.2.11, 2002).

$$A_{smin} = 0{,}26 * \frac{f_{ctm}}{f_{yk}} * b_t * d \ \geq 0{,}0013 * b_t * d \tag{15}$$

Where:

A_{smin}:Is the minimum permissible cross-sectional area of reinforcing bars

b_t:Is the width of tension zone

f_{yk}:Is the characteristic yield strength of reinforcement

f_{ctm}:Is the mean value of axial tensile strength of concrete

The minimum permissible cross sectional reinforcement area can also be calculated by using the expression 16 (Leskelä 2008, 381):

$$A_{smin} = 0{,}5 * \frac{f_{ctk}}{f_{yk}} * b * h \tag{16}$$

Where:

f_{ctk}:Characteristic value of axial tensile strength

b:width of beam cross section

h:Beam cross section depth

The minimum required cross-sectional reinforcement area can be calculated by using the expression 17:

$$As = \frac{Med}{Z * f_{yd}} \tag{17}$$

Where:

A_s: Is the minimum required cross-sectional reinforcement area

f_{yd}: Is the design yield strength of reinforcement, $f_{yd} = \frac{f_{yk}}{\Upsilon_s}$

Z: Is the lever arm of internal forces, which can be calculated by using the equation 18:

$$Z = \frac{d}{2}(1 + \sqrt{1 - 2\mu}) \tag{18}$$

The minimum permissible cross sectional reinforcement area ($\mathrm{A_{smin}}$) should be less or equal than the required cross sectional reinforcement area (As), this means that the maximum number from those two equations should be considered.

If $\mu \geq \mu_d$,then beam cross section must be increased or beam should be checked under compression. In this case, concrete has sufficient compressive strength, however while designing the beam in some cases the compression part of beam requires to be reinforced by steel. The calculations are done into the following steps:

1. **Calculation of lever arm using the value μ_d**

The lever arm of internal forces in the part of beam that is subjected to compression can be given by the following expression:

$$Z_c = \frac{d}{2}(1 + \sqrt{1 - 2\mu_d}) \tag{19}$$

2. **Calculating minimum required cross sectional compressive reinforcement area.**

The minimum required cross sectional compressive reinforcement area can be calculated as follow:

$$A_{s2} = \frac{(\mu - \mu_d) * f_{cd} * b * d^2}{\sigma_{sc} * (d - d_2)}, \text{Where } \sigma_{sc} = 700MPa * \frac{x - d_2}{x} \leq f_{yd} \tag{20}$$

Where:

X: Is the distance from cross section bottom to the neutral axis, for rectangular cross section beam is $\frac{h}{2}$

d_2: Is the effective depth of compression reinforcement, the distance from the top of beam cross section to the centre of compression reinforcement.

3. **Minimum required reinforcement area**

The minimum required cross sectional reinforcement area can be calculated by using the equation 21 (www.elementtisuunnittelu.fi 2021):

$$As = \frac{\mu_{d*} f_{cd} * b * d^2}{Z_c * f_{yd}} + A_{s2} \tag{21}$$

4.5.6 Deep beams

According to SFS-EN 1992,5.3.1 a deep beam is defined as the beam where the span is not less than 3 times the overall depth of the cross section, however according to Leskelä, 2008, p. 381 all beams of cross-section $\geq 800\text{mm}$ should be considered as a deep beam in reinforcement.

The reinforcement bars which are mostly concentrated in the bottom of the beam can not protect efficiently the concrete from cracking due to the tension force, even though maximum tension forces occur at the bottom of the beam. There are also small tension forces in flange of the beam that can cause cracking in the concrete, however this can be prevented by providing additional reinforcement with maximum bars spacing of 300mm. The additional cross-sectional reinforcement can be given by using the expression 22: (Leskelä 2008, 381).

$$\mathrm{A_s} \geq 0{,}12 * \frac{\mathrm{f_{ctk}}}{\mathrm{f_{yk}}} * \mathrm{b} * \mathrm{h} \tag{22}$$

As:Additional cross-sectional reinforcement area. *As* will be distrubuted over the depth of the beam. The figure 6 shows the distribution of additional cross-sectional reinforcement area over the depth of the beam.

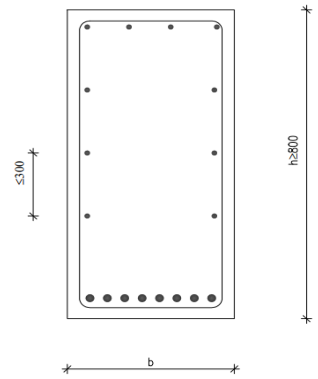


Figure 6: Additional reinforcement for deep beam when h≥800mm.

4.5.7 Anchorage of bottom reinforcement at the end of support.

Anchorage of bottom reinforcement at the end of support for simple supported beam with pinned or fixed end, the area of bottom reinforcement should be at least β_2 of the area of steel provided in the span of the beam. The recommended to use value of $\beta_2 = 0{,}25$ (SFS-EN 1992-1-1 2002, 9.2.14)

4.5.8 Beam resistance capacity to shear force

The shear reinforcement should form an angle alpha(α) which is between 45^0 and 90^0 to the longitudinal axis of the structural element. The shear reinforcement may consist of a combination of links enclosing the longitudinal tension reinforcement and the compression zone. The following figure shows the example of reinforcement (SFS-EN 1992-1-1 2002, 9.2.2).

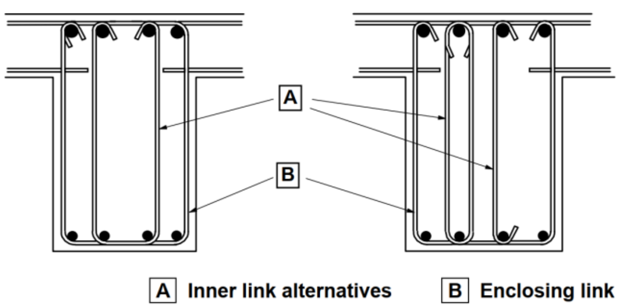


Figure 7: Example of shear reinforcement according to eurocode2. 1992.1.1

The shear reinforcement should be effectively anchored. A lap joint on the leg near the surface of the web is permitted and provided so that the link is not required to resist torsion. At least β_3 of necessary shear reinforcement should be in form of links. The recommended value β_3 is 0.5 (SFS-EN 1992-1-1, 9.2.11, 2002, 9.2.2)

4.5.9 Determing the design shear stress

V_{Edmax} is the maximum design value of the shear force. For a simply supported beam is the same as reaction support for the beam. The design value of shear stress V_{Ed} calculated at distance d from the edge to the support is given by using the equation 23:

$$V_{Ed} = \frac{V_{Edmax}}{Z * b_w} = \frac{V_{Edmax}}{0{,}9 * d * b_w} \tag{23}$$

where:

Z: Is the lever arm for internal forces, can be estimated as 0,9*d
d:Is the effective depth of beam cross section
V_{Ed}:Is the design value of shear force
b_w:Is the width of beam cross section

4.5.10 Resistance capacity of beam to shear stress.

The shear stress resistance capacity of a rectangular cross section beam depends on the type of concrete and its can be calculated by using the equation 24.

$$V_{Rd,max} = \frac{0{,}6 * b_w * Z * f_{cd}}{(cot\theta + tan\theta)}, where: 1 \leq cot\theta \leq 2 \tag{24}$$

The value of θ depends on shear force capacity, however in calculating the shear resistance capacity of the beam, it is recommended to use the minimal value of θ which is $21{,}8^0$ so that the capacity of beam cannot be overestimated. The beam reaches the higher resistance capacity when θ is 45^0. The design shear stress should be less than the shear stress resistance capacity of the beam, and if this condition is not satisfied then the beam´s cross section should be increased. (www.elementtisuunnittelu.fi 2021)

4.5.11 Shear reinforcement calculation

The minimum required shear reinforcement is given by the solving the expressions 25:

$$\frac{A_{sw}}{S} = \frac{V_{Ed}}{f_{yd} * cot\theta} \tag{25}$$

Where:

S:Is the unit length of beam (m)

A_{sw}:Is the required shear reinforcement cross-sectional area

The minimum permissible shear reinforcement cross-sectional area over one meter unit of length of the beam can be calculated by using the expression 24.

$$\frac{A_{sw,min}}{S} = b_w \frac{0{,}08 * \sqrt{f_{ck}}}{f_{yk}}, and\ bars\ spacing\ S_{1,max} \leq 0{,}75 * d \tag{26}$$

Where:

$A_{SW,min}$:Minimum permissible shear reinforcement cross-sectional area

$S_{1,max}$: Maximum shear reinforcement spacing

4.6 Design of Column

The position of column in the building frame affects its force magnitude, the edge columns have more bending moment than the columns in the middle of the perimeter.The eccentricity of edge and middle columns may differ due to their differents locations in the building frame (Leskelä 2008, 417).

4.6.1 Principles of column design

The columns should be designed in ultimate limit state (ULS), because the most important function of the columns is to transmit the loads from the other structures to the foundations. In the design of column two issues must be noted:

1. Cross-section of column must have sufficient resistance to stresses applied on it.
2. The pilar has sufficient rigidity so that it does not impair the overall stability of the building frame and the effect of geometric nonlinearity on forces does not become significant (Leskelä 2008, 418).

4.6.2 Slenderness of column and effective length of isolated members

The slenderness of the column (λ) is the ratio of effective length of the column to the least radius of gyration. This value represents the capacity of the column to carry the axial load. It can be determined by its length and cross-section, as it is shown in the expression 27:

$$\lambda = \frac{l_0}{i} \ where \ i = \frac{I_c}{A_c} \tag{27}$$

Where:

l_0: Buckling length of column, which depends on column boundary condition. The buckling length of column depends on it boundarie conditions as it shown in the Figure 8.

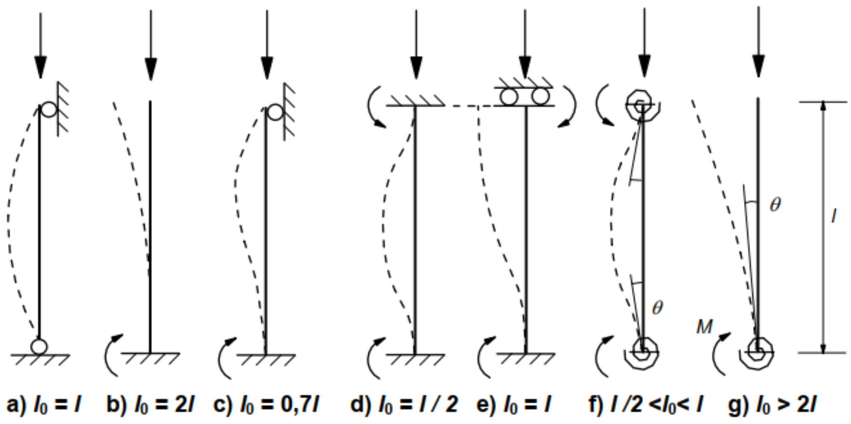


Figure 8: The example of different buckling modes of columns and their corresponding lengths (SFS-EN 1992-1-1 2002, 5.8.3.2)

i: Radius of gyration

I_c: Second moment of area, which is given by:

$$I_c = \frac{b * h^3}{12} \tag{28}$$

A_C: Area of column cross-section, b and h are width and height of column respectively.

4.6.3 Slenderness cretion for isolated members

The column slenderness ratio should be less than λ_{min}, otherwise the column should be considered as slender and second order effects must be considered. The minimum slenderness ratio is given by the expression 29 (SFS-EN 1991-1 2002, 5.8.3.1)

$$\lambda_{min} = \frac{20 * A * B * C}{\sqrt{n}} , Where, A = \frac{1}{1 + 0.2 * \Phi_{eff}}, \tag{29}$$

$$B = \sqrt{1 + 2 * \omega} , C = 1.7 - r_m \ and \ n = \frac{N_{Ed}}{A_c * f_{cd}}$$

if Φ_{eff} is not know the value of can be taken as 0.7

ω Is the mechanical reinforcement ratio, if ω is not know the value of B can be taken as 1,1. ω is calculated from the following expression:

$$\omega = \frac{A_s * f_{yd}}{A_c * f_{cd}}$$

A_s:Is the total area longitudinal reinforcement

A_C:Is the cross-sectional area of column

r_m:Is the moment ration, if r_m is not known the value of C can be taken as 0.7

4.6.4 Eccentricity of column

The column eccentricity shall be calculated according to the value of slenderness ratio. The slenderness should not exceed 140 ($\lambda \leq 140$), the column shall be classified as slender if $\lambda \geq 25$) or rigid if ($\lambda \leq 25$. The design value of eccentricity e_d is calculated according to concrete standards, e_d can be found from the following expression (Leskelä 2008, 421).

$$e_d = e_a + e_2 + e_0 \tag{30}$$

e_a: Basic eccentricity, which comes from the expression 31:

$$e_a = \frac{h}{20} + \frac{l_0}{500}\ , Where, \frac{h}{20} \leq 50mm \tag{31}$$

Basic eccentricity should be always considered even when the column is pinned.

e_2: Additional eccentricity, and can be calculated as:

$$e_2 = \left(\frac{A_{C*} {l_0}^2}{21025 I_C}\right) * h = \left(\frac{\lambda^2}{145^2}\right) * h \tag{32}$$

e_0: Eccentricity of the load

$$e_0 = \frac{M_{Ed}}{N_{Ed}} \tag{33.a}$$

In case the external moment in not constant, eccentricity of load can be calculated as:

$$e_0 = max\ (0.6 * e_{01} + 0.4 * e_{01}\ ;\ 0.4 * e_{01}) \tag{33.b}$$

The design value of eccentricity can be calculated as:

$$e_d = e_a + e_2 + e_0 \tag{34}$$

According to Leskelä 2008, p.419 the basic eccentricity always occurs even though e_0=0 and should always be considered because it includes the effect of casting position and uneven shrinkage as well as the eccentricity caused by installation of the reinforcement which cannot be eliminated in manufacturing process.

4.6.5 Column compressive strength

In design of rectangular cross section columns, the same mechanism that was used in beam design can also be used in column design. However, for column the depth of cross section is designated according to the position of normal force.

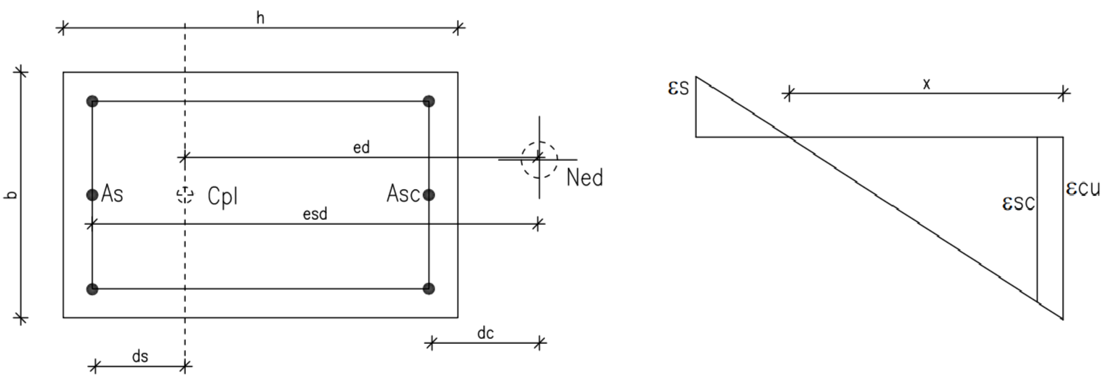


Figure 9: Stress distribution in columns (Leskelä 2008, 214)

The column compressive strength is sufficient to the normal load when the following condition is fulfilled: $N_{Ed} \leq N_{Rd}$

where:

N_{Ed}: Is the normal force design value

e_d: Is the eccentricity of axial load from where plastic moment occur for symmetrical reinforced column C_{pl} is the same as centre of gravity of cross section.

N_{Rd}: Is the resistance capacity of column which can be found from the following expression,

$$N_{Rd} = \frac{N_{Rd,0}}{1 + \frac{e_d}{e_{s.b}}\left(\frac{N_{Rd,0}}{N_{Rd,b}} - 1\right)}, \tag{35}$$

$$by\ using\ Whitney\ method, \qquad N_{Rd} = \frac{A_{sc} * f_{sd} * (d - d_2) + 0{,}4 * f_{cd} * b * d^2}{e_d + d - \frac{h}{2}}$$

$$N_{Rd.0} = (A_s + A_{sc}) * f_{sd} + b * h * f_{cd} \tag{36}$$

$N_{Rd.0}$:Is the characteristic value of column resistance under pure compressive load (in this case eccentricity of load is zero).

A_{sc}:Is the compressive column cross section area

A_s:Is the tensile reinforcement cross-sectional area.

$e_{s.b}$:Is the eccentricity of normal force from tensile reinforcement, it can be given from the following expression (Leskelä 2008, 214):

(37)

$$e_{s,b} = \frac{f_{cd*} y_b * b * \left(d - d_s - \frac{y_b}{2}\right) + A_{sc} * f_{sd}(d - d_c - d_s) + A_s * f_{sd} d_s}{N_{Rd,b}}$$

$$x_b = \frac{d * \mathcal{E}_{cu} * E_s}{f_{sk} + \mathcal{E}_{cu} E_s}, \qquad y_b = \lambda x_b, \qquad d_s = d - x_b$$

d: effective depth of column, for rectangular cross section

$$N_{Rd,b} = \lambda * b * d * f_{cd} \frac{\varepsilon_{cu} E_s}{f_{sk} + \varepsilon_{cu} E_s} + (A_{sc} - A_s) * f_{sd}, \tag{38}$$

$$\varepsilon_{cu} = 0{,}0035, \qquad \lambda = 0.8$$

for column which is symmetrically reinforced the expression can be written as:

$$N_{Rd,b} = \lambda * b * d * f_{cd} \frac{\varepsilon_{cu} E_s}{f_{sk} + \varepsilon_{cu} E_s} \tag{39}$$

4.6.6 Column resistance capacity to bending force

Tranverse reinforcement has a major role in increasing resistance capacity of the column to bending because, the insuficient transverse reinforcement will not prevent effectively the concrete from cracking when the column buckles. This will reduce the columns resistance capacity to bending as well as compressive force and also to the durability of the structure in general.

It is recommended to use at least the minimum cross-sectional area of the column transverse reinforcement. (Leskelä 2008, 213). The recommended diameter of transverse reinforcement should be at least 0.25 times the diameter of the longitudinal reinforcement bars.

4.6.7 Minimum reinforcement cross-sectional area

The minimum permissible reinforcement cross-sectional area is given by solving the expression 40:

$$A_{sw,min} = \omega * b * h * \frac{f_{cd}}{f_{yd}} \tag{40}$$

Where:

$A_{sw,min}$:Is the minimum reinforcement cross-sectional area *b*:Is the column cross-section width

h:Is the cross-section depth

ω:Is the relations between relative normal force (ν) and the relative moment(μ), see appendix 1.

4.6.8 Longitudinal reinforcement

The recommended minimum diameter of longitudinal reinforcement in a column depends on each country´s national annex, however the recommended minimum value is 8*mm*. The total amount of required longitudinal reinforcement cross-sectional area should be greater than $A_{s,min}$. and should not be greater than $A_{s,max}$. The reinforcement cross-sectional area can be calculated from the equation 41 (SFS-EN 1992-1-1 2002, 9.5.2)

$$A_{s,min} = max\left(\frac{0.1 * N_{Ed}}{f_{yd}}; 0.002 * A_{c0}\right) \tag{41}$$

4.6.9 Transverse reinforcement

The transverse reinforcement diameter (links, loops or helical spiral reinforcement) should be the great value between 6mm and the quarter of the maximum diameter of longitudinal reinforcement bars. (SFS-EN 1991-1 2002, 9.5.3).

The spacing of transverse reinforcement should not be greater than $S_{cl,max}$, the recommended value is minimum of the following values: (SFS-EN 1992-1-1 2002, 9.5.3)

4. 20 times the minimum diameter of longitudinal bar
5. The lesser dimension of column
6. 400mm

4.7 Design of intermediate floor

The intermediate floor structures for parking garage can be constructed using precast elements or construction site casting slab, however this thesis will be using precast elements as construction materials.

The intermediate floor materials can be hollow core slab and concrete casting slab of 80mm of thickness with reinforcement of 8Tk 150. In this case, the reinforcement of casting slab is to prevent the concrete from shrinking and cracking while hardening (Palolahti 2011, 16). It is possible to use TT slab instead of hollow core slab but since the parking is going to be used only by passenger car the imposed load is only 2,5kN/m2 (SFS-EN 1990 2002, 6.4.3.2). Based on the value of imposed load, hollow core slab would be much more economically sustainable and efficient option than TT slab. The figure 9. shows the bearing capacities of hollow slabs according to their spans and cross sections depths.

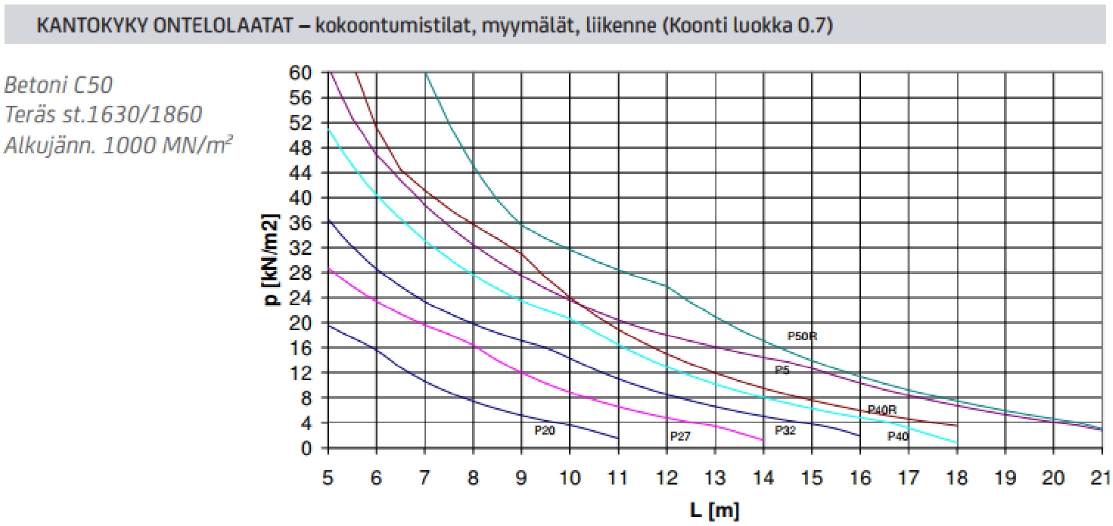


Figure 10: Hollow core slab bearing capacity when concrete class is C50, and exposure class is XC1 (Parma 2018, 14)

5 Conclusion

The aim of the thesis was studying and providing educational information about the design of concrete load-bearing structures. This thesis report outlined the design procedures of the concrete beam and concrete column which was the main objective of this thesis. It was impossible to examine accurately the design for some load-bearing structures such as foundation and ground floor, due to lack of information on where the parking garage is going to be build.

This thesis focused on the design of structures taking into consideration only the strength of concrete, however a structure designer`s decisions must not only be based on the strength of materials but also on the feasibility of the design as well as cost that can come from the chosen designing solution.

It is the responsibility of structure designer to assess every design decision from different perspectives, for effective design must be easy to implement, able to provide result of high quality at low cost.

References

InfraRyl 2021/1. 2021. *Päällysrakenteen osat ja radan alusrakennekerrokset.* Rakennustieto.

Kailajärvi, Karri. 2019. ”Diplomityö: Pysäköintilaitosten lattioiden pintakorjausmateriaalit ja-menetelmät.” Tampere.

Leskelä, Matti V. 2008. *Betonirakenteiden suunnittelu ja mitoitus 2008.* Suomen betoniyhdistys r.y.

Palolahti, Tuomas. 2011. *Pienrakentajan betoniopas.* Helsinki: Betoniteollisuus ry.

Parma. 2018. *parman ontelaatastot suunnitteluohje.*

RT 98-12367. 2016. *Pysäköintilaitos (Kuvat).* htttp://www.rt-kortisto.fi.

—. 2016. *Pysäköintilaitos.* http://www.rt-kortisto.fi.

RT 98-12367. 2016. *Pysäköntilaitokset.* Rakennustieto Oy. http://www.rt-kortisto.fi.

RT 98-12367. 2016. ”Pysäköntilaitokset.” http://www.rt-kortisto.fi.

RT 98-12367. 2016. ”Pysäköntilaitokset.” http://www.rt-kortisto.fi.

RT 98-12367. 2016. ”Rakennustito Oy.” http://www.rt-kortisto.fi.

RT RakMK-21716. 2016. ”YMPÄRISTÖMINISTERIÖN ASETUS LUMIKUORMIA KOSKEVISTA KANSALLISTA VALINNOISTA.”

SFS-EN 1990. 2002. *Eurocode, Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode, Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode: Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode: Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode: Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode: Basics of structure design.* Finnish standard association SFS.

SFS-EN 1990. 2002. *Eurocode: Basics of structure design.* Finnish standard association SFS.

SFS-EN 1991-1. 2002. ”Eurocode. Basics of structure design.”

SFS-EN 1991-1. 2002. ”Eurocode. Basics of structure design.”

SFS-EN 1991-1-3. 2003. ”Eurocode 1: Actions on structures-part 1-3: General actions-snows loads.”

SFS-EN 1992, 5.3.1. ei pvm. *Eurocode 2: The design of concrete structures: part-1-1: General rules and rules for buildings.* Finnish national standard association.

SFS-EN 1992-1-1. 2002. *Design of concrete structure- part 1-1 General rules and rules for buildings.* Finnish standards association SFS.

SFS-EN 1992-1-1. 2002. ”Eucode2.”

SFS-EN 1992-1-1. 2002. *EUROCODE 2: Design of concrete structures -part-1-1: General rules and rules for buildings.* Finnish national standards SFS.

SFS-EN 1992-1-1. 2002. *Eurocode 2: The design of concrete structures. part 1-1: General rules and rules for buildings.* Finnish standard association.

SFS-EN 1992-1-1. 2002. *EUROCODE: Design of concrete structures - part 1-1: General rules and rules for buildings.* Finnishi national standards SFS.

SFS-EN 1992-1-1. 2002. *EUROCODE: Designer of concrete structures -part 1-1 : General rules and rules for buildings.* Finnish national standards SFS.

SFS-EN 1992-1-1, 9.2.11,. 2002. *Eurocode 2: The design of concrete structures: Part 1-1: General rules and rules for buildings.* Confederation of finnish construction insdustries.

SFS-EN 1992-1-1; 4.4.1. ei pvm. *Design of concrete structures - part-1-1: General rules and rules for buildings.* Finnish standard association.

SFS-EN 206 +A2. 2021. *Concrete specification, perfomance,production and conformity.* Finnish standard association SFS.

SFS-EN 206. 2021. *Concrete specification, perfomance,production and conformity.* Finnish standards association SFS.

Smith, Paul. 2016. *Structure design of buildings.* Wiley John.

204. *Suomeen rakentamismääräyskokoelma B3.* Helsinki: Anja Nylund.

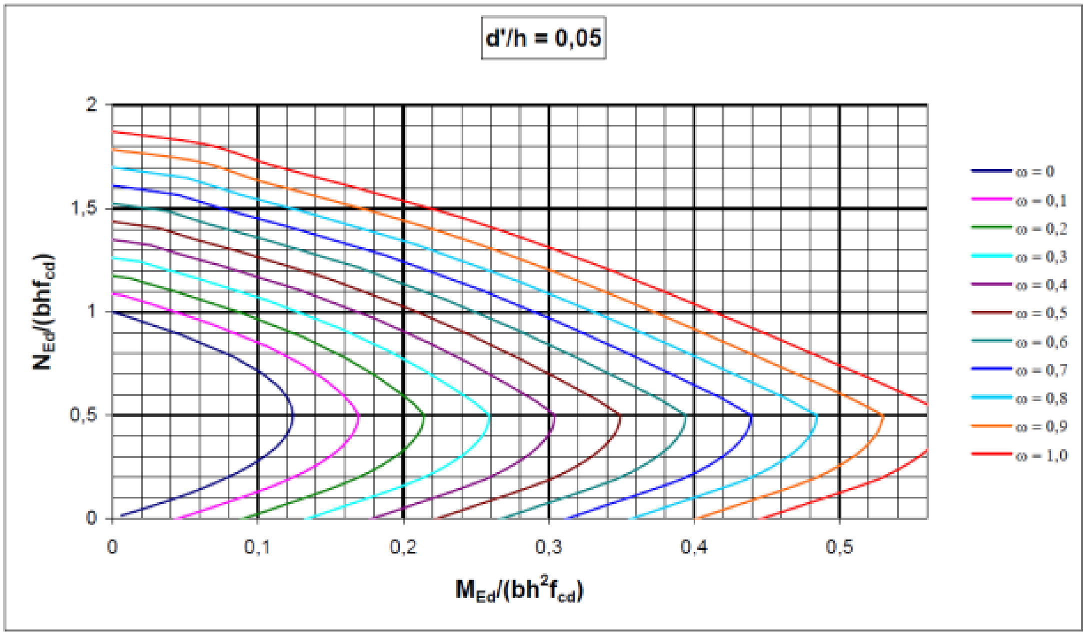
www.elementtisuunnittelu.fi. 2021. *https://www.elementtisuunnittelu.fi/runkorakenteet/palkit.* 03. 11.

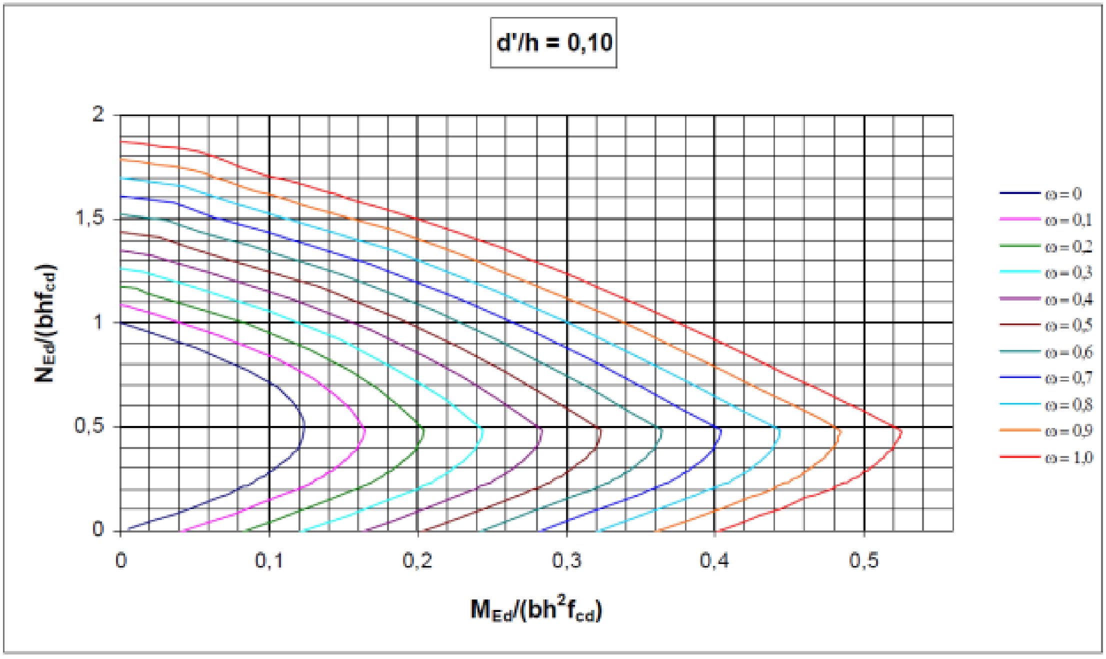
—. 2021. *https://www.elementtisuunnittelu.fi/runkorakenteet/palkit.* 03. 11.

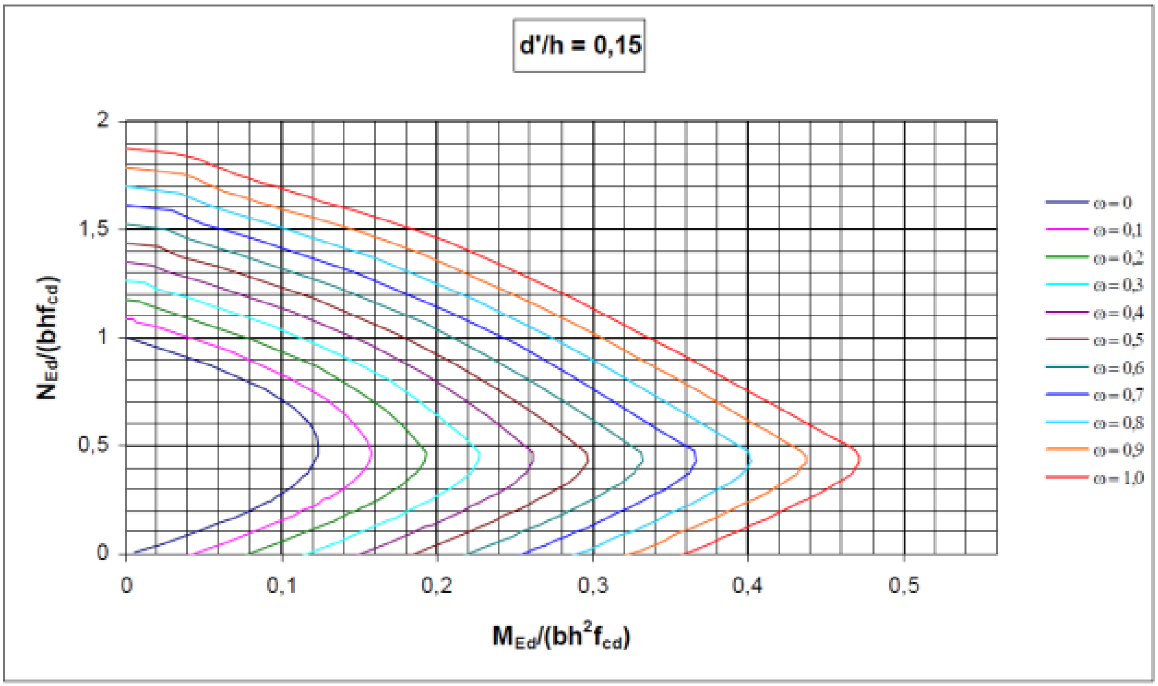
www.elementtisuunntittelu.fi. 2021. *https://www.elementtisuunnittelu.fi/runkorakenteet/palkit.* 4. 11.

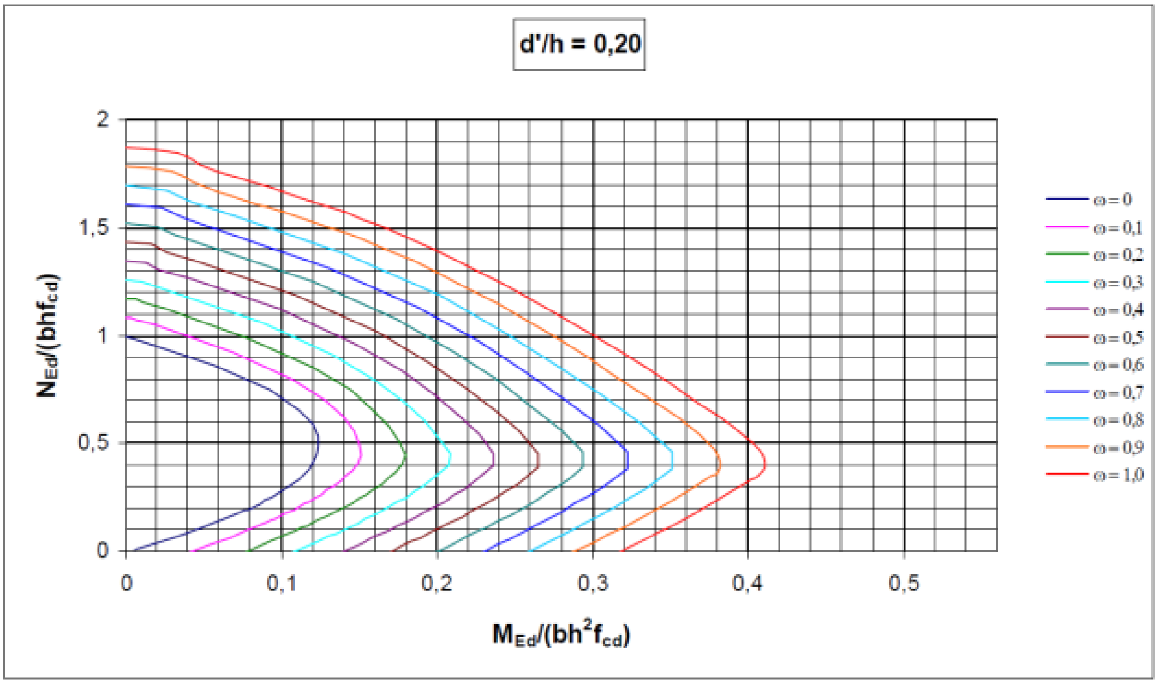
Appendix 1: Relationship diagrams between relative normal force (ν) and the relative moment(μ)

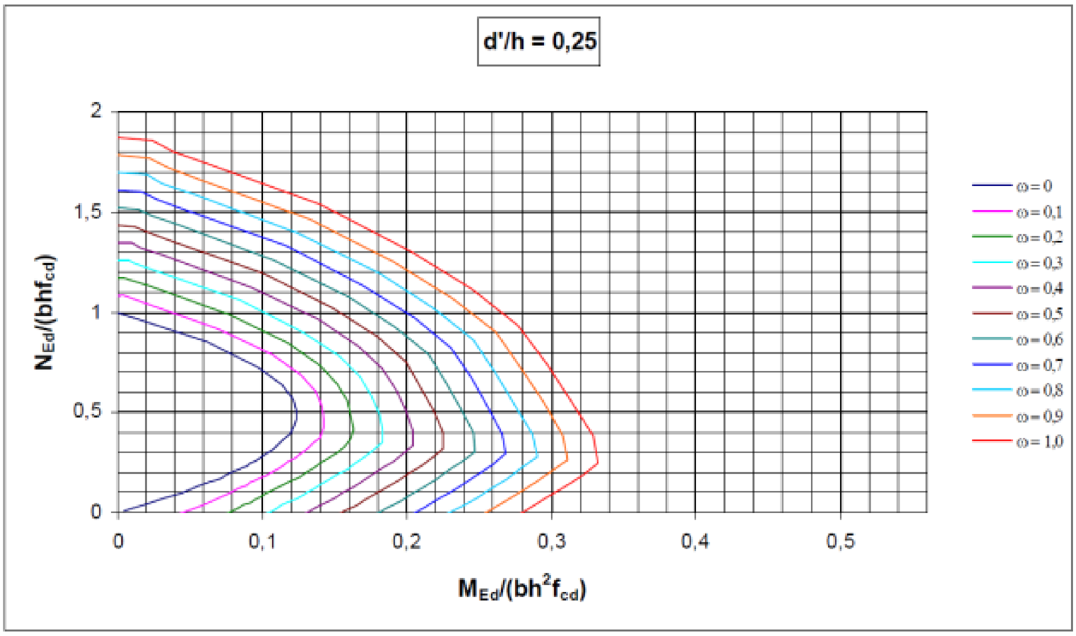
The following diagrams shows the relations between relative normal force (ν) and the relative moment(μ)






d'/h = 0,15
$N_{Ed}/(bhf_{cd})$
$M_{Ed}/(bh^2f_{cd})$
2
1,5
1
0,5
0
0
0,1
0,2
0,3
0,4
0,5
ω = 0
ω = 0,1
ω = 0,2
ω = 0,3
ω = 0,4
ω = 0,5
ω = 0,6
ω = 0,7
ω = 0,8
ω = 0,9
ω = 1,0


d'/h = 0,20
$N_{Ed}/(bhf_{cd})$
$M_{Ed}/(bh^2f_{cd})$
2
1,5
1
0,5
0
0
0,1
0,2
0,3
0,4
0,5
ω = 0
ω = 0,1
ω = 0,2
ω = 0,3
ω = 0,4
ω = 0,5
ω = 0,6
ω = 0,7
ω = 0,8
ω = 0,9
ω = 1,0


d'/h = 0,25
$N_{Ed}/(bhf_{cd})$
$M_{Ed}/(bh^2f_{cd})$
2
1,5
1
0,5
0
0
0,1
0,2
0,3
0,4
0,5
ω = 0
ω = 0,1
ω = 0,2
ω = 0,3
ω = 0,4
ω = 0,5
ω = 0,6
ω = 0,7
ω = 0,8
ω = 0,9
ω = 1,0

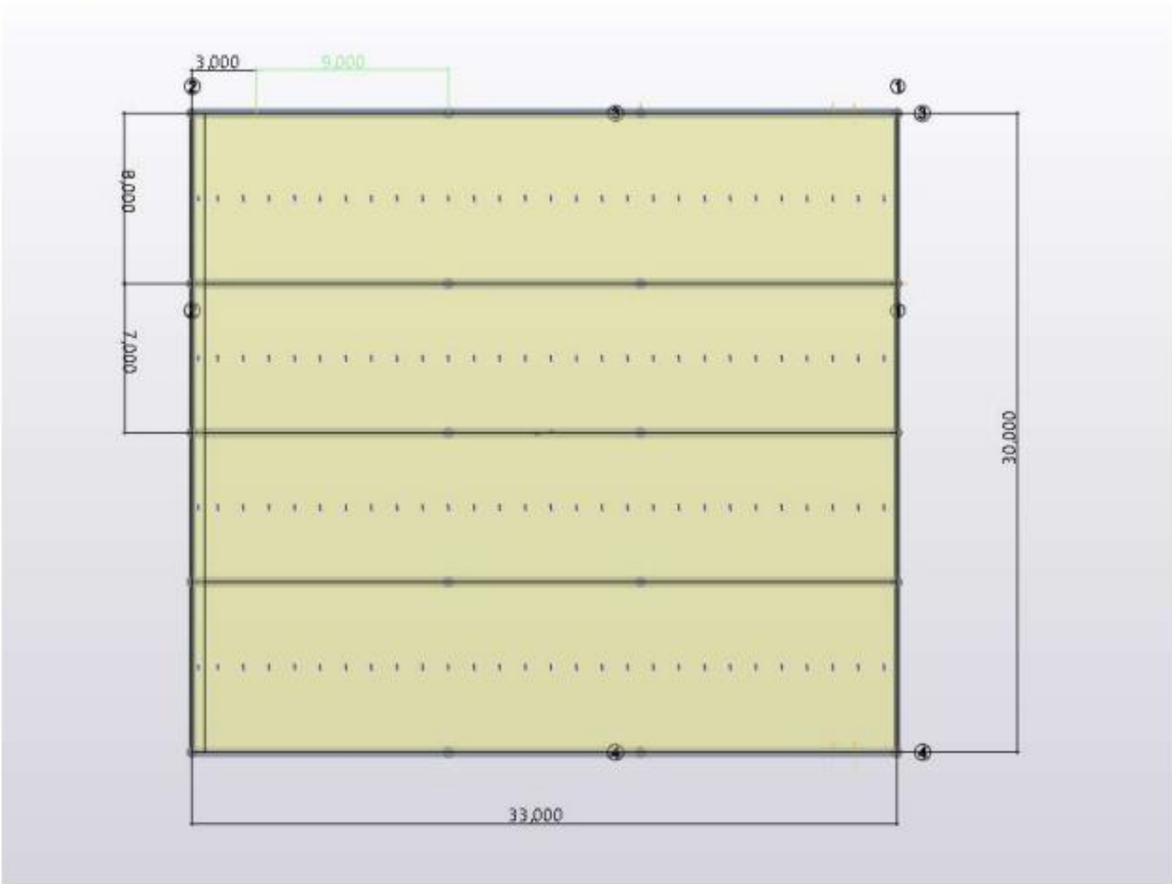
Appendix 2: Design of beam

Design of beam

1. INITIAL INFORMATIONS

$b := 480\ mm$ $h := 880\ mm$ Beam Dimension

Parking garage dimensions.



Longest beam span is 9m $L := 9\ m$

Properties of concrete

Characteristic value of concrete strenght $f_{ck} := 35\ \mathrm{MPa}$

Concrete partial safety factor $\gamma_c := 1,5$

Factor of long term-effect on concrete $\alpha_c := 0,85$

$$f_{cd} := \alpha_c \cdot \frac{f_{ck}}{\gamma_c} = 19,8333\ \mathrm{MPa}$$

Ultimate compressive strain in the concrete $\varepsilon_{cu} := 0,0035$

Maximum aggragate diamter in concrete $d_g := 20\ \mathrm{mm}$

The minimum distance between two reinforcement bars

$$a := \max\left(\left[\, d_g + 3\ \mathrm{mm} \quad 20\ \mathrm{mm} \,\right]\right) = 0,023\ \mathrm{m}$$

Reinforced concrete density: $\delta := 25\ \frac{\mathrm{kN}}{\mathrm{m}^3}$

Propertied of steel

Characteristic value of steel yield strength (*B500B*)

$$f_{yk} := 500\ \mathrm{MPa}$$

Steel partial safety factor $\gamma_s := 1,15$

Design value of steel strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434,7826\ \mathrm{MPa}$

Steel module of elesticity $E_s := 210\ \mathrm{MPa}$

Strain of reinforcement at maximum load $\varepsilon_u := 0,0025$

2. CONCRETE COVER

EXposure class see EN 1992-1-1, 4.4.1.2 Table 4.3N and Annex E T11.1.3.1

Concrete class is 6: C35/45,XC3

$$c_{nom} := c_{min} + \Delta c_{dev}$$

c_{nom} Normal concrete cover

c_{min} Minimum concrete cover

Δc_{dev} Design deviation

$$c_{min} := \max\left(\left[c_{min,b} \;\; c_{min,dur} + c_{min,\gamma} - c_{min,st} - \Delta Cdur \;\; 10\ mm \right]\right)$$

$c_{min,dur} := 35\ mm$ $\quad c_{min,\gamma} := 0\ mm$ $\quad c_{min,st} := 0\ mm$ $\quad \Delta Cdur := 0\ mm$

$c_{min,b} := 25\ mm$

$$c_{min} := \max\left(\left[c_{min,b} \;\; c_{min,dur} + c_{min,\gamma} - c_{min,st} - \Delta Cdur \;\; 10\ mm \right]\right) = 0,035\ m$$

$c_{nom} := c_{min} + \Delta c_{dev}$ $\quad \Delta c_{dev} := 10\ mm$ $\quad$ EN 1992-1-1, ·A2.1

$$c_{nom} := c_{min} + \Delta c_{dev} = 0,045\ m$$

Design of beam

$M_{Ed} := 1185\ kN\ m$ **From tekla designer**

This value was give from load combination of :

1. LOADS COMBINATION

Dead loads

Self weight of structures

Self weight of structures:

Beam cross section are: $A_b := b \cdot h = 0,4224\ m^2$

$$W_b := A_b \cdot \delta = 10,56 \ \frac{kN}{m}$$

Weight of P40 hollow core slab $W_h := 4,35 \ \frac{kN}{m^2}$

$$W_s := W_h \cdot \left(\frac{8}{2}\ m + \frac{7}{2}\ m\right) = 32,625 \ \frac{kN}{m}$$

Surface of Cast in place above slab 80mm:

$$A_c := \left(\frac{8}{2}\ m + \frac{7}{2}\ m\right) \cdot 0,08\ m = 0,6\ m^2$$

$$W_c := A_c \cdot \delta = 15 \ \frac{kN}{m}$$

Imposed load $q := 2,5 \ \frac{kN}{m^2}$ and

$$q_k := q \cdot \left(\left(\frac{8}{2}\ m + \frac{7}{2}\ m\right)\right) = 18,75 \ \frac{kN}{m}$$

Snow load $q_s := 2,75 \ \frac{kN}{m^2}$ $C_e := 0,8$ $\mu_1 := 1$

$$q_{sk} := C_e \cdot q_s \cdot \mu_1 \cdot \left(\frac{8}{2}\ m + \frac{7}{2}\ m\right) = 16,5 \ \frac{kN}{m}$$

Loads combinations

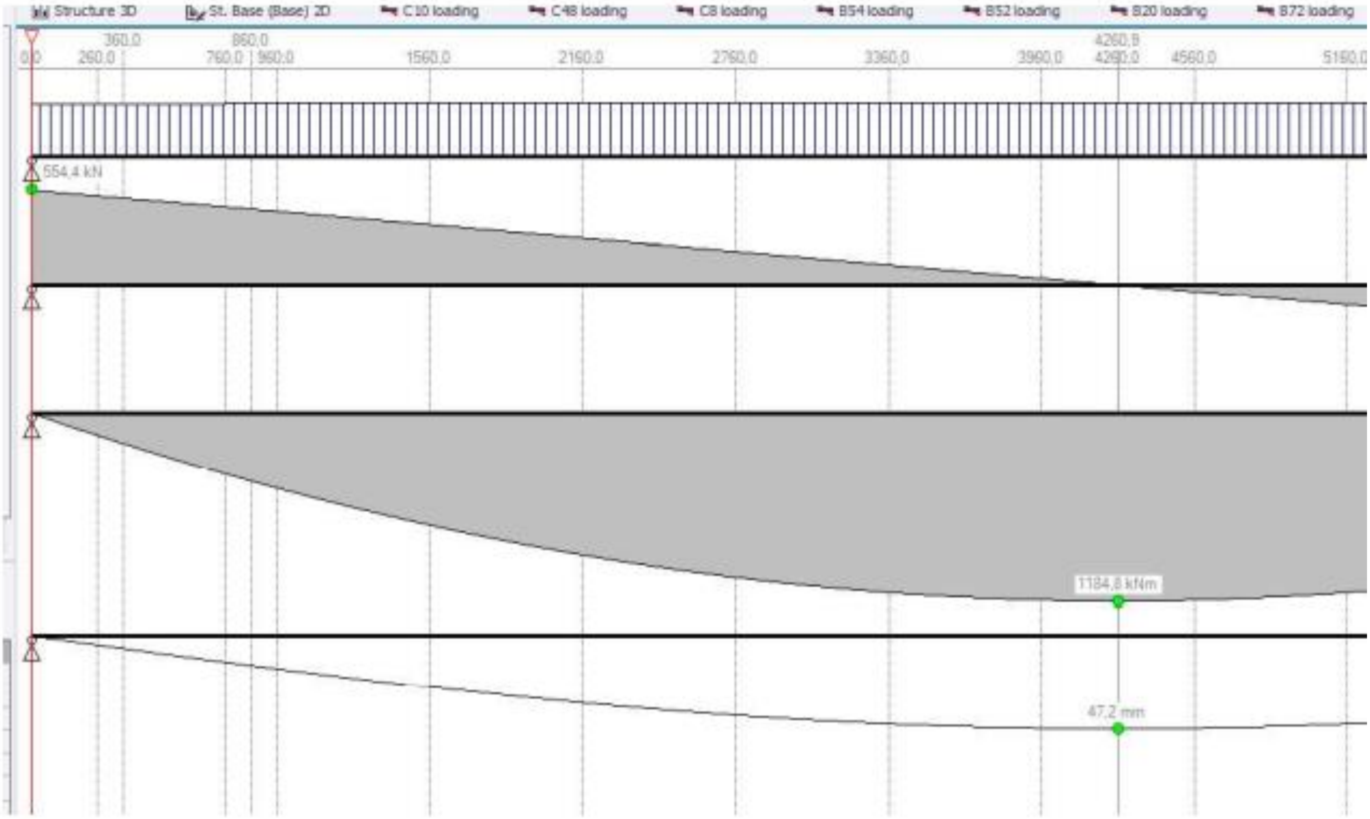
$$P_d := \max\left(\begin{bmatrix} 1,35 \cdot \left(W_b + W_s + W_c\right) \\ 1,15 \cdot \left(W_b + W_s + W_c\right) + 1,5 \cdot q_k + 1,05 \cdot q_{sk} \\ 1,15 \cdot \left(W_b + W_s + W_c\right) + 1,05 \cdot q_k + 1,5 \cdot q_{sk} \end{bmatrix}\right) = 112,3628 \ \frac{kN}{m}$$

Maximum moment

$$M_{Max} := \frac{P_d \cdot L^2}{8} = 1137,6728 \ kN\ m$$

By comparing this the value to that from tekla given below we can see that both values are in the same range.

$M_{Ed} := 1185\ kN\ m$ **From tekla designer**



Diameter of tension reinforcement bar $\phi_h := 25\ \text{mm}$

Diameter of shear reinforcement bar $\phi_s := 10\ \text{mm}$

$$d := h - \frac{\phi_h}{2} - \phi_s - C_{nom} = 812,5\ \text{mm}$$

Checking if cross section is sufficient to the applied bending moment

$$\mu := \frac{|M_{Ed}|}{b \cdot (d)^2 \cdot f_{cd}} = 0,1886$$

$$\beta := 1 - \sqrt{1 - 2 \cdot \mu} = 0,2108$$

$$\beta_d := \frac{0,8 \cdot \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_y}$$

$$\beta_d := \frac{0,8 \cdot \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_u} = 0,4667$$

$\beta_d \geq \beta$ The cross-section of beam is enough to support the above moment, we proceed to calculating the required reinfoncing bar.

Other procedure is calculating the value of design coefficient of friction μ_d

$$\mu_d := 0,960 \cdot \delta - 0,264 \cdot \delta^2 - 0,371 \qquad \delta := \frac{Z}{d}$$

Where, Z is moment lever arm

$$Z := \frac{d}{2} \cdot \left(1 + \sqrt{1 - 2 \cdot \mu}\right) = 0,7269 \text{ m}$$

$$\delta := \frac{Z}{d} = 0,8946$$

by solving the equation $\mu_d := 0,3$

$\mu_d \geq \mu$ The cross section of beam is enough to support the above moment,we can proceed to calculating the required cross-section area of reinfoncing bars.

3.TENSILE REINFORCEMENT

The minimum allowed cross-section area of reinforcement bars

$f_{ctm} := 3,21 \text{ MPa}$ Mean tensile strength of concrete

$$A_{smin} := 0,26 \cdot \frac{f_{ctm}}{f_{yk}} \cdot b \cdot d = 650,988 \text{ mm}^2$$

The minimum required cross-section area of reinforcement bars

$$A_{srd} := \frac{M_{Ed}}{Z \cdot f_{yd}} = 3749,6028 \text{ mm}^2$$

The minimum reinforcement is given by the maximum between A_{smin} and A_{srd}

$$A_s := \max\left(\left[\, A_{smin} \;\; A_{srd} \,\right]\right) = 0,0037 \text{ m}^2$$

Diameter of tension bar $\phi_h := 25 \text{ mm}$

$$A_{\phi h} := \frac{\pi \cdot \phi_h^{\ 2}}{4} = 490{,}8739\ mm^2$$

Number of tension bars

$$n := \frac{A_s}{A_{\phi h}} = 7{,}6386$$

Let choose for reinforcing bars is 8T25

$$n := 8$$

The maximum size of aggregate in concrete must be less than the distance between two reinforcing bars for concrete to provide good protection of the steel against corrosion

$$a_{req} := \frac{b - 2 \cdot C_{nom} - 2 \cdot \phi_h - 2 \cdot \phi_s}{n - 1} = 0{,}0457\ m \qquad a_{req} \geq a \qquad Ok$$

4. SHEAR REINFORCEMENT

$V_{Edmax} := 556\ kN$ **Design shear force (from tekla)**

Or manually the reaction can be calculated:

$$V_{Edmx} := P_d \cdot \frac{L}{2} = 505{,}6324\ kN$$

The resistance capacity opf beam under shear stress can be given:

$$V_{Rd,max} := \frac{0{,}6 \cdot b \cdot Z \cdot f_{cd}}{(cot\theta + tan\theta)}$$

Where $1 \leq cot\theta \leq 2$

Let choose the maximum value of cotθ so that we can not over estimate the beam capacity.

$$cot\theta := 2 \qquad tan\theta := \frac{1}{cot\theta}$$

$$V_{Rd,max} := \frac{0{,}6 \cdot b \cdot Z \cdot f_{cd}}{(cot\theta + tan\theta)} = 1{,}6608 \cdot 10^6\ N$$

Utilisation ratio

$$\frac{V_{Edmax}}{V_{Rd,max}} = 0,3348 \quad OK$$

The miminum reinforcement can be calculated using the following expression:

$$\frac{A_{sw,min}}{S} = \frac{0,08 \cdot \sqrt{f_{ck}}}{f_{yk}} \cdot b \qquad s := 1\ \text{m}$$

After calculation $\frac{A_{sw,min}}{S} = 0,5\ \frac{\text{mm}^2}{\text{m}}$

The max bar spacing $s_{1min} := 0,75 \cdot d = 0,6094\ \text{m}$

The miminum allowed required reinforcement can be calculated using the following expression:

$$\frac{A_{sw}}{S} = \frac{V_{Edmax}}{f_{yk} \cdot cot\theta} \qquad \frac{V_{Edmax}}{f_{yk} \cdot cot\theta} = 0,0006\ \text{m}^2$$

$$\frac{A_{sw}}{S} = \frac{0,0005\ \text{m}^2}{\text{m}}$$

2-pices

shear reinforcement is the max between $\frac{A_{sw}}{S}$ and $\frac{A_{sw}}{S}$

The shear reinforcement bar diameter T10

$$A_h := 2 \cdot \pi \cdot \frac{\phi_s^{\ 2}}{4} = 157,0796\ \text{mm}^2$$

Number of shear bars

$$n := \frac{\frac{V_{Edmax}}{f_{yk} \cdot cot\theta}}{A_h} = 3,5396$$

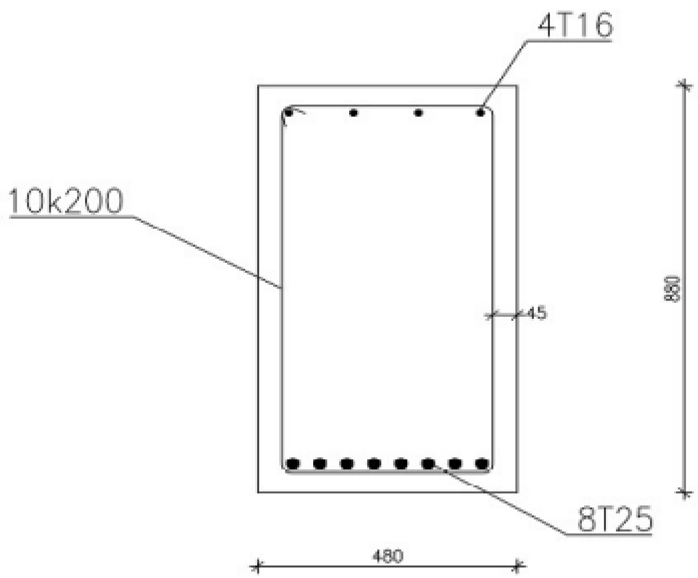
Means that we need 4 hooks in every every one meter.

The hooks spacing is given by

$$\frac{s}{4} = 0,25 \text{ m}$$

The beam depth is 880mm due to the effect of cracking in concrete we can reduce the spacing distance to 200mm

The shear reinforcement is **T10k200**



Appendix 3: Design of column

Design of column

1. INITIAL INFORMATION

$b := 880\ mm$ $h := 880\ mm$ $L := 6000\ mm$ $A_c := b \cdot h = 7,744 \cdot 10^5\ mm^2$

Properties of concrete

Characteristic value of concrete strenght $f_{ck} := 40\ MPa$

Concrete partial safety factor $\gamma_c := 1,5$

Factor of long-term effect on concrete $\alpha_c := 0,85$

$$f_{cd} := \alpha_c \cdot \frac{f_{ck}}{\gamma_c} = 22,6667\ MPa$$

Ultimate compressive strain in the concrete $\varepsilon_{cu} := 0,0035$

Maximum aggragate diamter in concrete $d_g := 20\ mm$

The minimum distance between two reinforcement bars

$$a := \max\left(\left[\, d_g + 3\ mm \quad 20\ mm \,\right]\right) = 0,023\ m$$

Propertied of steel

Characteristic value of steel yield strength (*B500B*) $f_{yk} := 500\ MPa$

Steel partial safety factor $\gamma_s := 1,15$

Design value of steel strength $f_{yd} := \frac{f_{yk}}{\gamma_s} = 434,7826\ MPa$

Steel module of elesticity $E_s := 210\ MPa$

Strain of reinforcement at maximum load $\varepsilon_u := 0,0025$

2. CONCRETE COVER

EXposure class see EN 1992−1−1, 4.4.1.2 Table 4.3N and Annex E T11.1.3.1

Concrete class is 6: C35/45,XC3

$$c_{nom} := c_{min} + \Delta c_{dev}$$

c_{nom} Normal concrete cover

c_{min} Minimum concrete cover

Δc_{dev} Design deviation

The preliminary main reinforcement bar size $\phi := 20\ mm$ $\phi_s := 6\ mm$

The reinforcement cross-sectional area is:

$$A_{s,max} := 0,04 \cdot A_c = 9216\ mm^2$$

$$c_{min} := \max\left(\left[\, c_{min,b}\ \ c_{min,dur} + c_{min,\gamma} - c_{min,st} - \Delta cdur\ \ 10\ mm \,\right]\right)$$

$c_{min,dur} := 35\ mm$ $c_{min,\gamma} := 0\ mm$ $c_{min,st} := 0\ mm$ $\Delta cdur := 0\ mm$

$$c_{min,b} := \phi = 0,02\ m$$

$$c_{min} := \max\left(\left[\, c_{min,b}\ \ c_{min,dur} + c_{min,\gamma} - c_{min,st} - \Delta cdur\ \ 10\ mm \,\right]\right) = 0,035\ m$$

$c_{nom} := c_{min} + \Delta c_{dev}$ $\Delta c_{dev} := 10\ mm$ EN 1992-1-1, A2.1

$$c_{nom} := c_{min} + \Delta c_{dev} = 0,045\ m$$

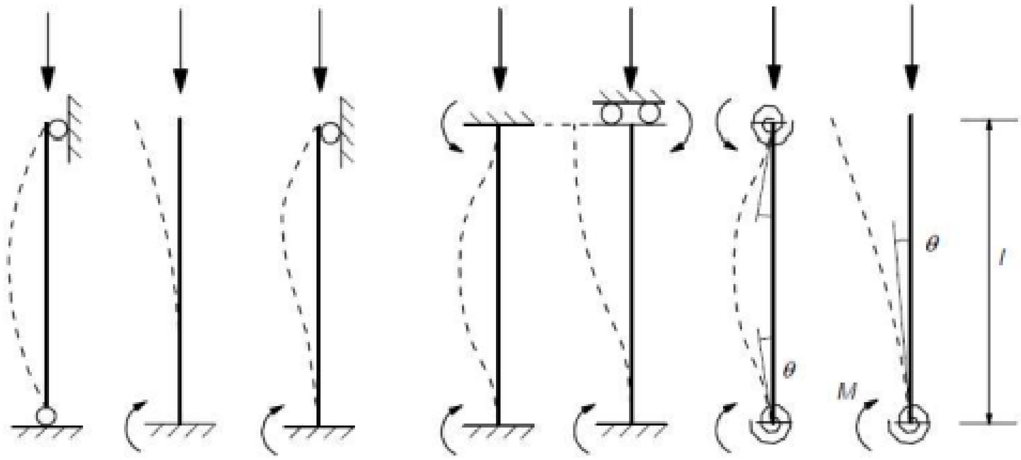
2. DESIGN OF COLUMN

Normal force $N_{Ed} := 1203\ kN$ From tekla designer.

$M_{Ed1} := 50\ kN\ m$

$$d := h - \frac{\phi}{2} - \phi_s - c_{nom} = 0,419\ m$$

Slenderness of column



a) $l_0 = l$ b) $l_0 = 2l$ c) $l_0 = 0,7l$ d) $l_0 = l/2$ e) $l_0 = l$ f) $l/2 < l_0 < l$ g) $l_0 > 2l$

Buckling effective length $L_{eff} := 0,7 \cdot L = 2,8$ m

$$I_c := \frac{b \cdot h^3}{12} = 4,4237 \cdot 10^9 \ \text{mm}^4$$

$$i := \sqrt{\frac{I_c}{A_c}} = 138,5641 \ \text{mm}$$

Slenderness ratio $\lambda := \frac{L_{eff}}{i} = 20,2073$

A. Let Column capacity under pure compression, when all load eccentricities are neglected.

$$N_{Rd,0} := \left(A_{sc} + A_s\right) \cdot f_{yd} + b \cdot h \cdot f_{cd}$$

A_{sc} compressive reinforcement cross-sectional area

A_s Tensile reinforcement cross-sectional area.

The column reinforcement is symmetrical therefore, $A_{sc} + A_s = A_{s,max}$

$$N_{Rd,0} := A_{s,max} \cdot f_{yd} + b \cdot h \cdot f_{cd} = 8,5766 \cdot 10^6 \ \text{N}$$

Utilisation ratio $UR := \frac{N_{Ed}}{N_{Rd,0}} = 0{,}1403$

B.Let check if column reistance capacity under bending force, this will likely happen due to the excentricity of load.

$$\lambda_1 := 0{,}8$$

$$N_{Rd,b} := \lambda_1 \cdot b \cdot d \cdot f_{cd} \cdot \frac{\varepsilon_{cu} \cdot E_s}{f_{yd} + \varepsilon_{cu} \cdot E_s} + \left(A_{sc} - A_s\right) \cdot f_{yd}$$

The column is symmetrically reinforced then, $A_{sc} - A_s = 0$

$$N_{Rd,b} := \lambda_1 \cdot b \cdot d \cdot f_{cd} \cdot \frac{\varepsilon_{cu} \cdot E_s}{f_{yd} + \varepsilon_{cu} \cdot E_s} = 5385{,}4572 \text{ N}$$

Utilisation ratio $UR := \frac{N_{Ed}}{N_{Rd,b}} = 223{,}3794$

$N_{Rd,b} < N_{Ed}$ and $N_{Ed} < N_{Rd,0}$

Means that column will have compression fracture,but transverse reinforcement can prevent this to happen.(see Betonirakentedein suunnittelu ja mitoitus p .419)

Calculating he minimum slenderness limit

$$\lambda_{lim} := \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}} \qquad A := 0{,}7$$

$$B := \sqrt{1 + 2 \cdot \omega}$$

Mechanical reinforcement ratio

Design value of eccentricity $e_d := e_a + e_2 + e_0$

Basic eccentricity: $e_a := \frac{h}{20} + \frac{L}{150}$ When $h \leq 50$ mm, h is zero.

$$e_a := 0 + \frac{L}{150} = 0,0267\ \mathrm{m}$$

Additional eccentricity

$$e_2 := \left(\frac{A_c \cdot L_{eff}^{\ 2}}{21025 \cdot I_c}\right) \cdot h$$

$$I_c := \frac{b \cdot h^3}{12} = 0,0044\ \mathrm{m}^4$$

$$e_2 := \left(\frac{A_c \cdot L^2}{21025 \cdot I_c}\right) \cdot h = 0,019\ \mathrm{m}$$

Eccentricity of the load

$$e_0 := \frac{M_{Ed1}}{N_{Ed}} = 0,0416\ \mathrm{m}$$

$$e_d := e_a + e_2 + e_0 = 0,0873\ \mathrm{m} \qquad d_c := 0$$

C. Column reistance to compressive strength

$$A_{sc} := \frac{A_{s,max}}{2}$$

$$N_{rd} := \frac{A_{sc} \cdot f_{yd} \cdot \left(d - d_c\right) + 0,4 \cdot f_{cd} \cdot b \cdot d^2}{e_d + d - \frac{h}{2}} = 5663,7325\ \mathrm{kN}$$

Utilisation ratio: $\frac{N_{Ed}}{N_{rd}} = 0,2124$

Required longitudinal reinforcement

$$A_{s,min} := \max\left(\begin{bmatrix} \frac{0,1 \cdot N_{Ed}}{f_{yd}} \\ 0,002 \cdot A_c \end{bmatrix}\right) = 460,8\ \mathrm{mm}^2 \qquad A_{s,max} > A_{s,min} = 1$$

$A_{s,max} > A_{s,min}$ ok

Geometric imperfections

Load eccentricity

$\theta_0 := \frac{1}{200}$ Basic value

$e_0 := \max\left(\begin{bmatrix} \frac{h}{30} \\ 20\ mm \end{bmatrix}\right) = 0,02\ m$ Basic eccentricity

$$\alpha_h := \begin{array}{|l} \text{if } \frac{2}{\sqrt{\frac{L}{m}}} < \frac{2}{3} \\ \quad \frac{2}{3} \\ \text{else} \\ \quad \frac{2}{\sqrt{\frac{L}{m}}} \\ \text{if } \frac{2}{\sqrt{\frac{L}{m}}} > 1 \\ \quad 1 \\ \text{else} \\ \quad \frac{2}{\sqrt{\frac{L}{m}}} \end{array} \qquad \frac{2}{\sqrt{\frac{L}{m}}} = 1$$

Height effect factor

$\alpha_h = 1$

$m_p := 6$ Number of column in a row

$\alpha_m := \sqrt{0,5 \cdot \left(1 + \frac{1}{m_p}\right)} = 0,7638$ Number of compents effectif factor

$\theta_i := \theta_0 \cdot \alpha_h \cdot \alpha_m = 0,0038$ Inclination

$e_i := \frac{\theta_i \cdot L_{eff}}{2} = 5,3463\ mm$ Load eccentricity

$H_i := \theta_i \cdot N_{Ed} = 4,594\ kN$ Horizontal compansating force for eccentricity.

The column is not slender the geometrical nonlinearity can be neglected.

$M_{Ed0} := N_{Ed} \cdot \left(e_0 + e_i\right) = 30,4916\ kN\ m$ Moment from eccentricity

Design moments and axial loads

$M_{Ed} := M_{Ed1} + M_{Ed0}$

$M_{Ed} = 80,4916$ kN m

$$\omega_{max} := \frac{A_{s,max} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0,8769$$

$$\omega_{s,min} := \frac{A_{s,min} \cdot f_{yd}}{A_c \cdot f_{cd}} = 0,0438$$

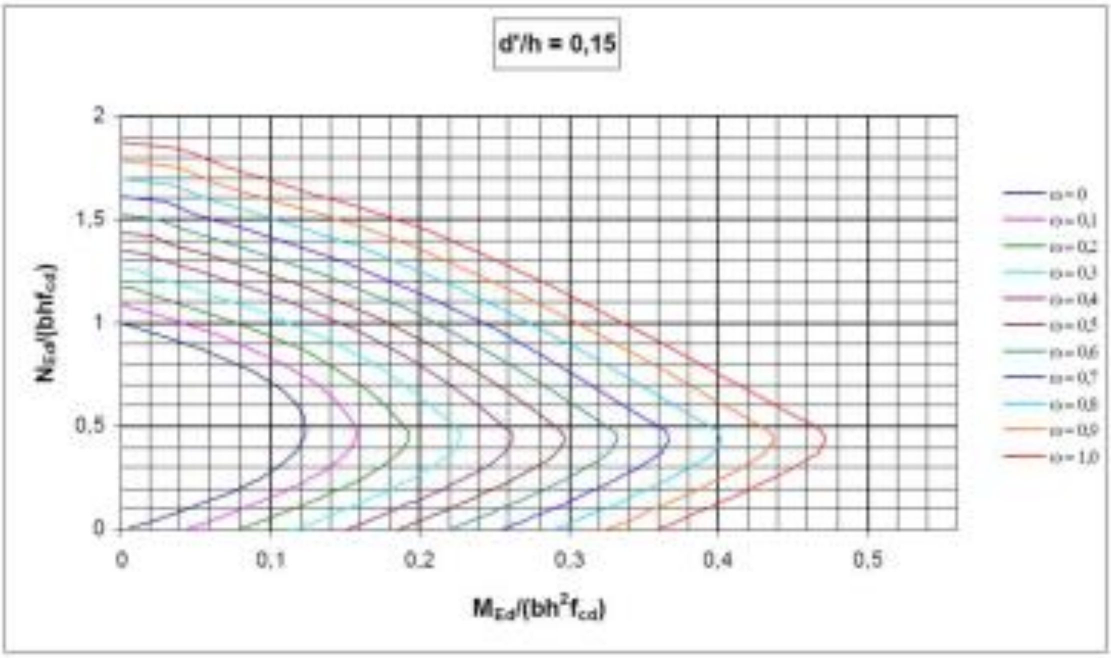
$$\mu := \frac{M_{Ed}}{b \cdot h^2 \cdot f_{cd}} = 0,0367$$

$$\nu := \frac{0,5 \cdot N_{Ed}}{b \cdot h \cdot f_{cd}} = 0,1316$$

$$d' := C_{nom} + \frac{\phi}{2} + \phi_s = 61 \text{ mm}$$

$b = 480$ mm

$$\frac{d'}{b} = 0,1271$$



$\omega := 0,0$

$\omega \le \omega_{s,min}$ Then the minimum permissible reinforcement

$$A_{srqmin} := \omega_{s,min} \cdot A_c \cdot \frac{f_{cd}}{f_{yd}} = 460,8\ mm^2$$

$$A_{srqu} := \omega \cdot A_c \cdot \frac{f_{cd}}{f_{sd}} = 0\ mm^2$$

The required cross -sectional area of longitudinal reinforcement

$$A_{srq} := \text{if } \omega \neq 0 \quad A_{srqu} \quad \text{else} \quad A_{srqmin} \qquad A_{srq} = 460,8\ mm^2$$

If $A_{s,max} \geq A_{srq}$ *OK* Otherwise the cross-section reinforcement must be increased.

Number of bars required

$$A_1 := \pi \cdot \left(\frac{\phi}{2}\right)^2 = 0,0003\ m^2$$

$$n := \frac{A_{srq}}{A_1} = 1,4668$$

Let choose longitudinal reinforcement bars as 4**T25**

<u>Transverse reinforcement</u>

Diamter of transverse bars is $\frac{\phi}{4} = 0,005\ m$

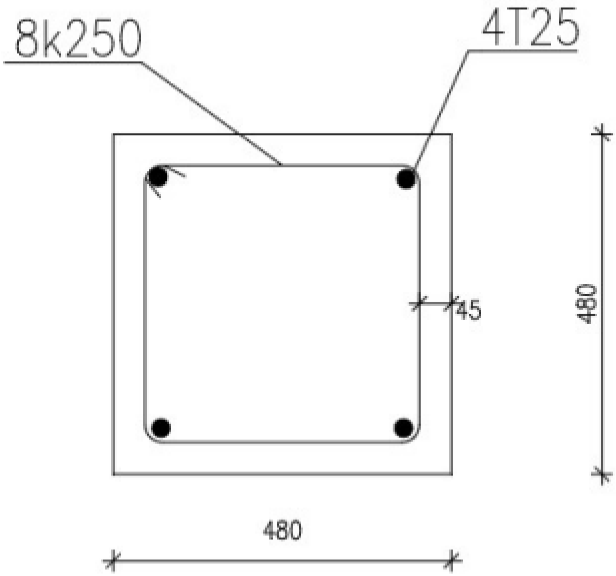
Let choose shear bars of 8 mm of diameter $\phi_s := 8\ mm$

transverse bars spacing: $S_{cl,max} := \min([\,20 \cdot 20\ mm\ \ 480\ mm\ \ 400\ mm\,])$

$$S_{cl,max} = 0,4\ m$$

For preventing concrete cracking we can reduce spacing distance to 250

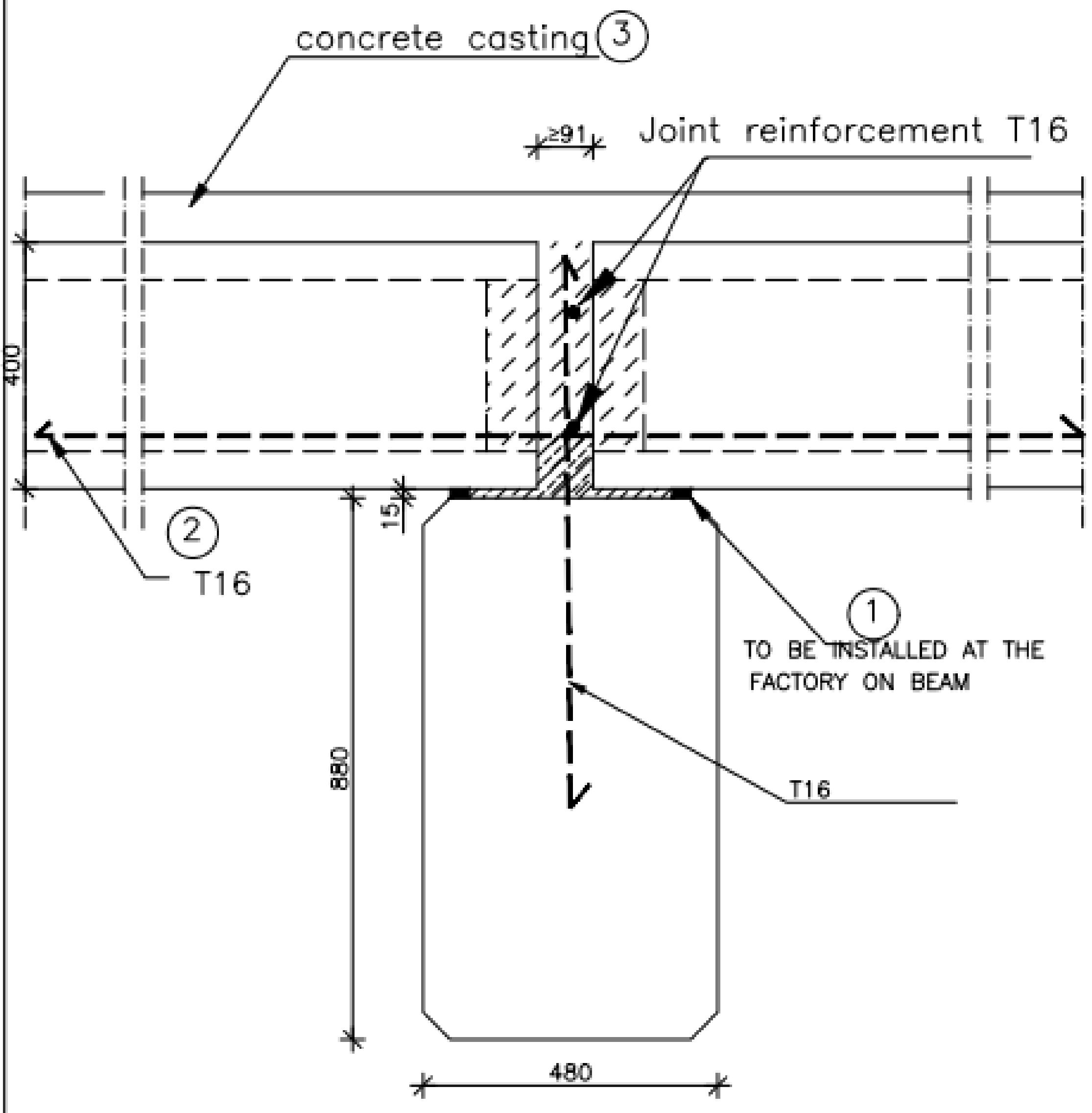
Therefore, transverse reinforcement is **T8k250**


8k250
4T25
45
480
480

Appendix 4: Connection details

	Content: HOLLOWCORE SLAB AND RECTANGULAR BEAM CONNECTION		
Designer:	work nro		D2
	Date:	Designer	

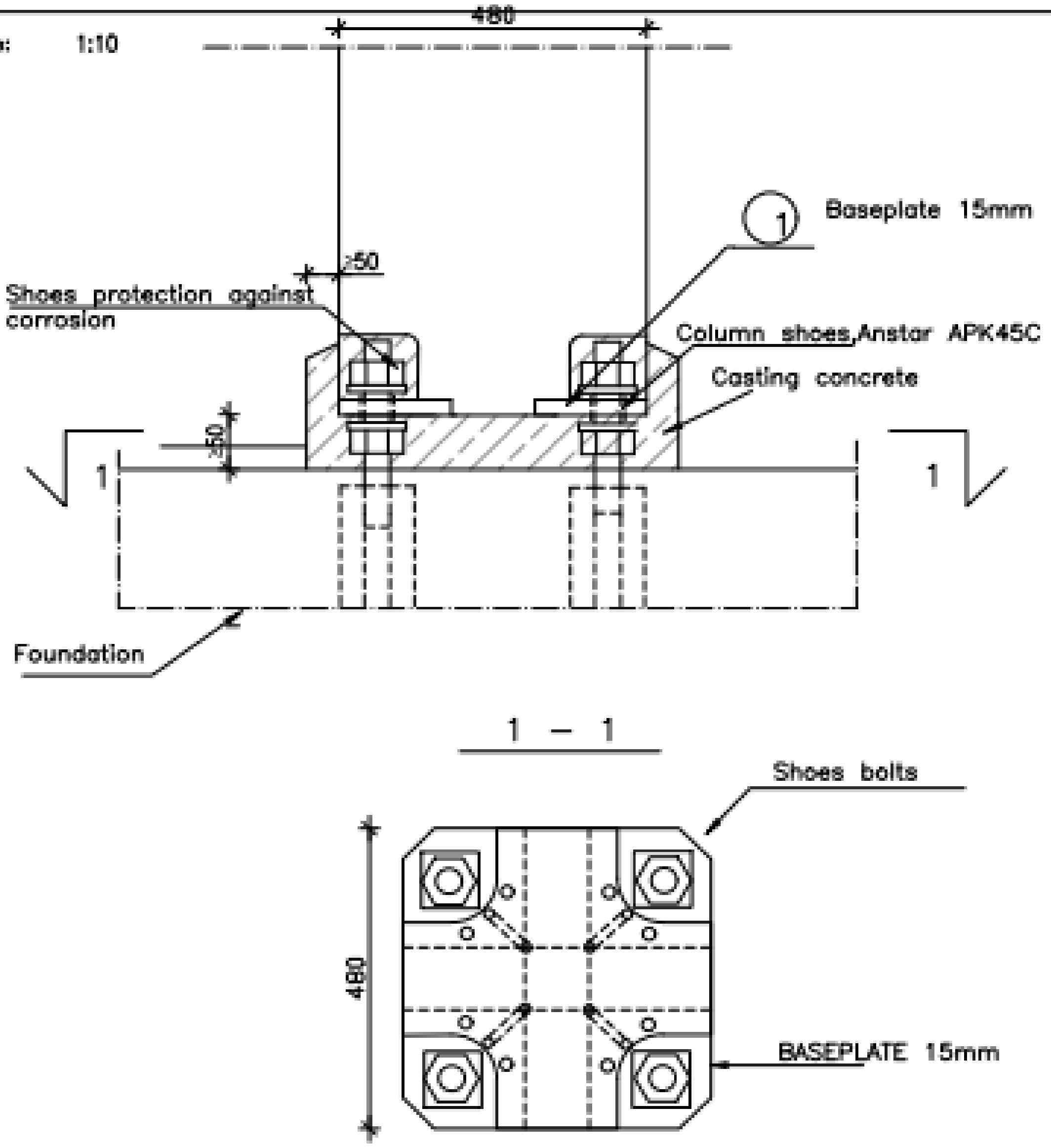
Scale: 1:10



MATERIALS LIST						CODE:	
RY	PO	MAT/TAR	TYPE	SIZE	QUALITY	QUANTITY	NOTICE
VKU	1	NEOPREN–NAUHA		20*10	SHORE 60		
RH	2	JOINT REINFORCEMENT T16			A 500 HW		
	3	Concrete castin reinforcement T8k150			A 500 HW		

	Content: COLUMN CONNECTION TO FOUNDATION COLUMN 580X580	
Designer:	Work nro	DT:1
	Date: / Designer:	

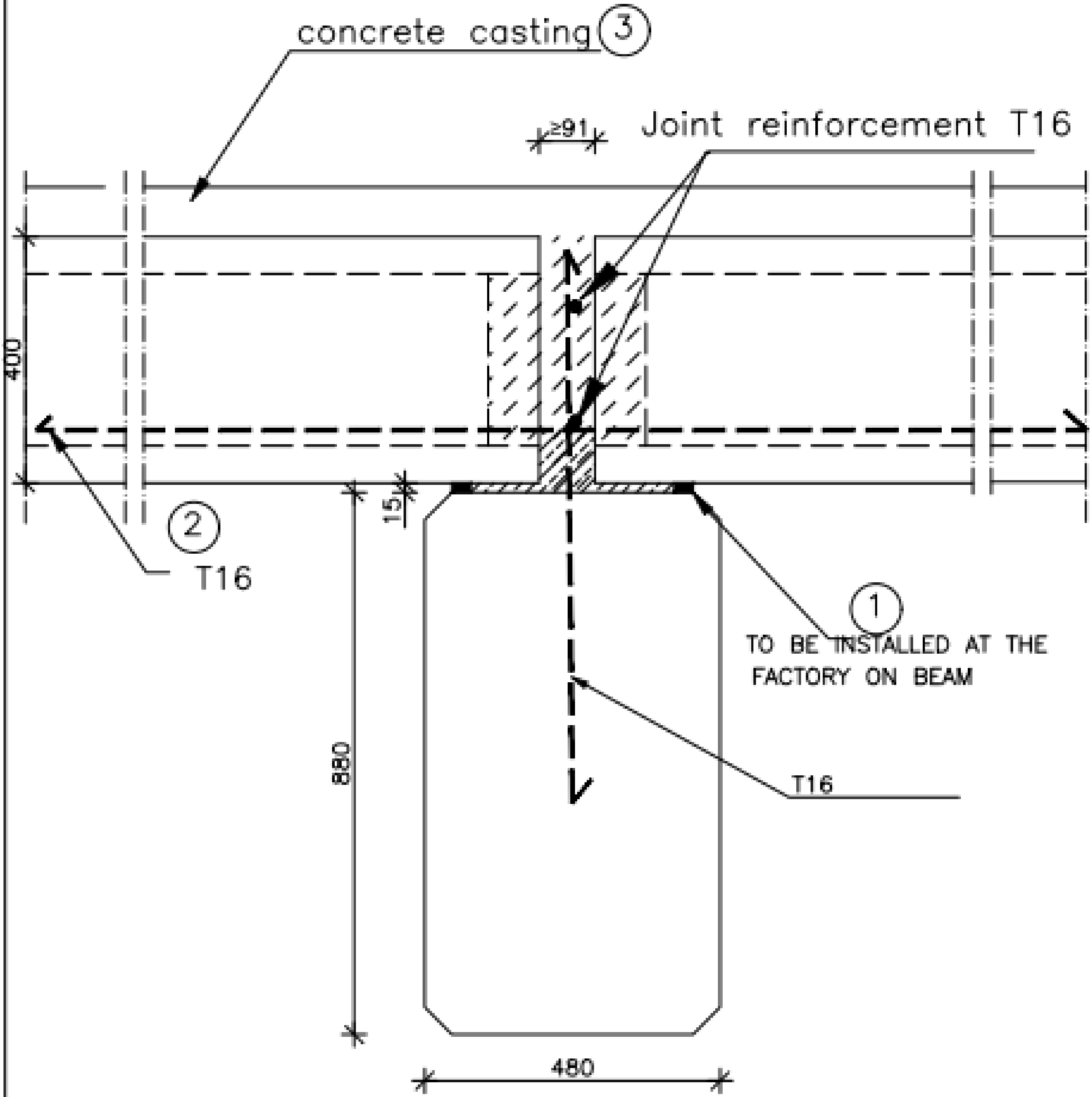
Scale: 1:10



MATERIALS				
			CODE:	
RY	PO	MAT/TAR TYPE size QUALITY	QUANTITY	NOTICE.
TXE		ANSTAR APK45C		
		Column reinforcement 16T25		
		Additional reinf. 2T12(vert.direction) and 5T12 Horiz.direction		

	Content: HOLLOWCORE SLAB AND RECTANGULAR BEAM CONNECTION
Designer:	work nro
	Date:	Designer	D2

Scale: 1:10



MATERIALS LIST					CODE:	
RY	PO	MAT/TAR TYPE	SIZE	QUALITY	QUANTITY	NOTICE
VKU	1	NEOPREN–NAUHA	20*10	SHORE 60		
RH	2	JOINT REINFORCEMENT T16		A 500 HW		
	3	Concrete castin reinforcement T8k150		A 500 HW		