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Author: Laxsha Thurupan

..................Laxsha............
(signature author)

Course coordinator: Mudiyan Nirosha Damayanthi Adasooriya

Supervisor(s): Mudiyan Nirosha Damayanthi Adasooriya

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This thesis was written in connection with the final bachelor project in the spring of 2022 at the Department of Mechanical Engineering, Building and Materials Technology at the University of Stavanger. I would like to thank my supervisor, Mudiyan Nirosha Damayanthi Adasooriya, for her good help and motivation during the assignment process.

The goal was to use this semester to learn about something new. The thesis advanced my understanding of the environment and costs in projects and gave me the opportunity to use my study background in concrete analysis. I chose a subject called “structural design” (BYG280) as a starting point to learn about slabs. This is a subject I have not had before and chose to learn myself. I have also used programs like LaTeX, Ove Sletten, and SAP2000 that I had not encountered before. In addition to that, I strengthened my knowledge of programs like ISY Calcus and Excel.

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Abstract

It is very popular to use prefabricated concrete elements instead of cast-in-situ slabs in many different construction buildings. The background of this thesis is to study how this apply to large construction buildings as well. The thesis has addressed three different influencing factors that have laid the foundation for choosing the right slab between flat slab and hollow-core slab. The main influencing factors are structural analysis, cost, and the environment. The purpose of this report is to formulate guidelines that make the choice between hollow-core slab and flat slab easier.

Each project will have different prerequisites. It is therefore expected that propulsion, cost, and environmental gas emissions will vary depending on the individual project. The thesis contains calculations of the process at the construction site, while the process outside the construction site is only discussed. Furthermore, the thesis refers to ideas on how it can be optimized about the influencing factors the thesis has addressed.

A case study was conducted where calculations and information obtained were used to analyze and select the best slab for an office building. To compare the results, the starting point was to design slabs with an area of 384 m^2. The case section presents the material-elements that were selected on ISY Calcus. Results from manual calculation and software's are added as an attachment. Except, results from ISY Calcus and Excel are shown through bar diagrams and tables.

A summary of the literature and calculation analysis shows that hollow core slabs provide lighter weight, longer spans, reduced winter costs, and less progress in the workplace. The disadvantages are factors that are more individual from project to project, and one of the primary factors is transportation. A flat slab will be the better choice for projects with a realistic completion plan and a simple foundation.

Through careful analysis of the calculations, it was concluded that the hollow-core slab would be the most profitable to use for an office building, despite the lowest environmental emissions and costs.

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b_t	width of the tension zone
c_{min}	minimum cover
$c_{min,b}$	minimum cover due to bond requirement
$c_{min,dur}$	minimum cover due to environmental conditions
c_{nom}	nominal cover for the slab
f_{cd}	the design value of concrete compressive strength
f_{ck}	characteristic cylinder strength
f_{ctm}	mean tensile strength
f_{yd}	the design yield strength of reinforcement.
f_{yk}	characteristic tensile strength
f_{yk}	characteristic tensile strength of steel
u_1	the length of the basic control perimeter
$V_{Rd,c}$	the design value of the punching shear resistance
w_1	corresponds to a distribution of shear as and is a function of the basic control
α_{cc}	coefficient account of long-term effects
γ_c	partial safety factor for concrete
γ_s	partial safety factors for reinforcing steel
ρ_{ly}, ρ_{lz}	the bonded tension steel in y- and z- directions respectively
Δc_{dev}	allowance in design for deviation
α.	the angle between the shear reinforcement and the plane of the slab.
A_{sw}	the area of one perimeter of the shear reinforcement around the column
b	the width of the considered potion of the slab
CO_2	Carbon dioxide
d	the effective depth
fck	characteristic cylinder strength in MPa
$F_{ywd,ef}$	the effective design strength of the punching shear reinforcement,
h	height
h	the design value of concrete compressive strength.
I/d	is the limit span/depth
K	factor

m	meter
m2	Square meter
mm	millimeter
N_{Ed}	longitudinal forces
Ø	the reinforcement bar size.
S_r	the radial spacing of perimeters of shear reinforcement.
u_0	the length of column periphery
V_{Ed}	the maximum shear stress
$V_{Rd,max}$	the design value of the maximum punching shear resistance.
$V_{Rd,max}$	should be in these limit
β	values varied depending on different cases.
κ	a coefficient dependent on the ratio between the column dimensions c1 and c2
ρ	required tension reinforcement ratio at mid-span to resist the moment due to the
ρ′	required compression reinforcement ratio at mid-span
ρ0	reference reinforcement ratio

1 Introduction

The construction industry generates 40% of the world's CO_2-emissions (splitkon,2019). A great amount of these emissions is caused by the concrete slabs that make up a significant portion of every building. There are several methods of producing concrete, and the traditional method is known as calcination. This creates enormous greenhouse gas emissions in the form of CO_2. But today, researchers are working on finding new climate-friendly solutions for concrete production. Global emissions can be reduced by up to 80% by using bio cement. This process uses bacteria instead of heating (Egge, 2021). Despite the fact that concrete production emits the most CO_2, other factors such as the amount of material used in slabs, and the construction method affect emissions.

An engineer has the opportunity to make choices that can reduce the project costs and, at the same time, choose better environmental solutions. For that reason, the purpose is to choose an eco-friendly and cost-effective slab for an office building. The thesis includes two types of slabs: flat slabs and hollow-core slabs. A flat slab is cast in situ, but a hollow-core slab is prefabricated. The concrete slabs are compared in terms of cost, and emissions of environmental gases based on material quantities and the size of the slab.

How can good planning and structural analysis impact the cost and CO_2-emissions? Time can be saved by utilizing the most appropriate and effective programs. However, in the thesis, software and manual calculations are used to design the slabs and calculate the amount of material needed for the slabs. Both calculation methods follow the Eurocode.

1.1 Limitations of the task

Slabs can be produced in different materials depending on the need for the project. The most commonly used materials are steel, wood, concrete, or a combination of them, but this thesis will only include an analysis of concrete slabs. Furthermore, the focus is only on dead load and characteristic live load. Other loads are considered more individual from project to project. Technical requirements will only be mentioned and discussed and will therefore lack some depth. The thesis analysis is directly related to the concrete slabs. Assumptions outside directly influencing factors will be different for each individual project.

1.2 Scope of the work

The objective of this thesis is to investigate which slab is best for an office building, considering the environment and cost. The assignment is divided into three main parts: literature review, calculation method, and case. The literature review section contains general information about concrete slabs and how these concrete slabs affect the climate and cost. The calculation part introduces manual calculation methods to design slabs, in addition to software's that can be used to analyze the slabs. In the case section, both the literature review and the calculation method will be used to find suitable solutions. This is done by first comparing the slabs separately and then together in the last part.

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Literature review

The objective of this chapter is to show the theory behind the slab, while simultaneously showing the advantages and disadvantages. Finally, demonstrate how these slabs affect CO_2-emissions and costs. All figures are hand-drawn by me.

2 Concrete as a building material

Because of its properties, concrete is the most popular material used for construction purposes. The material is made of water, cement, and aggregates that harden to a solid mass. The cement binds this mixture together due to an exothermic process in the reaction between water and cement, called hydration (Årtun et al., 2021). Additionally, the compressive strength of concrete is determined by the amount of cement and the water-cement ratio, w/c. Besides, the water-cement ratio is the number of liters per kilo of water in the cement mixture. The lower the w/c of the fresh concrete, the greater the strength when hardened (Glava, n.d.). Concrete has a long service life since it can withstand extreme weather and natural disasters. The material's adaptability allows construction companies to employ it in a variety of projects (Hanson, 2020). Strength is the primary reason why concrete has been used.

Strength of concrete

Compressive strength is one of the most important attributes of concrete. The concrete grade is quantified by the characteristic compressive strength. In general, concrete generally gains strength with time. In most designs, 28 days are used. The durability of a material is its ability to resist deterioration under the influence of the environment during its intended lifespan. Reinforced concrete has excellent durability. The durability of concrete depends on the ratio of its curing method and the skill level of its workers in mixing and pouring wet concrete. Concrete mixes can be adapted to specific environments. On the other hand, a bigger specimen could lead to a defect. Eurocode 2 (EC2) design rules are based on cylinder strengths (fck). According to EC2, a specimen with concrete grade 30 has a characteristic cylinder strength of 30 N/mm2 (Bhatt, 2011, p. 25).

Parameters influencing the compressive strength:

- Quality of raw material
- Water/cement ratio
- Coarse/fine aggregate ratio
- Age of concrete
- Compaction of concrete
- Relative humidity
- Curing of concrete

Measures for optimal hardening of the concrete

Plastic shrinkage occurs when the concrete surface evaporates and subsequently dries (SINTEF, 2007, p. 3). In circumstances when the temperature is over 20 degrees, a diffusion-tight layer can be applied over the concrete to prevent evaporation. When the surface has hardened sufficiently, water can be poured over it. If the temperature is below 20 degrees, adding accelerating chemicals or storing the concrete with the assistance of heaters and protective coverings can speed up the hardening process (Betongelement, n.d., p. 37).

3 Concrete slabs

Concrete slabs are combined with concrete and reinforcement. The reinforcement works with the concrete to carry the loads that the structure is subjected to. More specifically, concrete has high compressive strength but low tensile strength, so the steel reinforcement is designed to carry tensile forces. The amount of reinforcement needed and how it will be constructed depends on the building project. Flat slabs are cast-in-situ concrete slabs where the concrete is poured in situ. Although today, prefabricated elements have become more and more popular, such as hollow-core slabs (Thue, 2021).

3.1 Flat slab

A flat slab is a reinforced concrete slab supported directly by concrete columns without the use of beams. Figure 1 illustrates a building with flat slabs. It has many advantages. Firstly, it does not demand any beams, and this results in the height of the building (Jain, 2016). Mechanical and electrical services are easy to install, which reduces the construction time. Large bending moments and vertical forces exist in the zone of support in such structures. This leads to minimized material usage and a reduced cost (R.S.More, V. S. Sawant, 2015, p. 98).

The stability and rigidity of the structure vary as the height of the structure increases. In the case of flat slab buildings, two kinds of failure modes generally happen: punching shear failure and flexure failure.

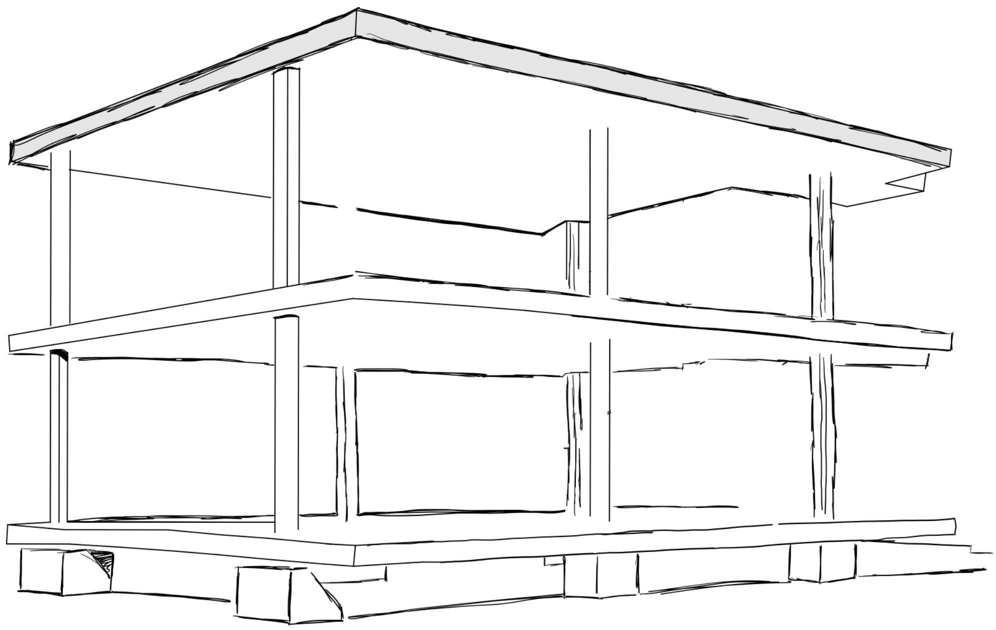


Figure 1 Flat slab

Construction in situ

Flat slabs are formed, reinforced, and cast on site. The advantage of cast-in-place slabs is that there is freedom to build in areas that are inaccessible for lifting prefabricated elements. It is easy to cast an irregular slab and change the reinforcement quantity before the cast if necessary.

Types of flat slab

There are various types of flat slab construction. Figure 2 depicts a flat slab with a drop panel, column heads, and both drop panels and column heads. Shear stresses are critical in flat slabs. Drop panels and column capitals are used to increase the shear resistance of the slab.

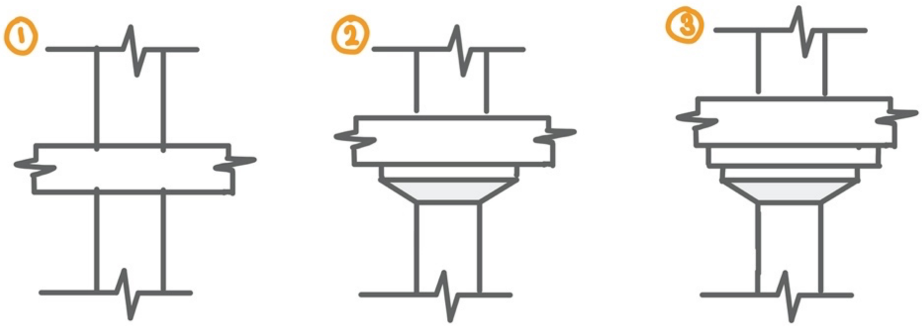


Figure 2 Floors without drop panel or column head (1), Floor panel with column head (2), Floor with drop panel and column head (3)

1. *Flat slab with drop panels*

Drop panels are efficient in decreasing shear stresses where the column is prone to punching through the slab because the moment in the slab is closer to the column. In addition, drop panels also create a higher moment of resistance where the negative moments are greater. Furthermore, it stiffens the slabs and decreases deflection (Mosley et al., 2012, p. 237).

2. *Flat slab with column head*

By widening the part of the column, it is possible to reduce the punching shear. Furthermore, by lowering the clear or effective span, it reduces the moments in the slab (Mosley et al., 2012, p. 237).

Propulsion

According to the “Betongelementboken”, it is mentioned that the reduction in construction time has a great impact on the building cost. The construction progress is influenced by several factors, such as illness, weather conditions, formwork defects, and material damage during transport. At the same time, there can also be good conditions that allow construction to progress faster than expected. Hence, laying the reinforcement is a process that is difficult to assess in the planning phase. Formwork can be built in different ways; the traditional one consists of a beam with formwork panels. Different types of formworks suitable for reuse have been developed, which also saves a lot of time.

3.2 Hollow-core slab

Hollow-core slabs are prefabricated elements that are normally used in offices, commercial buildings, homes, schools and hospitals (Forsèn et al., 2010, p. 103) . The standard width of hollow-core elements is 1200 mm. The dead load and live load have a significant influence on displacement. The element is available in several sizes. Elements with a thickness of 200 mm to 520 mm can be constructed for spans of roughly 6 m to 18 m, depending on the desired load capacity and span. Table 1 shows the properties of the standard elements. The most commonly used hollow-core elements are HD200 and HD265 (which have thicknesses of 200 mm and 265 mm) (Dekkesystemer AS, 2019, p. 7). Figure 3 depicts an example hollow-core element model.

Table 1. Hollow-core elements type (Helgeland Betong, n.d.)

Element	***Height***	***Weight (KN/m^2)***	***Max length (m)***
Hollow-core slab	200	2,5	10
	265	3,6	12
	320	4,0	15
	400	5,0	17
	500	6,4	21

Installation and production

Production in the factory takes place indoors, which means it is independent of the outside climate. This ensures a constant and high quality of the products all year round. The construction moisture is significantly lower because the elements have hardened before they arrive at the construction site. Thus, productivity and accuracy are much better in hollow-core elements than in cast-in-place slabs. Installation of concrete elements must be carried out in accordance with NS-EN 13670 + NA. There must also be documentation of lifting procedures, storage instructions, and installation descriptions (Hermod, 2018, p. 38)

When lifting the hollow core elements, they should be lifted with a yoke or lifting clips. Since the clamps can easily slip due to the horizontal forces. Lifting clips cannot be used if they have an irregular design (Reiersen, 2018). Prestressing strands are the only reinforcement in the slab. It is reinforced in the direction of the prestressed tension reinforcement.

Propulsion

Compared to flat slabs, prefabricated elements require only assembly and grouting. This leads to reduced work and time. Although the working hours on the construction site are shorter, transport can be a major obstacle. Transportation has a great influence on the construction site's progress. Since most construction sites do not have storage capacity for the elements, it usually depends on fast delivery. As a result, propulsion is contingent on the prefabricated elements arriving in the correct location and in the right order. For faster progress, a new crane needs to be installed, although this might be costly. To make the casting and forming process more efficient, more competent workers are required.

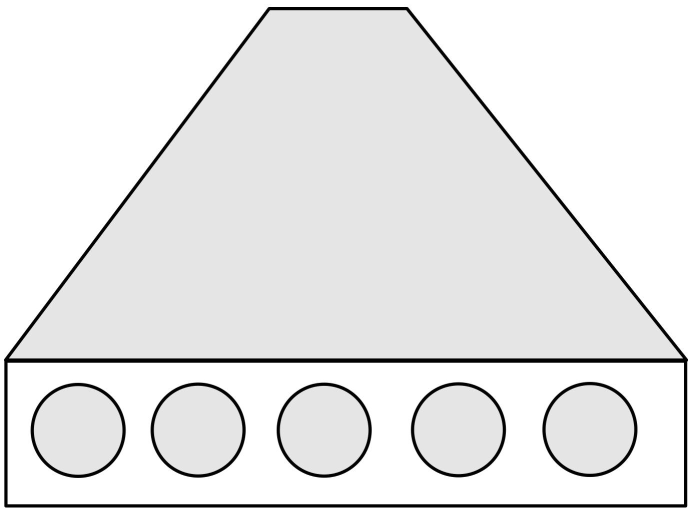


Figure 3 Hollow core slab

3.3 Advantages and disadvantages of slabs

Table 2 Advantages and disadvantages of hollow-core slab and flat slab

Slab type	***Advantage***	***Disadvantage***
Hollow core slab -	▪ Large spans with small construction height ▪ These are prefabricated elements, so the only work is to place them directly on the support system. ▪ Hollow core slab reduces the need for ceilings and can be painted directly.	▪ The disadvantage is that they don't meet the sound requirement and therefor action must be taken to allow this. These measures can be either laying floating floors or lowering the ceiling.
Flat slab	▪ It is possible to change as reinforcement until casting ▪ start working faster than with prefabricated elements ▪ Easy to cast irregular slab	▪ Time consuming

4 Cost and Environment

4.1 Importance of minimizing CO_2- emissions

The building industry has a significant impact on the environment. Heavy transport on the road causes 3.1 million tons of CO_2 equivalent (Byggsektor, 2015) .When it comes to new buildings and rehabilitation of old buildings, more and more builders are emphasizing the material environmental properties. It is a legal necessity to use materials that have the least possible impact on the environment. Additionally, use building materials with a low content of environmental toxins (Lavenergiprogrammet, 2020).

Global warming is when the earth's natural greenhouse effect is increased by environmental gases. The greenhouse effect is caused by the earth's atmosphere absorbing heat from the sun and warming the entire planet. The atmosphere contains of 78 percent nitrogen, 21% oxygen, and 1% other minor gases (NASA, 2021). CO_2 in particular intensifies the natural greenhouse effect and raises the earth's surface temperature (Buchanan, 2004). Figure 4 shows the emissions of the cement factory. It is important to minimize CO_2 emissions for several reasons, such as air quality and climate change. Smog is a more visible form of air pollution. When the temperature and humidity increase, carbon dioxide emissions lead to smog, which has negative effects on respiratory health (Somma, 2021).

How can we minimize greenhouse gas emissions in the building industry? Concrete production is a primary influencing factor on CO_2- emissions. In contrast to traditional concrete production, numerous ecologically friendly concrete options have been developed. Less CO_2 is emitted when ecologically friendly concrete is used. An Environmental Product Declaration (EPD) is a concise document that summarizes the environmental impact of a product. These publications detail the amount of greenhouse gas emissions that products emit during manufacture. Today, software's such as ISY Calcus is used. The software ISY Calcus offers CO_2 calculators that allow you to estimate how much environmental gas is emitted from the materials.

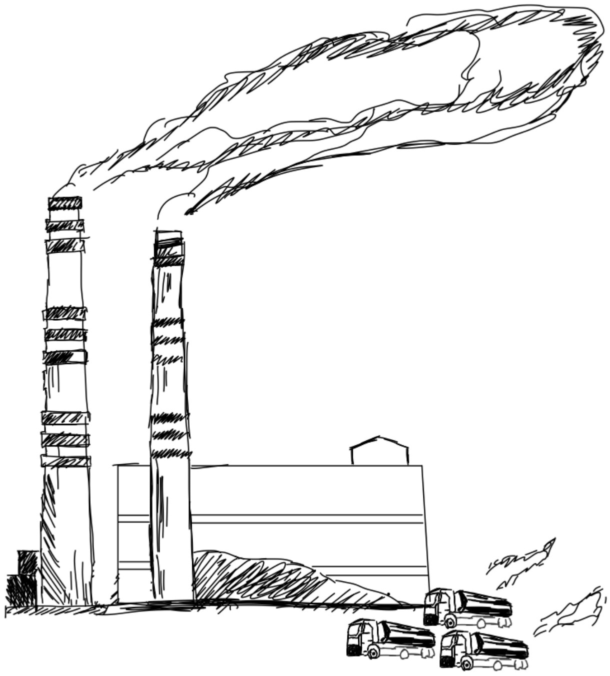


Figure 4 CO_2-emissions of a cement factory

4.2 Optimizing the construction cost

Table 3 shows the cost reducing strategies with use of hollow- core elements (Forsèn et al., 2010, p. 116). The cost of a flat slab versus a hollow-core slab is different. It can be used as reusable formwork to save time and money when using flat slabs. Due to the use of a traditional casting method, hollow core slabs offer numerous alternatives in terms of layout. It can be picked between short and long spans with various thicknesses.

Table 3 Cost reducing methods in hollow core slab

Cost reducing methods
Think elements from the beginning
Use as many standard elements as possible
Limit the number of variants
Use the dimensioning diagram when pre-projecting
Evaluate the performance of technical and mechanical systems at an early stage

Calculation

This chapter introduces the calculations needed to solve the problem. The structural analysis calculations affect the cost and environmental results. Furthermore, formulas and methods for designing flat slabs and hollow-core elements are described in detail. Additionally, this chapter introduces a variety of calculation programs, including SAP2000, E-bjelke, ISY-Calcus, and Excel.

5 Structural system

5.1 Loads

5.1.1 Mechanism of load transfer

The major load transfer mechanisms that develop in member slabs are introduced in tables 4 and 5 and are further used in the thesis.

Table 4 Description of the mechanism and their weaknesses (1)

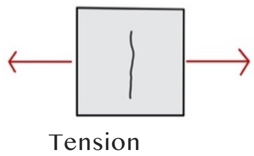
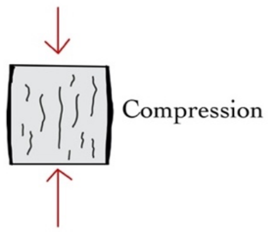
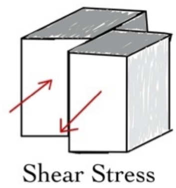
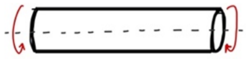
Tension	**Compression**	**Shear**
		
<u>Failiure mechanicm</u> When a member is stretched by forces parallel to its axis, it creates a stress known as tension. With reinforced concrete, the concrete cracks when exposed to the smallest tensile forces. Once this occurs, the tensile stress is exerted only by the reinforcement passing through the cracks (O'Brien et al., 2012, p. 15).	<u>Failiure mechanism</u> Compression occurs when the particles of a material are pressed against each other. Compression has some flexural rigidity to failure from buckling. The buckling strength of a compression member depends not only on its material properties but also on its length, its cross-sectional geometry, and the type of support (O'Brien et al., 2012, p. 15)	<u>Failiure mechanism</u> The introduction of transverse forces that are perpendicular to the member axis leads to the creation of shear stresses. Reinforced concrete is not homogenous and is neither linear nor elastic in shear failure (O'Brien et al., 2012, p. 15).

Table 5 Description of the mechanism and their weaknesses (2)

Flexure	**Torsion**
 Flexure	 Torsion
Failiure mechanism If the load is applied outside the axial plane of the member, the load will also be transmitted through the torsional force. Members that transfer lateral loads horizontally to one or more bearings do so, at least in part, by a shearing mechanism. In addition to these two devices, the transversely loaded members transmit their loads through bending. The center load creates a bending moment and causes the member to sag. Because of its minimum tensile strength, concrete is assumed to crack at its minimum tensile stress. In reinforced concrete members, the cracks during bending run through the tension zone to the neutral axis. The failure of the structure is prevented by the longitudinal steel, which overcomes the cracks and withstands the tensile force. Under the much larger final load for which the sections are designed, the stress distribution becomes non-linear. Unlike daily loads, reinforced concrete acts only as a linear elastic material (O’Brien et al., 2012, p. 17).	Failiure mechanism Torsion occurs in the member when an external transverse force is applied outside the plane comprising the member's axis. The impact of torsion is to cause a torsional action in the loaded member. The magnitude of the member, depends on the exerted torque (Pe), the length of the member (l), the cross sectional geometry, and the elastic shear modulus of the material. Torsional failure in concrete is pioneered by tensile cracks at the surface of the member. This is because the torsional stress on a section increases from zero at the center to a maximum at the edges. Torsional cracking is resisted in reinforced concrete by the closed connections. Overlapping of the links in this way ensures continuity of reinforcement around the section. (O’Brien et al., 2012, p. 17)

5.1.2 Types of structural load

Permanent and variable load

Permanent loads are self-loads on the structure that occur over time, whereas variable loads occur just temporarily or vary (Norsk standard, 2008). Since self-loads are defined as permanent loads, they include not only the load of the material itself, but also screeds, light walls, and technical systems. According to Byggteknisk forskrift, "the building shall be located, designed, and constructed in such a way that satisfactory safety against damage or significant inconvenience from natural stresses is achieved" (Byggteknisk forskrift (TEK17), 2017). Influences from people, equipment, snow loads, wind loads are all examples of variable loads. Variable loads on floors, balconies and stairs in buildings are defined in table 6 (Norsk standard, 2008)

Table 6 Characteristic values of imposed loads

Categories of loaded areas	*q_k [kN/m^2]*	*Q_k [kN]*
Category A	2,0	2,0
- *Floors*	2,0	2,0
- *Stairs*	3,0	2,0
- *Balconies*	4,0	2,0
- *Loft with low ceiling or limits access*	1,0	1,5
Category B	3,0	2,0
Category C		
- *C1*	3,0	4,0
- *C2*	4,0	4,0
- *C3*	5,0	4,0
- *C4*	5,0	4,0
- *C5*	5,0	4,0
Category D		
- *D1*	5,0	4,0
- *D2*	5,0	7,0

5.2 Ultimate limit state design

Failure at its most extreme can occur with little or no notice due to its fragility. Reinforcement fails in tension before the concrete fails in compression, and the member deforms significantly before it collapses completely. Building fire resistance is also an important safety consideration. The concrete functions as a barrier for the reinforcement. As a result, in the case of a fire, the concrete elements will maintain their strength long enough for the people to safely leave the structure.

Load factors

The ultimate limit state is a design that limits the stress to which materials are exposed to ensure the safety of a building and its inhabitants. The design loads in the ultimate limit state are obtained by multiplying the various loads with the coefficients in table 7 (Norsk Standard, 2008a).

Table 7 Ultimate limit state coefficients from Euro code, NS EN 1990

Ultimate limit state coefficients	
$\gamma = 1,2$	Load factor with unfavorable self-load
$\gamma = 1,35$	Load factor with unfavorable self-load
$\gamma = 1,5$	Load factor for the dominant payload
$\gamma = 1,05$	Load factor for other payloads

Ultimate limit state:

B1: rf = 1,35*q + 1,05*g

B2: rf = 1,2*q + 1,5*g

(q is self-load and g is payload)

5.3 Basis of calculation

Testing and calculations have helped to develop standards for the quantity of reinforcement and concrete that can be sized to give enough strength based on the concrete and reinforcement quality. Eurocode are European standards, abbreviated as EC. General rules and regulations for buildings are in Eurocode 1 section 1-1. To get a deeper insight into how slabs are designed, the equivalent frame method and finite element method can be used to calculate the flat slab. The equivalent frame method is generally used for rectangular slabs, while the finite element method is used for slabs with irregular plans (Lee & Kim, 2004, p. 1). For hollow core slabs, a dimensioning diagram and the software E-Bjelke can be used to determine the right element. Both hollow core slab and flat slab follow the NS-EN 1992-1-1:2004+NA:2008 and NS-EN 1990-1-1:2002+NA:2008.

5.4 Structural analysis of flat slabs

Two different methods have been introduced in this chapter: the equivalent frame method and the finite element method. A simplified method and software tools can be used to find the bending moment and deflection.

5.4.1 Equivalent frame method (manual calculation)

Flexure, punching shear, and deflection are all important while designing a flat slab. Unless there is a cantilever, the span is less than the interior span or moments are bigger in the end span and should be examined.

<u>Equivalent frame method</u>

The equivalent frame method uses an elastic frame analysis to compute the positive and negative moments in the various panels in the slab. Once the positive and negative moments are known, they are divided between middle strips and column strips. According to Eurocode 2, the vertical load and the stiffness of the slab are calculated using the full width of the slab. The bending moments obtained from the analysis are distributed across the width of the slab to account for variations in the bending moment. Eurocode 2 (EC2) does this using the concept of column strips and middle strips.

These are strips within the slab which are reinforced to withstand different fractions of the total moment (Bhatt, 2011, p. 326).The column strip resists directly on the columns and absorbs a greater proportion of the total moment than the middle strip. However, for the sag moment, the effect is less pronounced as the flexible middle strip support causes the hog to decrease and the sag to increase. Table 8 shows the apportionment of bending moment for a flat slab (Norsk Standard, 2008b).

Table 8 Simplified apportionment of bending moment for a flat slab.

	Column strip	*Middle strip*
Hogging moment (neg)	60-80%	40-20%
Sagging moment (pos)	50-70%	50-30%

Design procedure for flat slabs based on Eurocode 2

The main reference is NS-EN 1992-1-1:2004+NA:2008. All figures and pictures are referred to by Eurocode 2.

Concrete cover

The cover is necessary to provide adequate durability, a safe transfer of bond force, and fire resistance. It is also referred to as a nominal cover. The concrete cover is the distance between the surface of the reinforcement closest to the nearest concrete surface. (Including links, stirrups, and surface reinforcement where relevant).

The nominal cover for slabs is calculated by:

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

$$c_{min} = Max\left\{c_{min,b}\ ;\ c_{min,dur}\ ;\ 10mm\right\}$$

Δc_{dev} is allowance in design for deviation =10mm (NA 4.4.1.3)
c_{min} is the minimum cover (NA 4.4.1.2)
$c_{min,b}$ minimum cover due to bond requirement (NA4.4.1.2 and Table NA.4.2)
$c_{min,dur}$ minimum cover due to environmental conditions (NA 4.4.1.2 and Table NA 4.4N) depends on exposure class and durability class.

Effective depth of slabs

The effective depth has a major impact on a panels ability to resist a bending moment. The depth of the slabs does not determine the overall depth of the structures. However, a small increment in slab depth contribute greatly to the self-weight of the structure that significantly increases the bending moments in the slab itself and adds load to all columns and foundations which support the slab. The effective depth requirements for flat slabs can also be governed by punching shear.

$$D = h - c_{nom} - \frac{\emptyset_l}{2} \text{ (longitudinal direction, long span)}$$

$$D = h - c_{nom} - \emptyset_l - \frac{\emptyset_t}{2} \text{ (Transverse direction, shorter span)}$$

h is the design value of concrete compressive strength.
$\emptyset$ is the reinforcement bar size.
c_{nom} nominal cover for the slab. *(refer table 2.1N)*

Material properties

Strength class for concrete

$$f_{cd} = \frac{\alpha_{cc} * f_{ck}}{\gamma_c}$$

f_{cd} is the design value of concrete compressive strength
α_{cc} is 0,85 and taking coefficient account of long-term effects *(refer section NA.3.1.6).*
f_{ck} characteristic cylinder strength *(refer table 3.1)*
γ_c partial safety factor for concrete *(refer table 2.1N)*

Strength class for steel

$$f_{yd} = \frac{f_{yk}}{\gamma_s}$$

f_{yd} is the design yield strength of reinforcement.

γ_s partial safety factors for reinforcing steel , see table 9.

f_{yk} characteristic tensile strength (Annex C and Table NA.3.5 (901)).

Table 9 Partial factors for materials for ultimate limit states

Design situations	γ_c *for concrete*	γ_s *for reinforcing steel*	$\boldsymbol{\gamma_s}$ *for prestressing steel*
Persistent & Transient	1,5	1,15	1,15
Fatigue	1,5	1,15	1,15
Accidental	1,2	1,0	1,0

Design for flexure

The total bending moment obtained from the analysis should be distributed over the width of the slab. In elastic analysis, negative moments tend to be concentrated towards the centerlines of the column (N. Adasooriya, personal communication, September 8, 2020). In Figure 7, the panels are divided into column and middle strips, and the bending moments should be determined.

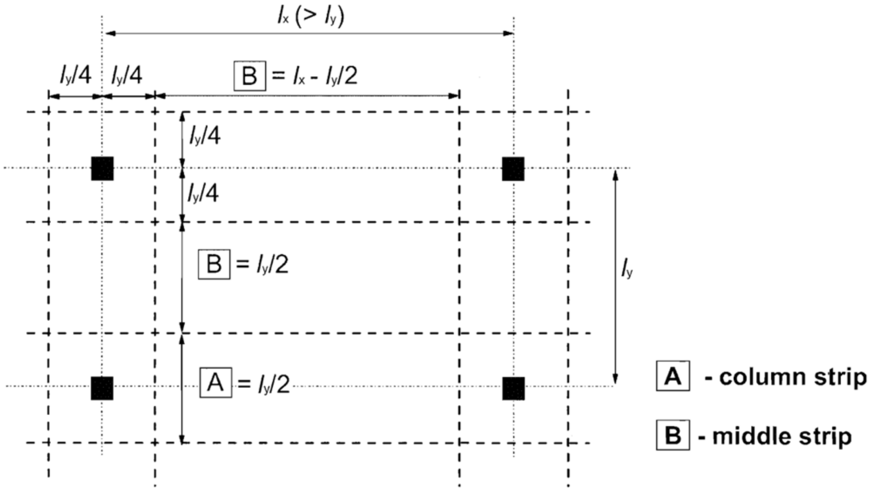


Figure 5 Division of panels in flat slabs (section 1.1.2 in EC2)

Design flexural reinforcement

The moment capacity for single r/f balanced sections in concrete is class B20-B45 (Mcd).

$$M_{cd} = 0{,}293 f_{cd} b d^2$$

$$\text{Lever arm } z = \left(1 - 0{,}18 \frac{M_{Ed}}{M_{cd}}\right) d. \quad and \quad z \leq 0{,}95d$$

$$Design\ reinforcement\ \ A_s = \frac{M_{Ed}}{f_{yd}}$$

d is the effective depth
b is the width of the considered potion of the slab

Minimum reinforcement area

$$A_{s,min} = 0{,}26\frac{f_{ctm}}{f_{yk}}b_t d. \quad \text{but not less than } 0{,}0013 b_t d$$

b_t is the width of the tension zone
f_{yk} is characteristic tensile strength of steel
f_{ctm} is mean tensile strength *(Table 3.1)*
d is effective depth

Maximum reinforcement area

$$A_{s,max} = 0{,}04A_c$$

Maximum spacing of reinforcement bars ($S_{max.slab}$)

The reinforcement that is required structurally is principal reinforcement.

For slabs following maximum spacing rules :
The principal reinforcement, $3h \leq 400mm,\ h \text{ is the total depth of the slab}$
Secondary reinforcement, $3{,}5h \leq 450mm$

In areas with concentrated load or maximum moment:
The principal reinforcement, $2{,}5h \leq 250mm$
Secondary reinforcement, $3{,}5h \leq 400mm$

Deflection control

In general, deflection control can be achieved by the span/depth ratio. There are two ways to check the deflection control: the simplified method and the rigorous method. The rigorous method is the most useful method for calculating deflection. It is an appreciated technique to define an actual deflection assumption but should only be used with computer simulations. This will be calculated with the finite element method using linear elastic analysis. The deflection of the beam or slab is determined at multiple points along the length of the beam or slab and is repeated for all loading stages (Eigelaar, 2010, p. 73)The simplified method is a method of controlling the deflection without structural analysis. The maximum deflection under quasi-permanent loads is normally limited to span/250. In this subchapter, the simplified method formulas are presented.

Control of deflection without structural analysis:

$$\frac{l}{d} = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3,2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \leq \rho_0$$

$$\frac{I}{d} = K\left[11 + 1,5\sqrt{f_{ck}}\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if } \rho > \rho_0$$

I/d is the limit span/depth
K is the factor to consider in the different structural systems. Recommended values for K are given in table 10.
ρ0 is the reference reinforcement ratio = $\sqrt{fck} * 10^\wedge - 3$
ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)
ρ′ is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)
fck characteristic cylinder strength in MPa

Factor K

Table 10 Recommended values for K (table 7.4N)

Structural system	K
Simply supports beam, one – or two-way spanning simply supported slab	1,0
End span of continuous beam or one-way continuous slab or two- way spanning slab continuous over one long side	1,3
Interior span of beam or one-way or two-way spanning slab	1,5
Slab supports on columns without beams (flat slab) (based on longer span)	1,2
Cantilever	0,4

Factors F1,F2 and F3

Table 11 Factors influencing the actual limiting values of a flat slab (Section 7.4.2)

F1	1,0 (for flat slabs bf=bw)
F2	1,0 (assume no brittle partitions)
F3	$\frac{\boldsymbol{As,prov}}{\boldsymbol{As,req}} \leq \mathbf{1,5}$

Figure 6 is a graph of the limiting value for span-effective-depth ratios. L/d can be calculated by dividing the reinforcement percentage by the "actual L/d".

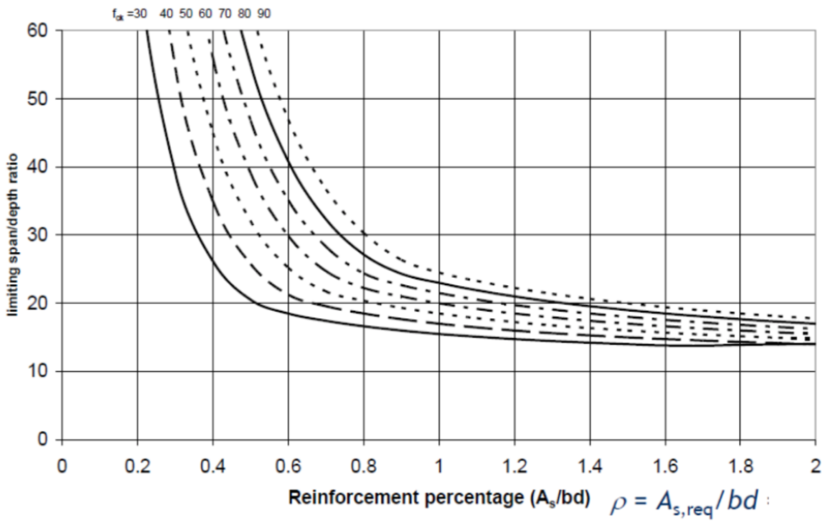


Figure 6 Limiting value for span-effective-depth ratios, it's a graphical form of equation.

The deflection is controlled if it conforms to the formula :

$$\left(\frac{Acutal\ limiting\ value\ of\ span}{effective\ depth}\right) > \frac{L}{d}$$

Punching shear

Punching failure is the shear failure of a structural member due to a high-concentrated load on a relatively small area of the structure. Punching shear failures in flat slabs develop in areas such as columns, capitals, or walls. A critical section is located at a distance of d/2 from the face. We are providing capital columns and drop panels to reduce the stress. Depth is the governing factor in punching strength. Small thickness with a bigger column or the opposite will lead to punching failure.

<u>Checks</u>

At the column perimeter, or the perimeter of the loaded area, the maximum punching shear stress should not be exceeded.

$$V_{Ed} \leq V_{Rd,max}$$

V_{Ed} is the maximum shear stress
$V_{Rd,max}$ is the design value of the maximum punching shear resistance.
$V_{Rd,max}$ should be in these limit

$$V_{Rd,max} = 0{,}4 \cdot v \cdot f_{cd} \leq 1{,}6 \cdot V_{Rd,c} \cdot \frac{u_1}{(\beta \cdot u_0)}$$

Where v is a factor determined as:

$$v = 0{,}6 \cdot (1 - \frac{f_{ck}}{250})$$

No punching shear reinforcement is required if:

$$V_{Ed} \leq V_{Rd,c}$$

Where $V_{Rd,c}$ is the design value of the punching shear resistance of a slab without punching shear reinforcement, and will be designed accordance to 6.4.4 in EN 1992-1-1. Shear reinforcement is required if this condition is not satisfied. Then, it should be designed in accordance with section 6.4.5 in EN 1992-1-1.

Load distribution and basic control perimeter

The basic control perimeter u_1 may normally be taken to be at a distance of 2,0d from the loaded area and should be constructed to minimize its length. Figures 7 and 8 show the typical control perimeters.

The effective depth of the slab is assumed constant and may typically be taken as:

$$d_{eff} = \frac{d_y + d_z}{2}$$

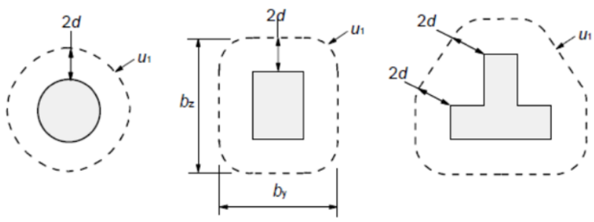


Figure 7 Typical basic control perimeter around loaded area

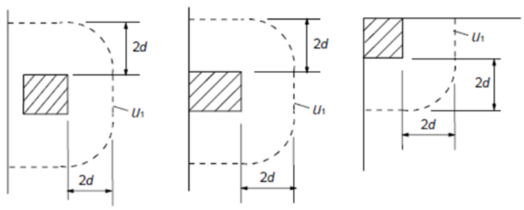


Figure 8 Basic control perimeters for loaded areas close to or at edge or corner

Factor β

Due to asymmetric loading, there is always a moment transfer from the slab to the column, which affects the shear stress distribution around the critical control section. The value considers the unbalance moment and, at the same time, takes the geometry into account (Blažek, 2012). The design shear stress along the control section is therefore increased by multiplying this value by:

$$\beta = 1 + \kappa \cdot \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{w_1}$$

κ is a coefficient dependent on the ratio between the column dimensions c1 and c2 : its value is function of the proportions of the unbalanced moment transmitted by uneven shear and by bending and torsion.

β values varied depending on different cases.

$\frac{M_{Ed}}{V_{Ed}}$ is the eccentricity of the load

u_1 is the length of the basic control perimeter

w_1 corresponds to a distribution of shear as and is a function of the basic control perimeter u_1

The approximate values for interior, edge and corner columns may only be used if all the following conditions are met:

- Lateral stability does not depend on frame action
- Adjacent spans do not differ by more than 25% (section 6.4.3)
- The slab is only under uniformly distributed loads
- The moment transferred to edge and corner columns are not larger than

$$M_{td,max} = 0{,}25 b_e \cdot d^{\,2} \cdot f_{cd}$$

The approximated values are given in the figure 9:

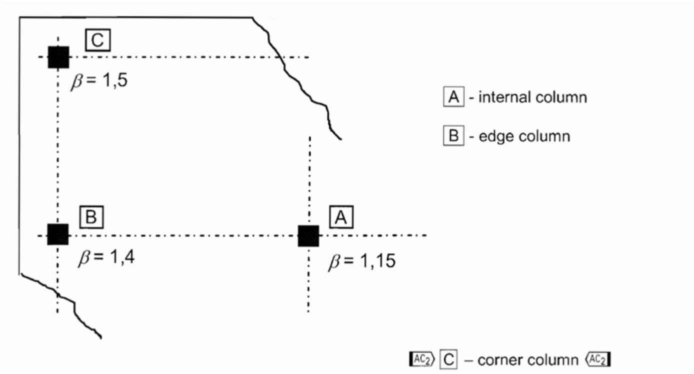


Figure 9 Recommended values for β

Maximum shear stress

$$V_{Ed} = \beta \cdot \frac{V_{Ed}}{u_1 \cdot d}$$

Punching shear resistance of slabs and column bases with shear reinforcement

$$v_{Rd,cs} = 0{,}75 \cdot v_{Rd,c} + 1{,}5\left(\frac{d}{s_r}\right) \cdot A_{sw} \cdot f_{ywd,ef} \cdot \left(\frac{1}{u_1 \cdot d}\right) \cdot sin\alpha \leq k_{max} \cdot V_{Rd,c}$$

A_{sw} is the area of one perimeter of the shear reinforcement around the column

S_r is the radial spacing of perimeters of shear reinforcement.

$F_{ywd,ef}$ is the effective design strength of the punching shear reinforcement, according to

$$f_{ywd,ef} = 250 + 0{,}25 \cdot d \leq f_{ywd}$$

d is the mean of the effective depths in the orthogonal directions (mm).

α. is the angle between the shear reinforcement and the plane of the slab.

$$V_{Ed} = \beta \cdot \frac{V_{Ed}}{u_0 \cdot d} \leq V_{Rd,max}$$

u_0 is the length of column periphery

Punching shear resistance of slabs without shear resistance

$$V_{Rd,c} = C_{Rd,c}k\,(100\rho_1 f_{ck})^{\frac{1}{3}} + 0{,}10\sigma_{cp} \geq v_{min} + 0{,}10\sigma_{cp}$$

Where,

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0$$

$$\rho_1 = \sqrt{\rho_{ly} + \rho_{lz}} \leq 0{,}02$$

Where ρ_{ly}, ρ_{lz} is the bonded tension steel in y- and z- directions respectively. ρ_{ly} and ρ_{lz} should be calculated as mean values considering a slab width equal to the column width plus d each side.

$$\sigma_{cp} = \frac{\sigma_{c,y} + \sigma_{c,z}}{2}$$

σ_{cy} *and* σ_{cz} are the normal stresses in the critical section in y- and z- directions.

$$\sigma_{c,y} = \frac{N_{Ed,y}}{A_{c,y}} \text{ and } \sigma_{c,y} = \frac{N_{Ed,y}}{A_{c,y}}$$

Where $N_{Ed,y}$ and $N_{Ed,z}$ are the longitudinal forces across the full bay for intern columns and the longitudinal force across the control section for edge columns. The force may be from a load or prestressing action. A_c is the area of concrete according to the definition of N_{Ed}.

5.4.2 Finite element method : SAP2000

Finite element method

Linear elastic methods of analysis are based on the assumption of linear elastic behavior. The use of finite element analysis has increased significantly in the last few years, and the analyses can provide a more accurate study of a structure (Pacoste, C. et al., 2013, p. 4).The major advantage is that the model can be more flexible because there are no restrictions on column support. The results from the software include moment and shear envelopes and contours of structural deformation.

How it works:

The slab is idealized as a collection of discrete slab elements joined only at the nodes. This model can be viewed as a special case of the stiffness method. The moment will change rapidly, especially at the supports. Therefore, it should be used with a reasonably large number of elements.

SAP2000

SAP2000 stands for Structural Analysis Program2000 and is a structural analysis and design software produced by Computer and Structures Incorporated (CSI). The program provides an analysis of the structure, and it is possible to implant columns on the designed slab and check the design in the software. Additionally, the structural analysis software is based on the finite element method, which can be used to analyze linear and nonlinear models (Rivera et al., 2015). Figure 10 shows the deflection of a slab in the SAP2000 software.

How to use it

A user guide is included with the program (Pyle, 2004). The source list includes a reference to the manual. The manual outlines what is required to accomplish the desired results.

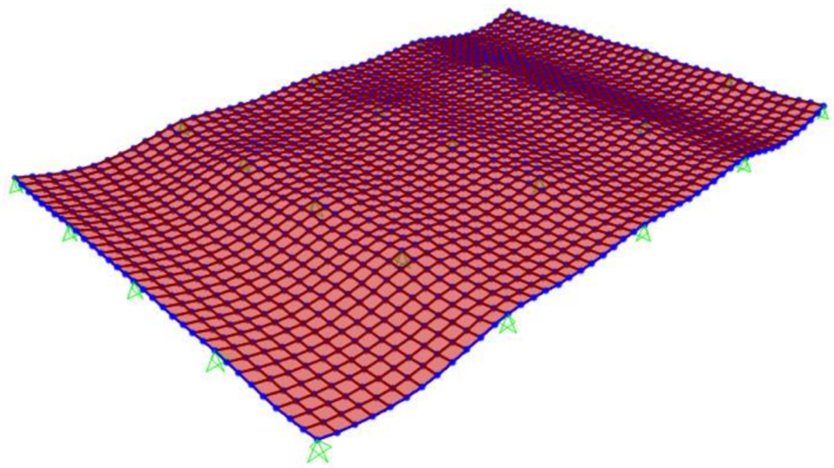


Figure 10 SAP2000

5.5 Structural analysis of Hollow core slab

Dimensioning diagram and E-bjelke are two methods introduced in this subchapter. Both approaches may be used to determine if the element is capable of withstanding the loads.

5.5.1 Dimensioning diagram (manual calculation)

Figure 11 illustrates how much load may be added in addition to the components' self-weight. The maximal reinforcement is limited by the stated load-bearing capacity.

Calculation method:

The load consists of self-load (g) and live-load (p). The weight of the slab elements has already been corrected, and it is therefore not included in the formula.

$$0.9g + p \leq \ limit\ of\ use$$

The accurate calculation of the load capacity depends on the reliability class, the load combination, the fire class, and the ration between dead load and variable load. The shear stress can be dimensioned between the red and blue curves. Moreover, the stripped line needs to be examined more closely for deformation (Forsèn et al., 2010, p. 105)

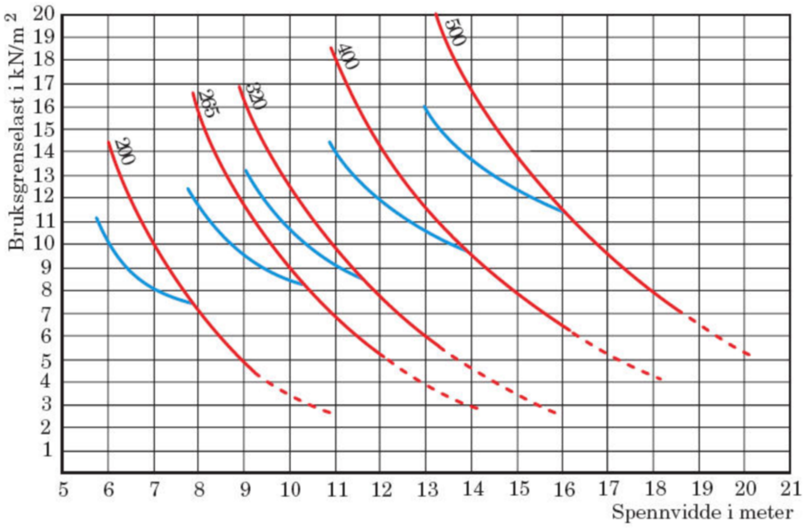


Figure 11 Dimensioning diagram (Forsèn et al., 2010, p. 105)

5.5.2 E-bjelke

Ove Sletten is a software package created by engineer Ove Sletten that includes multiple applications for various components. E-bjelke is included in the Ove Sletten package, and the software is shown in figure 12. The program is used to calculate prestressed or slack-reinforced beams. This software can also be used to design and analyze hollow core elements. The program conforms to European standards. Furthermore, programs are simple to use and deliver quick and precise results.

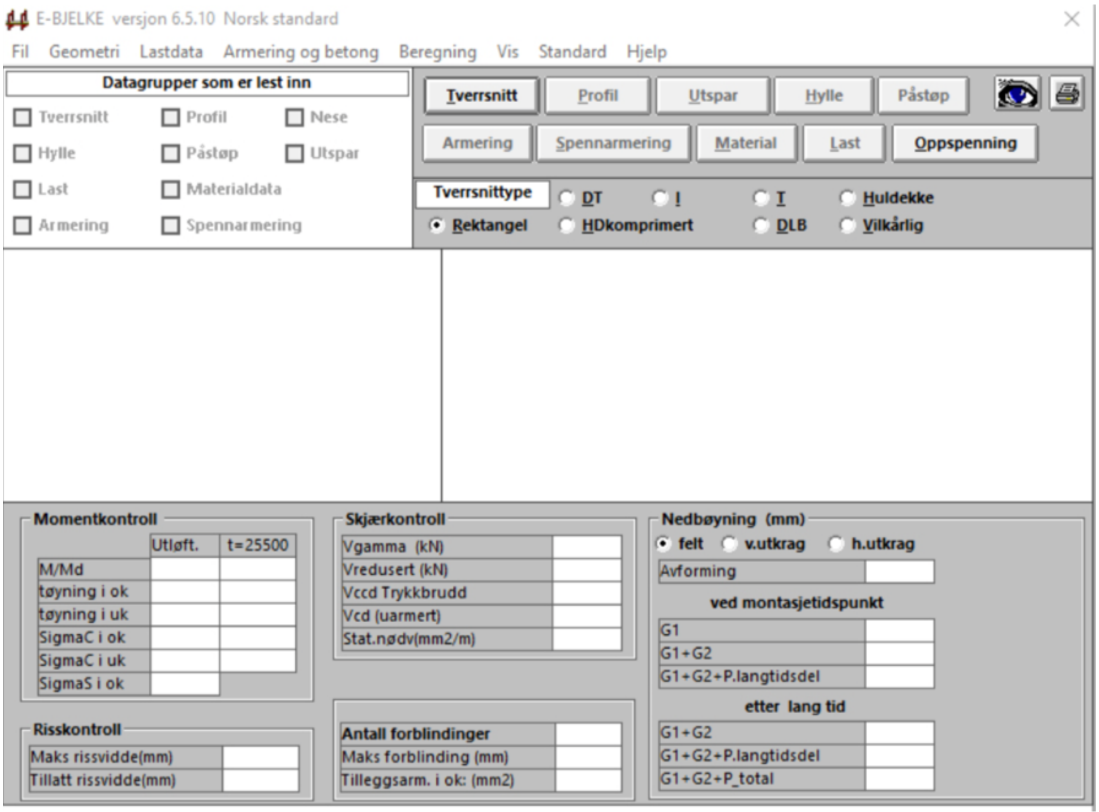


Figure 12 E-bjelke

How to use it

A user guide is included with the program. The source list includes a reference to the manual (Sletten, 2010). The manual outlines what is required to accomplish the desired results.

6 Cost and Environment analysis by software's

6.1.1 ISY Calcus

The comparison of materials and items based on greenhouse gas emissions is a good starting point when selecting slabs. Builders, planners, architects, and contractors utilize ISY Calcus as a computation tool. The software follows the continuous development in the construction industry. Estimated cost, CO_2-emissions, and life cycle cost tools are included in the software (Nois, n.d.). The costs and CO_2 emissions are the focus of this thesis. There are several types of the same element in the program, each with a different price and emission level. The CO_2 emissions and costs of the slab can be calculated by entering the required quantity and dimension from the structural analysis. In that case, the result from ISY Calcus is determined by the structural analysis result. Figure 13 shows an ISY Calcus file produced in the work process.

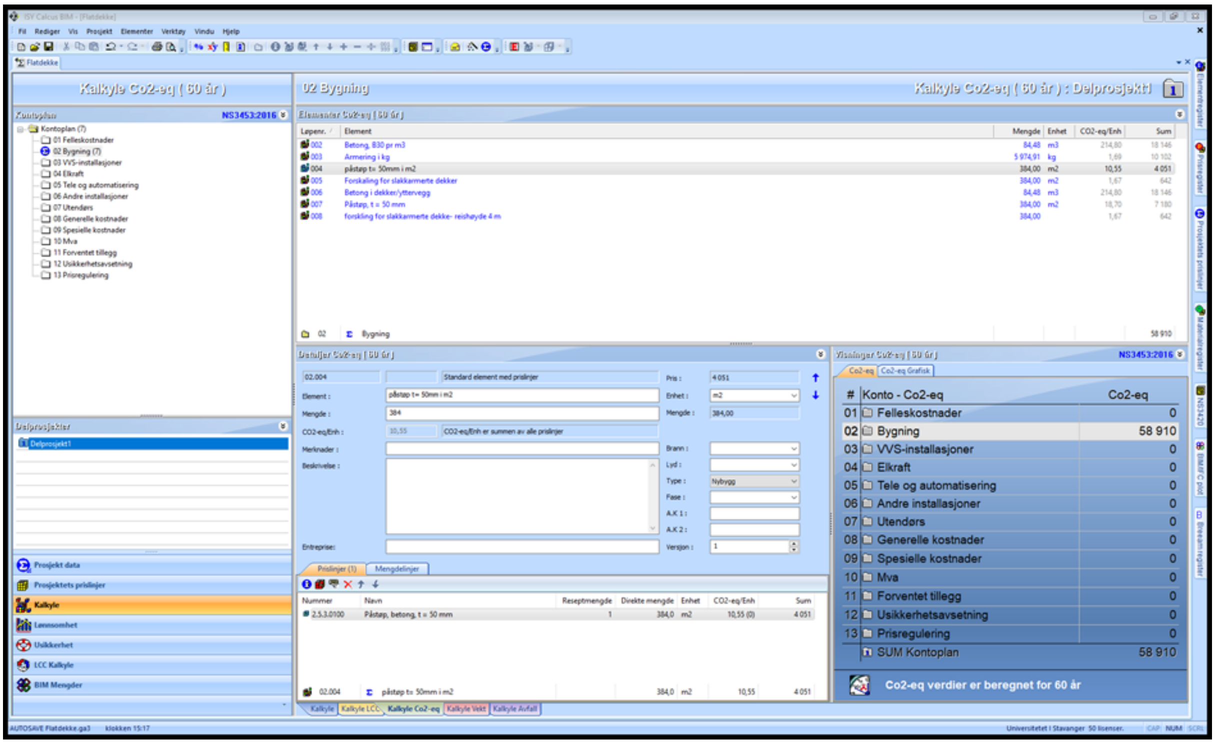


Figure 13 ISY Calcus

How to use it:

A user guide is included with the program. The source list includes a reference to the manual (Johansen, 2016). The manual outlines what is required to accomplish the desired results.

6.1.2 Excel

Excel is a Microsoft software tool that utilizes tables to organize numbers and dates with the use of formulae and functions (*Excel Definition*, n.d.). The program is a useful tool for arranging the results and comparing them. Additionally, an excel file can be created in advance for any changes in the calculation. Figure 14 depicts an excel file produced in the work process.

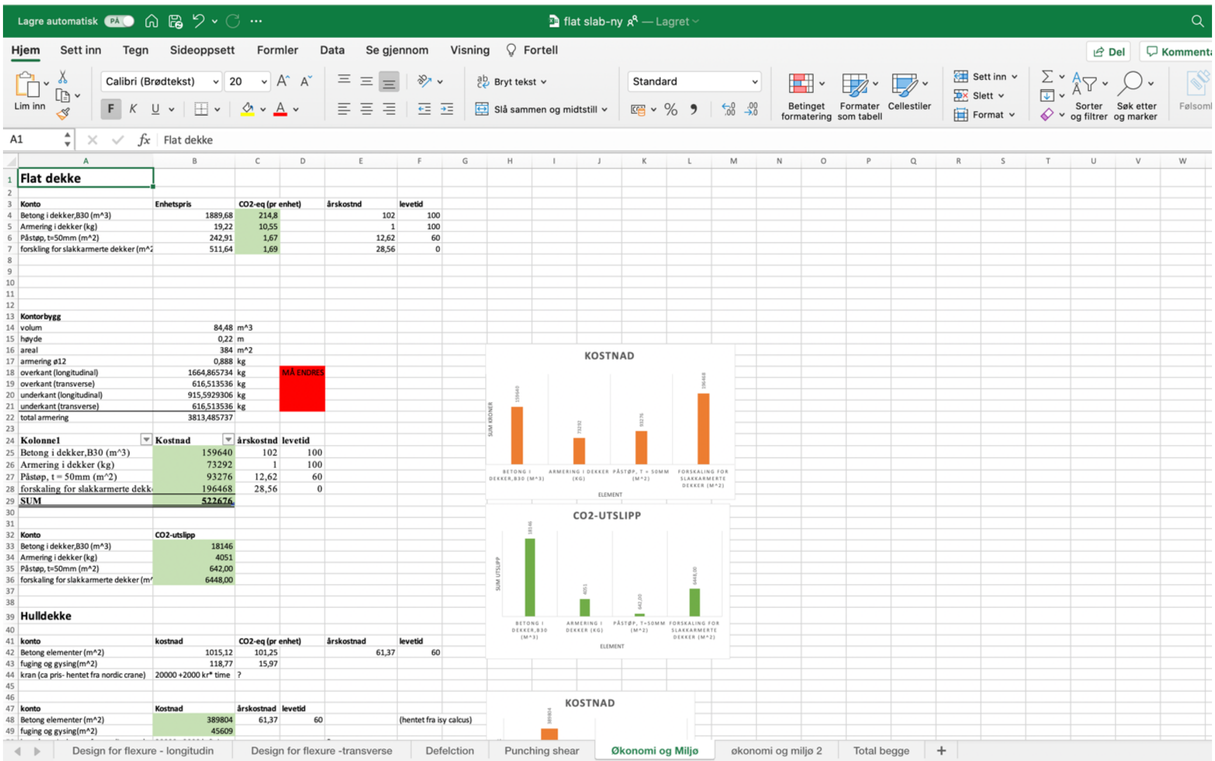


Figure 14 Excel

7 Method

7.1 Research methods

By using the right research method, you will have a systematic and structured way of obtaining data and information. The methods used in the thesis help to shape the result. There are three methods to choose from; quantitative, qualitative and a mixed method. A quantitative method is used to analyze a quantity rather than a detail. It is used to test and confirm theories and assumptions. The common quantitative methods include experiments, observations recorded as numbers, and surveys with closed-ended questions. A qualitative method entails the researcher delving deeply into a narrow field. The data material used can be collected by interviews, observations described in words, or literature reviews (Scribbr, 2019). With the use of a mixed method, it is drawn from both qualitative and quantitative methods.

7.2 Applied method

After careful planning of the design of the thesis, it was chosen to use a mixed method to achieve a representative result. This mixed method contained literature review, structural analysis, cost estimation, and environmental estimation. A literature review was used to obtain basic knowledge about the slab systems and concrete in general. This was actively used to evaluate the calculations and guide me to my goal. Calculations in accordance with European standards were performed to find the necessary reinforcement and concrete quantities for the slabs. At the same time, the properties of the designed slabs were analyzed with the help of software's and manual calculations. The required quantities from structural analysis results were used to find cost-effective and environmentally friendly solutions.

7.3 Source criticism

It was considered that while retrieving information from a source, we should consider how relevant the source is to our problem. Critical decisions were made about the sources. The articles were found in databases that had relevant and original sources. Libraries were also used to locate relevant literature for the thesis.

8 Case study

The goal is to find the right cover in terms of cost and environment for an office building with a size of 24x16m. Figure 15 illustrates hollow-core slab and flat slab with same area. Problem will be solved using knowledge from the literature and calculation methods. For flat and hollow-core slabs, almost the same material properties (subchapters 8.1 and 8.2) will be used to find the fairest possible result. Further, find out if new ideas can help influence the result.

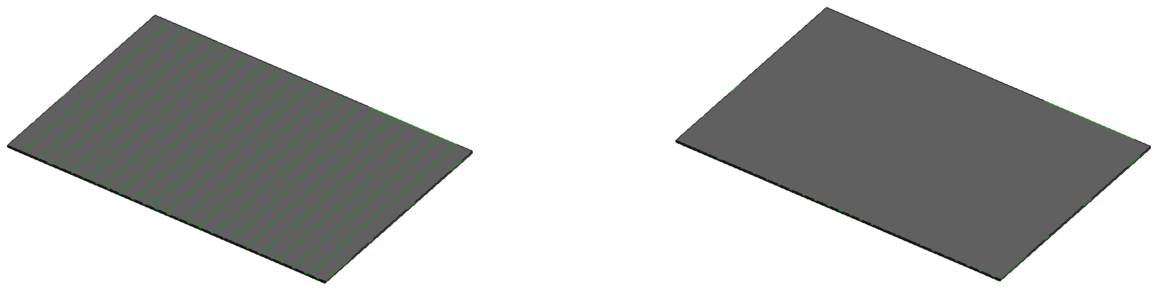


Figure 15 Hollow-core slab and flat slab, drawn in Revit

8.1 Material

Table 12 Material characteristics for concrete

Concrete		
	Strength of class	B30 -> C30/37
	Concrete cover	25mm
	Exposure class	X0
	f_{cd}	17Mpa
	γ_c	1,5

Table 13 Material characteristics for steel

Steel		
	f_{yk}	500Mpa (B500NC)
	f_{yd}	434,8Mpa
	γ_s	1,2

Table 14 Reinforcement characterizes (Celsea-steelservice, 2008, p. 2)

Reinforcement	
Diameter (mm)	12
Mass (kg/m)	0,888
Area (mm^2)	113
Circumference (mm)	37,7

8.2 Loadings

The relevant loads that are taken into account are dead load and live load. The self-weight of a flat slab is composed of a concrete density of $25kN/m^3$ with a slab thickness of 230mm and a screed thickness of 50 mm. The self-weight calculation of a hollow core element is different, so it will be calculated with software to get a more accurate value. According to Eurocode 1, NA.6.1, office buildings are in category B, and the characteristic live load for the office building will be:

$$q = 4 \frac{KN}{m^2}$$

9 Results and Discussion

9.1 Effective ideas

9.1.1 Planning phase : for flat slab and hollow core slab

For flat slabs, planning can be done mainly in the construction phase. Under the construction phase, time for transportation can be considered as one of the main concerns. Transporting concrete and pumping it out can take time, and therefore it may be better to use this slab type in projects that are not in a hurry. When constructing irregular slabs, flat slab construction may be advantageous. Additionally, can the reinforcement be changed before casting. Using a flat slab makes it easier to implement, but it takes a long time to install. However, hollow-core elements also have their advantages. Planning which elements to use and how to place them helps to influence the result enormously. Using standard elements and a few variations of elements leads to reduced cost.

In the planning phase of the case section, it was decided to analyze two types of hollow core elements before deciding to proceed with the HD400. This element was selected using the dimensioning diagram (Appendix G). As I mentioned in the method section (chapter 7) , the same dimension and properties are used in both slabs to get a fair result in the comparison. Therefore, it was chosen to use an element of 230 mm, the same thickness as a flat slab. Why one item was chosen above the other is discussed in the next section.

1. *Planning idea for HD230;*

Two different solutions have been developed for how we can place the elements on the slab to save as much money and time as possible. The first solution is to use short spans. As said, a non-standard element with a thickness of 230 mm was chosen. HD230 has a 6-meter span. The green lines in figure 16 show how the elements are divided. How many elements are required on a column in a slab can be calculated by dividing the vertical length by 1.2m. We need a total of 13 elements in a column, and the last element will have a width of 1600 mm, which is not the standard width. By multiplying the 13 elements by four columns, we get about 54 elements.

$$\frac{16m}{1,2m} = 13,33\ elementer$$

$$13,33 * 4 = 54,32 \approx 54\ elementer$$

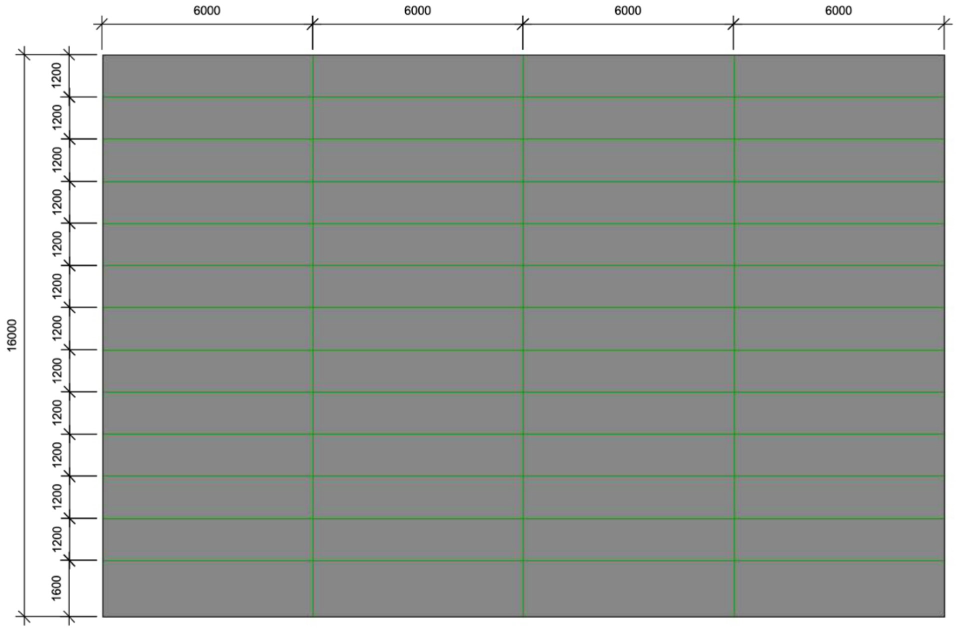


Figure 16 The idea of placing the HD230 elements

As a result, if we decide to use different widths than the standard width in the project, this will result in an additional cost.

2. *Planning idea for HD400*

The HD400 has a maximum span of 17 meters in principle, but in this case, 16 meters are utilized. The green lines in figure 17 show how the elements are divided. In comparison to the HD230, the elements have much larger spans. We may calculate the total number of components required by dividing the vertical length by 1.2 m. As a result, we only require 20 elements in total.

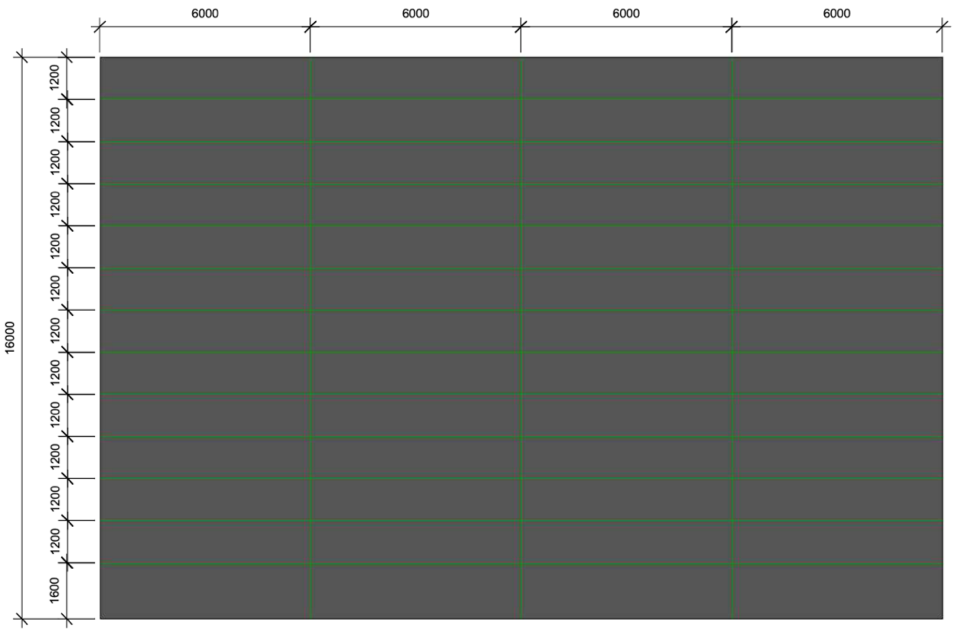


Figure 17 The idea of placing the HD400 elements

$$\frac{24m}{1{,}2m} = 20\ elementer$$

The advantage is that HD400 is a standard element. It will therefore cost less if standard elements are used and there are few variations. This also leads to fewer elements to transport than the HD230 elements. Further, this results with less CO_2-emissions from transportation and lower transportation and crane costs. Hollow-core elements with large spans are a good solution, both in terms of propulsion and cost. Transportation can be a problem if planning is poor or if it gets damaged along the way. When we use a hollow-core slab, it is impossible to change after something has been misplaced. Planning is therefore very important, and at the same time, knowing which materials are needed and how they will affect each other.

9.1.2 Planning phase : choose suitable material elements in ISY Calcus

It has conducted a thorough analysis of which option is the most cost-effective and emits the least CO_2. The following solution description includes a brief comparison of the material elements. The products chosen from the comparison are the green items. The flat slab is shown in Table 15, and the hollow-core slab is shown in Table 16.

Table 15 Comparison of flat slab elements

Flat slab	Unit price	CO_2 -emmissions
Concrete , B30 pr	1889,68 NOK	214 pr unit
Concrete, B30	1837,98 NOK	214 pr unit
Solution description	In terms of cost per cubic meter, they are virtually identical. When the total volume is 84,48 m3, the total price difference will be 4367 NOK. Thus, there is not a very big difference in the price.	Both emit the same amount of CO_2 per unit. The last element will be chosen because it was cheaper.
Reinforcement	19,22 NOK	1,69 pr unit
Solution description	This was the only reinforcement option that was suitable for the slab.	This was the only reinforcement option that was suitable for the slab.
Screed, t=50mm	242,91 NOK	10,55 pr unit
Screed, t=50mm	663,26 NOK	18,70 pr unit
Solution description	The total price difference is 161416 NOK. The price difference is huge if we think in percentage terms. Therefore, the first solution is the best solution.	The first one is better because it has a much lower CO_2-emission.
Forwork-system	511,64 NOK	1,67 pr unit
Formwork system – height e ca 4m	564,55 NOK	1,67 pr unit
Solution description	The price difference is 20319 NOK, so the best solution is the first one.	Both emit the same amount of CO_2 per unit, so the first element will chosen.

Table 16 Comparison of hollow core elements

Hollow core slab	**Unit price**	**CO_2 -emmisions**
Hollow core slab, HD400	1015,12	101,25
Hollow core slab, HD400	1039,44	101,25
Solution description	In percentage terms, the price difference is significant. As a result, the first option is better.	The first choice is more profitable for cost reasons, even if they emit the same amount of CO_2.
Grouting concrete elements	118,77 NOK	15,97 NOK
Solution description	This was the only grouting choice that was suitable for the slab.	This was the only grouting choice that was suitable for the slab.

9.2 Structural analysis

The calculation can be found in the appendices. The results of the flat slab are shown in Appendices A, B, and C. Appendix D contains the total reinforcement for the flat slab, which was computed manually using the equivalent frame method. The program E-bjelke determines the overall reinforcement necessary for the elements. For HD230 and HD400, Appendix E and F show a full analysis of the elements. Table 17 contains the results of the computations.

Table 17 Result of calculation

	Flat slab 24x16 (manual)	**Flat slab 24x16 (SAP2000)**	**HD230**	**HD400**
Thickness of slab	230mm	230	230mm	400mm
screed	50mm	50	50mm	50mm
cover	25mm	25	30mm	30mm
Strength of class	B30/37	B30/37	B30/37	B30/37
Safety factor	1,2+1,5	1,2+1,5	1,2+1,5	1,2+1,5
Loads	Egenvekt: 6,9 KN/m^2 Permanent last (screed): 1,25 KN/m^2 Nyttelast: 4 KN/m^2	Egenvekt: 6,9	Egenvekt: 5,07 KN/m Permanent last :1,5 KN/m^2 Nyttelast: 4,8 KN/m	Egenvekt: 6,68 KN/m Permanent last (screed): 1,5 KN/m^2 Nyttelast: 4,8 KN/m
F_{cd},f_{yk},f_{yd} (mpa)	17 ; 500 ; 434,8		17 ; 500 ; 434,8	17 ; 500 ; 434,8
support	4		2	2
Maximum moment	53,86 KNM		63,4KNm	530KNm
Deflection	30,15mm		14mm	28mm

9.2.1 Why do different values exist?

The manual calculations and software's can result in different values. There could be several reasons for that. In equivalent frame method, four pin supports are used. Whereas in the Ove Sletten, only two support is used. That can lead to different results in flexure and deflection. When calculating the deflection of a flat slab, a simplified method is used. This gives an approximate answer as we use a chart to find the deflection. Also, we multiply by coefficients to be on the safe side since approximations are then made. The rigorous method from SAP2000 would have given a more accurate answer (Appendix I).

Because of missing information, the hollow core elements have been calculated in software. The hollow-core slab was determined automatically by the program E-bjelke, and some programs multiply with factors that may be different in other software's . The task focuses only on the dead load and characteristic live load. Other factors, such as wind and snow, can affect the outcome. Moreover, technical aspects such as fire and sound requirements are neglected in the task. This can also affect the result.

It has been chosen to use the same concrete class on both slabs. B30 concrete must withstand 30 MPa before it breaks. If we had taken a higher concrete class, it could have led to a smaller number of reinforcements. Too strong or too weak strength classes can have adverse effects on the environment, quality, and duration of the construction. The reduced dead weight enables hollow core slabs to have a greater span than other concrete slabs of the same thickness. Less concrete on the slabs leads to less cost and less concrete in the foundation. The span length of HD230 and the flat slab is 6 m. In table 16, it shows that the hollow core slab weighs 1,83 KN/m less than the flat slab.

9.3 Cost and environment

In this chapter, the results from flat slab and hollow-core slab are presented and discussed. It is used the equivalent frame method from the structural analysis to find the results for cost and CO_2-emmisions. Table 18 shows the cost and CO_2-emission per m^2.

9.3.1 Flat slab

Table 18 Prices and CO_2 - emissions per m^2

Material	Unit price	CO_2 - eq (pr m^2)
Concrete element,B30	1837,98 ,-	214,8
Reinforcement	19,22 ,-	10,55
Screed, t=50mm	242,91 ,-	1,67
Formwork	511,64 ,-	1,69

Cost

In this thesis, the focus is only on the slab itself; the actual reinforcement, screed, concrete, and formwork. The rest is neglected. Additionally, the cost of concrete varies depending on quality and additives. The total price is calculated manually and on ISY calculus, and the method for manual calculations is entered in Appendix G.

Table 19 Costs of Flat slab

Material	Cost (NOK)
Concrete,B30	159640-,
Reinforcement	130014,-
Screed, t = 50mm	93276,-
Formwork	196468,-
SUM	**579398 ,-**

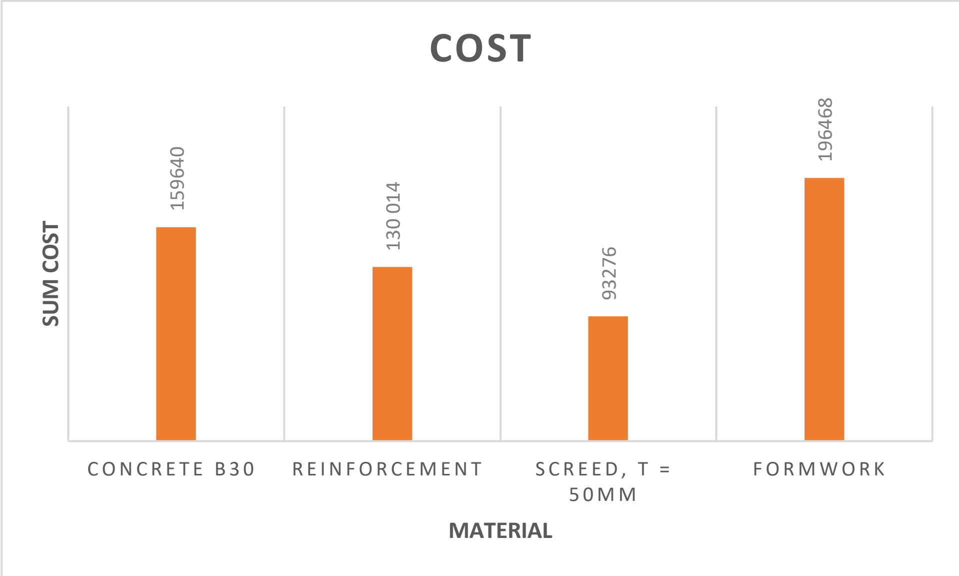


Figure 18 The cost of a flat slab: concrete and screed together cost the most.

Figure 18 and table 19 depicts the total material costs. Furthermore, reinforcing and concrete are priced according to the amounts required to achieve the design capacity. The amount of reinforcement required for the specified concrete class is calculated in Appendix D. According to the calculations in Appendix G, a slab thickness of 230 mm requires a total of 5974.90 kg of reinforcement. If the concrete in the slab and screeds were added together, the total price for concrete would be 252916 NOK. It will also cost more to transport concrete to a work site. It will be utilized by a concrete mixer and pumps on the worksite, but it is not included because each project is different.

Environment impact

Table 19 and figure 17 shows the total CO_2-emissions of the material elements. Unlike the hollow core slab, there is more work at the site, which can cause additional emissions related to the construction. However, this was not selected for analysis in this work, as it is individual for each project. Steel and transportation will also affect emissions in different grades, but these have not been calculated due to insufficient information.

Table 19: CO_2-emissions of a flat slab

Elements	CO_2-utslipp
Concrete B30	18146
Reinforcement	11438
Screed, t = 50mm	642,00
Formwork	6448,00
SUM	**36674**

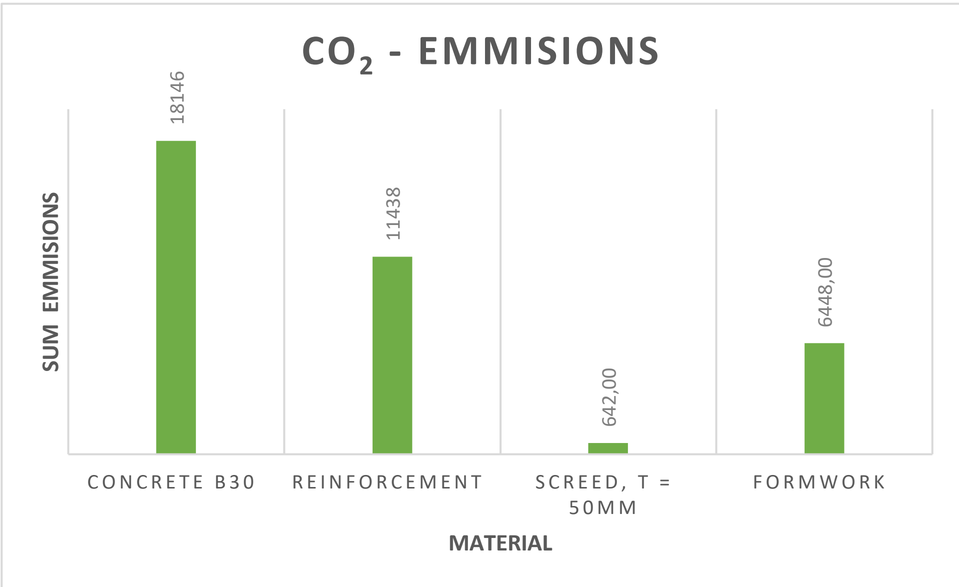


Figure 17: Diagram of CO_2-emissions of a flat slab: Concrete and screed together release a lot of CO_2

9.3.2 Hollow core slab

Table 20 shows the cost and CO_2-emission per m^2. The thickness of the hollow core elements is chosen according to the recommended element type from the dimensioning diagram. The calculations are presented in Appendix G. Based on the comparison of the elements (subchapter 9.1.1), a thickness of 400 mm is utilized. The slab is 384 square meters and requires 20 hollow core elements with a standard width of 1.2 meters.

Table 20 Prices and CO_2- emissions per m^2

Material	Cost	CO_2-eq (pr m^2)
Concrete element	1015,12,-	101,25
Grouting	118,77,-	15,97
Crane (Nordic crane)	30000 +2000-, pr hour	-
Screed, t=50mm (m^2)	242,91,-	1,67

Cost

The cost of hollow core elements varies due to the thickness of the element, customizations to the element, and market demand. In ISY Calculus, neither the price nor CO_2 emissions are given on the crane. It needed specific information to find the exact price, but an approximate price was given by Frank Bjørheim, who works as operations manager at Kranringen. He stated that mobilization is NOK 30,000 (within 50 km), in addition to an hourly rate of NOK 2,000. Table 21 and figure 19 shows the total CO_2-emissions of the material elements.

Table 21 Costs of Hollow-core slab

Material	Cost (NOK)
Concrete element	389804 ,-
grouting	45609 ,-
Crane (Nordic crane)	30000 +2000 ,- pr hour
Screed, t=50mm (m^2)	93276,-
SUM	**528689 (+30 000 +2000 pr hour) ,-**

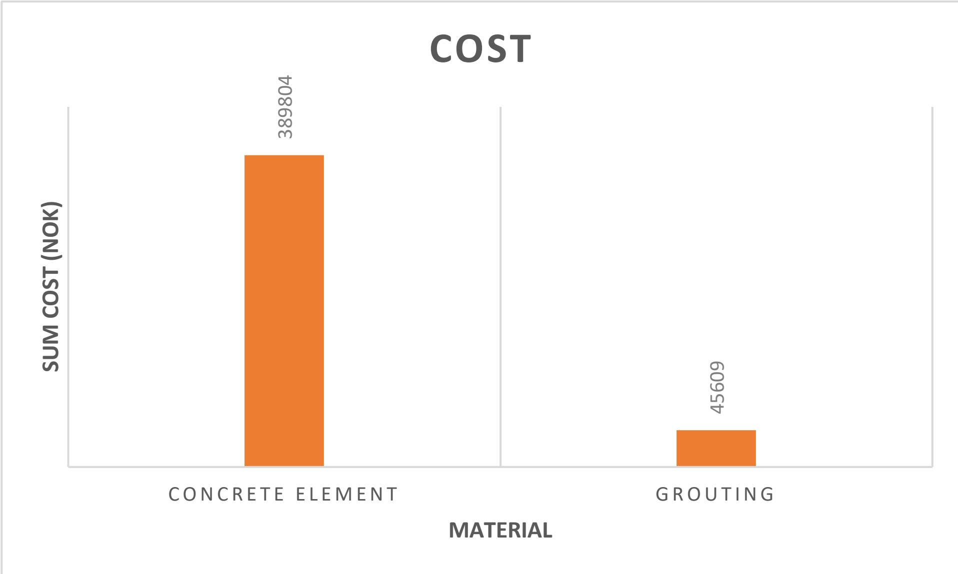


Figure 19 Diagram of Costs of hollow-core slab : concrete elements costs most

Environment

Table 22 and Figure 20 show the CO_2-emissions of the concrete elements and the grouting. The concrete elements release much larger amounts of CO_2 than the actual grouting. Since prefabricated elements weigh less than cast-in-place concrete, they also require less concrete in the foundation. This means that the hollow-core slab will emit fewer emissions than the flat slab.

Table 22 CO_2 -emissions of hollow core slab

Material	CO_2-emmissions (pr $m^{2)}$
Concrete element	8553
Grouting	6131
Screed, t=50mm (m^2)	642,00
SUM	**14 684**

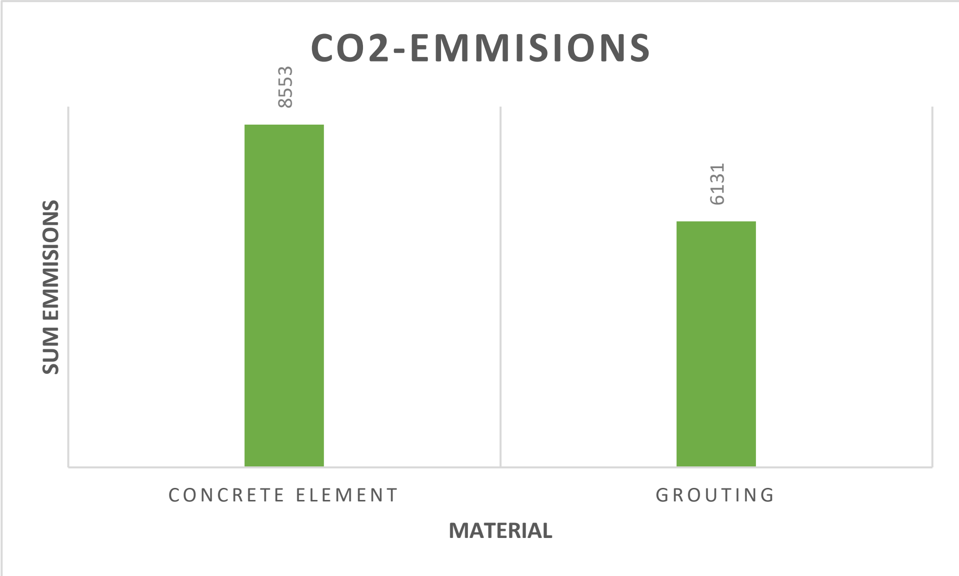


Figure 20 CO_2 emissions from a hollow core slab diagram: concrete elements emit the highest CO_2.

9.3.3 Comparison

In a modern society with a lot of CO_2-emissions, it is important to consider how much this can affect the environment. Cost and the environment are the most important factors in choosing a slab type. The thesis has therefore focused on how we can achieve a successful project while taking these factors into account. Figure 21 shows a summarized diagram of the total costs and CO_2-emmisions.

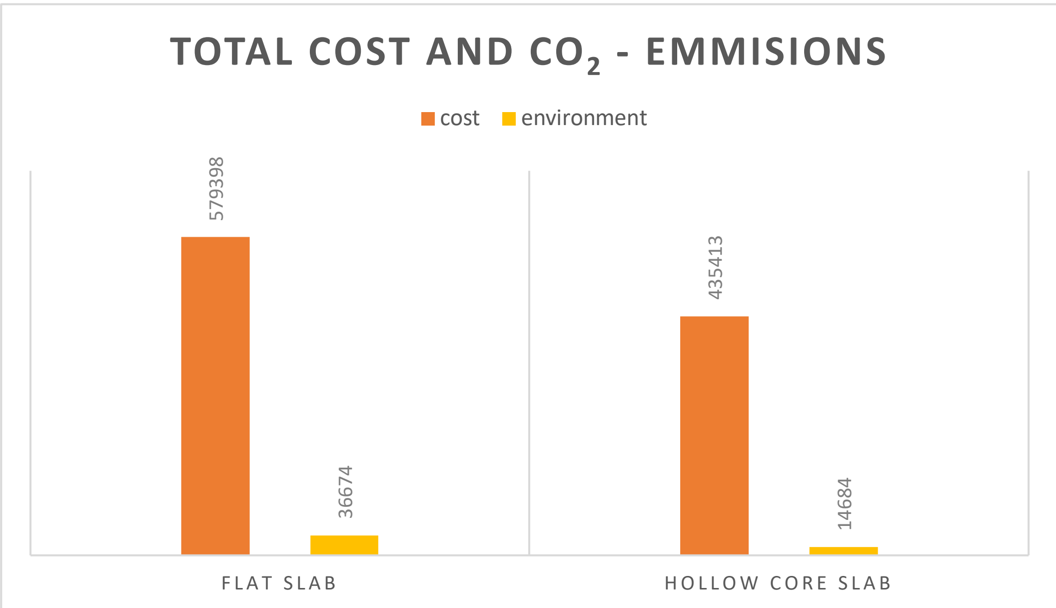


Figure 21 Flat slab and Hollow-core slab comparison of cost and environment

Cost

Prefabricated elements rely largely on assembly. Multiple cranes can make this more efficient, although crane setup and hourly rates have been proven to be very costly. The total price for the various slabs is shown in Table 23. For a slab of 24 x 16 m, hollow-core slabs with large spans are substantially less expensive than flat slabs.

Table 23 Total costs of Flat slab and Hollow-core slab

Slab	Total cost (NOK)
Flat slab	579 398,-
Hollow core slab	558 689 (+2000 pr hour for crane)
Result	Flat slab is more expensive than hollow core slab.

Environment:

Table 24 shows that the CO_2 emissions can vary greatly depending on the construction. Prefabricated elements emit approximately 22,000 equivalents less CO_2 than flat slabs. Since concrete is the largest contributor to environmental emissions, reducing this amount has resulted in the greatest reduction in the total CO_2 emissions.

Table 24 Total CO_2 -emissions of flat slab and Hollow-core slab

Slab	Total CO_2 - equivalents
Flat slab	36 674
Hollow-core slab	14 684
Result	The hollow core slab emits much less than the flat slab. Even if we include the CO2 emissions for transportation, it is still less than the flat slab emissions.

The curing of concrete in flat slabs is different. Hardening can be more efficient by adding accelerating additives or storing concrete with the help of a heater. There is a greater quantity of concrete to harden compared to the progress of the hollow core slab. In connection with this, it will affect the propulsion and the cost. On the other hand, production of hollow core elements takes place inside, so it has time to harden without affecting the progress. The grout needs extra freezing point suspending substances during cold periods. This may incur an additional cost. However, both hollow-core slabs and flat slabs need freezing point suspending substances during cold periods, but in terms of quantity, the hollow-core slab is a better choice.

Although hollow-core slabs take less time on site than cast-in-situ slabs, transportation can be a major obstacle. To keep the costs within plan, it is important that transport is not an obstacle. More items lead to more crane and transport use. Elements can be damaged during transport. It may take more time to assemble all the elements, and therefore more workers will be needed on site. This thus increases the cost. Installation will be easier, but if things go wrong during the progress, it may take longer than expected.

The transport industry can cut large amounts of emissions through efficiency improvements. It is possible to use, for example, zero-emission vehicles or electric cars. Coordination between suppliers, transporters, and construction sites are also significant challenges that can be changed. Poor communication and lack of planning can lead to several cars arriving at the same time, which will cause a queue. A lot of working time can be expected in the goods receipt, and this in turn leads to additional costs in construction projects. Not only that, but it will also emit unnecessary CO_2 gases and cause local pollution. It is important to have better space utilization, which means less fuel consumption.

As a starting point, a building that has already been built or is being built may be beneficial. By determining the building site, the results become more precise, and it is then possible to compute loads such as snow and wind. Also, determine the cost and CO2-emissions associated with transportation. Additionally, it is easier to determine which kind of foundation is appropriate for a given structure's location. Building a foundation takes time, and as previously stated, time equals money.

10 Conclusion

The purpose of the thesis was to find eco-friendly and cost-effective slab for an office building. Furthermore, find adequate guidelines for choosing the right slab for future entrepreneurs and engineers. It is difficult to find a conclusive answer as to which slab type is best to utilize at any given time. Based on the comparisons that have been made against the influencing factors, it can be concluded that the underlying claims can be good guidelines for the choice between flat slab and hollow core slab.

In order to assess prices and CO_2-emissions, the starting point was a limited quantity description in ISY Calcus. In the quantity description, the focus was on the items with the greatest impact on price and CO_2. The strategy consisted of selecting the appropriate slab type and quality for a specific area to get the most equitable result.

Prefabricated elements are largely dependent on the assembly for a good result. This can be made more efficient by good communication between the transporters. Adding more cranes will lead to more costs as the set-up and hourly rates are high. Prefabricated elements with both short and long spans were compared in the thesis, and the results show that it is better to use long spans as this requires less transport and crane use. Efficient propulsion of prefabricated elements depends on continuous supply and an available crane. A smaller span is more likely to be used in a residential project than in an office building. It also states that it is better to use hollow core slabs on larger construction projects as increased progress is only desirable if it is at an artificial cost. Even if we use means of transport without CO_2 emissions, it will still affect the cost if the waiting time is long on the way to the construction site. Because time is money. It is a good idea to use long spans as it requires less transport. But compared to flat slab, hollow core slab releases less CO_2, because the biggest factor, which is concrete, is reduced.

Flat slab will be a good choice in projects where the time schedule is reasonable for completion and the foundation is not very complicated. In these cases, the choice may be made primarily based on aesthetics, quality, and adaptability. There are several factors that affect the progress of a project. The advantage of an in-situ concrete slab is that you can start immediately and change when something is uncertain before casting. The unique thing is that an irregular slab is easier to cast and assemble in a flat slab than in a hollow-core slab.

During the winter period, both hollow-core slabs and flat slabs require freezing point suspending chemicals, although hollow-core slabs are the preferable choice in terms of quantity. The results show that it is very economical and environmentally friendly to use prefabricated elements.

The calculations show that the prefabricated elements perform best in terms of CO_2-emissions and cost. Some emissions from screed and grouting are to be expected, but the total emissions are still significantly lower than flat slab. A hollow-core slab will contribute to a lighter construction. This will lead to savings in the foundation costs compared to a flat slab. My preliminary pre-design recommendation for this office building will be a hollow-core slab rather than a flat slab based on the assumptions.

Further review

The utilization of life cycle costs (LCC) and life cycle analysis (LCA) are two further areas that may be of relevance for future research beyond this topic. We may see the problem from a different perspective if we look at the slab's complete life cycle. This tool might be useful for gaining a deeper view of various environmental effect categories such as global warming, acidification, toxicity, reuse, and ozone layer depletion. In this way, one will gain an understanding of the consequences the slab inflicts.

The choice of a slab system is a large and complex topic, and it would be wise to obtain internal information through interviews. In this way, we gain more knowledge by hearing the experiences and opinions of those who work with slab systems on a daily basis. It is also possible to test this by replacing the concrete slab with wood. It would probably have given better results in terms of the environment and cost, but would it have been a better solution in terms of structural analysis as well?

It is also possible to analyze how the parking basement had affected these factors. Had it given better results with a combination of both flat slab and hollow-core elements?

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12 Appendices

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Manual calculations and data sheets

The results from the manual calculations and from the software's *E-bjelke* and *SAP2000* are attached below.

Flat slab

Appendix A Flat slab - Flexure

Appendix B Flat slab - Deflection

Appendix C Flat slab - Punching shear

Appendix D Total reinforcement in Flat slab.

Appendix I Sap2000- Deflection and Bending moment

Hollow-Core slab

Appendix E Ove sletten HD230.

Appendix F Ove sletten HD400.

Appendix G Dimensioning Diagram

Common

Appendix H Quantity calculation for ISY Calcus.

Flat slab Calculation - Longitudinal

Choosed sizes
Size: 24m x 16m
Imposed load: 4kN/m2
Thickness of slab: 230mm
Screed: 50mm
Strength of class: B30
Panel-width :1,2m (same as hollow core slab)
The calculation is followed the "calculation method" in the thesis

1. Initial sizes

Concrete cover

$$Cnom = 15mm + 10mm = 25mm$$

Cover thickness =25mm

15 mm is from the exposure class for flat slabs X0 , XC1 because of low humidity.
Effective depth
Longitudinal:

$$d = 230mm - 25mm - \frac{12}{2}$$

d=199 mm

Transverse:

$$d = 230mm - 25mm - 12 - \frac{12}{2}$$

d=187 mm

2. Material Properties

strength class of concrete

$$f_{cd} = \frac{0,85 \cdot 30}{1,5}$$

$\mathbf{f_{cd} = 17}$ **Mpa**

strength class of steel

$$f_{yd} = \frac{500}{1,15}$$

$\mathbf{f_{yd} = 434{,}8}$ **Mpa**

3. Design for flexure

Load calculation
Permanent load :

$$g_f = 25\frac{KN}{m^3} \cdot (230 + 50) \cdot 10^{-3}m \cdot 1,2m$$

$$g_f = 8,4\frac{kN}{m^2}$$

Imposed load:

$$q_f = 4\frac{KN}{m^2} \cdot 1,5$$

$$q_f = 6\frac{kN}{m^2}$$

LONGITUDINAL DIRECTION
Moment factors obtained from lecture notes of Knut Erik Kismul, University of Bergen
Sagging moment

$$MEd_1 = (0,077 \cdot 8,4\frac{kN}{m^2} \cdot 6^2m^2) + (0,100 \cdot 6\frac{kN}{m^2} \cdot 6^2m^2) \cdot 1,2m$$

$$MEd_1 = \textbf{53,86 KNm}$$

Hogging moment

$$MEd_1 = (0,107 \cdot 8,4\frac{kN}{m^2} \cdot 6^2m^2) + (0,121 \cdot 6\frac{kN}{m^2} \cdot 6^2m^2) \cdot 1,2m$$

$$MEd_1 \approx \textbf{70,19 KNm}$$

Moment capacity for singly r/f balanced section

$$M_{cd} = (0,293 \cdot 17Mpa \cdot 1200mm) \cdot 199^2mm^2$$

$$M_{cd} = \textbf{236,70 KNm}$$

No compression reinforcement is needed when $M_{cd} > M_{Ed}$

Calculation of longitudinal r/f for column strip

have to check for :

$$As_{min}, As_{max}, A_{h,max}$$

$$Ac = 1200mm \cdot 199mm$$

$$AC = 238800mm$$

$$As_{min} = 0,26 \cdot 199 \cdot 10^-3 \cdot \frac{2,9Mpa}{500Mpa}$$

$$As_{min} \approx 360,11mm^2$$

$$Check : 360mm^2 > 0,0013 \cdot 1200 \cdot 199 = 310,44mm^2\textbf{OK}$$

Maximum r/f area

$$As_{max} = 0,04 \cdot 1200mm \cdot 199mm$$
$$As_{max} = 9552mm^2$$

Maximum spacing for principal r/f

$$S_{min} = \{460, 250\}$$
$$S_{h,max} = 250mm$$

1. For sagging

$$z = (1 - 0,18 \cdot \frac{53,86kNm \cdot 1,1}{197,3kNm}) \cdot 199mm$$
$$z = 190,03$$
$$190,848 > 0,95d = 189,05mm$$

choose $0,95d = 189,05mm$
Required steel area;

$$A_S1 = \frac{53,86KNm \cdot 1,1 \cdot 10^6 mm^2}{0,95 \cdot 199mm \cdot 434,8\frac{N}{mm^2}}$$
$$A_S1 = 720,82mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{720,82\frac{mm^2}{m}}$$
$$S = 156,77mm \approx 150 < Sh, max$$

Ø12C150 uk

Provided steel area

$$A_Sprov = \frac{113mm^2 \cdot 1000}{150}$$
$$A_Sprov = 753,33\frac{mm^2}{m}$$

1. For Hogging

$$z = (1 - 0,18 \cdot \frac{1,33 \cdot 70,19kNm}{197,3kNm}) \cdot 199m$$
$$z = 184,87$$
$$185,07mm > 189,05mm$$

choose 0,93d = 184,87 mm
Required steel area;

$$A_S1 = \frac{70,19KNm \cdot 1,33 \cdot 10^6 mm^2}{0,93 \cdot 199mm \cdot 434,8Mpa}$$

$$A_S1 = 1135,76mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{1135,76\frac{mm^2}{m}}$$

$$S = 99,49 \approx 90 < Sh,max$$

Ø12C90 ok

Provided steel area

$$A_{S1prov} = \frac{113mm^2 \cdot 1000}{90}$$

$$A_{S1prov} = 1255,56\frac{mm^2}{m}$$

Calculation of longitudinal r/f for middle strip

1. For sagging

$$z = (1 - 0,18 \cdot \frac{53,86kNm \cdot 0,9}{197,3kNm}) \cdot 199mm$$

$$z = 191,66$$

$$191,66 > 0,95d = 189,05mm$$

choose $0,95d = 189,05mm$
Required steel area;

$$A_S1 = \frac{53,86KNm \cdot 0,9 \cdot 10^6 mm^2}{0,95 \cdot 199mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_S1 = 589,76mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{589,76\frac{mm^2}{m}}$$

$$S = 191,60mm \approx 190 < Sh,max$$

Ø12C190 uk

Provided steel area

$$AS1 = \frac{113mm^2 \cdot 1000}{190}$$

$$AS1 = 594,74\frac{mm^2}{m}$$

1. For Hogging

$$z = (1 - 0,18 \cdot \frac{0,67 \cdot 70,19kNm}{197,3kNm}) \cdot 199m$$

$$z = 191,88$$

$$191,88mm > 189,05mm$$

choose 0,95d = 189,05 mm
Required steel area;

$$A_S1 = \frac{70,19KNm \cdot 0,67 \cdot 10^6mm^2}{0,95 \cdot 199mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_S1 = 572,15mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{572,15\frac{mm^2}{m}}$$

$$S = 197,50 \approx 190 < Sh,max$$

Ø12C190 uk

Provided steel area

$$A_{S1prov} = \frac{113mm^2 \cdot 1000}{190}$$

$$A_{S1prov} = 594,74\frac{mm^2}{m}$$

Flat slab Calculation - Transverse

TRANSVERSE DIRECTION
Moment factors obtained from lecture notes of Knut Erik Kismul, University of Bergen

Sagging moment

$$MEd_1 = (0,077 \cdot 8,4\frac{kN}{m^2} \cdot 4^2m^2) + (0,100 \cdot 6\frac{kN}{m^2} \cdot 4^2m^2) \cdot 1,2m$$

$$MEd_1 = \textbf{23,94 KNm}$$

Hogging moment

$$MEd_1 = (0,107 \cdot 8,4\frac{kN}{m^2} \cdot 4^2m^2) + (0,121 \cdot 6\frac{kN}{m^2} \cdot 4^2m^2) \cdot 1,2m$$

$$MEd_1 \approx \textbf{31,20KNm}$$

Moment capacity for singly r/f balanced section

$$M_{cd} = (0,293 \cdot 17\frac{N}{mm^2} \cdot 1200mm) \cdot 187^2mm$$

$$M_{cd} = \textbf{209,02 KNm}$$

No compression reinforcement is needed when $M_{cd} > M_{Ed}$

Calculation of longitudinal r/f for column strip

have to check for :

$$As_{min}, As_{max}, A_{h,max}$$

$$Ac = 1200mm \cdot 187mm$$

$$AC = 224400mm$$

$$As_{min} = 0,26 \cdot 187 \cdot 10^{-3} \cdot \frac{2,9Mpa}{500Mpa}$$

$$As_{min} \approx 338,40mm^2$$

$$Check : 338,40mm^2 > 0,0013 \cdot 1200 \cdot 187 = 291,72mm^2\textbf{OK}$$

Maximum r/f area

$$As_{max} = 0,04 \cdot 1200mm \cdot 187mm$$

$$As_{max} = 8976mm^2$$

Maximum spacing for principal r/f

$$S_{min} = \{460, 250\}$$

$$S_{h,max} = 250mm$$

1. For sagging

$$z = (1 - 0,18 \cdot \frac{23,94kNm \cdot 1,1}{209,02kNm}) \cdot 187mm$$

$$z = 182,76mm$$

$$182,76 < 0,95d = 177,65mm$$

choose $0,95d = 189,05mm$
Required steel area;

$$A_S1 = \frac{23,94KNm \cdot 1,1 \cdot 10^6 mm^2}{0,95 \cdot 187mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_S1 = 340,92mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{340,92\frac{mm^2}{m}}$$

$$S = 331,46mm \approx 332mm > Sh,max$$

kortere spacing på lengre bredde

Ø12C250 uk

Provided steel area

$$A_S1 = \frac{113mm^2 \cdot 1000}{250}$$

$$A_S1 = 452\frac{mm^2}{m}$$

1. For Hogging

$$z = (1 - 0,18 \cdot \frac{1,33 \cdot 31,20kNm}{209,02kNm}) \cdot 187m$$

$$z = 180,32$$

$$180,32mm > 177,65mm$$

choose 0,95d = 177,65 mm
Required steel area;

$$A_Sb = \frac{31,20KNm \cdot 1,33 \cdot 10^6 mm^2}{0,95 \cdot 187mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_Sb = 537,17mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{537,17\frac{mm^2}{m}}$$

$$S = 210,36 \approx 210mm < Sh,max$$

Ø12C200 ok

Provided steel area

$$A_{S1prov} = \frac{113mm^2 \cdot 1000}{200}$$

$$A_{S1prov} = 565\frac{mm^2}{m}$$

Calculation of longitudinal r/f for middle strip

1. For sagging

$$z = (1 - 0,18 \cdot \frac{23,94kNm \cdot 0,9}{209,02kNm}) \cdot 187mm$$

$$z = 185,53$$

$$185,53 > 0,95d = 177,65mm$$

choose $0,95d = 177,65mm$
Required steel area;

$$A_S1 = \frac{23,94KNm \cdot 0,9 \cdot 10^6 mm^2}{0,95 \cdot 187mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_S1 = 278,94mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{278,94\frac{mm^2}{m}}$$

$$S = 405,11mm \approx 405mm > Sh,max$$

kortere spacing på lengre bredde

Ø12C250 uk

Provided steel area

$$A_S1 = \frac{113mm^2 \cdot 1000}{250}$$

$$A_S1 = 452\frac{mm^2}{m}$$

1. For Hogging

$$z = (1 - 0,18 \cdot \frac{0,67 \cdot 31,20kNm}{209,02kNm}) \cdot 187mm$$

$$z = 183,63mm$$

$$183,63 > 177,65mm$$

choose 0,95d = 177,65 mm
Required steel area;

$$A_Sb = \frac{31,20KNm \cdot 0,67 \cdot 10^6 mm^2}{0,95 \cdot 187mm \cdot 434,8\frac{N}{mm^2}}$$

$$A_Sb = 270,61mm^2$$

spacing;

$$S = \frac{113mm^2 \cdot 10^3}{270,61\frac{mm^2}{m}}$$

$$S = 417,58 \approx 418 > Sh,max$$

Ø12C250 ok

Provided steel area

$$A_{S1prov} = \frac{113mm^2 \cdot 1000}{250}$$

$$A_{S1prov} = 452\frac{mm^2}{m}$$

Flat slab Calculation - Deflection

Simplified method 1: -

(Actual Limiting value of span / effective depth) > $\frac{L}{d}Actual$

Actual limiting value of span / eff depth = k x (limiting value of span/eff depth x $F_1 x F_2 x F_3$)

Step 1: Actual $\frac{L}{d}$

$$\frac{6000mm}{199mm} = \mathbf{30{,}15}$$

The deflection is also dependent on the depth.

Step 2: Deflection check of column strip

$$K = 1,2 \ (for\ flat\ slabs)$$

$$F_1 = 1,0 \ (forflat\ slabs\ b_f = b_w)$$

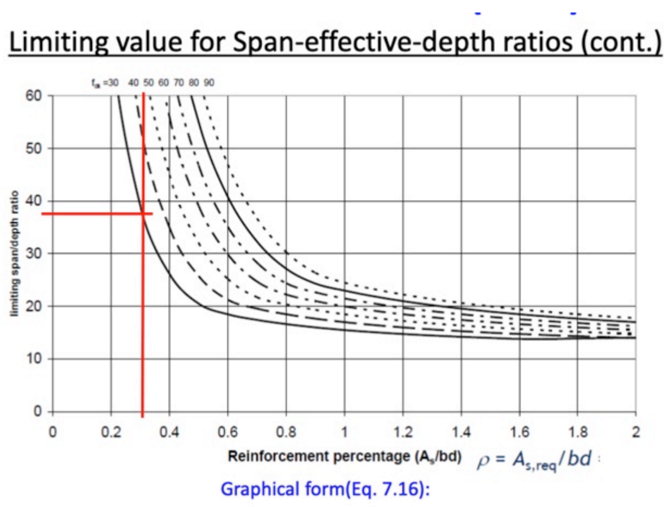
$$F_2 = 1,0 \ (Assume\ no\ brittle\ partitions)$$

$$F_3 = \frac{753,33\frac{mm^2}{m}}{720,82mm^2} = 1,05 < 1,5OK$$

To find limiting value of span/eff depth required tension reinforcement ratio.

$$\rho = \frac{720,82mm^2 \cdot 100\%}{1200mm \cdot 199mm}$$

$$\rho = 0,301\%$$



From graph ; limiting value of span / eff depth $\frac{L}{d} = 37$ (less value is better)
Actual limiting value of span/ effective depth

$$= 1,2 \cdot 37 \cdot 1,05$$

$$46,62 > actual \cdot \frac{l}{d} = 30,15$$

Deflection is ok for column strip

Step 3: Deflection check of middle strip

$$K = 1,2 \; (for \; flat \; slabs)$$

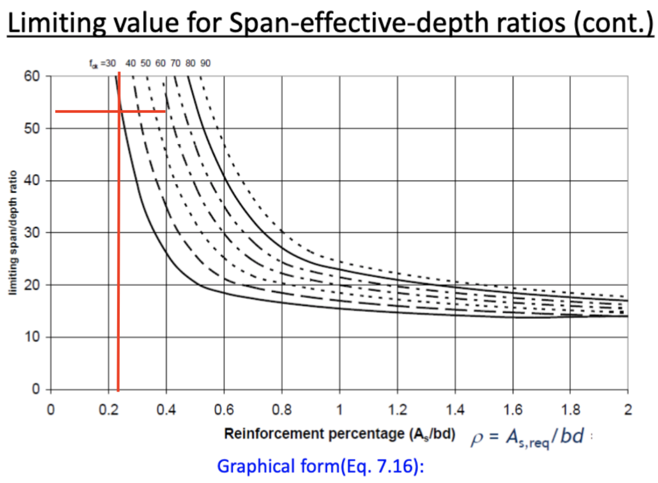
$$F_1 = 1,0 \; (forflat \; slabs \; b_f = b_w)$$

$$F_2 = 1,0 \; (Assume \; no \; brittle \; partitions)$$

$$F_3 = \frac{594,74\frac{mm^2}{m}}{589,76mm^2} = 1,008 < 1,5OK$$

$$\rho = \frac{589,76mm^2 \cdot 100\%}{1200mm \cdot 199mm}$$

$$\rho = 0,25\%$$



From graph ; limiting value of span / eff depth $\frac{L}{d} = 53$ (less value is better)
Actual limiting value of span/ effective depth

$$= 1,2 \cdot 53 \cdot 1,001$$

$$64,10 > actual \cdot \frac{l}{d} = 30,15$$

Deflection is ok for column strip
How to manage, if it is not staisfied:
If deflection is not satified , you can change d of the slab.

Flat slab Calculation - Punching shear

Step 1: Ultimate column reaction VEd

$$K = \frac{700mm}{700mm} = 1 \ gives \ 0,6$$

Step 2: Shear check at perimeter of the column

Shear at perimeter of the column 1: -

$V_{Ed} < V_{Rd,max}$

$$shear \ stress \ V_{Ed} = \beta \cdot \frac{V_{Ed}}{u \cdot d}$$
$$fck = 30Mpa$$
$$d = \frac{199 + 187}{2} = 193mm$$
$$u_0 = 4 \cdot 700 = 2800mm$$

$$ved = (6 \cdot 4)(8,4 \cdot 16) = use \ 350KN$$

$$\beta = 1,15 \ for \ internal \ columns$$

$$V_{Ed} = 1,15 \cdot \frac{350 \cdot 10^3}{2800 \cdot 193} = 0,74Mpa$$

$$V_{Rd,max} = 0,4 \cdot 0,6(1 - \frac{30}{250} \cdot 17) = 3,59Mpa$$

Punching shear resistance of slabs without shear r/f No longitudinal forces

$$CRd,c = \frac{0,18}{1,5} = 0,12$$

$$k = 1 + \frac{200}{193} = 2,02 > k = 2.0$$

$$\rho_x = \frac{120 \cdot \pi \cdot \frac{12^2}{4}}{24000 \cdot 193} = 0,00293$$

$$\rho_y = \frac{67 \cdot \pi \cdot \frac{12^2}{4}}{6000 \cdot 193} = 0,00654$$

$$\rho_i = \sqrt{0,00293 \cdot 0,006544} = 0,00437 < 0,02$$

No Punching shear r/f required

$$V_{Rd,c} = 0,12 \cdot 2 \cdot (100 \cdot 0,00437 \cdot 30)^{1/3} = 0,565Mpa$$

$$V_{min} = 0,035 \cdot 2^{3/2} \cdot 30^{1/2} = 0,542$$

$$\underline{VRd,c > Vmin}$$

$$u_1 = 2 \cdot (700 + 700) + (2\pi \cdot 2 \cdot 193) = 5225mm$$

$$u_0 = 2 \cdot (700 + 700) = 2800mm$$

$$\beta = 1,15$$

$$V_{Rd,max} 0,4 \cdot f_{cd} = 3,59Mpa > 1,6 \cdot V_{Rd,c} \cdot \frac{u_1}{\rho \cdot u_0} = 1,41Mpa$$

$$V_{Rd,max} 0,4 \cdot f_{cd} = 3,59Mpa > 1,6 \cdot 0,565 \cdot \frac{5225}{1,15 \cdot 2800} = 1,47Mpa$$

No shear failiure at perimeter of the coloumn

Step 3: Shear check at basic control perimeter

$$V_{Ed} = 1,15 \cdot \frac{350 \cdot 10^3}{5225 \cdot 193} = 0,4Mpa$$

$$V_{Rd,c} = 0,12 \cdot 2 \cdot (100 \cdot 0,00437 \cdot 30)^{1/3} = 0,565Mpa$$

$$\underline{VEd < VRd_c}$$

No shear at basic control perimeter and no Punching shear r/f required

It is checked for internal columns, corner columns, and edge columns. Table 1 shows the results based on the values in the exel sheet

Table 1 :No shear failiure at :

	Perimeter of the column	**Basic control perimeter**
Internal column	0,74 $\leq 1,41$	0,4 $\leq 0,542$
Corner column	0,97 $\leq 1,08$	0,52 $\leq 0,542$
Edge column	0,91 $\leq 1,16$	0,49 $\leq 0,542$

Flat slab Calculation

Reinforcement - OK (overkantarmering

LONGITUDINAL
For column strip : Ø12C150 OK give $\frac{2000 \cdot 3}{150} \approx 40$
For middle strip : Ø12C190 OK give $\frac{10000}{190} \approx 53$
TRANSVERSE
For column strip middle strip : Ø12C250 OK give $\frac{24000}{250} \approx 96$

Reinforcement - UK (underkantarmering

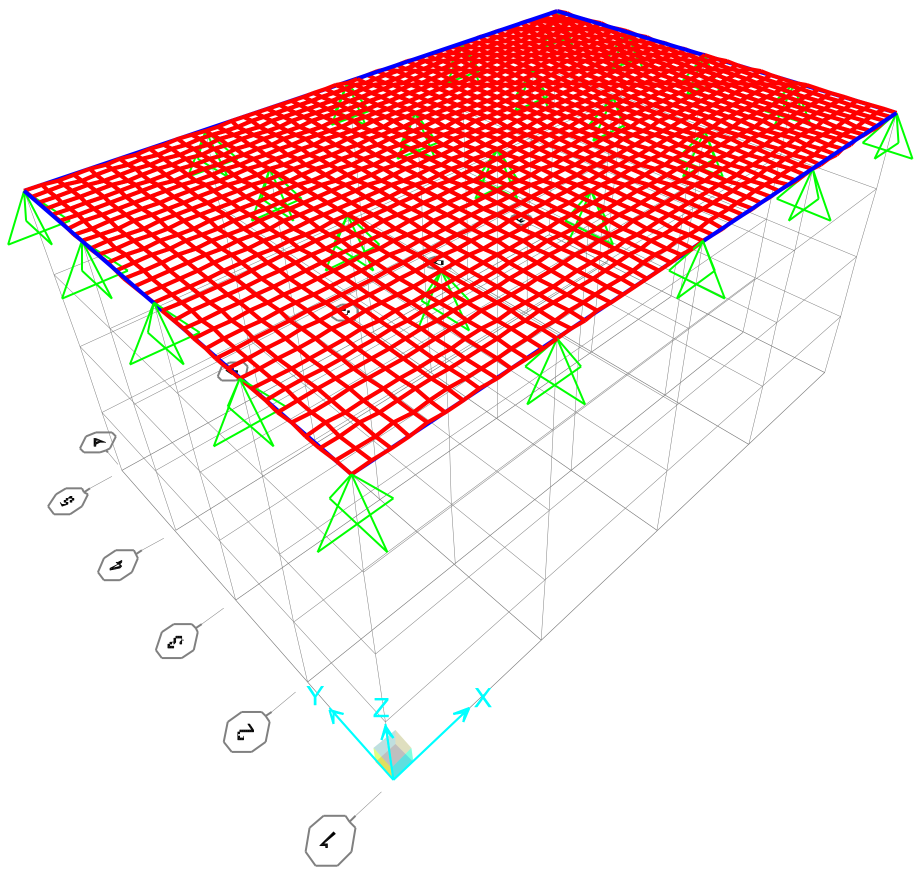
LONGITUDINAL
For column strip : Ø12C90 OK give $\frac{6000}{90} \approx 67$
For middle strip : Ø12C190 OK give $\frac{10000}{190} \approx 53$
TRANSVERSE
For column strip : Ø12C200 OK give $\frac{24000}{200} \approx 120$
For middle strip : Ø12C250 OK give $\frac{24000}{250} \approx 96$

Appendix I

SAP2000

1. Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity. Both manual calculation and software use the same material properties.



Figur 1 Finite element method

2. Bending moment

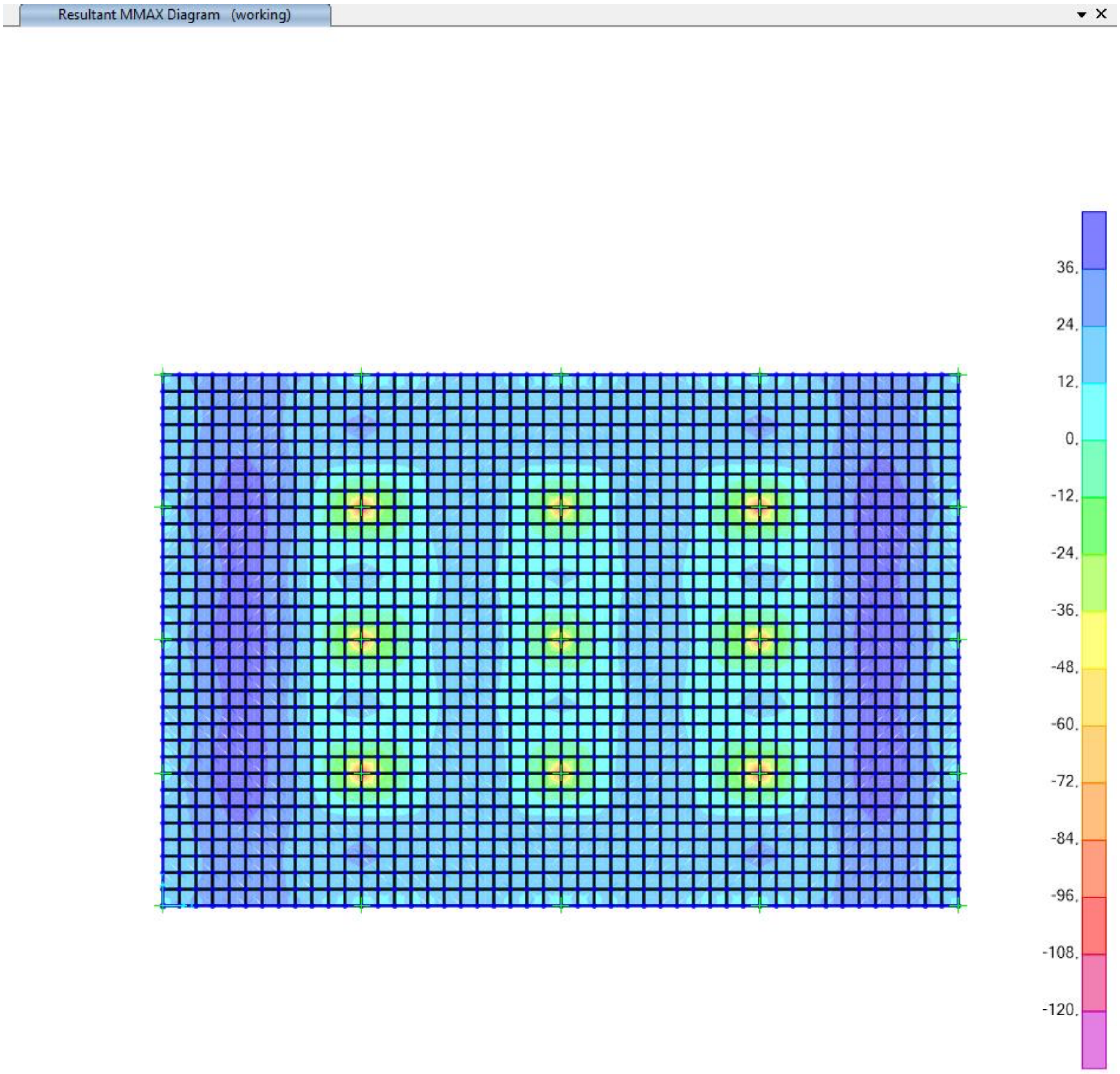


Figure 2 Bending moment

The bending moments at various points in the slab are shown in Figure 1. The bending moment in the column strip is substantially higher than in the middle strip, as seen in Table 1.

Maximum bending moment	-126,864 KNm
Minimum bending moment	40,687 KNm

Tabell 1 Maximum and Minimum bending moment

3. Deflection

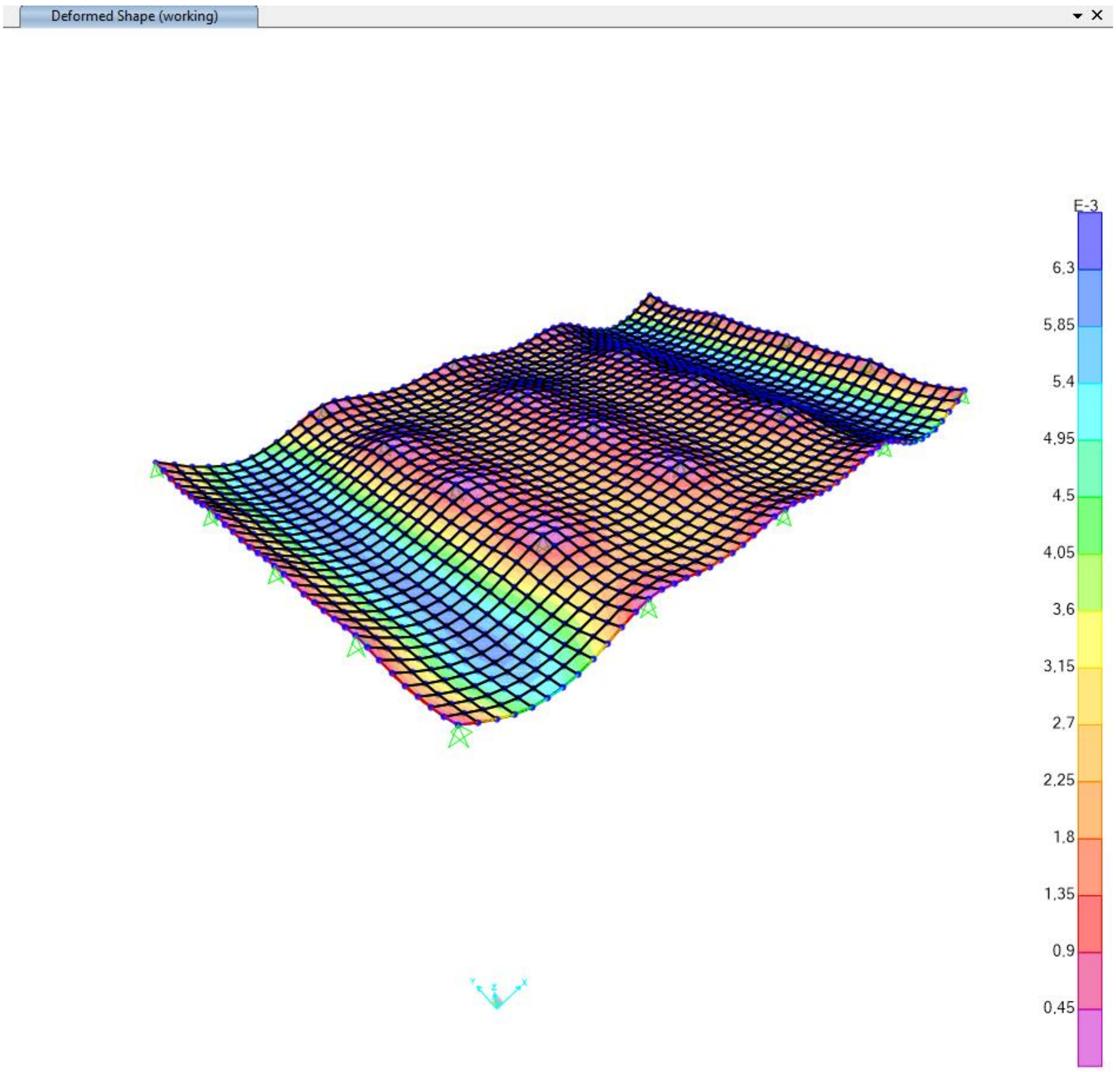


Figure 2 Deflection

Figure 3 shows deflection at the various points in the slab. In the middle strip the deflections are much higher than in the column strip. The maximum and minimum deflections are shown in Table 2.

Minimum deflection	0mm
Maximum deflection	6mm

Tabell 2 : Maximum and minimum deflection

HD 230, med fugevekten inkludert i romvekten.

Tittel Appendix E			Side 1
Prosjekt bachelor oppgave	Ordre	Sign	Dato 06-04-2022

Dataprogram: E-BJELKE versjon 7.1 Laget av Sletten Byggdata
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002+A1:2005+NA:2016

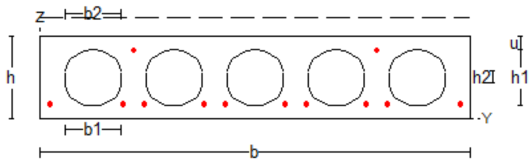
INNHOLD

1.0 Materialdata

Korreksjonsfaktor for Emodul pga tilslag	1	**Data vedr. spennarmert element**		
Materialkoeffisient betong	1,5	Strekkfasthet N/mm2 (fpk)		1860
Materialkoeffisient stål	1,15	0.1 % strekkgrense N/mm2 (fp0.1k)		1640
Betongkvalitet	B30(C30/37)	Forlengelse ved største belastning (euk)		0,037
Densitet (kg/m3)	2400	Spennarmering. Emodul		195000
Sement i fasthetsklasse (R / N / S)	R	Sylindertrykkfasthet ved avspenning (fckj)		28
Armering flytegrense	500	Sylindertrykkfasthet ved transport(fckj)		30
Bøyler flytegrense	500	Betongens alder ved avspenning (døgn)		1
Relativ fuktighet i lagringsperioden %	70			
Relativ fuktighet i ferdig bygg %	40	Eksponeringsklasser	uk:XC1	ok:XC1
Betongens alder ved pålastning (døgn)	28	Korrosjonsømfintlig armering		
Effektiv høyde, h0 (EN1992-1-1 3.1.4(5))	360	Dimensjonerende levetid		50
Korttids Emodul, Ecm	32800	**Min. overdekning (mm)**	**uk**	**ok**
Dimensjonerende trykkfasthet, fcd	17	*)Min. krav for spennarmering	25	25
Aksial strekkfasthet, fctm	3	Toleranse	5	5
Dimensjonerende strekkfasthet, fctd	1,15	Nominell overdekning	30	30
		*)Krav til overdekning for bøyler er 10 mm mindre		
Kryptall, FI 0_28	0,95	Svinntøyning, 0_28		-0,00007
Kryptall, FI 28_9000	2,23	Svinntøyning, 0_9000		-0,00056

NA.6.2.2(1) Minst 1 av følgende krav til tilslag i betongen er IKKE oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

1.1 Tverrsnitt (med samvirkepåstøp vist stiplet)



Antall hull	5	
b	1200	mm
h	230	mm
b1	155	mm
b2	155	mm
h1	155	mm
h2	33	mm
u	40	mm

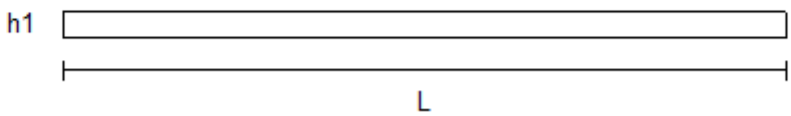
Zt	116	mm
Areal	1,76E+05	mm2
Ic	1,06E+09	mm4
Sc	6,25E+06	mm3
Wo	9,34E+06	mm3
Wu	9,11E+06	mm3
bw	425	mm

Symboler: Se NS-EN 1992 (6.4)

1.2 Armeringsdata

Kant	Lag nr	Kantavstand	Slakkarmering	Spennarmering
ok	1	40		2d 11.3- 100 mm2
uk	1	40		10d 11.3- 100 mm2

1.3 Bjelkeprofil



L	6000	mm
h1	230	mm
h2	230	mm

Elementvekt: 3,0 tonn

Utkragerlengde (mm)

	Venstre ende	Høyre ende
Utløfting	500	500
Lagring	500	500
Transport	500	500
Ferdig montert	100	100

Minste effektive oppleggsbredde: 80 mm

1.4 Lastfaktor og pålitelighetsklasse

	Lastfaktor Nedbøyning	Risskontroll	Bruddgr. B1	BENYTTES: Bruddgr. B2
Permanent last	1,00	1,00	1,35	1,20
Variabel last	0,50	0,50	1,05	1,50

Pålitelighetsklasse	3
PSI -faktor	Kategori B : kontorer
Krav til maks. nedbøyning	Konstruksjoner der det pga bruk eller utstyr stilles krav

Formsug ved avforming	0,00 kN/m
Elementets romvekt	2880 kg/m3
Horisontalkraft i oppleggspunkt (H/N)	0,20

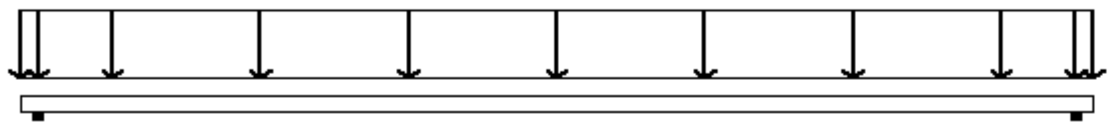
1.5 Spennkabler

Spennkraft pr kabel i ok, m. standard diameter	50,0	kN	(499 N/mm2) d=11,30 mm
Spennkraft pr kabel i uk, m. standard diameter	90,0	kN	(897 N/mm2) d=11,30 mm
Avspenning: MYK			

1.6 Egenvekt, permanent last og nyttelast

Jevnt fordelt last (kN/m)

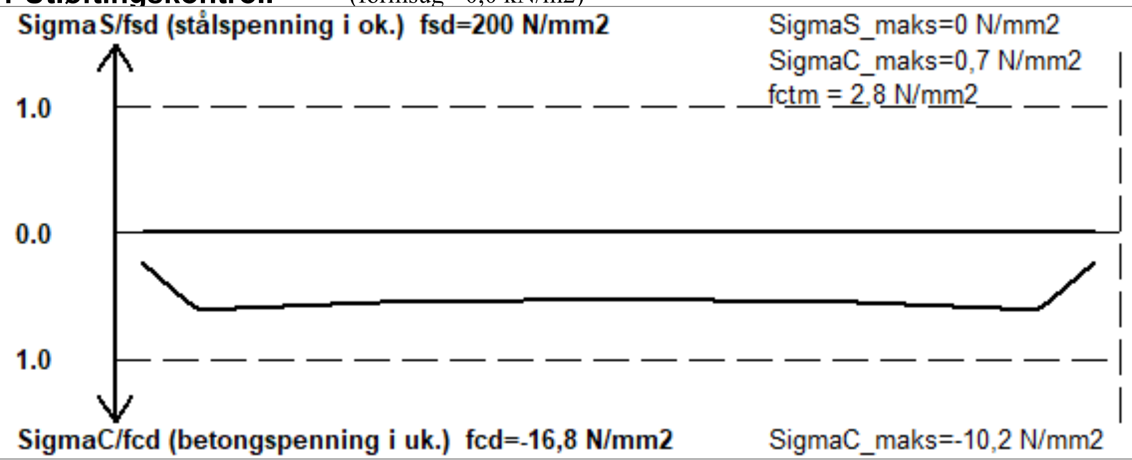
	v. utkrager	midtfelt	h. utkrager
Egenvekt	5,07	5,07	5,07
Permanent last	1,50	1,50	1,50
Variabel last	4,80	4,80	4,80



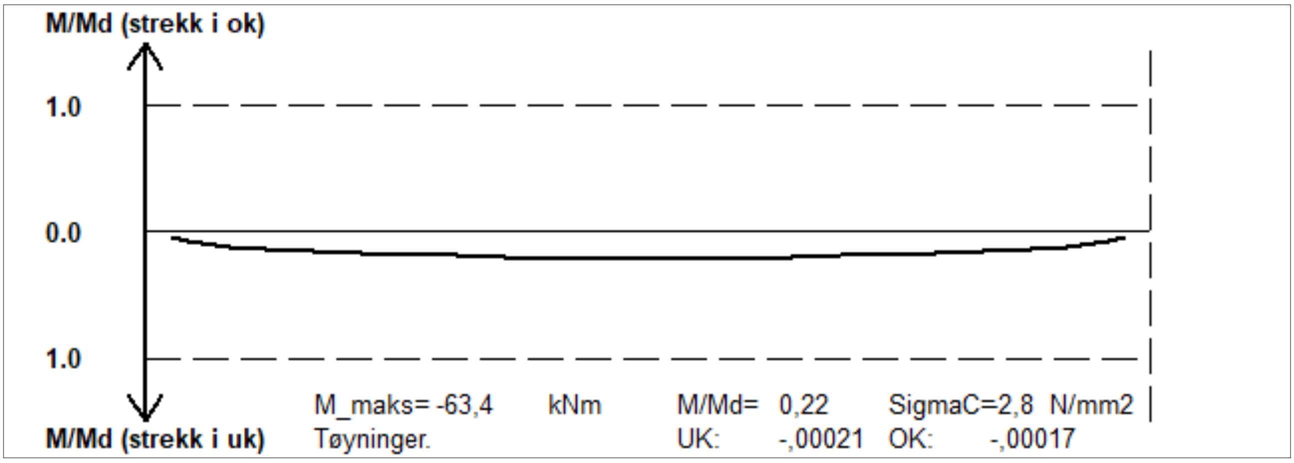
1.7 Samvirkepåstøp (med dynamisk last)

Bredde av påstøp	1200	mm	Betongkvalitet	B30(C30/37)
Tykkelse av påstøp, tp	50	mm	Antall armeringsjern	0
Fra ok bjelke til uk påstøp	0	mm		
			Fugetype:	Ru
Påført egenvekt: Lastandel etter samvirke	0,0		Effektiv fugebredde	1200 mm

5.1 Utløftingskontroll (formsug =0,0 kN/m2)



5.2 Momentkontroll



5.3 Risskontroll

Maks rissvidde=,000 mm Tillatt rissvidde=,260 mm

5.4.1 Skjærkraftkontroll

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert skjærkraft (kN)	Vrd,max trykk kap. (kN)	Vrd,c ** (kN)	Statisk nødvendig skjærarmer. (mm2/m)
255	-41,4	-40,5	453,7	112,8	0
426	-38,8	-38,8	457,8	132,3	0
500	-37,7	-37,7	457,8	139,9	0
712	-34,5	-34,5	457,8	143,9	0
998	-30,2	-30,2	457,8	150,1	0
1284	-25,9	-25,9	457,8	150,1	0
1570	-21,6	-21,6	457,8	150,1	0
1856	-17,3	-17,3	457,8	150,1	0
2428	-8,6	-8,6	457,8	150,1	0
3000	0,0	0,0	457,8	150,1	0
3572	8,6	8,6	457,8	150,1	0
4144	17,3	17,3	457,8	150,1	0
4430	21,6	21,6	457,8	150,1	0
4716	25,9	25,9	457,8	150,1	0
5002	30,2	30,2	457,8	150,1	0
5288	34,5	34,5	457,8	143,9	0
5500	37,7	37,7	457,8	139,9	0
5574	38,8	38,8	457,8	132,3	0
5745	41,4	40,5	453,7	112,8	0

(**)Beregningene forutsetter faste opplegg vertikalt. (Ikke moment på tvers av elementet.)
(**)EN 1168 4.3.3.2.2.3 er benyttet
Skjærameringen helningsvinkel med bjelkeakse: 90 grader
Trykkdiagonalens helningsvinkel med bjelkeakse: 45 grader

5.5.1 Skjærarmering gjennom støpeskjøt (Fordeling: Se NS-EN 1992 Figur 6.10)

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert Vrd,max (N/mm2)	Statisk nødvendig skjærarmer.	Minimums-armering (mm2/m)	Maks bøyleavstand (mm)
160	-42,9	0,13	0	466	500
255	-41,4	0,12	0	466	500
426	-38,8	0,11	0	466	500
500	-37,7	0,11	0	466	500
712	-34,5	0,10	0	466	500
998	-30,2	0,09	0	466	500
1284	-25,9	0,08	0	466	500
1570	-21,6	0,06	0	466	500
1856	-17,3	0,05	0	466	500
2428	-8,6	0,03	0	466	500
3000	0,0	0,00	0	466	500
3572	8,6	0,03	0	466	500
4144	17,3	0,05	0	466	500
4430	21,6	0,06	0	466	500
4716	25,9	0,08	0	466	500
5002	30,2	0,09	0	466	500
5288	34,5	0,10	0	466	500
5500	37,7	0,11	0	466	500
5574	38,8	0,11	0	466	500
5745	41,4	0,12	0	466	500
5840	42,9	0,13	0	466	500

Minimumsarmering: basert på NS-EN 1992(NA.9.5N). Maks bøyleastand: basert på tidligere praksis (NS 3473 12.7.2)

5.6 Forankringsarmering (på grunn av skjærkraft og horisontalkraft i oppleggspunkt)

Beregningene forutsetter fctd ved avspenning

Forankringskraft i v. ende, underkant	0,0	kN
Forankringskraft i h. ende, underkant	0,0	kN

6.1 Nedbøyning (mm)

(G1=egenvekt av bjelken G2=påført permanent last P=variabel last)

	V. utkrager	Midtfelt	H. utkrager
Avforming		-6	
G1: ved montasje		-9	
G1+G2: ved montasje		-9	
G1+G2+P.langtidsdel ved montasje		-8	
G1+G2 etter lang tid		-14	
G1+G2+P_langtidsdel etter lang tid		-13	
G1+G2+P_total etter lang tid		-13	

7.1 Oppleggskrefter (kN) (alle lastfaktorer = 1 i bruksgrense)

	------------- Bruksgrense -------------			------------- Bruddgrense -------------		
	Permanent last	Variabel	All last	Permanent last	Variabel	All last
v. opplegg	19,7	14,4	34,1	23,7	21,6	45,3
h. opplegg	19,7	14,4	34,1	23,7	21,6	45,3

8.1 Brannteknisk dimensjonering, basert på NS-EN 1992-1-2 og NS-EN 1168

Tverrsnitt data

Tykkelse med påstøp	h=	280	mm
Areal	Ac=	176188	mm2
Bredde	b=	1200	mm
2 armeringslag i uk			
lag nr 1: Armeringsdybde	a1=	40	mm
Indre momentarm	z1=	229	mm
armeringsareal	As1=	1003	mm2
lag nr 2: Armeringsdybde	a2=	80	mm
Indre momentarm	z2=	189	mm
armeringsareal	As2=	0	mm2

Temperatur og reduksjonsfaktorer for spennarmering.   Med brannmotstand: REI 120

Dim. spenning ved t=20 grader: fs = fp0.1k <= 0.9*fpk (se materialdata) fs = 1640 N/mm2

armeringslag nr 1	t1=	470	grader	EN 1992-1-2 fig A.2
reduksjosfaktor	kp1=	0,280		EN 1992-1-2 fig 5.1, kurve 3
Dim. spenning (kp1*fs)	fs1=	459	N/mm2	
a50%=71,6 mm	t,a50=	270,9	grader	EN 1168 fig G.1 og EN 1992-1-2 fig A.2
armeringslag nr 2	t2=	265	grader	EN 1168 G.1.2: interpolering
reduksjosfaktor	kp2=	0,703		EN 1992-1-2 fig 5.1, kurve 3
Dim. spenning (kp2*fs)	fs2=	1153	N/mm2	

Kontroll av utnyttelse i mest påkjente snitt

Krav til brannmotstand etter EN 1168, tabell G.1 er oppfylt
Lastfaktor for egenlast: 1.0   Lastfaktor for nyttelast: 0,30

Momentkontroll: Mfi: moment fra påførte laster: Md,fi: tverrsnittets momentkapasitet ved brann
Mfi = 34 kNm   Md,fi = fs1*As1*z1 + fs2*As2*z2 = 105 kNm   Mfi/Md,fi = 0,32 < 1   OK

Skjærkontroll: Vfi: skjærkraft fra påførte laster: Vrdc,fi: tverrsnittets skjærkapasitet ved brann
Avstand fra v.ende til kontrollsnitt for skjærkraft: 255 mm
Reduksjonsfaktor for skjærkapasitet (EN 1168 tabell G.2) kpV = 0,55
Vrdc = 113 kN   Vrdc,fi =Vrdc*kpV = 62 kN
Vfi   = 22 kN Vfi/Vrdc,fi = 0,35 < 1   OK
EN 1168 tabell G.2 forutsetter at det er lagt inn forangkringsarmering i fuge eller hull: 226 mm2/element

Dataprogram: E-BJELKE versjon 7.1 Laget av Sletten Byggdata
Beregningene er basert på NS-EN 1992-1-1 og NS-EN 1990:2002+A1:2005+NA:2016
Data er lagret på fil: F:\bachelor\HD400-5hull-Stjørdal-def2.ebj

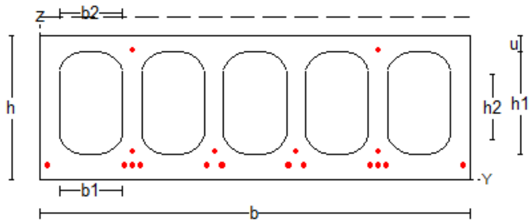
INNHOLD

1.0 Materialdata

Korreksjonsfaktor for Emodul pga tilslag	1	**Data vedr. spennarmert element**		
Materialkoeffisient betong	1,5	Strekkfasthet N/mm2 (fpk)		1860
Materialkoeffisient stål	1,15	0.1 % strekkgrense N/mm2 (fp0.1k)		1640
Betongkvalitet	B30(C30/37)	Forlengelse ved største belastning (euk)		0,037
Densitet (kg/m3)	2400	Spennarmering. Emodul		195000
Sement i fasthetsklasse (R / N / S)	R	Sylindertrykkfasthet ved avspenning (fckj)		28
Armering flytegrense	500	Sylindertrykkfasthet ved transport(fckj)		30
Bøyler flytegrense	500	Betongens alder ved avspenning (døgn)		1
Relativ fuktighet i lagringsperioden %	70			
Relativ fuktighet i ferdig bygg %	40	Eksponeringsklasser	uk:XC1	ok:XC1
Betongens alder ved pålastning (døgn)	28	Korrosjonsømfintlig armering		
Effektiv høyde, h0 (EN1992-1-1 3.1.4(5))	360	Dimensjonerende levetid		50
Korttids Emodul, Ecm	32800	**Min. overdekning (mm)**	**uk**	**ok**
Dimensjonerende trykkfasthet, fcd	17	*)Min. krav for spennarmering	25	25
Aksial strekkfasthet, fctm	3	Toleranse	5	5
Dimensjonerende strekkfasthet, fctd	1,15	Nominell overdekning	30	30
		*)Krav til overdekning for bøyler er 10 mm mindre		
Kryptall, FI 0_28	0,95	Svinntøyning, 0_28		-0,00007
Kryptall, FI 28_9000	2,23	Svinntøyning, 0_9000		-0,00056

NA.6.2.2(1) Følgende krav til tilslag i betongen er oppfylt:
1. Største tilslag etter NS-EN 12620: D>=16 mm (D= 22 mm)
2. Det grove tilslaget >=50% av total tilslagsmengde
3. Grovt tilslag skal ikke være av kalkstein eller stein med tilsvarende lav fasthet

1.1 Tverrsnitt (med samvirkepåstøp vist stiplet)



Antall hull	5	
b	1200	mm
h	400	mm
b1	174	mm
b2	174	mm
h1	288	mm
h2	180	mm
u	44	mm

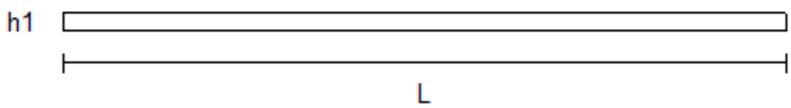
Zt	189	mm
Areal	2,50E+05	mm2
Ic	4,97E+09	mm4
Sc	1,62E+07	mm3
Wo	2,35E+07	mm3
Wu	2,63E+07	mm3
bw	330	mm

Symboler: Se NS-EN 1992 (6.4)

1.2 Armeringsdata

Kant	Lag nr	Kantavstand	Slakkarmering	Spennarmering
ok	1	40		2d 11.3- 100 mm2
uk	1	40		12d 11.3- 100 mm2
uk	2	80		4d 11.3- 100 mm2

1.3 Bjelkeprofil



L	16000	mm
h1	400	mm
h2	400	mm

Elementvekt: 10,7 tonn

Utkragerlengde (mm)

	Venstre ende	Høyre ende
Utløfting	1000	1000
Lagring	500	500
Transport	500	500
Ferdig montert	100	100

Minste effektive oppleggsbredde: 80 mm

1.4 Lastfaktor og pålitelighetsklasse

	Lastfaktor			BENYTTES:
	Nedbøyning	Risskontroll	Bruddgr. B1	Bruddgr. B2
Permanent last	1,00	1,00	1,35	1,20
Variabel last	0,50	0,50	1,05	1,50

Pålitelighetsklasse	3
PSI -faktor	Kategori B : kontorer
Krav til maks. nedbøyning	Konstruksjoner der det pga bruk eller utstyr stilles krav
Formsug ved avforming	0,00 kN/m
Elementets romvekt	2670 kg/m3
Horisontalkraft i oppleggspunkt (H/N)	0,20

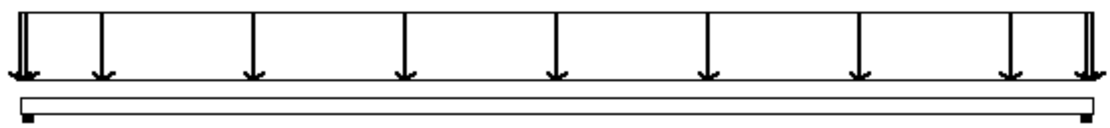
1.5 Spennkabler

Spennkraft pr kabel i ok, m. standard diameter	50,0	kN	(499 N/mm2) d=11,30 mm
Spennkraft pr kabel i uk, m. standard diameter	90,0	kN	(897 N/mm2) d=11,30 mm
Avspenning: MYK			

1.6 Egenvekt, permanent last og nyttelast

Jevnt fordelt last (kN/m)

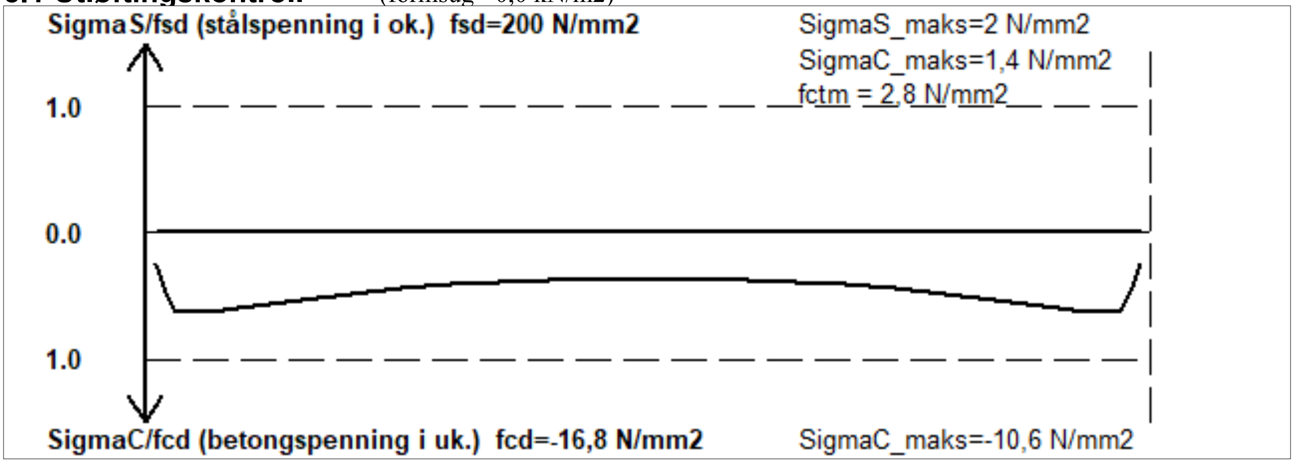
	v. utkrager	midtfelt	h. utkrager
Egenvekt	6,68	6,68	6,68
Permanent last	1,50	1,50	1,50
Variabel last	4,80	4,80	4,80



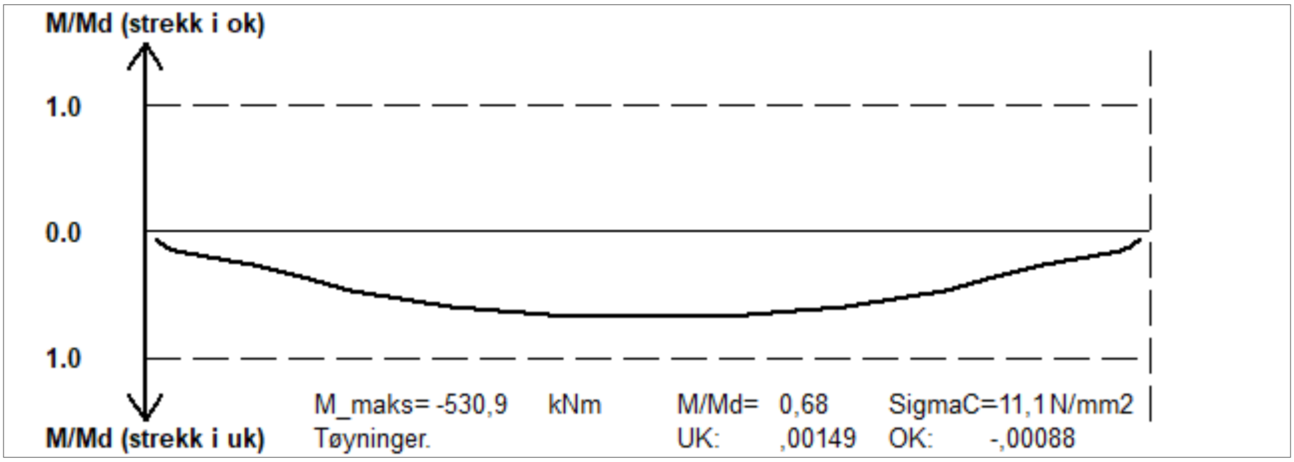
1.7 Samvirkepåstøp (med dynamisk last)

Bredde av påstøp	1200	mm	Betongkvalitet	B30(C30/37)
Tykkelse av påstøp, tp	50	mm	Antall armeringsjern	0
Fra ok bjelke til uk påstøp	0	mm		
			Fugetype:	Ru
Påført egenvekt: Lastandel etter samvirke	0,0		Effektiv fugebredde	1200 mm

5.1 Utløftingskontroll (formsug =0,0 kN/m2)



5.2 Momentkontroll



5.3 Risskontroll

Maks rissvidde=,000 mm Tillatt rissvidde=,260 mm

5.4.1 Skjærkraftkontroll

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert skjærkraft (kN)	Vrd,max trykk kap. (kN)	Vrd,c ** (kN)	Statisk nødvendig skjærarmer. (mm2/m)
340	-130,3	-127,8	592,4	168,9	0
500	-127,6	-126,5	592,4	193,1	0
1000	-119,1	-119,1	592,4	207,5	0
1712	-107,0	-107,0	592,4	207,5	0
2498	-93,6	-93,6	592,4	207,5	0
3284	-80,2	-80,2	592,4	207,5	0
4070	-66,9	-66,9	592,4	157,0	0
4856	-53,5	-53,5	592,4	157,0	0
6428	-26,7	-26,7	592,4	157,0	0
8000	0,0	0,0	592,4	157,0	0
9572	26,7	26,7	592,4	157,0	0
11144	53,5	53,5	592,4	157,0	0
11930	66,9	66,9	592,4	157,0	0
12716	80,2	80,2	592,4	207,5	0
13502	93,6	93,6	592,4	207,5	0
14288	107,0	107,0	592,4	207,5	0
15000	119,1	119,1	592,4	207,5	0
15500	127,6	126,5	592,4	193,1	0
15660	130,3	127,8	592,4	168,9	0

(**)Beregningene forutsetter faste opplegg vertikalt. (Ikke moment på tvers av elementet.)
(**)EN 1168 4.3.3.2.2.3 er benyttet
Skjærameringen helningsvinkel med bjelkeakse: 90 grader
Trykkdiagonalens helningsvinkel med bjelkeakse: 45 grader

5.5.1 Skjærarmering gjennom støpeskjøt (Fordeling: Se NS-EN 1992 Figur 6.10)

Avst. til v. ende (mm)	Maks skjærkraft (kN)	Redusert Vrd,max (N/mm2)	Statisk nødvendig skjærarmer.	Minimums-armering (mm2/m)	Maks bøyleavstand (mm)
165	-133,3	0,19	0	361	500
340	-130,3	0,19	0	361	500
500	-127,6	0,19	0	361	500
1000	-119,1	0,17	0	361	500
1712	-107,0	0,16	0	361	500
2498	-93,6	0,14	0	361	500
3284	-80,2	0,12	0	361	500
4070	-66,9	0,10	0	361	500
4856	-53,5	0,08	0	361	500
6428	-26,7	0,04	0	361	500
8000	0,0	0,00	0	361	500
9572	26,7	0,04	0	361	500
11144	53,5	0,08	0	361	500
11930	66,9	0,10	0	361	500
12716	80,2	0,12	0	361	500
13502	93,6	0,14	0	361	500
14288	107,0	0,16	0	361	500
15000	119,1	0,17	0	361	500
15500	127,6	0,19	0	361	500
15660	130,3	0,19	0	361	500
15835	133,3	0,19	0	361	500

Minimumsarmering: basert på NS-EN 1992(NA.9.5N). Maks bøyleastand: basert på tidligere praksis (NS 3473 12.7.2)

5.6 Forankringsarmering (på grunn av skjærkraft og horisontalkraft i oppleggspunkt)

Beregningene forutsetter fctd ved avspenning

Forankringskraft i v. ende, underkant	0,0	kN
Forankringskraft i h. ende, underkant	0,0	kN

6.1 Nedbøyning (mm)

(G1=egenvekt av bjelken G2=påført permanent last P=variabel last)

	V. utkrager	Midtfelt	H. utkrager
Avforming		-5	
G1: ved montasje		-2	
G1+G2: ved montasje		5	
G1+G2+P.langtidsdel ved montasje		8	
G1+G2 etter lang tid		14	
G1+G2+P_langtidsdel etter lang tid		27	
G1+G2+P_total etter lang tid		28	

7.1 Oppleggskrefter (kN) (alle lastfaktorer = 1 i bruksgrense)

	------------- Bruksgrense -------------			------------- Bruddgrense -------------		
	Permanent last	Variabel	All last	Permanent last	Variabel	All last
v. opplegg	65,4	38,4	103,8	78,5	57,6	136,1
h. opplegg	65,4	38,4	103,8	78,5	57,6	136,1

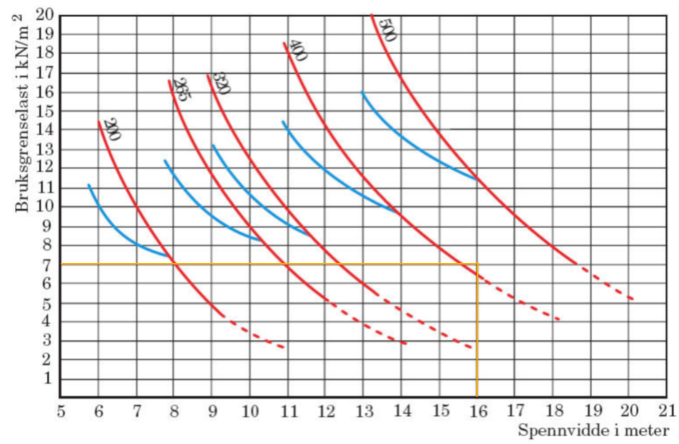
Hollow-core elements - Dimensioning diagram

Step 2: Deflection check of column strip

When dimensioning , it must be checked that the current load state as 0,9g + p is less than the load-bearing capacity stated in the diagram. The self-weight is not included in the applied load.

$$\mathbf{g} = 1,25KN/m^2 \quad \mathbf{p} = 4KN/m^2$$

$$Apllied\ load : 0.9 * 1,25 + 4 = 5,125KN/m^2$$



$$Required\ dimension : HD400$$

CHECK:

$$Maximum\ loading\ of\ use\ is\ bigger\ than\ apllied\ load\ (OK)$$

Flat slab - Quantity Calcualtion

Hollow core slab only needed the area to calculate the price and CO_2 emmision in ISY Calcus. Flat slab needed the volume, area and Kg to calculate the total price and CO_2 emmisions. Below, is it inserted methods for calculate these;

VOLUME: CONCRETE SLAB 1: -
Height x Width x Length = Volume

Betong (concrete) : $220 \cdot 10^{-3} = 84,48m^3$
Double check with hand calculation in addition to ISY Calcus
$-Price : 84,48m^3 \cdot 1889,68\frac{kr}{m^3} = 159640$

AREA: CONCRETE SCREED AND FORMWORK 1: -
Height x Width = Area

Påstøp (screed) : $384m^2$ Double check with hand calculation in addition to ISY Calcus
$-Price : 384m^2 \cdot 242,91\frac{kr}{m^2} = 93276$

KILOGRAM: REINFORCEMENT 1: -
Total reinforcement pr direction x mass of one piece of reinforcing bar x spacing x length of the direction)= kg for longitudinal/ transverse

Armering i dekker (Reinforcement) :
To calculate price and CO2 emissions, it was asked to enter quantity in ISY Calcus. These quantities are taken from the Flat slab calculation. In the Reinforcement file, the number of reinforcement per direction and the spacing are added. The method of calculation is given above;

Top Reinforcement
Longitudinal :
$(40 * 0,888 * 753,33) + (53 * 0,888 * 594,74) * 10^-3 * 24 = 1313,978999kg$
Transverse :
$96 * 0,888 * 452 * 10^-3 * 16 = 616,513536kg$
Bottom reinforcement
Longitudinal :
$(67 * 0,888 * 1255,56) + (53 * 0,888 * 594,74) * 10^-3 * 24 = 2464,599387kg$
Transverse :
$(120 * 0,888 * 565) + (96 * 0,888 * 452) * 10^-3 * 16 = 1579,815936kg$

$$Total\ Reinforcement : 5974,907858kg$$