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MASTER'S THESIS

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Abstract

Hollow core slabs (HCS) are pre-stressed concrete elements, normally supported by steel or reinforced concrete/pre-stressed concrete beams. Studies have shown that the deflection of the supporting beam may be the reason for decreased shear capacity of the HCS, leading to premature failure of the HCS. Since shear tension failure is a common failure in HCS, there is a need for measures to prevent damage. There are measures to increase the shear capacity of the HCS which have been tested in experiments by other researchers, which will be discussed in this thesis.

When the supporting beam deflects, it also leads to a corresponding rotation in the connection zone. Cracks may appear in the connection as a result of this, and in addition with shrinkage and creep in the concrete, large cracks may form and damage the above flooring. Brittle flooring material, such as ceramic tiles, easily crack when there is movement and cracking in the structure underneath. Calculations will be done to analyze different measures to decrease the cracking in the connection joints, and measures to limit the rotation in the support zone. The results will be compared and analyzed to see the effect.

Steel fiber reinforced hollow core slabs have shown in studies, done by other researchers, promising result when it comes to controlling the damages resulting from deflection in the supporting beam. Since the HCS does not have shear reinforcement, the steel fibers would act as shear reinforcement. Using fiber in the HCS also increased the flexural strength. Filling the voids with concrete have shown in studies not to have as good effect as hoped, being a questionable solution according to some studies.

When using brittle flooring materials in hollow core slab flooring systems, it is recommended from calculations to use smaller spans, continuous supporting beams and thin HCS to reduce rotation due to deflection of the supports. Adding extra reinforcement in the connection joint has shown to be a good solution to control cracking. Normally reinforcement is not placed in the connections to control cracking, however, results from calculations have shown this measure have a positive impact on the crack width calculations, by decreasing the cracks up to 21,9%.

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Preface

This thesis ends my master's degree at University in Stavanger, the Faculty of Science and Technology. It has been educational and interesting to work with this thesis.

The work on this thesis took place between February 15th and June 15th, 2021. I have learned a lot during this period, and I am grateful for taking this experience with me into the working life.

I wish to thank my supervisor, Samindi Samarakoon, for being helpful throughout the process. I am also grateful for my husband Ola and daughter Jenny, for being my mental support and understanding.

14th of June 2021, Stavanger

Tables

Equations

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Notations

Latin letters

B_w	Width of the cross-section at the centroidal axis, allowing for the presence of ducts in accordance with expressions (6.16) and (6.17) in EC2.
C	Cover to the longitudinal reinforcement.
d	Expected deflection
E_s	Design value of modulus of elasticity of reinforcing steel
E_{cm}	Secant modulus of elasticity of concrete.
F_{cm}	Mean compressive strength
F_{cmo}	= 10MPa
$F_{ct,eff}$	Mean value of the tensile strength of the concrete at the time cracking is expected to happen.
I	Second moment of intertia
K_1	A coefficient which takes account of the bond properties of the bonded reinforcement. = 0.8 for high bond bars = 1.6 for bars with an effectively plain surface (e.g. prestressing tendons)
K_2	A coefficient which takes account of the distribution of strain: = 0.5 for bending = 1.0 for pure tension
K_3	= 3.4
K_4	= 0.425
K_t	A factor dependent on the duration of the load. K_t = 0.6 for short term loading. K_t = 0.4 for long term loading.
L	Span length
RH	Ambient relative humidity (%)
RH_0	= 100%
S	First moment of area above and about the centroidal axis
$S_{r,max}$	Maximum crack spacing.
W_k	Crack width

Greek letters

α	$= l_x/l_{pt2} \leq 1.0$ for pretensioned tendons.
α_{ds1}	A coefficient which depends on the type of cement.
	= 3 for cement Class S
	= 4 for cement Class N
	= 6 for cement Class R
α_{ds2}	A coefficient which depends on the type of cement
	= 0.13 for cement Class S
	= 0.12 for cement Class N
	= 0.11 for cement Class R
ε_{cs}	Total shrinkage strain.
ε_{cd}	Drying shrinkage strain.
ε_{ca}	Autogeneous shrinkage strain.
ε_{cm}	Mean strain in the concrete between cracks.
ε_{sm}	Mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered.
ξ	Ratio of bond strength of prestressing and reinforcing steel
σ_s	Stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\sigma_p$ the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level.
l_x	Distance of section considered from the starting point of the transmission length.
l_{pt2}	Upper bound value of the transmission length of the prestressing element according to Expression (8.18)
σ_{cp}	Concrete compressive stress at the centrodal axis due to axial loading and/or prestressing.
φ	Bar diameter.
φ_p	Bar diameter or equivalent diameter of the bar.
φ_s	Biggest bar diameter.
θ	$= \frac{16d}{5L}$

Glossary

Transverse reinforcement	Reinforcement going across hollow core slab joints
Longitudinal reinforcement	Reinforcement going along hollow core slab joints
HCS	Hollow core slab
BB-B	Betongelementboka Bind B
BB-C	Betongelementboka Bind C
Concr. Inv.T.	Concrete inverted T-beam
S-value	Theoretically calculated crack width

1 Thesis description

1.1 Introduction

Hollow core slabs (HCS) are pre-stressed concrete elements, normally used as floors and roof in residential and commercial buildings. Steel or reinforced concrete/pre-stressed concrete beams will act as a flexural support to the HCS and will deflect when load is applied. HCS will fail when overloaded, and four principal modes in which the HCS fails are pure flexural failure, anchorage (bond slip), shear compression and shear tension (web shear failure). The latter is the failure mode mostly seen in HCS, and studies have shown that the deflection of the supporting beam may be the reason for decreased shear capacity of the HCS, leading to premature failure of the HCS. There are measures to increase the shear capacity of the HCS which have been tested in experiments by other researchers. A summation of various studies will take place, as well as a discussion in Chapter 6.

When the supporting beam deflects, it also leads to a corresponding rotation in the connection zone. Cracks may appear in the connection as a result of this, and in addition with shrinkage and creep in the concrete, large cracks may form and damage the above flooring. Brittle flooring material, such as ceramic tiles, easily crack when there is movement and cracking in the structure underneath. Calculations have been done to limit the cracking in the connection joints using different measures, and to limit the rotation in the support zone. The results will be compared and analyzed to see the effect.

1.1 Objectives of this thesis

The objective of this study is to carry out literature review on design guidelines for HCS, and study research carried out from researchers regarding damages in HCS that can appear when the supporting beams deflects, and possible solutions to avoid these damages.

Calculations are done in FEM-design to study the resulting effect from different measures to reduce the crack width in the joint connections in HCS, as well as reducing the rotation in the support.

1.2 Limitations

There are certain limitations attached to the calculation method and the results. First of all, prestressing losses in the hollow core slabs were not considered in the calculations, nor were prestressed concrete beams. Second of all, linear finite element analysis has been used.

2 Literature review

2.1 Pre-cast concrete members: Hollow Core Slabs (HCS)

Hollow core slabs are pre-stressed concrete elements used world-wide, normally used as floors and roof in multistory buildings such as office- and business buildings, schools, and hospitals. High tensile strength prestressed strands are placed within the element in the manufacturing process, which prestresses the HCS-elements making an upward deflection, to minimize the downward deflection when load is added to the HCS. How much prestressing is required, are designed individually for every structure.

Hollow core slabs have many advantages and diverse application. Some of the advantages are assured quality, quick and easy installation, excellent fire resistance, high load capacity and rigidity, and reduced self-weight [1].

Hollow core slabs come in various span lengths, normally from five meters to 14 meters, and the width is normally 1200mm. The most common thicknesses are 200mm to 320mm, but it can go as high as 500mm. Normal supporting structures for the hollow core slabs are pre-cast concrete beam, concrete wall and steel beams.

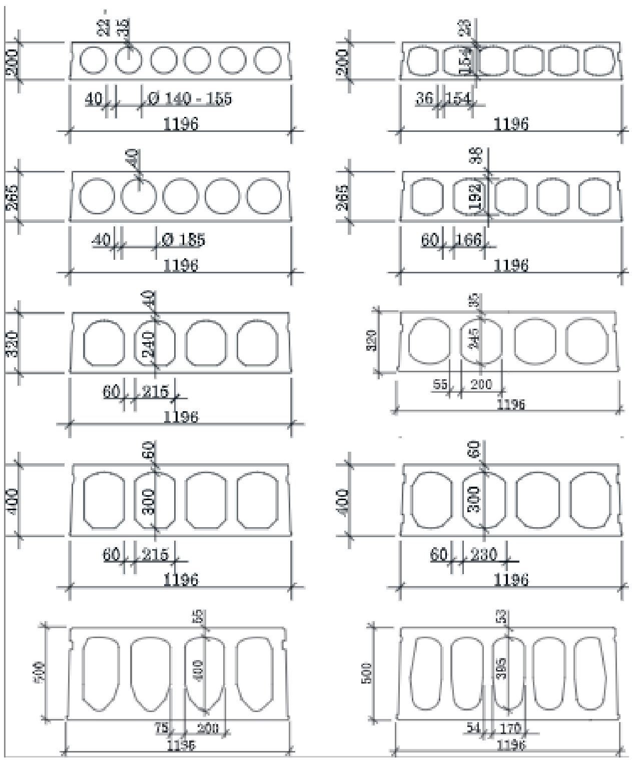

Figure 2-1 Different cross-sections of hollow core slabs [2]

The longitudinal voids in the HCS leads up to 50% reduction of concrete compared to a in situ slab, and due to the lower self-weight, it cuts the amount of prestressing steel by 30% [1]. The low weight cause smaller transport costs and easy on-site handling, and large production volumes and the ability to move the casting bed after just 6-8 hours causes big money savings.

The hollow core slab is reinforced with high-strength steel strands that are pre-stressed before the concrete hardened. Usual reinforcement is 7-wire strands 9.3mm diameter. The strands are located as shown in Figure 2-2. Elastic modulus of the standard strands is 195GPa, compared to regular structural steel which has an elastic modulus of 210GPa.

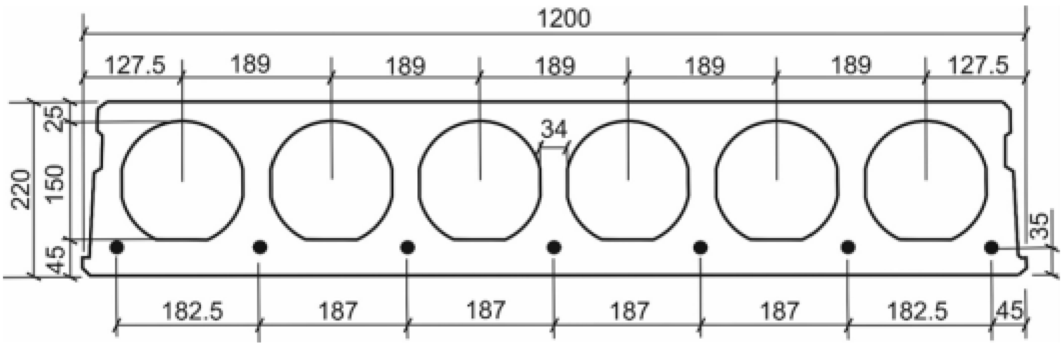

Figure 2-2 Reinforcement strands in hollow core slab [4]

2.1.1 Application to buildings

There are guidelines and recommendations for hollow core slabs in matter of how to support it on bearing structures. This applies to support length and necessary reinforcement in the connection to ensure the HCS and bearing structures are properly attached.

Betongelementboka Bind B (BB-B) [3] and Betongelementboka Bind C (BB-C) [2] are used as a guideline in this thesis when it comes to HCS connections. Betongelementboka is a Norwegian special literature about precast concrete elements, which is a guideline made for engineers, architects and others involved in the building projects. These books are also found online.

2.1.1.1. Support of the hollow core slab

A proper support length is important to ensure loads applied on the HCS can be transferred to the supporting structures without damaging the elements involved. BB-C suggest for concrete flange beams (LB and DLB) and concrete walls, that the supporting lengths should be:

For HCS 200 – HCS 400: support width 110mm and joint 40mm
For HCS 500 - HCS 520: support with 150mm and joint 50mm

For steel beams:
For HCS 200 – HCS 340: support width 80mm and joint 30mm
For HCS 380 – HCS 520: support width 100mm and joint 40mm

Between the HCS and support there is a rubber band. This is to establish a connection with room for movement. Read more about important qualities for connections in chapter 2.1.1.2.

Figure 2-3 and Figure 2-4 shows how to form a good connection between concrete wall/beam and steel beams, respectively.

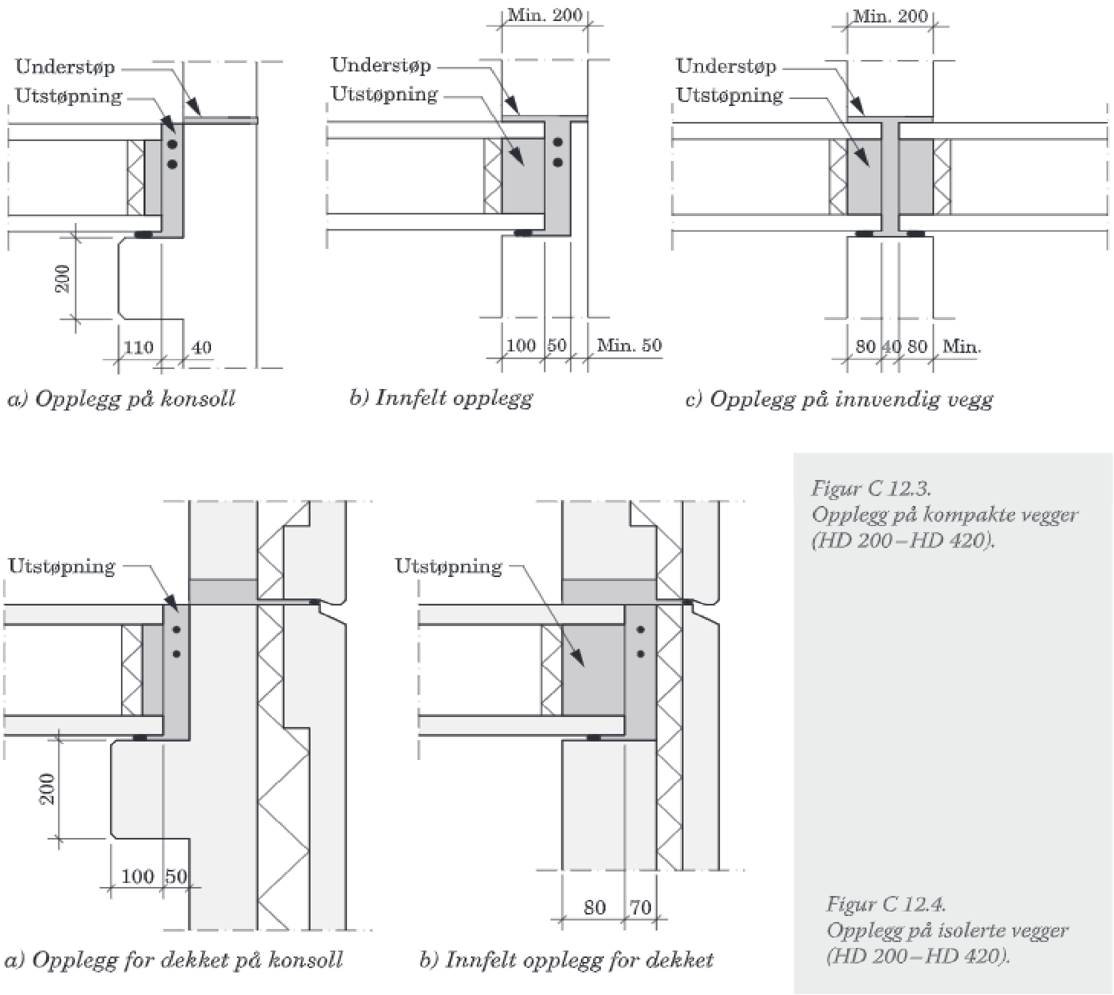

Figure 2-3 Support of hollow core slab on walls [2]

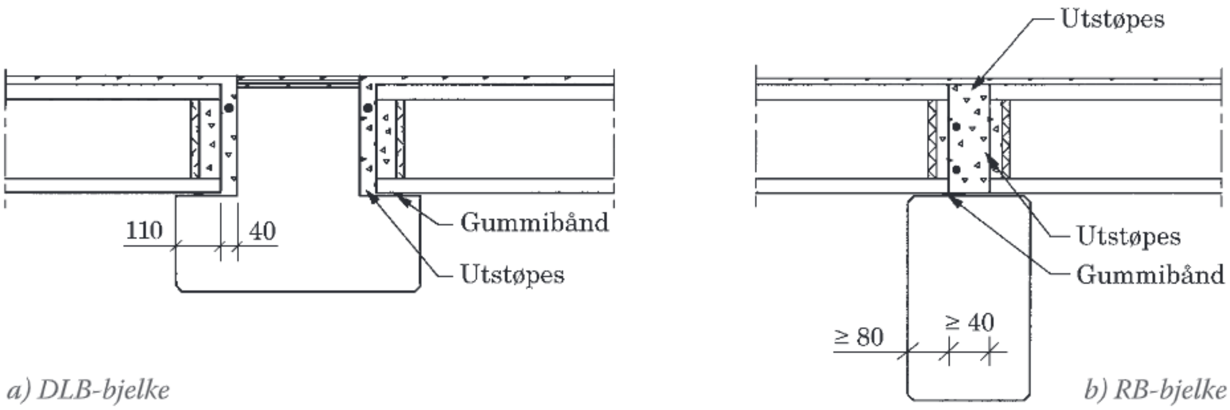

Figure 2-4 Support of hollow core slab on beam [2]

2.1.1.2. Joints and connections

The connections and joints are responsible for strength and stability for precast structures [5]. It is necessary to establish the difference between a joint and a connection. A joint is the interface or a purposely gap between two or more structural elements, where the action forces such as tension, shear, compression, and moments may occur. A connection is an assembly, involving one or more interfaces and parts of adjoining elements, designed to resist present forces [6].

Connections and joint fail for many reasons, for instance due to

- Improper detailing of reinforcement in elements
- Inadequate design of precast element
- Type of concrete use and its mix design
- Low quality of material used at the time of production.

Other points may be low quality of material, wrong assembly, and ignorance by the workers [5].

A connection should not be detailed in a singular matter, by only consider it as a detail. It should be designed in a way that considers the global stability. All connections between precast wall element and floor elements need to have a tensile force capacity, meaning that all the applied reinforcement needs to have a sufficient anchorage. Connections should also have a ductile behavior, giving the connection chance to have a relatively large plastic deformation before failure. Figure 2-5 shows the mechanical behavior a connection should have, by withstanding further deformation after concrete failure, rather than going straight into failure [6].

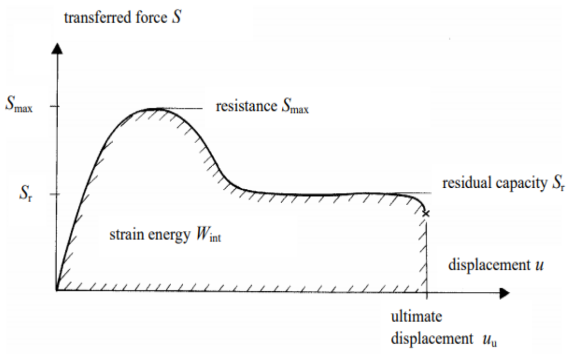

Figure 2-5 Mechanical behavior of a structural connection can be characterized by a load-displacement relationship for the primary action [6]

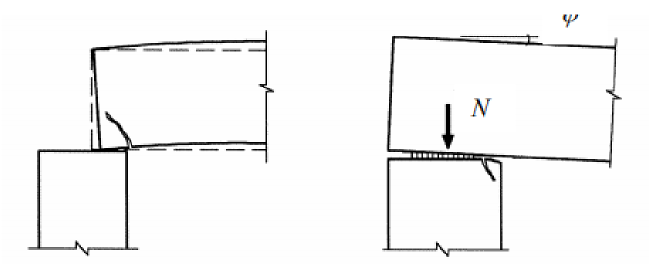

Figure 2-6 Movement in the precast elements damaging the connection [6]

A connection must withstand movement in the system, otherwise there will be a risk of damage. Examples of movement could be applied load, shrinkage and creep of the concrete, long-term deflection, and temperature variation.

Problems that can occur in connections of precast elements.

Even though handbooks and catalogues have standard solutions for connections between HCS and supporting beams/walls, they are not always optimal. One problem with precast elements such as HCS, is that all the support are considered hinged. However, when there is for example a bearing wall on top of the connection is clamps the ends of the HCS and restraining rotation. This unintentional partial fixing can cause cracks and reduce shear capacity of the elements [6][7].

Another reason for unintentional partial fixing is the location of the reinforcement in the connection. Often voids in the HCS are opened, filled with concrete and reinforcement to increase the shear capacity of the moment, or to withhold horizontal forces that flows through the slab. The placement of the reinforcement should not be accidental, but as high in the cross section as possible. The reason for this is that the reinforcement will absorb the tensile forces as high in the connection as possible, leading to (most likely) less cracking of the concrete.

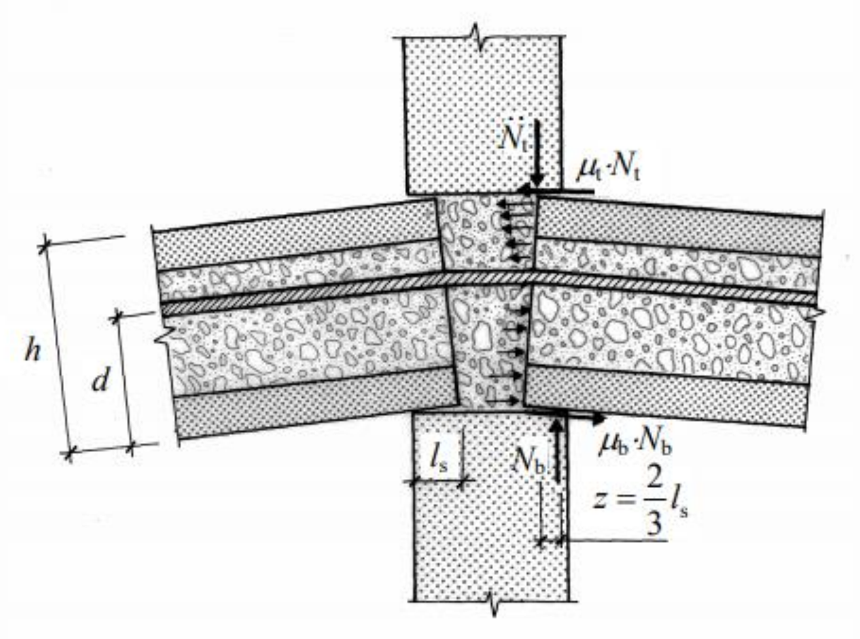

Figure 2-7 Example of cause of unintended restrain in connection [6]

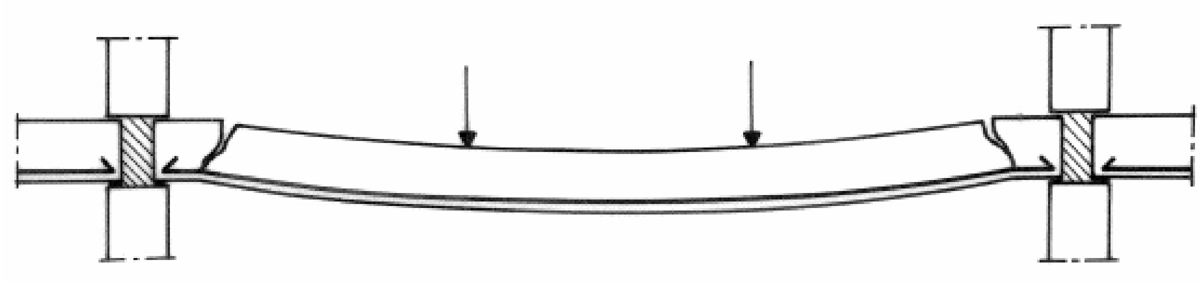

Figure 2-8 Restraining of the connection, leading to a negative moment at the support [6]

Figure 2-8 illustrate cracking that may happen due to negative bending moment at support.

“fib: Guide to good practice” [6] explains some aspects that need to be considered when it comes to the reinforcement in the connection zone:

- The reinforcement used for strengthening should be placed in the upper part of the section in addition to the ordinary tie bars, which are placed in the lower part of the section.
- The additional reinforcement bars should be anchored so that they are able to transfer their yield load in the assumed crack section.
- The amount of reinforcement in the upper part should be limited so that no further flexural cracks (from a negative bending moment) appear in unfavorable locations after formation of the first crack in the preferred location. [6]

Another problem that may arise in the connection, is the possibility of concrete cracking before the reinforcement has started to yield, making the structure brittle and delicate for failure. Using smaller cross section and more tie bars, rather than big cross section is favorable.

2.1.1.3. Seam reinforcement

To connect the hollow core slabs to the supporting structure and connecting them to themselves, calculations for required reinforcement needs to be done. The HCS works as a diaphragm for the building, transferring horizontal forces that appear throughout the building and down to the fundaments. Horizontal forces may be from wind or earth pressure acting on the outer walls, or an earthquake. The connections must be strong and well planned to be able to resist these horizontal forces, and it is important to understand how these forces flow through the connections.

According to BB-C [2] there are three things to consider when reinforcing the connections: anchoring in the ends of the HCS (in the voids and joints), anchoring the sides of the HCS and dimension the entire slab for shear forces. The latter is dimension for shear forces when considering all the HCS together as a slab, see Figure 2-9 for how the shear forces flow through the slab.

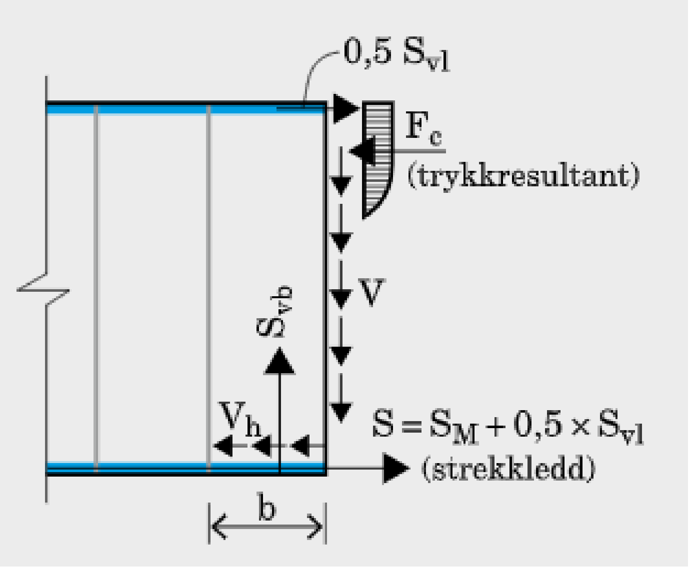

Figure 2-9 Shear forces in a slab [3]

Anchoring in voids and joints

Required anchoring in the voids are calculated from local tensile forces, such as negative pressure from wind. It is possible to anchor for these forces in the joints, in the voids or a combination. It is done by opening one or more voids, placing reinforcement, then place concrete. How much reinforcement is needed, depends on the magnitude of the present force, and the capacity of the anchoring. See Appendix A for tables describing the maximum anchoring of reinforcement steel bars in joints (Figure 9-1) and in voids (Figure 9-2).

BB-C [2] states that long anchoring and high steel stresses have problems mobilizing the big anchoring length before failure in the first canal. Therefore, the steel stresses need to be reduced, ensuring small strains. It is recommended to use 2/3 of the steel capacity, meaning $f_{yd} = 291 \frac{N}{mm^2}$. This means that the calculated required reinforcement will be higher than before.

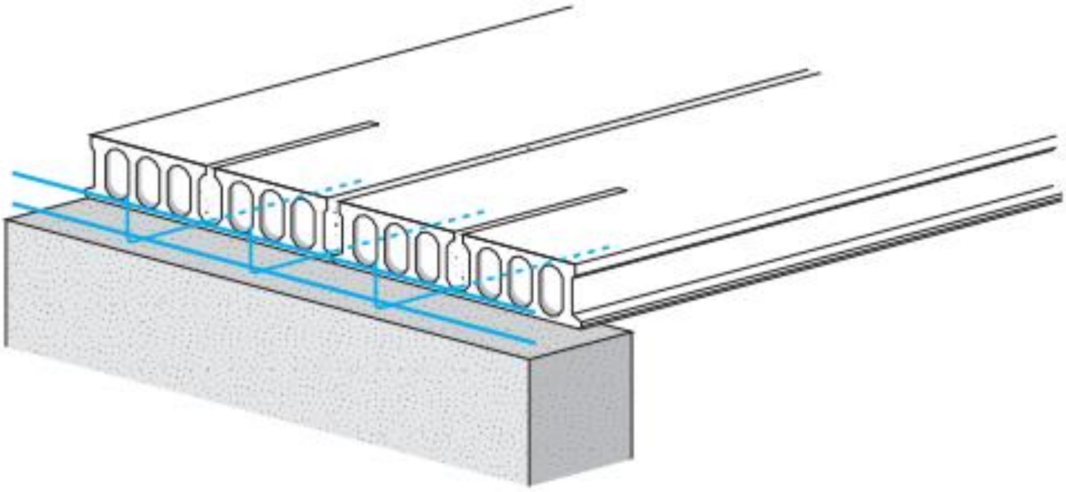

Figure 2-10 Example of reinforcement in the connection between hollow core slab and supporting beam [8].

Shear reinforcement for the slab

To ensure the slab work as a tall beam [3], reinforcement for the shear forces working in the slab needs to be calculated. The magnitude of the horizontal forces can be very high, due to forces created by earthquake.

Calculating shear in connections is done by equation:

$$V = n * \mu * (S - \frac{M}{z})$$

Equation I: Shear force in diaphragm

n	is the number of joints with longitudinal reinforcement.
μ	is a factor dependent on the surface.
S	is the longitudinal force.
M/z	is moment generated in the connection divided by the moment-arm z. See Appendix E for tables explaining the given values.

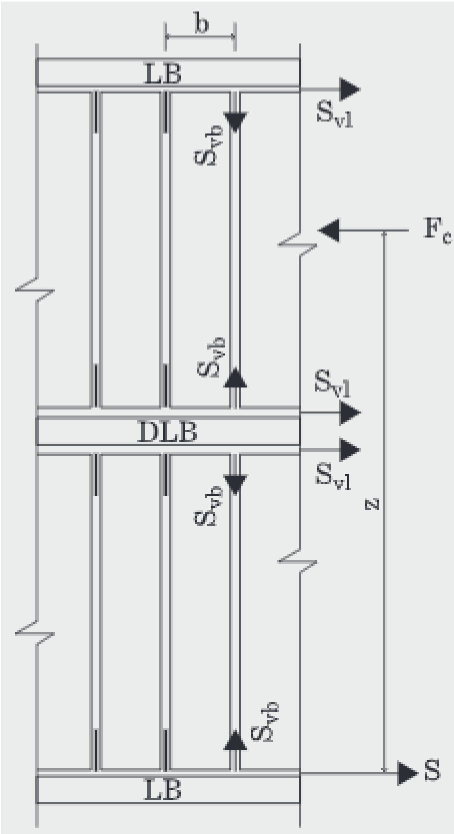

Figure 2-11 Horizontal forces in hollow core system [2]

Some examples of regular seam reinforcement are given in the Appendix E. This is used as a guideline in this thesis, as there is not going to be calculations of required reinforcement due to horizontal forces.

2.2 Damages in Hollow Core Slabs

2.2.1 Failure modes

Walraven and Mercx from Delft University in the Netherlands [9] gathered information of two series of tests that were carried out: in 1979 there where twelve tests performed, and in 1981, where thirty tests were performed. They observed the failure modes, and therefore determined four principal modes in which the HCS fails:

- Pure flexural failure
- Anchorage failure
- Shear tension
- Shear compression

<u>Pure flexural failure</u>

This is a failure mode due to behavior in bending, and the reason for this failure is cracking in the tension areal. The cracking is severe due to the small cross-section of the steel strands. This is a common failure type for HCS with uniformly distributed load, that have reached the ultimate load. Figure 2-13 illustrates the flexural cracks.

<u>Anchorage failure</u>

Anchorage failure is also a failure mode due to behavior in bending, also called bond slip. The reason for this failure mode is the steel strands in the HCS which have lost the sufficient anchorage grip due to cracking in the HCS have reached too near the support. When the strands lose their anchorage grip, the strands slip and the concrete will collapse in this cross-section. Figure 2-12 illustrates the crack that develops.

<u>Shear tension failure</u>

Shear tension failure, also called web-shear failure is a failure due to behavior in shear. When the tensile stress becomes too high near the supports in the uncracked section, an inclined crack occurs and propagates both in the upward and downwards direction resulting in failure. Figure 2-15 illustrates the damage that occur from this.

Shear compression failure

This failure mode is also due to behavior in shear. Flexural cracks can develop into shear cracks. An increase of the load can subsequently result in failure of the compression zone, by crushing or by splitting. Figure 2-14 illustrates the damages.

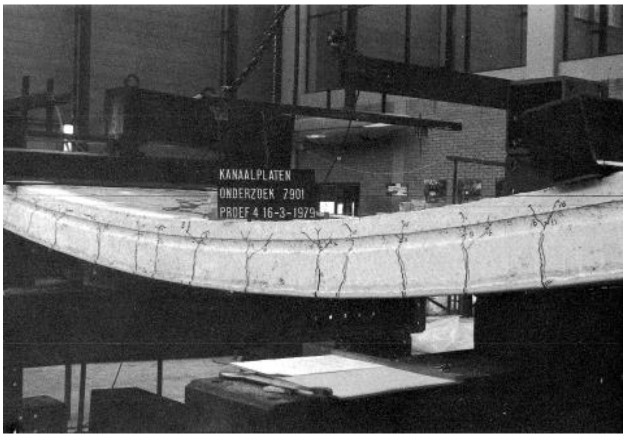

Figure 2-13 Pure flexural failure [9]

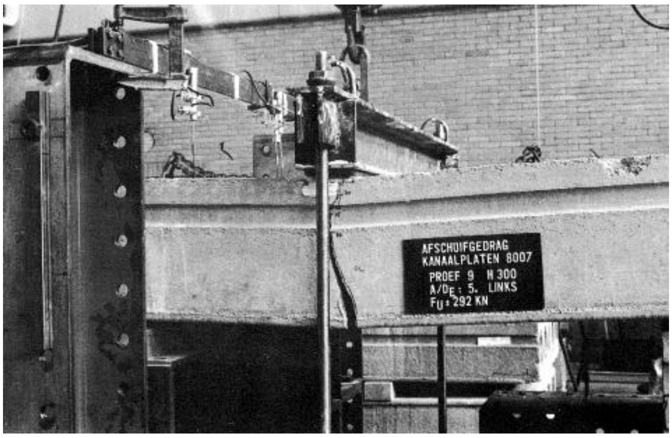

Figure 2-12 Anchorage failure [9]

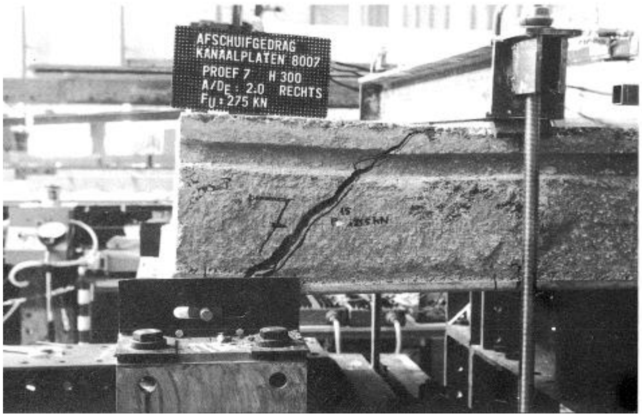

Figure 2-15 Shear tension failure [9]

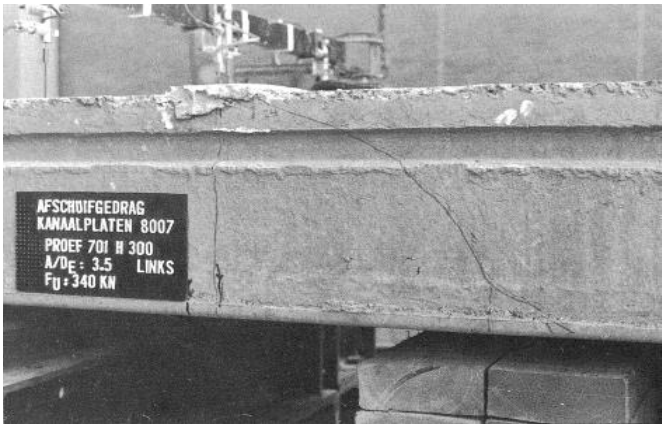

Figure 2-14 Shear compression failure [9]

Cracking in HCS can reduce the shear capacity, expose the steel reinforcement leading to corrosion, affect the load distribution, or even lead to failure. Cracks in the anchorage zone can be harmful since they have a great impact on bonding of the prestressing tendons. Tensile forces as bursting, splitting and spalling are the general reasons for cracking in the transmission zone [10].

This thesis is focusing on damages in the HCS due to deflection of the supporting beam. Many studies have shown that deflection of the supporting beam may decrease the shear capacity of the HCS, and since the shear failure is shown to be the dominant failure in many experiments, this is an important factor to investigate. The shear capacity calculations have shown to overestimate the shear capacity of the HCS, and it is believed that it has to do with the fact of not considering the extra forces and stresses appearing in the HCS due to flexural support. The following chapters will go over various research done by other researches to investigate this problem, and what measure are done to control this type of damage.

2.2.2 Shear tension failure

2.2.2.1. General

When a hollow core slab suffers from shear tension failure, an inclined crack appears near the support, see Figure 2-15. This is caused by diagonal shear tension. The hollow core slab does not have shear reinforcement and the support length is short. The shear strength of HCS is decided by the web-shear capacity at the ends, meaning the shear capacity of the member is defined when the cross-section reaches the cracking strength of concrete [11].

Tawadrous and Morcus specifies factors that can affect the web-shear strength of HCS [12]:

- Load and support configuration
- Shear span-depth ratio (a/d)
- Prestressing level
- Concrete compressive strength
- Geometry of cross section
- Overall unit depth

Failure from immediate rupture may happen in the webs of HCS when the tensile stress becomes too high in the region uncracked in bending. The prestressing force is not fully developed in this region, leaving the vertical stresses only to the concrete in the HCS.
Shear tension failure occurs of the tensile stress in the concrete reaches a critical value, i.e. tensile strength [7].

The shear tension failure occurs in the area uncracked by bending. An area uncracked by bending is a defined area in the HCS near the support, where the flexural cracking from bending do not occur. According to BB-C, the l_1 is a place between 4xh and 5xh, where the h is the height of the HCS.

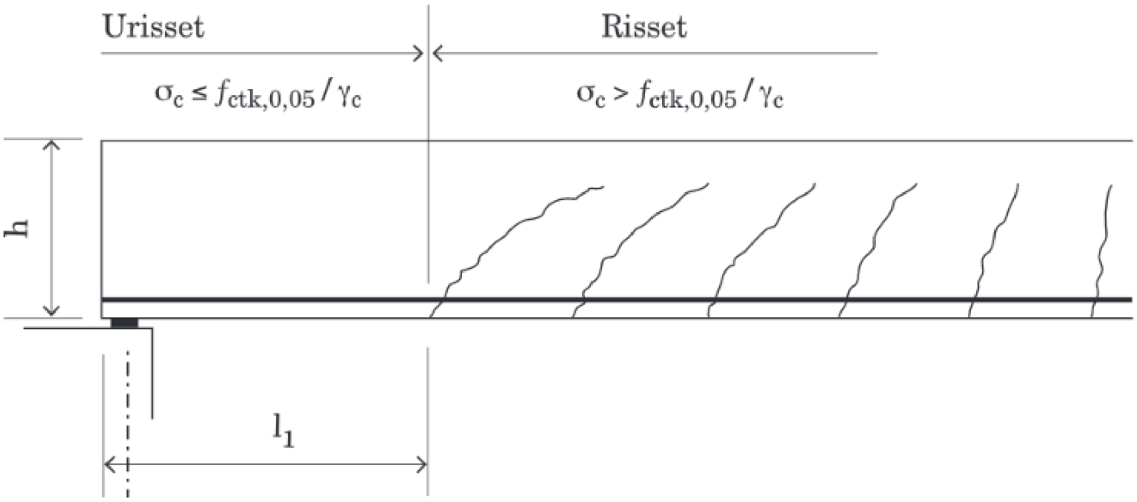

Figure 2-16 Area uncracked by bending [2]

Studies have shown when the HCS are placed on flexible support, in this context means beams that deflect, it is seen that the shear capacity calculations according to EC2 gives a too high value. There can be seen a shear reduction in the HCS, as various studies have seen [13][14][16][17]. The following chapter will explain how to calculate the shear capacity of the HCS, as well as the reasons for why the shear capacity calculations on flexural supports may be overestimated.

2.2.2.2. Shear capacity calculation of hollow core slab

There are two things to consider when calculating the shear capacity of HCS. First is the shear capacity of the HCS itself, and second is the shear capacity of the concrete filled joints.

To calculate the shear capacity of the hollow core slab, EK2-1-1 [15] clauses 6.2.2 and 6.2.3 are used. Since the HCS does not have shear reinforcement and the area near support is considered an uncracked by bending, section 6.2.2 (2) equation 6.4 is used to calculate the shear capacity for the HCS.

$$V_{RD,c} = \frac{I * b_w}{S} * \sqrt{f_{ctd}^2 + \alpha * \sigma_{cp} * f_{ctd}}$$

Equation II: Shear resistance in regions uncracked by bending

I	is the second moment of inertia
b_w	is the width of the cross-section at the centroidal axis, allowing for the presence of ducts in accordance with expressions (6.16) and (6.17) in EC2.
S	is the first moment of area above and about the centroidal axis
α	$= l_x/l_{pt2} \leq 1.0$ for pretensioned tendons.
l_x	is the distance of section considered from the starting point of the transmission length.
l_{pt2}	is the upper bound value of the transmission length of the prestressing element according to Expression (8.18)
σ_{cp}	is the concrete compressive stress at the centrodal axis due to axial loading and/or prestressing.

The reason for premature shear tension failure, may be the shear capacity calculation being overestimated for HCS on flexural support. Studies have shown that Equation II in fact is overestimated. Walraven and Mercx [9] performed 41 tests on HCS, and from this it was introduced a reduction factor for the shear capacity, due to premature failure. The paper claimed the reasons for why the shear capacity is overestimated could be:

- Concrete quality differs throughout the member, leaving the shear capacity to the weakest section, which could be very small.
- The biaxial tension-compression stress combination may result in a reduced tensile strength.
- The usage of prestressing fore at the inner edge at support (it is shown that the prestressing forces are weaker near the support)
- The period between the start of a test and failure is some hours. It is known that the tensile strength decreases under sustained load, which could even after a short time result in lower strength [9].

Derkowski [16] introduces a model by Roggendorf, where the equation for shear capacity of HCS on flexible support is extended by the components of stresses, including cracking in the joint and their consequences. It is important to consider this because when cracking in the interface between the beam and HCS occurs, or HCS and filling concrete, it increases the deflection of the beam. This leads to an increase in the cracks of the joints, creating a compression force c in the upper elements, which again creates moving of the slabs in longitudinal axis of the beam, creating transverse stresses. Equation III presents how the components of stresses, including cracking in their joint and their consequences are introduced in the standard equation from EC2:

$$V_{Rd.ct.bw} = f * \frac{I_y * b_w}{S_y * \left(1 + \alpha_{comp} * \beta_f * k_{1,c} * m * \mu\right)} * (\sqrt{f_{ctd}^2 - \alpha_1 * \sigma_{cd} * f_{ctd} - \left(\sqrt{1 - \frac{\alpha_1 * \sigma_{cd}}{f_{ctd}}} * k_v * \tau_{2c}\right)^2} - \alpha_p * \tau_{cpd}$$

Equation III: Reduction of the shear capacity for regions uncracked by bending

Without explaining all the factors, it is seen that the addition of the components of stresses will decrease the calculated shear capacity.

In addition to calculating the shear capacity of the HCS, the shear capacity of the interface between concrete should be calculated.

According to EC2-1-1, the capacity of the interface between concrete cast at different times can be calculated from clause 6.2.5, equation 6.25 (see Equation (5)). Following should satisfy the equation:

$$v_{Edi} \leq v_{Rdi}$$

$$v_{Edi} = \frac{\beta V_{Ed}}{z * b_i}$$

Equation IV: Design value of the shear stress

β	is the ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered.
V_{Ed}	is the transverse shear force.
z	is the lever arm of composite section.
v_{Rdi}	is the design shear resistance at the interface and is given by:

$$v_{Rdi} = c * f_{ctd} + \mu * \sigma_n + \rho * f_{yd}(\mu * sin\alpha + cos\alpha) \leq 0{,}5 * v * f_{cd}$$

Equation V: Design shear resistance at the interface

$c \text{ and } \mu$	are factors which depend on the roughness of the interface.
f_{ctd}	is dimensioning tensile strength of concrete.
σ_n	stress unit area caused by the minimum external normal force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0.6 f_{cd}$ and negative for tension. When σ_n is tensile c f_{ctd} should be taken as 0.
ρ	$= A_s / A_i$

A_s is the area of reinforcement crossing the interface, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface.

A_i is the area of the joint.

α is defined in figure 6.9 in EC2-1-1.

v is a strength reduction factor (see EC2-1-1 clause 6.2.2 (6)).

The EC2-1-1 clause 10.9.3 (12) states

> *In diaphragm action between precast slab elements with concreted or grouted connections, the average longitudinal shear stress* v_{Rdi} *should be limited to 0.1MPa for very smooth surfaces, and to 0.15MPa for smooth and rough surfaces. See 6.2.5 for definition of surfaces.*

These values are very low, and the reason according to BB-B [3] is the insecurity with cracking in joints, because slabs have prevented possibilities for contraction.

2.2.3 Consequences for large deformations in the supporting beam.

A topic that often arises when looking at the effect of deflection for the supporting beam of hollow core slab is reduction of the shear capacity of the HCS. When HCS are supported on beams, the beams and the slabs deflect, subjecting the HCS to both vertical and transverse shear, called interaction effect. This effect may lead the webs in the slabs to fail near the support [13]

Pajari [17] performed test on 20 hollow core slabs between the years 1990 – 2006, with different dimensions for the HCS and supporting beam, different load distributions and some of the HCS were strengthened with concrete filled in the voids. The experiments showed that the HCS went to shear tension failure before the deflection of the supporting beam reached the allowed deflection. Shear resistance have been proven to lose 23-60% shear resistance due to deflection of the supporting beam. Deflections as small as L/1000-L/300 have shown to cause considerable reductions in shear resistance in the HCS with strengthened slab ends [13].

Large deflections of supporting beams have also shown to increase the risk of longitudinal cracking of the slabs supported in the middle of the beam (where the deflection is greatest) [14]. Longitudinal cracking can appear at the voids and at the web. These types of cracking can influence the load distribution for concentrated loads for slabs without concrete screed.

In addition, large deflections of the supporting beams cause rotation in the supporting/connection zone, leading to cracks in the concrete. These cracks can move upwards in the added flooring on the slabs, damaging the floor. For brittle floor finishes such as ceramic tiles, rotation and movement in the structure below leads to stresses in the floor and cracking the tiles. This topic is more described in chapter 2.2.4.

However, there are other parameters to consider when looking at reduction in shear capacity due to deflection of the supporting beam, and the decrease in shear strength is not entirely connected to the deflection in the supporting beam [14][12].

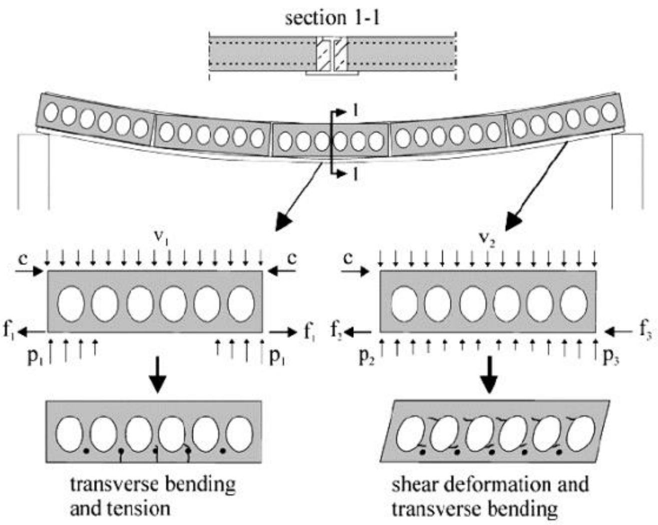

Figure 2-17 Stresses in hollow core slab on flexible support [10] .

Supporting structures have restrictions to how much deflection is allowed. The maximal allowable deflection of supporting structures is according to EC2 clause 7.4.1 (4):

> *The appearance and general utility of the structure could be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250.*

BB-C have their own guidelines for allowable deflections for supporting structures for HCS, and because of the reduction of the shear capacity in the HCS due to deflection, other limits are considered for HCS [2]:

- When the deformation of the beam is less than L/350, the shear capacity can assume to be intact.
- The shear capacity of the HCS can assume to be 50 % reduced if the deformation of the beam is l/150
- The value above can interpolate.

2.2.3.1. Recommended solutions to increase the shear capacity.

This chapter will undergo some of the solution for how to increase the strength of the cross-section of HCS near the support. Some of the solutions are from the BB-C, and others are from researchers. Mainly for these solutions, is to strengthen the shear capacity of the member in the uncracked zone, due to this being a weak point.

Concrete filled voids

Filling in the voids with concrete can reduce the chance of failure due to shear stresses, reason being it gives the HCS more shear strength in the uncracked zone, by increasing the amount of concrete in the cross-section. The uncracked zone is assumed to be the distance l_1 from the support (approximately 4xh to 5xh) [2]. Figure 9-3 in Appendix A shows the increase in shear strength when filling in the voids according to the BB-C. The increase in shear strength is showed by the equation:

$$\Delta V_{Rd,c} = \left(\frac{2}{3}\right) n b_c d f_{ctd}$$

Equation VI: Shear strength calculations for concrete filled voids

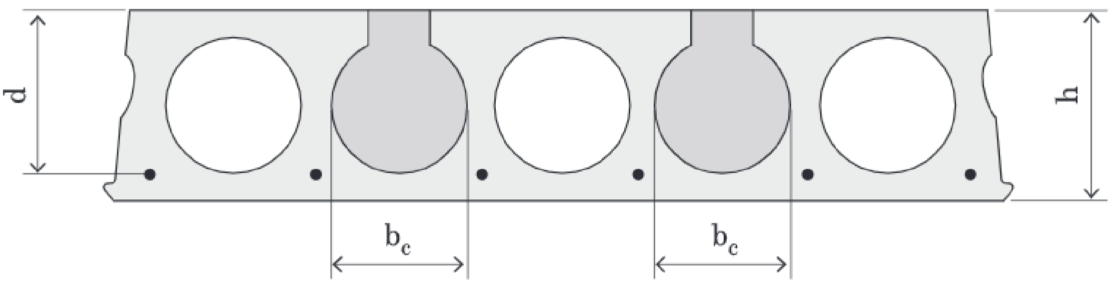

Figure 2-18 Cross-section of a hollow core slab [2].

According to some studies [18][19], it has been observed increased web-shear strength of core filled specimen. Palmer and Schultz commented in one case where it was only 50% as efficient as the rest of the cross section in resisting web shear [18]. Why this was the case, could be the loss of prestress due to anchorage loss for the strands if the HCS webs fail before the filled cores. The papers stated that it was unclear whether it applies to HCS generally or just in this case. Hegger observed no increase of the shear capacity for concrete filled voids for slim floor constructions, however it was indicated that shear deformation resistance may be developed [14].

Using fiber-reinforced concrete

Using steel fiber in concrete is a normal practice with many advantages, such as control cracking due to plastic and drying shrinkage, and making the concrete more ductile, leaving room for more displacement after cracking instead of going straight to failure.

Cuenca and Serna, Chao, Dudnik and Palmer and Schultz did experiments with adding steel fibers to the HCS [20][21][18][29]. Adding of steel fibers have shown to improve the shear capacity compared to HCS without steel fibers. In addition, it also increased the elements strength in flexural capacities. Palmer and Schultz observed fibers that were still engaged in a wide crack, suggesting the fibers has a big role in the strength mechanism of the HCS.

It was possible to produce fiber-reinforced concrete without technical problems [20]. Due to the impossibility of using transverse reinforcement in hollow core slabs, the usage of steel fibers is a clever way to strengthen the shear capacity. The increased tensile strength means greater web-shear strength, and the fiber reinforced specimens had the largest web-shear strength.

Some problems with this method are steel fibers poking out of the HCS. This means a levelling layer must always be applied to smoothen out the fibers. Also, the test specimen where wet cast, compared to normal HCS which is normally dry cast. This means adding of steel fibers would change the whole casting process of HCS casting distributers.

Figure 2-19 Steel fibers in hollow core slab [20].

Cross-section and material

Hollow core slabs come with different shapes for the voids, either circular or square-like. According to shear capacity calculations in BB-C, the higher b_w value, the higher the shear capacity. Circular voids have higher shear capacities compared to square-like voids, but it increases the HCS deadload.

The formula for shear resistance according to BB-C [2]:

$$V_{Rd,c} = 1{,}025 * b_w$$

Equation VII Shear calculations from BB-C

The formula is from EC2 section 6.2.2 (2) equation 6.4, but with following assumptions:

$\sigma_{cp} = 0$ and I/S ≈ 0,67*h for rectangular cross-section, $b_w = t_o + t_u$ and $f_{ctd} = 1{,}53MPa$ (for normal HCS with concrete strength class B45).

A study concluded with the circular core shape had ultimate strength greater than the square shape by 13,4%, and when increasing the compressive strength of the concrete from 38MPa to 48MPa increased the ultimate strength in the HCS by 23,6% [22]. From Equation VII it is seen that the shear capacity also relies on the concrete strength, meaning the shear strength also would get an increase by increasing the compressive strength.

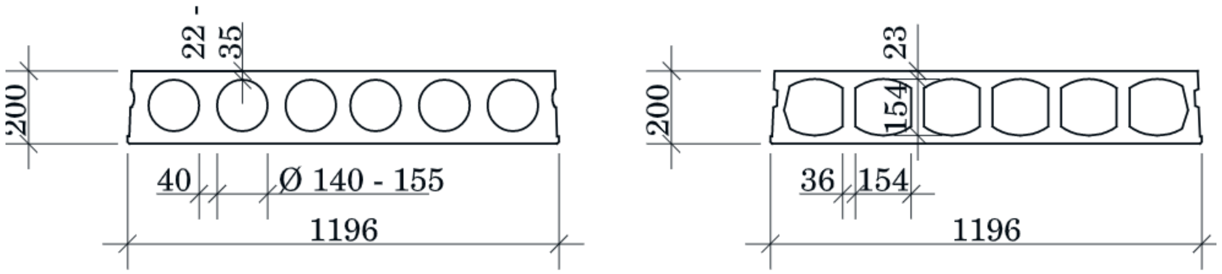

Figure 2-20 Different shape of cross-section in hollow core slab [2]

2.2.4 Damages caused by rotation in the support zone

Tiles are a commonly used flooring, but it comes with challenges regarding rotation and deformation of the structure. Ceramic tiles are a brittle material that cannot absorb rotation or much movement. This means measures needs to be done to ensure the floor is flowing almost freely from the structure movement, by building a multiple leveled floor or restricting the movement in the structure. The Masonry Catalogue Guidelines P14 [23] will be used as a guideline for limits in the structural movement to avoid cracking of tiles. Only this will be considered, as other guidelines regarding this topic were not found.

Figure 2-21 Cracking in tiles [23]

Figure 2-22 Picture from a local shopping mall, where cracking in tiles could be due to structural movement.

The deformation over a support zone causes a rotation, which results in a crack in the longitudinal joints between the hollow core slabs as in 2S, and this value are calculated using Equation VIII:

$$S = \theta(h_D + h_{NA})$$

Equation VIII: S-value

Where:

$\theta = 16d/5L$

d = expected deflection

L = span

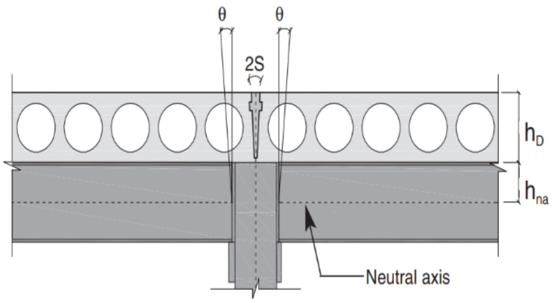

Figure 2-23 Element rotation at support [23]

It is recommended for the estimated total end rotation from 50% loading (imposed load) and deformation over long time should not exceed 2mm. This means a total deformation should stay within L/1200 for beam, except for inverted t-beams.

The masonry catalogue describes in three ways how the rotational movement can be absorbed [23]:

- Using movement joints
- Using concrete topping on the antifriction layer
- Combination of the two above

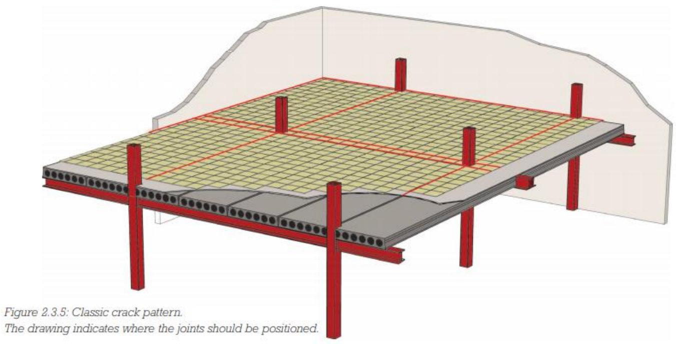

Figure 2-24 Cracking pattern in hollow core slab. The lines indicate where cracks most likely happen [23].

Suggested solutions to do in floor structure to avoid cracking

1 Different floor constructions

The Masonry catalogue [23] suggest different floor constructions to restrict the tiles from cracking. The choice depends on the individually need. The different floor constructions are listed:

- Tiles are fastened directly to the levelled floor. This is also called firm anchoring.
- Tiles on a base that is separated from the floor slabs with an antifriction layer and concrete topping, also called a floating floor.
- LFF-method. A floating construction with a thin antifriction layer.
- Tiles on elastic support (tension release mats).

Fasten the tiles directly to the levelled floor means that any deformation and rotation in the supporting structure will be transferred to the tiles. Movement joints are important to use, and placing them right to avoid unexpected cracking of the tiles.

Tiles separated from the floors with an antifriction layer ensures the base and tile layer can be able to move independent of one another. The antifriction layer is two layers of 0.15 mm plastic foil, or a combination of the foil and a fiber material.

An alternative method for floors with antifriction layers is the LFF-method (Thinly constructed floating floors). The layers in this constructure can take up to 90% of the shrinkage of the screed, and a resistant glass fibre net is laid in addition to the fibres with an overlap in the joints. This should ensure the floor to move freely during and after the curing period.

Tensile release mats prevent tension between the tiles and base. Spacers create a gap with the base and gives the grouting an opportunity to dry out, so that it is possible to start gluing the tiles before the concrete has reached its final humidity level.

2 Movement joints

Movement joint can prevent cracking of the tiles, leaving room for the tiles to move a little bit. Precast concrete with screed will have shrinkage up to 0,6-0,8% during the first month because of the drying of concrete. Stresses occurs between the tiles because of shrinkage, creep and temperature variation of the concrete.

Movement joints can be placed in edge joints, section joints, support joints and construction joints. The distance between movement joints can be calculated due to the shrinkage process and sudden temperature changes.

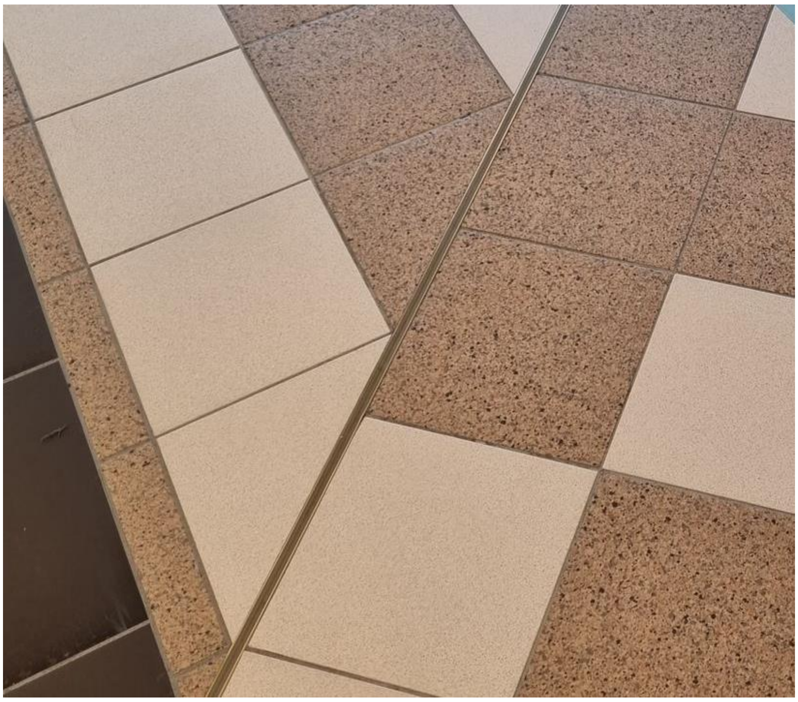

Figure 2-25 Movement joint in a local shopping mall.

2.3 Crack width

2.3.1 General

Cracks in concrete will occur due to applied loading and drying shrinkage and creep. Since concrete has low tensile strength and tensile capacity, reinforcement is added to control the tensile behavior and limit crack widths. One needs to calculate the crack width to prevent damaging the function and durability and giving the concrete an acceptable look.

Big crack widths can accelerate the reinforcement corrosion. In severe environments, such as a pool, there are strict requirements for the crack width to protect the reinforcement from the harsh environment [15].

Neglecting the effects of creep, shrinkage and cracking in concrete, the analysis can lead to miscalculations of crack widths, deflections, and support reactions. It is therefore important to consider these factors when dimensioning with reinforced concrete structures.

2.3.2 Crack width calculation

In Eurocode 2-1-1 clause 7.3.1 (9):

> *Crack widths may be calculated according to 7.3.4. A simplified alternative is to limit the bar size or spacing according to 7.3.3.*

Section 7.3.4 is a guide to direct calculation of the crack width, and clause 7.3.3 is crack width calculation without direct calculations.

EC2 gives a maximum value of crack width, w_{max}, from clause 7.3.1 (5) based on the exposure class of the concrete, as well as the reinforcement type. Allowable crack width is calculated according to table NA.7.1N in EC2.

Table 1 Allowable crack width

Exposure class	Reinforced members and prestressed members with unbonded tendons		Prestressed members with bonded tendons	
	Load combination	Limit value	Load combination	Limit value
X0	Quasi-permanent	0,4[1]	Quasi-permanent	0,3 k_c
XC1, XC2, XC3, XC4	Quasi-permanent	0,3 k_c	Quasi-permanent	0,2 k_c
			Quasi-permanent	0,2 k_c
XD1, XD2, XS1, XS2	Quasi-permanent	0,3 k_c	Quasi-permanent	Pressure relief[2]
XD3, XS3	Frequent load	0,3 k_c	Frequent load	Pressure relief[2]
XSA	Assessed separately		Assessed separately	

The crack width must be controlled for all reinforcement in both directions.

EC2 clause 7.3.4 shows the calculation of the crack width w_k. This crack width calculation is extensive, and a lot of assumptions and simplifications needs to be done. Following equations are used to calculate the predicted crack width:

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm})$$

Equation IX: Crack width calculation

$$s_{r,max} = k_3 c + k_1 k_2 k_4 \varphi / \rho_{p,eff}$$

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \frac{f_{ct,eff}}{\rho_{p,eff}}(1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0,6 \frac{\sigma_s}{E_s}$$

$$\rho_{p,eff} = \frac{A_s + \xi_1^2 * A_p'}{A_{c,eff}}$$

$$\xi_1 = \sqrt{\xi \frac{\varphi_s}{\varphi_p}}$$

$$\alpha_e = \frac{E_s}{E_{cm}}$$

W_k	crack width
$S_{r,max}$	is the maximum crack spacing.
ε_{cm}	is the mean strain in the concrete between cracks.
ε_{sm}	is the mean strain in the reinforcement under the relevant combination of loads,

including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered.

σ_s	is the stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by $\Delta\sigma_p$ the stress variation in prestressing tendons from the state of zero strain of the concrete at the same level.
K_t	is a factor dependent on the duration of the load. $K_t = 0.6$ for short term loading. $K_t = 0.4$ for long term loading.
$F_{ct,eff}$	is the mean value of the tensile strength of the concrete at the time cracking is expected to happen.
φ	is the bar diameter.
C	is the cover to the longitudinal reinforcement.
K_1	is a coefficient which takes account of the bond properties of the bonded reinforcement. = 0.8 for high bond bars = 1.6 for bars with an effectively plain surface (e.g. prestressing tendons)
K_2	is a coefficient which takes account of the distribution of strain: = 0.5 for bending = 1.0 for pure tension
K_3	3.4
K_4	0.425
E_s	is the design value of modulus of elasticity of reinforcing steel
E_{cm}	is the secant modulus of elasticity of concrete.
φ_s	is the biggest bar diameter.
φ_p	is the bar diameter or equivalent diameter of the bar.
ξ	is the ratio of bond strength of prestressing and reinforcing steel

3 Materials

3.1 Concrete

3.1.1 General

Concrete is a material commonly used for construction work in buildings due to its strength, how easy it is to create and form into shapes and sizes. If correctly designed, the structure can face earthquakes, hurricanes, and tornadoes [24]. Concrete is made from cement, aggregates (sand and gravel), water, and additives. The mixing proportions between these can determine a certain strength class and workability of the concrete.

The compressive strength is one of the most important quality criteria for concrete. Properties of concrete are often expressed in compressive strength in codes and design rules. Rules for material composition have therefore also mainly been related to strength. The definition of strength is given by force divided by area, on a 100 mm x 100 mm cylinder: $f_{ck} = F/A$. For $f_{ck} = 35\ N/mm^2$ the strength class is B35 [25].

Concrete is strong in compression but weak in tension. To strengthen the concrete in tension, reinforcement is added, which as a high tensile strength. The adding of reinforcement also helps controlling the crack width in the concrete.

Hollow core slabs are usually strength class B45, and the concrete for filling voids are usually B25 – B35.

3.1.2 Creep and shrinkage

The creep and shrinkage of concrete depends on the ambient humidity, the dimension, and the composition of the concrete. Following is explanation of these two phenomena, and how to calculate them according to EC2-1-1.

Concrete influenced by compression over a period will keep deforming additional to the momentaneous deformation when the load is applied. This additional deformation is called creep [28]. Creep is dependent on the magnitude of the load, and the time the load is applied.

Creep can be defined as:

$$\varepsilon_{cc}(t, t_0) = \varphi(t, t_0) * \frac{\sigma_c}{E_c}$$

Equation X: Calculation of creep

Where:

t	is the age of concrete in days.
T_0	is the concrete age when load is applied.
$\varphi(t, t_0)$	is the creep coefficient.
E_c	is the tangent E-module of the concrete.

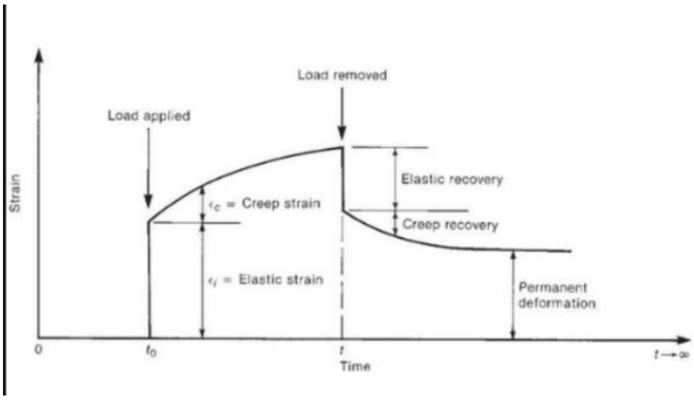

Figure 3-1 Creep in concrete

The creep coefficient can be calculated using this formula from EC2-1-1 Annex B – Creep and shrinkage strain:

$$\varphi(t, t_0) = \varphi_0 * \beta_c(t, t_0)$$

Equation XI: Creep coefficient

$$\varphi_0 = \varphi_{RH} * \beta(f_{cm}) * \beta(t_0)$$

$$\varphi_{RH} = \left[1 + \frac{1-\frac{RH}{100}}{0.1*\sqrt[3]{h_0}}\right] \text{ for } f_{cm} \leq 35\text{MPa}$$

$$\varphi_{RH} = \left[1 + \frac{1-\frac{RH}{100}}{0.1*\sqrt[3]{h_0}} * \alpha_1\right] * \alpha_2 \text{ for } f_{cm} > 35 \text{ MPa}$$

$$\beta(f_{cm}) = \frac{16.8}{\sqrt{f_{cm}}}$$

$$\beta(t_0) = \frac{1}{(0.1+t_0^{0.2})}$$

$$h_0 = \frac{2A_c}{u}$$

$$\beta_c(t, t_0) = \left[\frac{(t-t_0)}{\beta_H+t-t_0}\right]^{0.3}$$

$\beta_H = 1.5[1 + (0.012RH)^{18}]h_0 + 250 \leq 1500$ for $f_{cm} \leq 35$

$\beta_H = 1.5[1 + (0.012RH)^{18}]h_0 + 250\alpha_3 \leq 1500\alpha_3$ for $f_{cm} > 35$

$$\alpha_1 = \left[\frac{35}{f_{cm}}\right]^{0.7} \quad \alpha_2 = \left[\frac{35}{f_{cm}}\right]^{0.2} \quad \alpha_3 = \left[\frac{35}{f_{cm}}\right]^{0.5}$$

φ_{RH}	is the notional creep coefficient.
RH	is the relative humidity of the ambient environment in %.
F_{cm}	is the mean compressive strength of concrete in MPa at the age of 28 days.

$\beta(t_0)$	is a factor to allow for the effect of concrete age at loading on the notional creep coefficient.
H_0	is the notional size of the member in mm.
A_c	is the cross-sectional area.
U	is the perimeter of the member in contact with the atmosphere.
$\beta_c(t, t_0)$	is a coefficient to describe the development of creep with time after loading.
T	is the age of concrete in days at the moment considered.
T_0	is the age of concrete at loading in days.
$t-t_0$	is the non-adjusted duration of loading in days.
β_H	is a coefficient depending on the relative humidity (RH in %) and the notional member size (h_0 in mm).
$\alpha_{1,2,3}$	are coefficients to consider the influence of the concrete strength.

Shrinkage is a result of drying of the concrete, and is independent of the loading, unlike creep. Shrinkage consists of two contributions: drying shrinkage ε_{cd} and autogenous shrinkage ε_{ca}. [25]. Drying shrinkage is a result of moisture transport through hardened concrete and is developing slowly, while autogenous shrinkage develops with the concretes strength development. EC-1-1 clause 3.1.4(5) describes how to calculate the total shrinkage strain with the following formula:

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca}$$

Equation XII: Total shrinkage strain

EC2-1-1 describes in Annex B.2 how to calculate the drying shrinkage:

$$\varepsilon_{cs,0} = 0{,}85\left[(220 + 110\alpha_{ds1}) * exp\left(-\alpha_{ds2} * \frac{f_{cm}}{f_{cmo}}\right)\right] * 10^{-6} * \beta_{RH}$$

Equation XIII Drying shrinkage

ε_{cs}	is the total shrinkage strain.
ε_{cd}	is the drying shrinkage strain.
ε_{ca}	is the autogeneous shrinkage strain.
F_{cm}	is the mean compressive strength
F_{cmo}	=10MPa
α_{ds1}	is a coefficient which depends on the type of cement.
	= 3 for cement Class S
	= 4 for cement Class N
	= 6 for cement Class R
α_{ds1}	is a coefficient which depends on the type of cement
	= 0.13 for cement Class S
	= 0.12 for cement Class N
	= 0.11 for cement Class R
RH	is the ambient relative humidity (%)
RH_0	= 100%

3.1.3 Durability and cover

The durability of concrete is taken care of in the standards by defining requirements to exposure classes, durability classes, a combination of exposure classes and durability classes, chloride classes, classes for concrete cover, structural design and detailing, execution of the work, control, supervision and inspection, and special measures (stainless steel, coating) [25]

To meet certain criteria of the EC2, the concrete should meet the dimensional service life, as well as be able to face the environment it is placed, to handle such as humidity, frost, chloride etc. To manage this, the concrete is constructed after the exposure classes from EC2 Table 4.1. The exposure classes can be split into 6 groups, and Table 2 describes 2 of them:

Table 2: Exposure classes [15]

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk or corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freezing/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal: very dry.	Concrete inside buildings with very low air humidity.
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity. Concrete permanently submerged in water.
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact. Many foundations.
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity. External concrete sheltered from rain.
XC4	Cyclic wet and dry	Concrete surfaces subjected to water contact, not within exposure class XC2.

A minimum concrete cover shall be provided to ensure safe transmission of bond forces, an adequate fire resistance and protection of the steel against corrosion. The latter being the most important factor. The more aggressive the environment, the more cover needed. The cover is dependent on the national measures, exposure classes and the diameter of the reinforcement. The cover is calculated using equation 4.1 in EC2 clause 4.4.1.1:

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

Equation XIV: C_{nom} calculation

Where $\Delta c_{dev} = 10mm$ according to NA.4.4.1.3 (1).

$$c_{min} = \max\,(c_{min,b}; c_{min,dur}; 10)$$

$$c_{min,b} = \max\,(\emptyset_s; 10)$$

$C_{min,dur}$ is obtained from table NA.4.4N in Eurocode 2.

Table 3: $C_{min,dur}$ [15]

Exposure class	Durability class	$c_{min,dur}$ (in millimeter)	
		50 years design life	100 years design life
X0	M90	$c_{min,b}$	$c_{min,b}$
XC1	M60	15	25
XC2, XC3, XC4	M60	25	35
XD1, XS1	M45	40	50
XD2, XD3, XS2	M40	40	50
XS3	M40	50	60

For exposure class XC1 with reinforcement Ø12:

$c_{min,b} = \max(\emptyset_s; 10) = 12mm$

$c_{min} = \max\big(c_{min,b}; c_{min,dur}; 10\big) = 15mm$

$c_{nom} = c_{min} + \Delta c_{dev} = 15mm + 10mm = 25mm$

The reinforcement should have a cover of 25mm in concrete inside buildings.

3.2 Reinforcement

Concrete is strong in compression, but weak in tension. Adding reinforcement strengthens the concrete in tension, making a more ductile material capable of deformation, and getting smaller crack width.

The reinforcement should have a sufficient ductility, which is defined as the ratio between the tensile strength and yield strength. The characteristic yield strength of the reinforcement f_{yk} is 500MPa. According to Eurocode 2, a dimension yield strength should be used, and is found with this equation:

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

Equation XV: Dimension yield strength

Where γ_s is the material factor 1.15. This gives us the dimensioning yield strength:

$$f_{yd} = \frac{500MPa}{1.15} = 434{,}8MPa$$

However, as mentioned in Chapter 2.1.1.3, reinforcement used in the voids and joints of HCS, should have the dimensionin yield strength limited to 2/3:

$$f_{yd} = 291MPa$$

4 Structure analysis modeling

4.1 FE-analysis using FEM-design.

FEM-design is an advanced modeling software for finite element analysis and design of load-bearing concrete, steel, timber, and foundation structures according to Eurocode with NA. The structural model is easily created in 3D with intuitive CAD-tools or imported from BIM-software. Results are shown in a variety of 3D-graphs, contour lines, color palettes or sections. It is a very popular calculation program used by engineers [26].

4.2 Modeling in FEM-design

The modelling process are shown in the flow chart below. For a more detailed review of the modelling process, as well as the dimensions for used beams and hollow core slabs, see Appendix D.

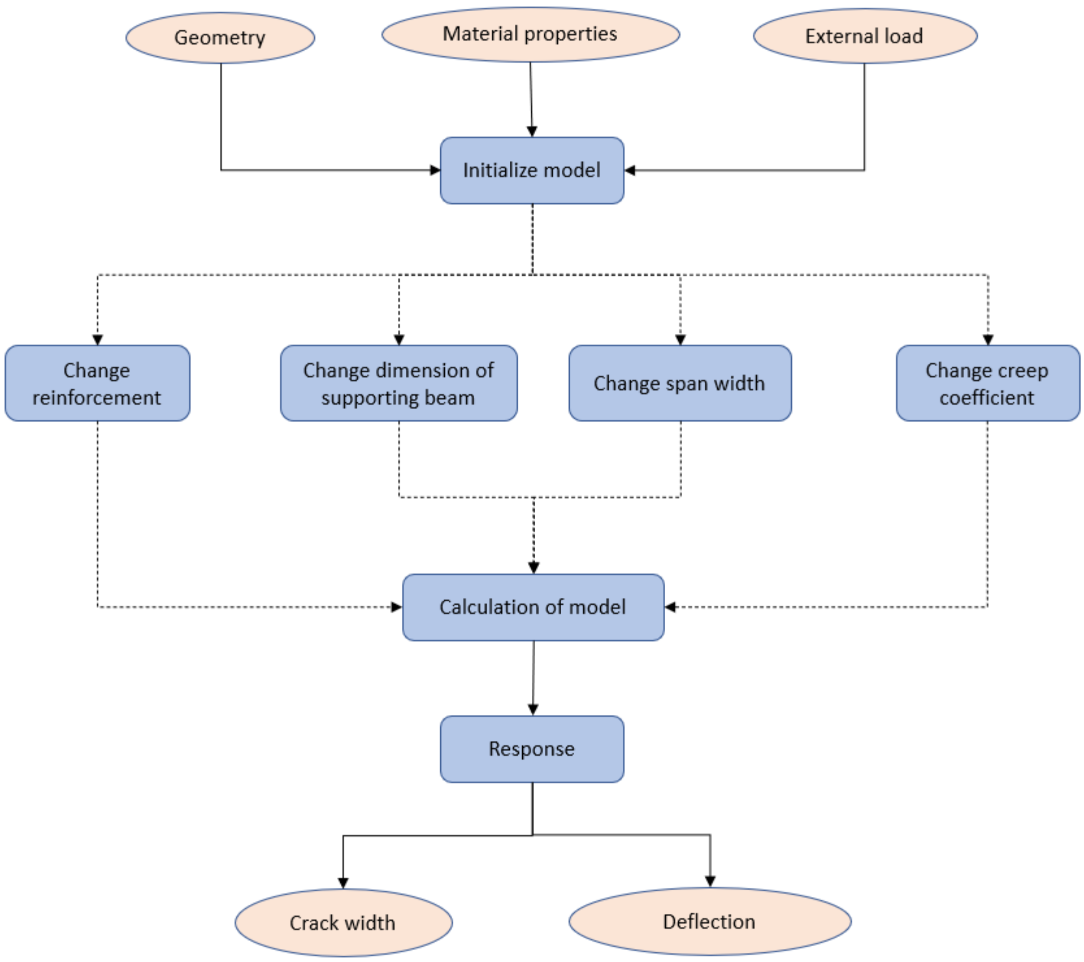

Figure 4-1 Flow chart of modelling process in FEM-design

4.3 Mesh

Meshing is applied to the shells in FEM-design by using a generating tool. This will define the minimum division numbers and the average element sizes to generate a balanced mesh.
The phases of the mesh generator are defining the vertices of the elements, creating a triangular mesh using the vertices, converting the triangle mesh to mixed quadrate-triangle mesh, optimizing the coordinates of the nodes in the mesh (smoothing) and setting the middle points of the element sides [27].

Peak smoothing is used over the columns to avoid singularity problems. Over columns there will be certain places that get infinite inner force according to the theory. Smoothing over the columns will then produce a more reliable result when it comes to crack width calculations.

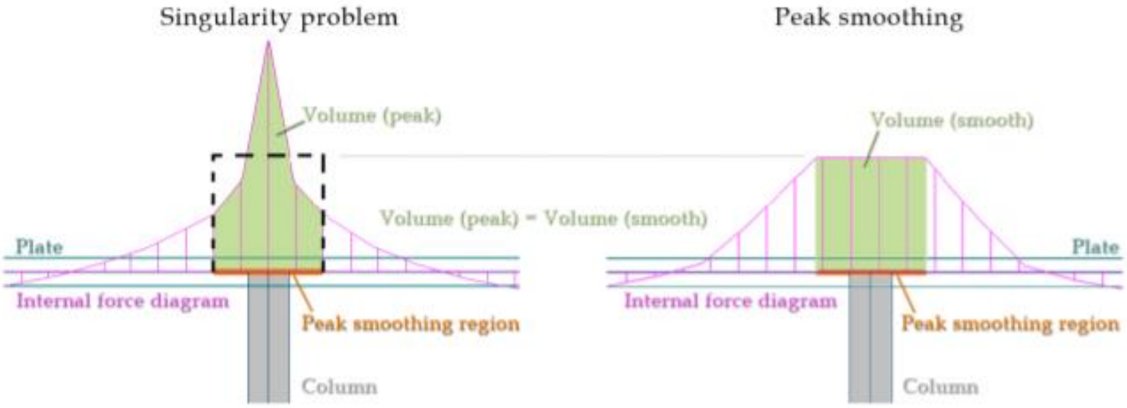

Figure 4-2 Peak smoothing [26]

5 Analysis of the model

This chapter is divided in two parts – first there will be hand calculations done by the author to both check if the results obtained from the calculation model are reliable, as well as calculations of factors like allowable crack width and creep coefficient. Second, there will be analysis of the results from FEM-design, of measures done to reduce cracking in the connection, as well as measures to decrease rotation.

5.1 Pre-calculations

It is important to check the model with a few hand-calculations, to be sure the model can be trusted and is modelled correctly. Creep coefficient are also calculated to insert the right parameters in the model. To check the obtained crack widths in the FEM-design model, it must be compared to the allowable crack width. The allowable crack width is also calculated.

5.1.1 Deflection of supporting steel beam

To see if the model can be trusted, a hand calculation is done to compare the value given in FEM-design. The middle support beam is chosen, dimension D32-400 with 1.5kN/m^2 surface load. The load combination is shown in Appendix D, and the hand calculation is shown in Appendix B.

Table 4 sums up the results. The hand calculated value and the value from FEM-design, differ 0.629 mm. This is considered acceptable, and the model can be trusted.

Table 4: Deflection of steel beam

Deformation [mm]		**Error (mm)**
Hand calculation	FEM-design	
6,517	5,888	0.629

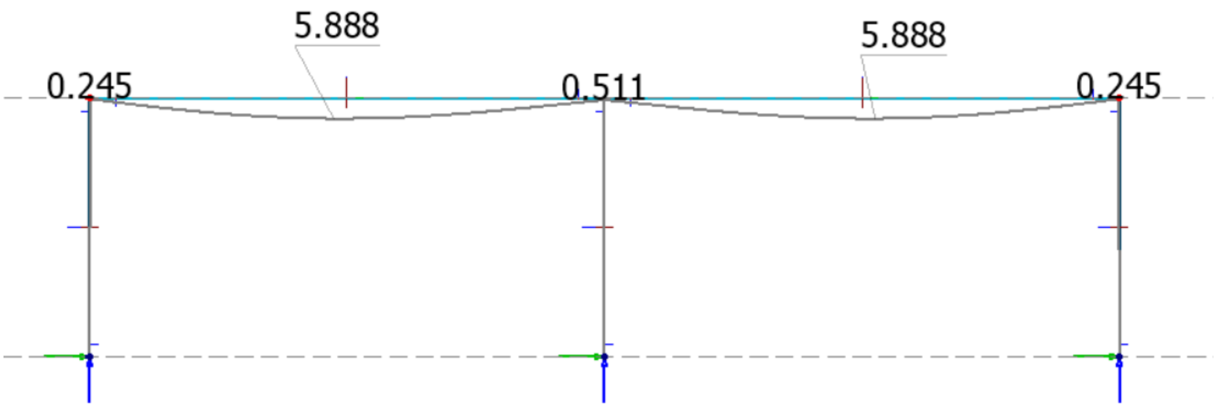

Figure 5-1 Deflection of supporting beam in FEM-design.

5.1.2 Allowable crack width

To analyze the crack width in the rotation zone, a maximal value of crack width must be calculated beforehand. The maximal value is from EC2-1-1 Table NA.7.1N (see more about allowable crack width in chapter 2.3 - Crack width.)

For concrete exposure class XC1, which is concrete in building with low humidity, the allowable crack width is calculated from EC2 clause NA.7.3.1:

$$k_c = \frac{c_{nom}}{c_{min,dur}} \leq 1{,}3$$

$$k_c = \frac{35}{15} = 2{,}33$$

$$w_k = 0.3 * 1.3 = 0.39$$

The value for c_{nom} is calculated in chapter 3.1.3.

The maximal crack width is 0.39 mm. In the graphs in chapter 5.2, this is denoted by a red, dotted line in the results graphs.

5.1.3 Creep coefficient

The creep coefficient effects the crack width, meaning calculating the value for a creep coefficient that are as precise as possible, can make crack width calculations more predictable and precis. Creep coefficient was calculated by hand, and since the calculations are very extensive, they are found in Appendix C.

FEM-design has a function that automatically calculates the creep coefficient, where the user can edit the input themselves. The table below presents the results from hand calculation, and what FEM-design suggests without editing the input in FEM-design calculations:

Table 5: Comparing creep coefficient

Creep coefficient	Hand-calculations	FEM-design
$\boldsymbol{\varphi(t, t_0)}$	2.072	2.081

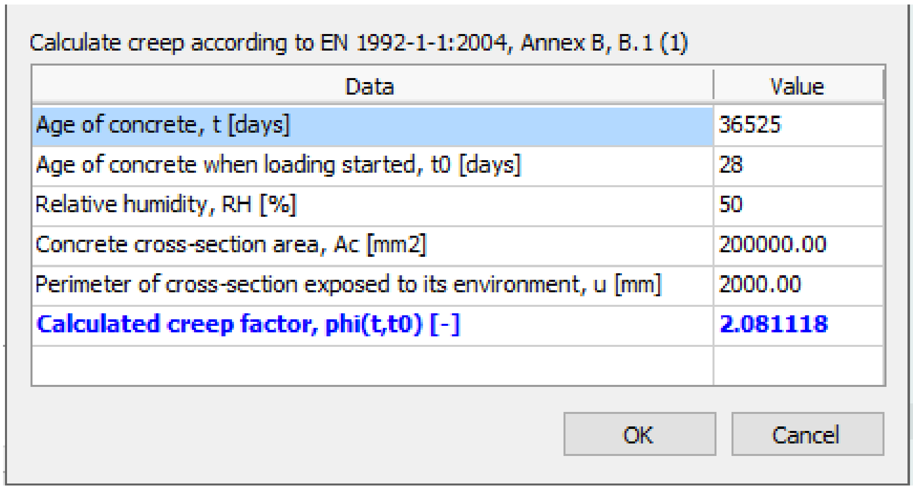

Figure 5-2 Creep coefficient data in FEM-design

There is a small difference in the values. When checking the input data in FEM-design, it was observed that the value chosen for t (which is age of concrete in days) was set to 100 years. When dimensioning a building, it is most commonly to use 50 years of design life, which is the value the author used in the creep coefficient calculations. By editing from 50 year of design life to 100 years, the same creep coefficient as the FEM-design was obtained.

5.2 Results

This chapter will present the results obtained from FEM-design. This is divided into two parts – first there are calculations done to see different measures to reduce the crack width in the connection zone. Second there are calculations to see which measures decrease the rotation in the support by a satisfying amount. The results are explained and discussed in each part chapter, and the final discussion and conclusion will be found in Chapter 6.

As mentioned, the modelling process is showed in Appendix D.

To get a reliable result as possible when it comes to crack width, the seam reinforcement is added to the model. Since this thesis does not calculate required amount of seam reinforcement, a suggestion from BB-C on regular amount of seam reinforcement is used, see Appendix E.

5.2.1 Analysis of measures to reduce cracking in connection zone.

Two ways to reduce the crack width in theory that is examined here, is adding of reinforcement and changing creep coefficient. Different reinforcements are added in the connection where crack occurs to see the affect. Changing the creep constant is also look at, to see how much this affects the crack width calculations.

5.2.1.1 Adding extra reinforcement

Figure 2-24 shows where crack may appear, and in the calculations, two different size reinforcement, Ø12 and Ø16, are added in the longitudinal crack developing from the column, to see the effect on the crack width. When modelled in FEM-design, an anchorage length of 50 x Ø is added to each side of the bar. This is a standard anchorage length for reinforcing steel.

Figure 5-4 illustrates the crack width in connections in HCS resting on a D32-400 steel beam under different loading. The effect of adding an extra Ø12, or Ø16, in the joints where cracks may appear, is shown in the figure. There is a reduction of 10,5% in crack width after load is applied with the extra Ø12 reinforcement, and for the extra Ø16 there is a reduction of 17,5%.

Figure 5-3 illustrates crack width in connection in HCS resting on a concrete inverted T-beam (600x500). This example has a higher crack width compared to Figure 5-6, due to larger deflection causing more tension in the concrete in the connection. Adding an extra Ø12 bar has shown to reduce the crack width by an average of 13,6% and adding an extra Ø16 bar reduce the crack with by an average of 21,9%.

Due to little spacing in the joints for extra reinforcement, the Ø12 might be the best option in constructional matter. Calculations have shown that there is a positive effect adding reinforcement in the joints, and adding of extra reinforcement in the connections could be a measure for structures that needs to prevent the cracking in connections.

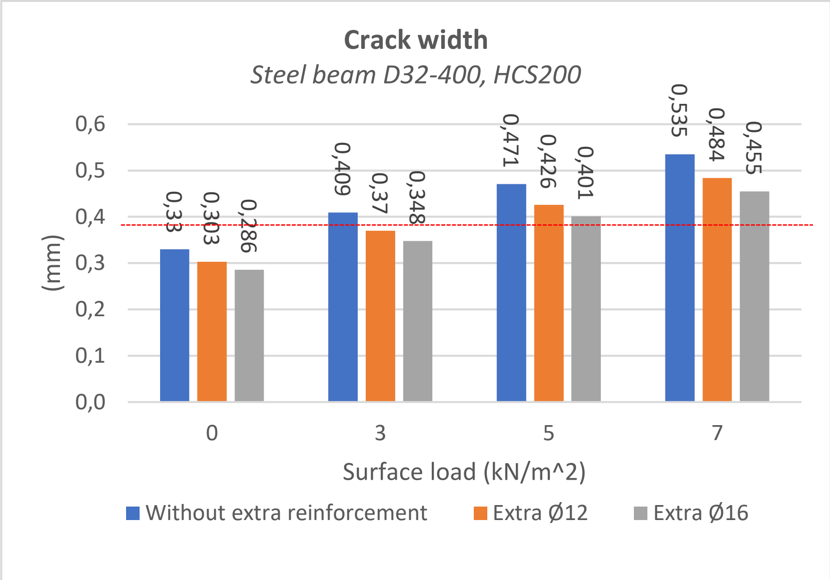

Figure 5-4 Crack width for steel beam D32-400, HCS200

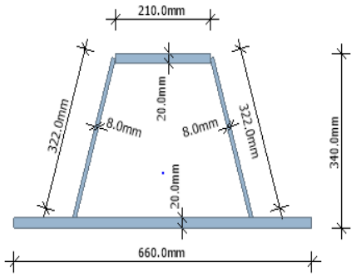

Figure 5-3 Steel beam D32-400

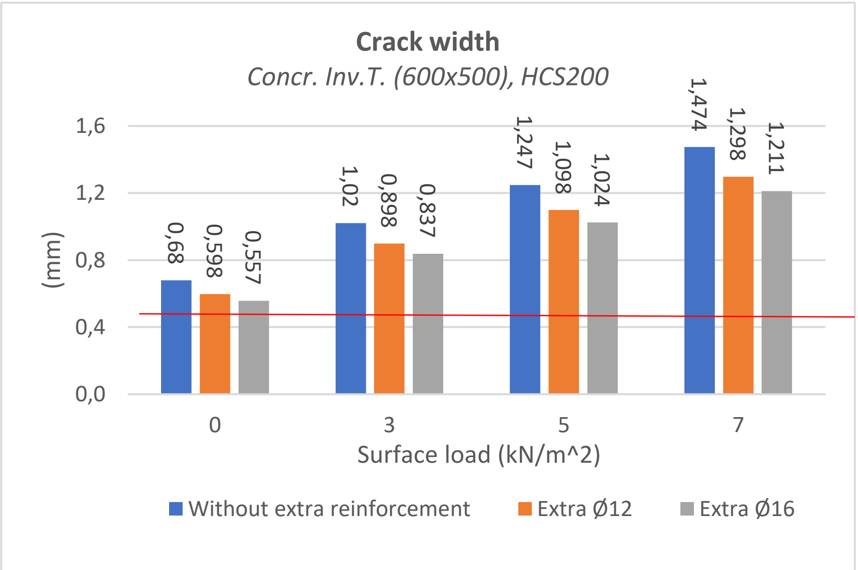

Figure 5-5 Crack width for concr.inc.T (600x500),, HCS200

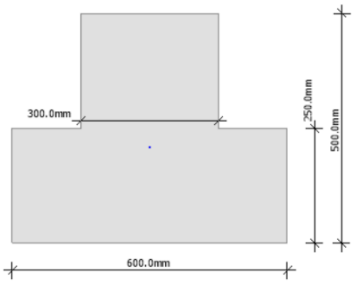

Figure 5-6 Concrete inverted T-beam (600x500)

5.2.1.2 Changing the creep coefficient

The creep of concrete is dependent on the humidity and temperature of the environment, dimension of the concrete and its composition. This means the creep coefficient can be determined in advance to a certain level. The calculations for looking at the effect for changing the creep coefficient with the crack within the connection, a HCS 200 on a D32-400 supporting steel beam is investigated. The results are presented in Figure 5-7.

It is seen a decrease in the crack width as the creep coefficient increases. To get a higher creep coefficient, the strength class of the concrete and the relative humidity of the surroundings could be increased, and the concrete age before loading should not exceed 28 days.
The observed decrease in the crack width were relatively small, meaning measures done to get a higher creep coefficient may be too costly for the very small change that was found here. However, being consistent with the concrete strength class, the humidity of the surrounding and time applied before loading show its importance. If some of these factors are considered in the calculations, but not done in practice, larger crack width compared to the calculated ones will appear.

It was also observed that the S-value increased by a small percentage when increasing the creep coefficient. This is correlated to the small increase in deflection that occurred. However, the increase was so small, it is assumed that it is not important to consider. For example, 3 kN/m^2 loading and changing the creep coefficient from 2 to 2,5, increased the S-value 0.08mm.

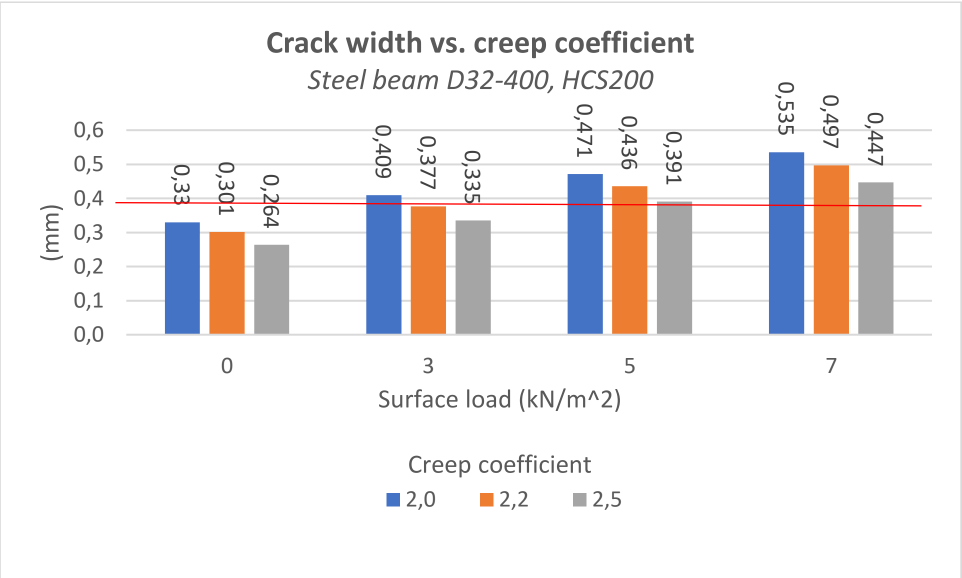

Figure 5-7 Crack width calculations with different creep coefficient

5.2.2 Analysis of measures to decrease rotation

As previously mentioned, rotation in the connection can cause damage to the hollow core slab, in addition causing cracks in the connection, that can damage the flooring. When using brittle flooring material, such as tiles, the rotation should be limited.

When comparing different measures to decrease the rotation, calculated S-values are compared. According to Masonry catalogue, the estimated total end rotation from 50% imposed loading and deformation over long time, should not exceed 2mm when using tiled concrete flooring. This means, other than inverted T-beams, a guideline for a maximum deflection of L/1200 is suggested (see Chapter 2.2.4). The following equations are used:

$$S = \theta(h_D + h_{NA})$$

$$2S \leq 2mm$$

$$\theta = \frac{16d}{5L}$$

Where d is the deflection calculated in FEM-design, when using 50% imposed load, and L is the span length of the supporting beam. The limit of 2mm is illustrated in the graphs by a red line.

5.2.3.1 Continuous beam vs. hinged beam

Using a continuous beam instead of hinged will decrease midspan the deflection, leading to a bending moment force in the support. The effect of using continuous beam to hinged beam are compared in this chapter.

Figure 5-8, Figure 5-9, and Figure 5-10 shows how the S-value changes for switching a hinged beam to a continuous steel beam. There is an average of 66% increase in the S-value for the D32-400 steel beam, 64% increase for the D37-500 and 69,9% increase for the concrete beam. When using a brittle flooring material, using continuous flexural support beams will reduce the rotation in the support zone a lot, making it less likely to get large cracks.

However, when analyzing the S-values in this case, it was observed that when the S-value reached 2mm, only a deflection of approximately L/1350 was measured in the beam. This means the guideline for keeping the deflections in the supporting beams to a L/1200 does not suit the S-value (of 50% loading long term deflection) being limited to 2mm. It is suggested to use stiffer beams if the deflection is too high, but finding a beam that gets the allowable deflection to less than L/1350, means using a beam that is way too over dimensioned.

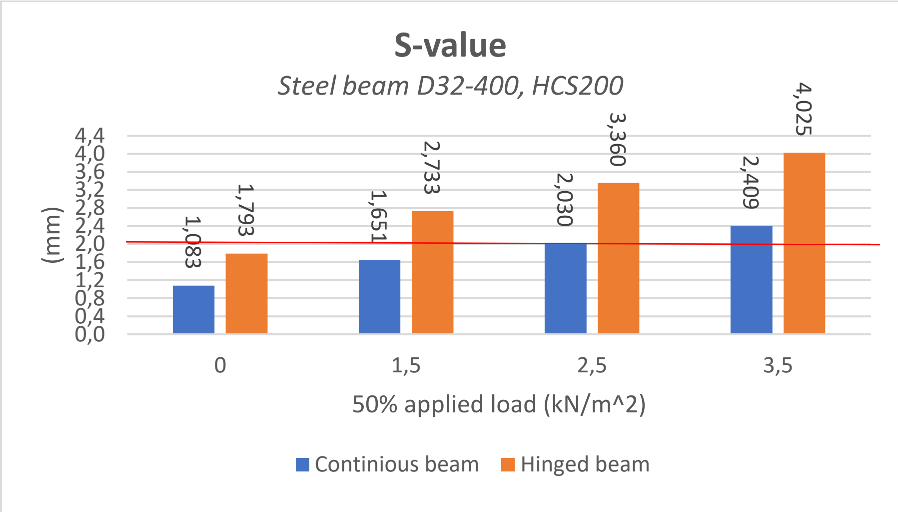

Figure 5-8 S-value for steel beam D32-400

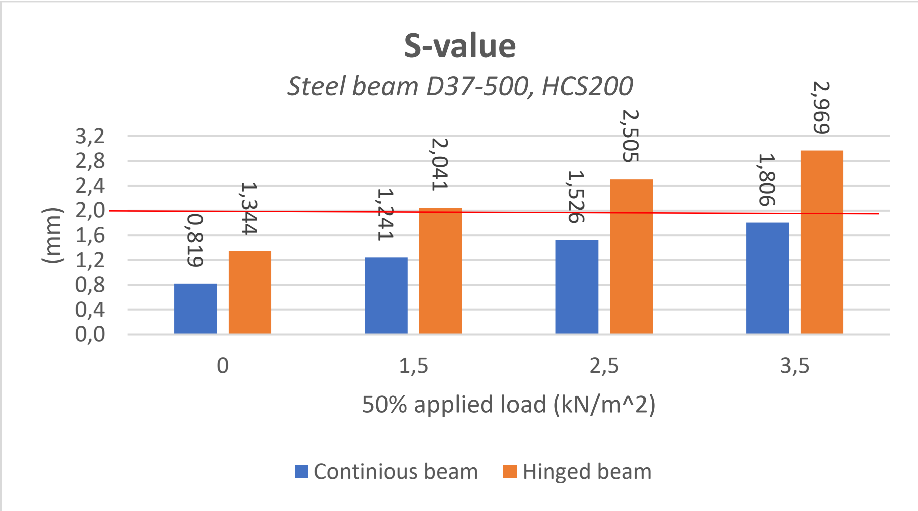

Figure 5-9 S-value for steel beam D37-500

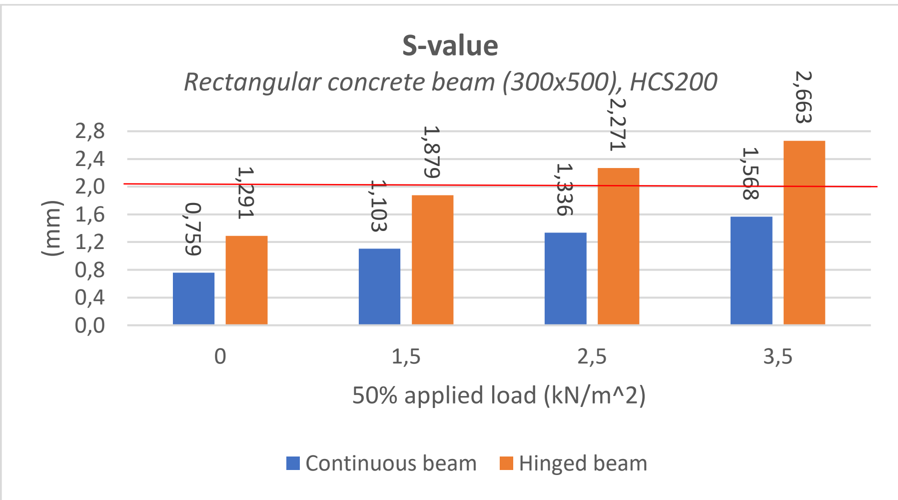

Figure 5-10 S-value for rectangular concrete beam (300x500)

5.2.3.2 Different dimension of hollow core slab

A different size of the hollow core slab is used, where the original HCS 200 is switched with a HCS 265. The calculated S-value is done for different supporting beams: Steel beams D40-500 and D50-60, and concrete beams 300x500mm and 300x800mm. The results are presented in Figure 5-11.

The S-value increases considerably when the dead load of the HCS increases. As seen in Figure 5-11, the S-values are over 2mm for almost every beam before adding load to the structure. The deflections of the beams were maximum L/1140, except for the concrete beam 300x500, which had a deflection of approximately L/540. This is without any loading.

The reason for why the S-value increases remarkably, is due to the new value of h_D generating a higher S-value. Also, when using a stiffer beam, the h_{NA} naturally increases, which also increase the S-value. It might seem the S-value limitation is not a good approach when it comes to dimensioning the supporting beams for HCS when the HCS are over 200mm thick. Using D steel beams instead of rectangular concrete beams showed best result.

The reason for not including the DLB in these calculations, are that the limit of L/1200 for the beams did not apply to the inverted T-beams.

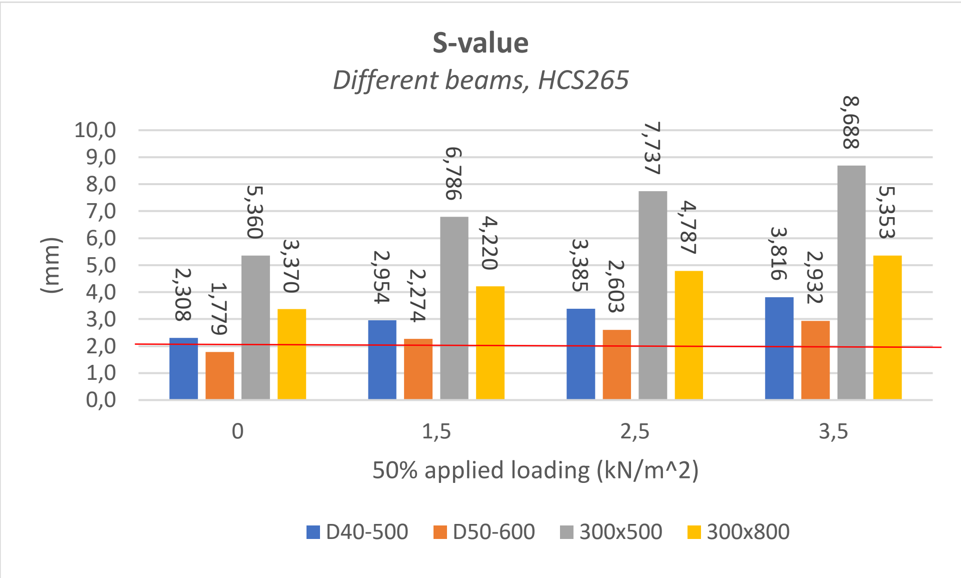

Figure 5-11 S-value for different beams with a HCS 265

5.2.3.3 Changing the span length

Another solution to decrease the rotation, is changing the span length. Following calculations will show how large effect reducing the span length will have on the deflection. Since the HCS 265 gave not satisfying results for the S-value, a decrease in the span length of the beam will be done here with a D40-500 beam to see if the S-value goes below 2mm. The results are presented in Figure 5-12.

A decrease of an average 24,7% decrease was found in the S-value. As seen in Figure 5-11, the S-value are still above 2mm, meaning the rotation is still too large.

Nonetheless, reducing the span length 9% (0,5 meters) gave an average reduction in S-value of 24,7%. This shows that a small change in the span length reduces the deflection of the supporting beam 2.7 times more in this case, decreasing the rotation in the support.

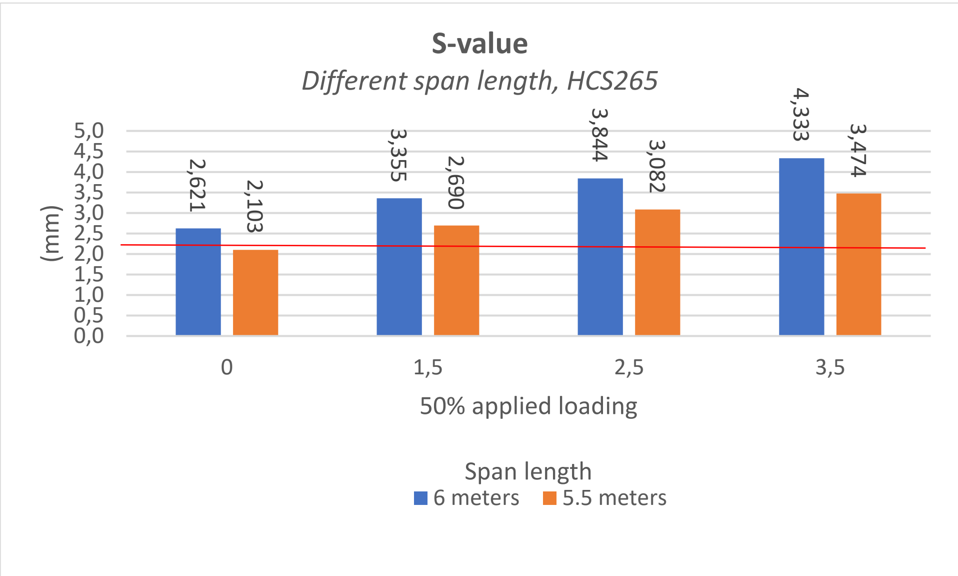

Figure 5-12 S-value for different span length

6 Discussion and Conclusion

6.1 Discussion

This thesis has investigated multiple studies carried out by other researchers, regarding the damages in hollow core floor systems when the supporting beam deflects and suggested solutions to control damage.

Studies have found that there is a reduction in the shear capacity in the HCS when it is supported on flexural support. This has shown that there is a need for a reduction factor in the shear capacity calculations, and a reduction factor between 0.75 and 0.8 have been observed by other researchers to be sufficient. Other researchers have emphasized that the reduction in the shear capacity do not solely dependent on the deformation of the supporting beam, but also other factors such as geometry of the HCS, overall unit depth, concrete compressive strength, the composition of the concrete and more. Nonetheless, deformation in the supporting beam has shown to increase web stress in the HCS, leading to shear tension failure.

There are some measures to increase the shear capacity to the HCS. Studies carried out by other researchers have found that concrete filled voids, fiber reinforced HCS and HCS with a higher compression strength all could increase the shear strength. Using a higher compression strength might not be the most suitable solution, due to the HCS normally having a high concrete class (B45), and the shear strength of concrete has shown only to be as strong as the cross-section’s weakest point. This means that if the distribution of the concrete in the HCS is not optimal, the shear strength will reduce. Filling in the voids had conflicting results from researchers, where some noticed an increased shear capacity, while others didn’t. Studies regarding the use of steel fibers in the HCS have shown promising results, with both increase in the shear strength and flexural strength of the HCS. It also showed to increase the ductility of the HCS, controlling the cracks.

Deflection of the support leads to rotation in the support zone, which can cause cracking in the connections. When using a brittle flooring material, rotation, movement and cracking generally needs to be restricted in a structure.
Calculations showed the effect of measures to decrease the cracking. Adding reinforcement bars of Ø12 and Ø16 in the connection where cracking could occur, could reduce the cracks between 10%- 21,9%. Due to the small spaces in the connections between HCS, Ø12 might

produce the best results. Using Ø12 have shown to have a positive effect, meaning the adding of reinforcement in the connections where cracks might happen, is ideal for structures with limited possibility for cracking.

Changing the creep coefficient affected the crack width by a very little percentage. Increasing the creep coefficient showed decrease in the crack width. To increase the creep coefficient, one needs to use a higher strength class of the concrete, have low ambient humidity of the surrounding and loads of the structure should be applied within 28 days after casting. These are all measures that can be done to prevent increase in crack width due to a low creep coefficient.

The Masonry catalogue describes measures and guidelines to follow when constructing a building with HCS on flexural support to minimize the cracking in the flooring. The calculated S-value should not exceed 2mm for 50% added load, which means a restriction for L/1200 for the supporting beams. Calculations were done to compare different measures to restrain the movement. It was found that increasing the dimensions of the supporting beam had a great influence, as well as decreasing the span length to minimize the deflection and using thin HCS sections to avoid large dead load.
However, when comparing the results to the S-value that should not exceed 2 mm, it was found that the deflection of the supporting beam could be as small as L/1350 when the S-value was 2 mm. This means a stiffer beam need to be used, and there is a danger for over dimensioning. It is suggested to use continuous beam and smaller span lengths to decrease the rotation and cracking in building with brittle flooring material.

Adding steel fibers in the concrete in the connection may be a possible solution to control cracking, due to the positive results from the HCS. Steel fibers have proven to make the concrete more ductile, meaning making the connection more applicable for hollow core slab flooring systems.

6.2 Conclusion

As mentioned in the discussion, steel fiber hollow core slabs show promising result when it comes to controlling the damages resulting from deflection in the supporting beam. Since the HCS does not have shear reinforcement, the steel fibers could act as shear reinforcement, being a good measure to prevent premature shear tension failure.

When using brittle flooring materials in hollow core slab flooring systems, it is recommended to use smaller spans, continuous supporting beams and thin/light HCS to prevent rotation due to deflection of the supports. Adding extra reinforcement in the connection joint where cracking might happen might be a good solution to control cracking. Normally reinforcement is not placed in the connections to control cracking, but results from calculations have shown to have a positive impact on the crack width calculations.

7 Suggestion for further work

It is suggested that an experimental study should be done, to see the correlation between the deflection of supporting beam of HCS, to the appearance of cracks in the connection. This thesis has only shown the modelling and calculational effects of this, and it is of interest to see how it will react in real life. Calculations of crack widths are difficult to compare to real life situations, because of mesh-generating in FE-design and all the assumptions that are done not are 100% realistic.

It is also of interest to see the other solutions for controlling cracking in connections. It is recommended that it should be tested with fiber reinforced concrete in the connections, to see if the ductile behavior has a positive effect on the system.

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9 Appendix

Appendix A – Anchoring in Hollow Core Slabs

Armeringsstål B500NC		Fasthetsklasse		
		B25 $\gamma_c = 3{,}6$ $f_{bd} = 0{,}956$ MPa	B30 $\gamma_c = 3{,}6$ $f_{bd} = 1{,}062$ MPa	B35 $\gamma_c = 3{,}6$ $f_{bd} = 1{,}169$ MPa
Ø (mm)	$N_{Rd,s}$ (kN)	Forankringslengde $l_{bd} = (N_{Rd,s} \times 0{,}7) / (\pi \times Ø \times f_{bd})$		
8	22	640	577	524
10	34	792	713	648
12	49	952	857	778

Figure 9-1 Anchoring in joints [2]

Stangtype	Dimensjon (mm)	$N_{Rd,s}$ (kN)	Maksimalt strekk (N_{maks1}) (kN)					l_{bd} (mm)	$l_0 \approx l_{bd} + 130$ (mm)
			h = 200	h = 265	h = 320	h = 400	h = 500		
Kamstål B500NC	8	22	22	22	22	22	22	320	450
	10	34	34	34	34	34	34	369	500
	12	49	49	49	49	49	49	476	610
	16	87	50*	70*	75*	87	87	633	770
	20	137	-	-	-	90*	115*	670	800
Gjengestang K4.8	M16	45	45	45	45	45	45	388	520
	M20	71	50*	70*	71	71	71	490	620
	M24	102	50*	70*	75*	90*	102	586	720
	M30	162	-	-	-	90*	115*	529	660
Gjengestang K8.8	M16	90	50*	70*	75*	90	90	776	910
	M20	141	-	-	-	90*	115*	793	930

* Øvre grense, N_{maks1} i tabell C 12.3, er dimensjonerende.

Figure 9-2 Anchoring in voids [2]

Hulldekke			Utstøpt kanalende $\Delta V_{Rd,c} = (2/3) \times n \times b_c \times d \times f_{ctd}$ (kN pr. kanal)		
h (mm)	b_c (mm)	d (mm)	B25 $\gamma_c = 1{,}8$ $f_{ctd} = 0{,}85$ MPa	B30 $\gamma_c = 1{,}8$ $f_{ctd} = 0{,}94$ MPa	B35 $\gamma_c = 1{,}8$ $f_{ctd} = 1{,}04$ MPa
200	140	160	12,7	14,0	15,5
	155		14,1	15,5	17,2
265	166	225	21,2	23,4	25,9
	185		23,6	26,1	28,9
320	200	269	30,5	33,7	37,3
	215		32,8	36,2	40,1
400	215	347	42,5	46,7	51,7
	230		45,2	50,0	55,3
500	170	446	43,0	47,5	52,6
	200		50,5	55,9	61,8

Figure 9-3 Shear capacity in concrete filled voids [2]

Appendix B – Hand Calculations

Handcalculation to vertify the model

Deflection in steel beam:

$b := 8\ m$ $\qquad E := 210 \cdot 10^3\ \dfrac{N}{mm^2}$

$L := 6\ m$ $\qquad I := 3.9948 \cdot 10^8 \cdot mm^4$ For D32-400

Variable load

$$q_1 := \frac{1.5\ \dfrac{kN}{m^2} \cdot b}{2} + \frac{1.5\ \dfrac{kN}{m^2} \cdot b}{2} = 12\ \frac{kN}{m}$$

Deadload hollow core slab

$$q_2 := \frac{2.55\ \dfrac{kN}{m^2} \cdot b}{2} + \frac{2.55\ \dfrac{kN}{m^2} \cdot b}{2} = 20.4\ \frac{kN}{m}$$

Total loading:

$$q := q_1 + q_2 = 32.4\ \frac{kN}{m}$$

Calculated deflection

$$w_2 := \frac{5}{384} \cdot \frac{q \cdot L^4}{E \cdot I} = 6.517\ mm$$

Appendix C – Calculation of Creep Coefficient

Creep coefficient

Creep coefficient is calculated using formula from Eurocode 2:

$$\varphi(t, t_0) = \varphi_0 * \beta_c(t, t_0)$$

Necessary data

For B35	$f_{cm} := 43$
The concrete age in days when applying load:	$t_0 := 28$
The concrete age in days at considered time (50 years):	$t := 36525$
The relative humidity in percentage:	$RH := 50$
Cross-section area (in mm^2):	$A_c := 200000$
Part of constructions circumference exposed to drying in contact with atmosphere:	$u := 2000$

$$h_0 := \frac{2\ A_c}{u}$$

$$\alpha_1 := \left(\frac{35}{f_{cm}}\right)^{0.7} = 0.866 \qquad \alpha_2 := \left(\frac{35}{f_{cm}}\right)^{0.2} = 0.96 \qquad \alpha_3 := \left(\frac{35}{f_{cm}}\right)^{0.5} = 0.902$$

$$\varphi_{RH} := \left(1 + \frac{1 - \frac{RH}{100}}{0.1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right) \cdot \alpha_2 = 1.67$$ for fcm > 35MPa

$$\beta_{fcm} := \frac{16.8}{\sqrt{f_{cm}}} = 2.562$$

$$\beta_{t0} := \left(\frac{1}{\left(0.1 + {t_0}^{0.2}\right)}\right) = 0.488$$

$$\varphi_0 := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0}$$

$$\beta_H := 1.5 \cdot \left(1 + (0.012 \cdot RH)^{18}\right) \cdot h_0 + 250 \cdot \alpha_3$$

$$\beta_{ct.t0} := \left(\frac{(t - t_0)}{(\beta_H + t - t_0)}\right)^{0.3} = 0.996$$

$$\varphi(t, t_0) := \varphi_0 \cdot \beta_{ct.t0} = 2.081$$

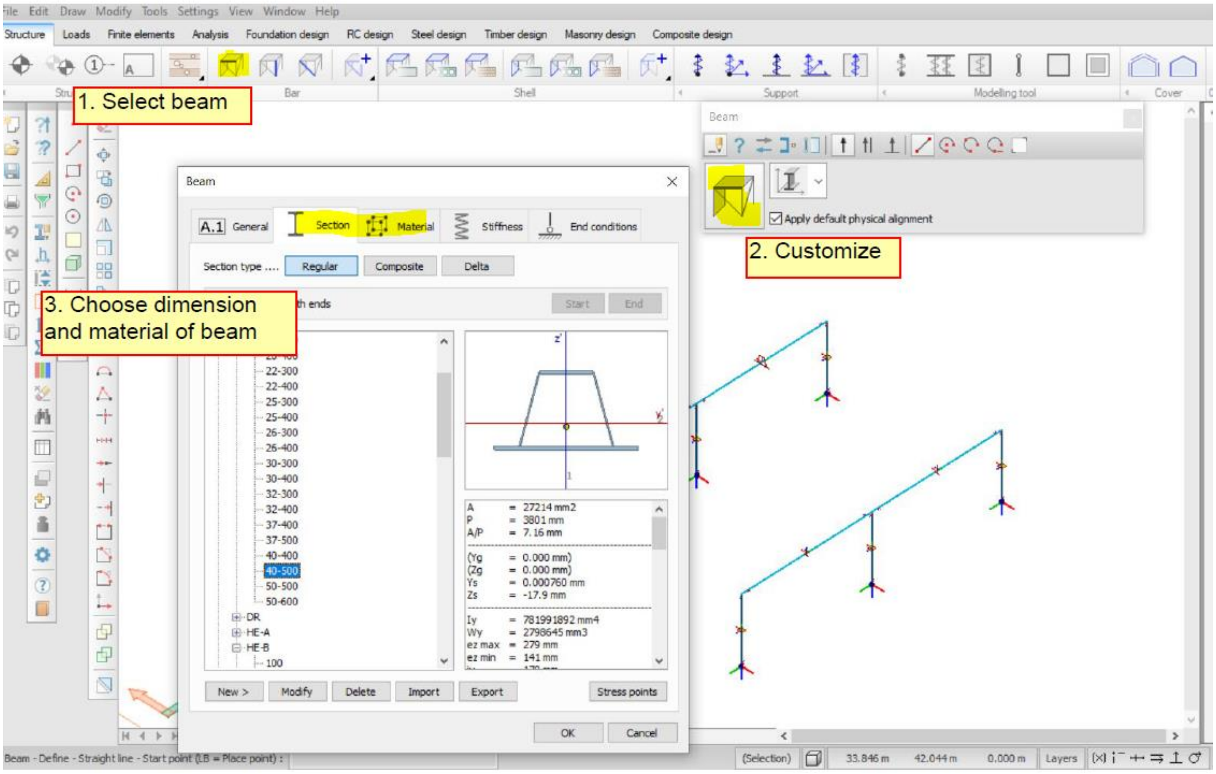
Appendix D – Modelling Process in FEM-design

This section will the designing procedure of the model in FEM-design.

The order in calculation in FEM-design, is given in the flow chart below:

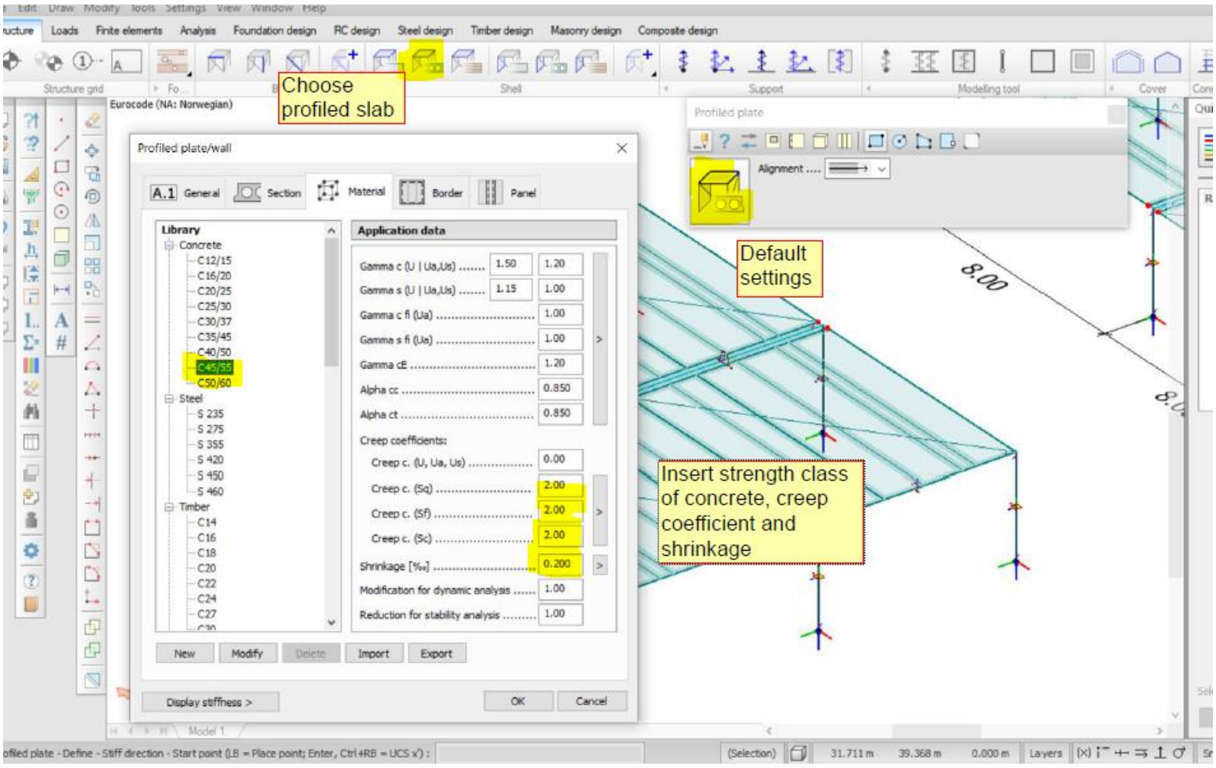
1. Structure

Start with constructing the building with beams and columns. Selected span width for supporting beams are 6 meters, and the span for the HCS are 8 meters. Add hinged support to the columns.

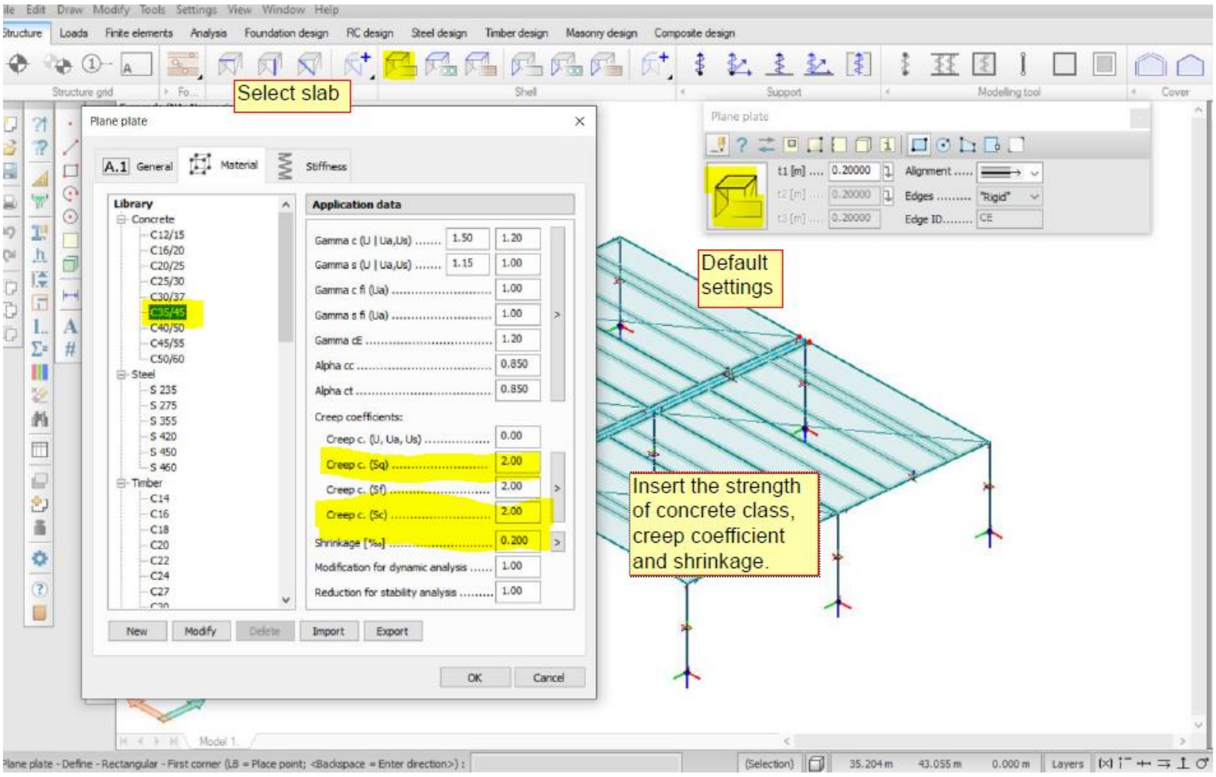


Steel quality S275 is used for steel beams, concrete strength B35 are chosen for the concrete beams and B45 is the concrete class for the HCS.

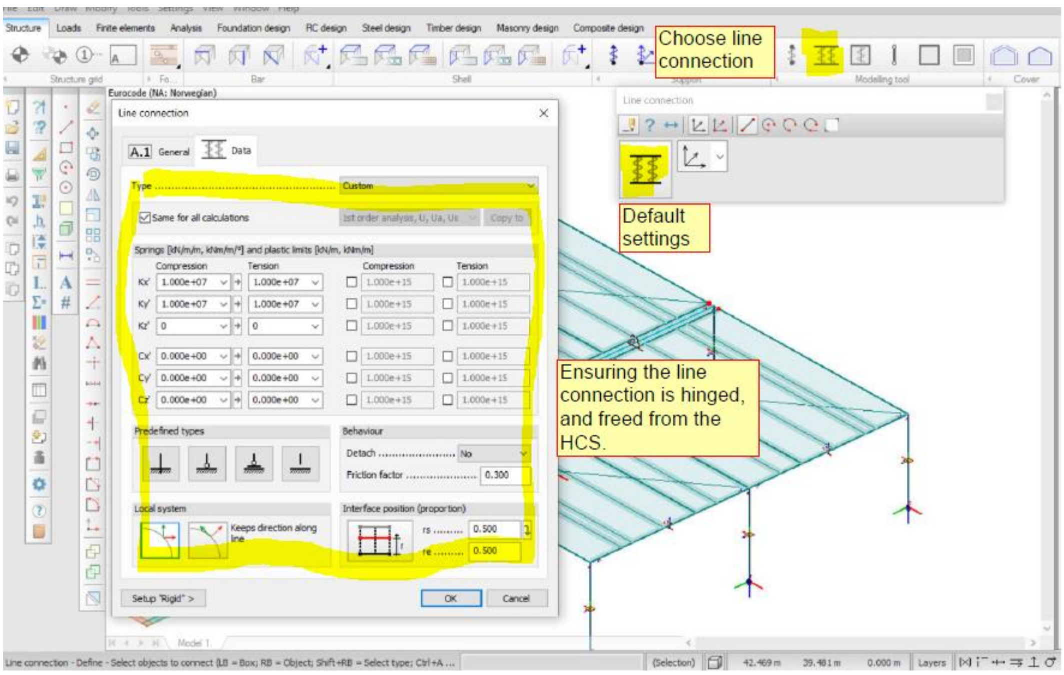
Select the hollow core slab option and add all the parameters below. Under "section" in the default settings, thickness of the hollow core slab is set to 200mm.



A concrete filling is laid on top of the supporting beam, where the cracking generates. The thickness is set to 400mm, to match the thickness of the HCS, and the width is set to 400mm. This width is chosen due to support lengths and the size of the beam. For example, for a steel beam D32-400: 2 x 110mm (support lengths on both sides) + 210mm (width of beam) = 430mm.

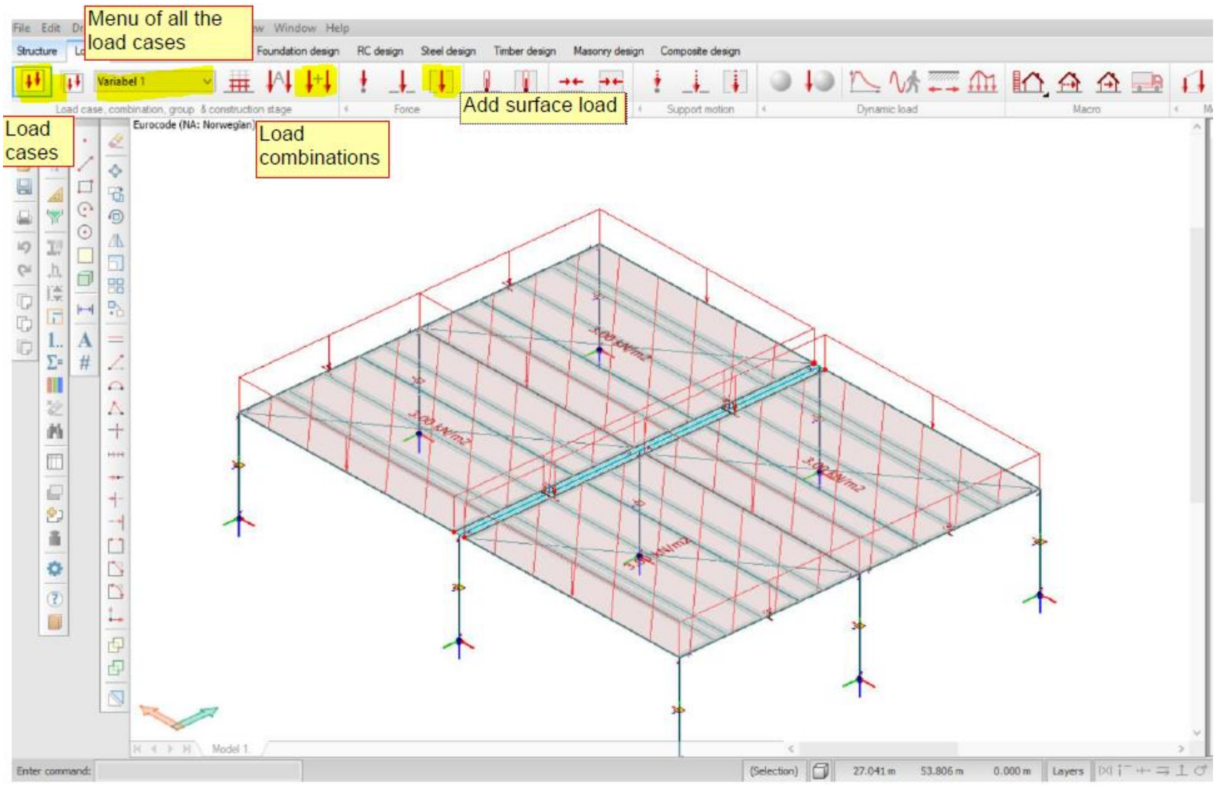


A connection line is inserted between the HCS and the supporting beam. This is to ensure that the concrete filling on top of the supporting beam does not support the HCS. The connection line ensures the forces are transported from the HCS to the supporting beam. Clicking Structure->Line connection->Default settings.



2. Loads

When adding the loads to the model, these options are used:



Load cases and load combinations are generated.

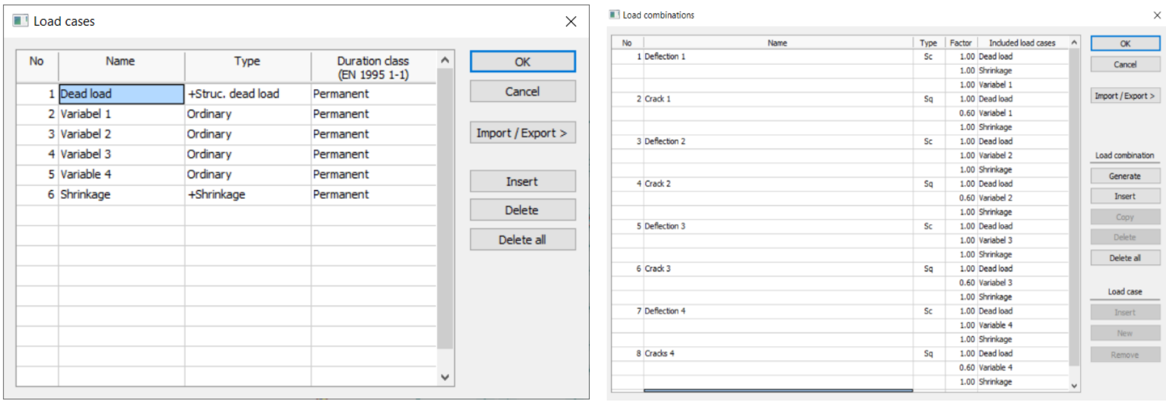


Figure 9-4 Load cases and load combinations

The load combinations are listed in Table 2. When calculating deflection, characteristic load combination is used, and for calculating the crack width, quasi-permanent equation is used.

Table 6: Load factors for SLS combinations

Lastkombinasjoner som skal påvises	Permanente laster	Dominerende variabel last	Ander variable laster	
Karakteristisk (Sc)	1.0	1.0	$1.0\psi_0$	
Ofte forekommende (Sf)	1.0	$1.0\psi_1$	$1.0\psi_2$	
Tilnærmet permanent (Sq)	1.0	$1.0\psi_2$	$1.0\psi_2$	

Category C and D are considered, due to tiled flooring usually happens in these situations.

Tabell NA.A1.1 – Verdier for ψ-faktorer for bygninger

Last	ψ_0	ψ_1	ψ_2
Nyttelastkategorier i bygninger (se NS-EN 1991-1-1)			
Kategori A: boliger	0,7	0,5	0,3
Kategori B: kontorer	0,7	0,5	0,3
Kategori C: forsamlingslokaler, møterom	0,7	0,7	0,6
Kategori D: butikker	0,7	0,7	0,6
Kategori E: lager	1,0	0,9	0,8
Kategori F: trafikk- og parkeringsarealer for små kjøretøyer (kjøretøyvekt ≤ 30kN og høyst 8 seter utenom førersete)	0,7	0,7	0,6
Kategori G: trafikk- og parkeringsarealer for mellomstore kjøretøyer, 30kN < kjøretøyvekt ≤ 160kN på to akslinger	0,7	0,5	0,3
Kategori H: tak	0	0	0
Snølaster (se NS-EN 1991-1-3)	0,7[1)]	0,5[1)]	0,2[1)]
Vindlaster (se NS-EN 1991-1-4)	0,6[1)]	0,2[1)]	0[1)]
Temperatur (ikke brann) i bygninger (se NS-EN 1991-1-5)	0,6[1)]	0,5[1)]	0[1)]

[1)] Eventuell modifisering for ulike geografiske områder kan kreves av lokale myndigheter

Figure 9-5 Ψ-factor

Table 7: Load combinations

No.	Name	Type	Factor	Load cases
1	Deflection 1	Characteristic	1.000	Dead load (+Struc. dead load)
			1.000	Shrinkage (+Shrinkage)
			1.000	Variabel 1
2	Crack 1	Quasi-permanent	1.000	Dead load (+Struc. dead load)
			0.600	Variabel 1
			1.000	Shrinkage (+Shrinkage)
3	Deflection 2	Characteristic	1.000	Dead load (+Struc. dead load)
			1.000	Variabel 2
			1.000	Shrinkage (+Shrinkage)

4	Crack 2	Quasi-permanent	1.000	Dead load (+Struc. dead load)
			0.600	Variabel 2
			1.000	Shrinkage (+Shrinkage)
5	Deflection 3	Characteristic	1.000	Dead load (+Struc. dead load)
			1.000	Variabel 3
			1.000	Shrinkage (+Shrinkage)
6	Crack 3	Quasi-permanent	1.000	Dead load (+Struc. dead load)
			0.600	Variabel 3
			1.000	Shrinkage (+Shrinkage)
7	Deflection 4	Characteristic	1.000	Dead load (+Struc. dead load)
			1.000	Variable 4
			1.000	Shrinkage (+Shrinkage)
8	Cracks 4	Quasi-permanent	1.000	Dead load (+Struc. dead load)
			0.600	Variable 4
			1.000	Shrinkage (+Shrinkage)

There are 4 different types of variable loads. They are listed in the Table 3. The values are from EC1-1-1 Table NA.6.2 The value 7 kN/m^2 are not found in tables and are quite high, but usualy there are more loads on a slab then the imposed load. It could be dry walls, heavy equipment or screed. Therefore, this surface load is considered.

Table 8: Load cases

Load case	**Surface load (kN/m^2)**
Variable 1	3
Variable 2	5
Variable 3	7
Variable 4	0

Tabell NA.6.2 – Nyttelaster på gulv, balkonger og trapper i bygninger

Kategorier for belastede områder		q_k [kN/m²]	Q_k [kN]
Kategori A			
-	Gulv	2,0	2,0
-	Trapper	3,0	2,0
-	Balkonger og verandaer[1)]	4,0	2,0
-	Loft med liten takhøyde eller begrenset adgang	1,0	1,5
Kategori B		3,0	2,0
Kategori C			
-	C1	3,0	4,0
-	C2	4,0	4,0
-	C3	5,0	4,0
-	C4	5,0	7,0
-	C5	5,0	4,0
Kategori D			
-	D1	5,0	4,0
-	D2	5,0	7,0

[1)] Takterrasse, se kategori I

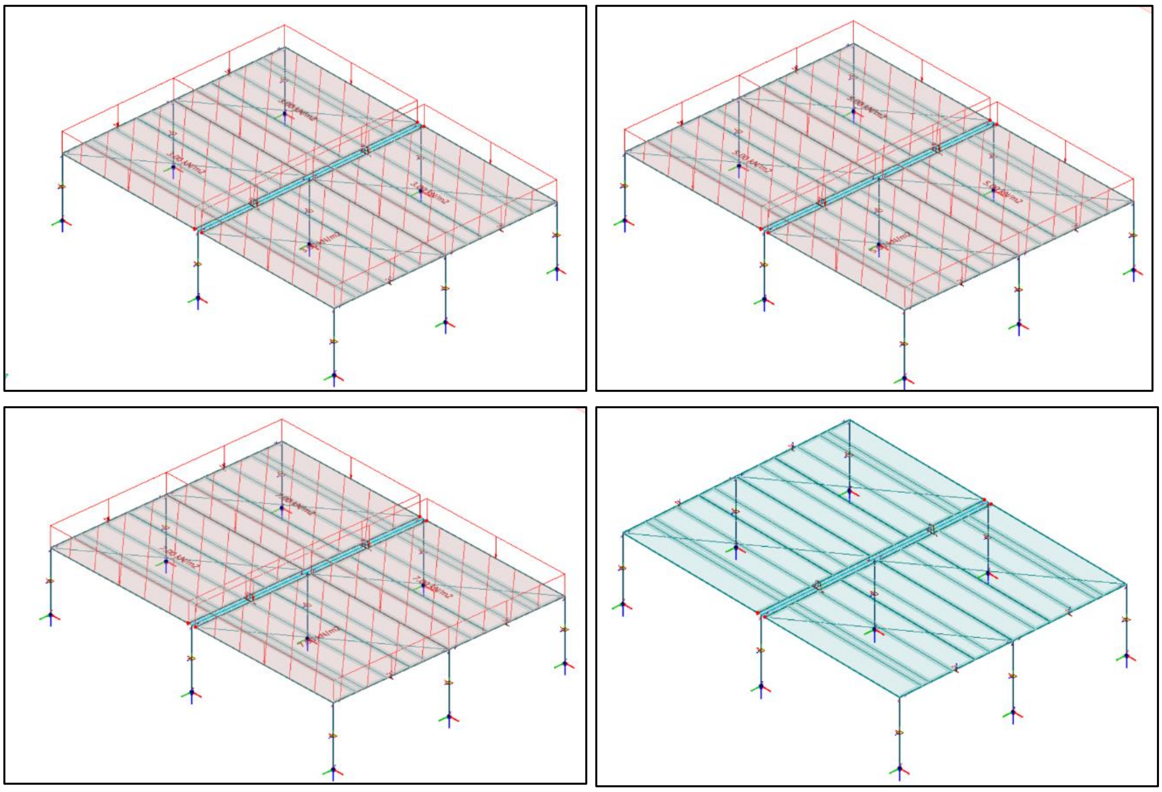


Figure 9-6 Added surface load on the HCS.

3. Finite elements

FEM-design automatically generates an evenly distributed mesh. It is done by clicking Finite Elements->Generate mesh and selecting the whole model. Over the columns (orange circles) are the smoothing of the mesh. See Chapter 4.3.

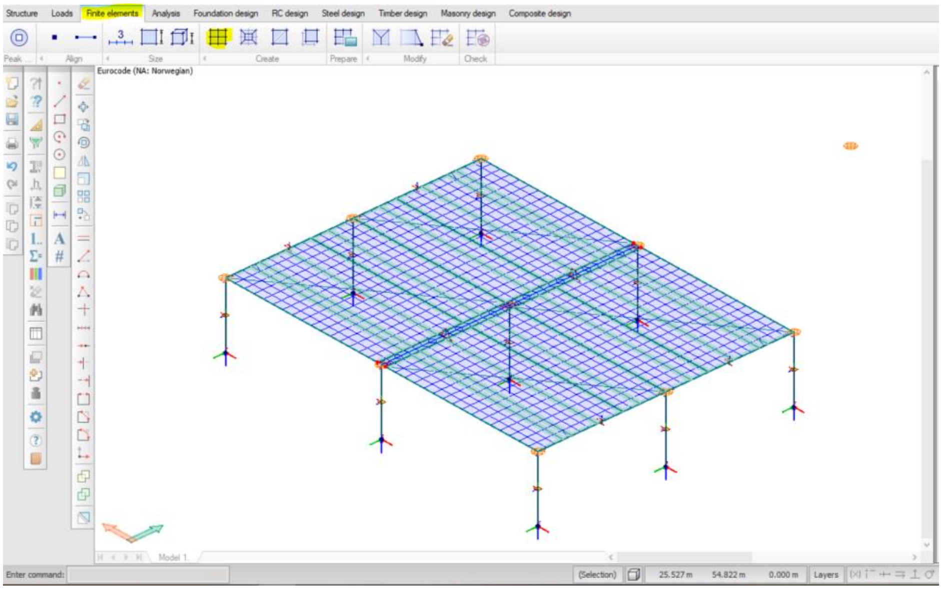
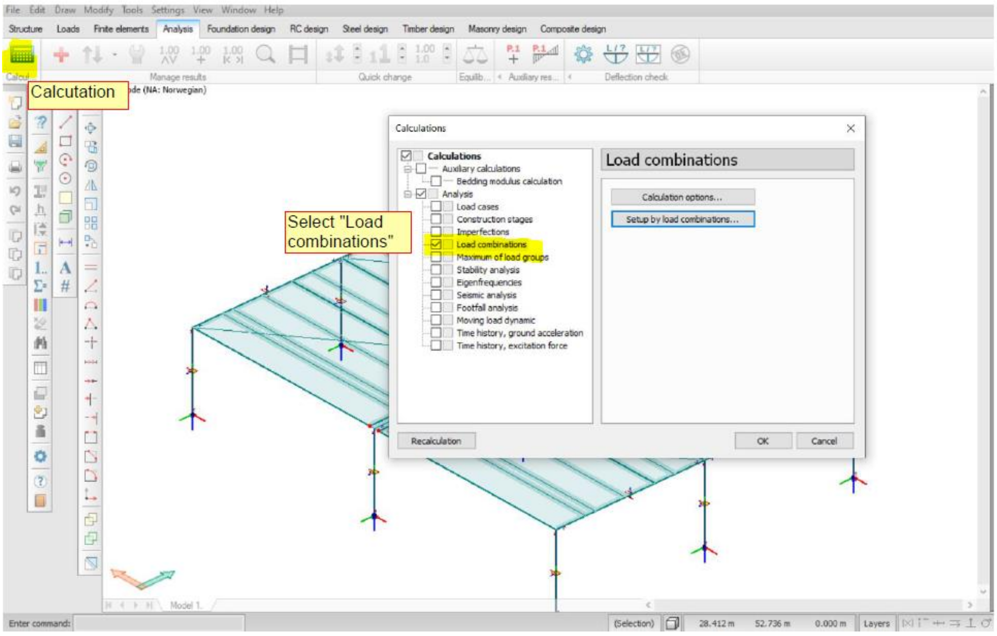


Figure 9-7 Meshing of the model

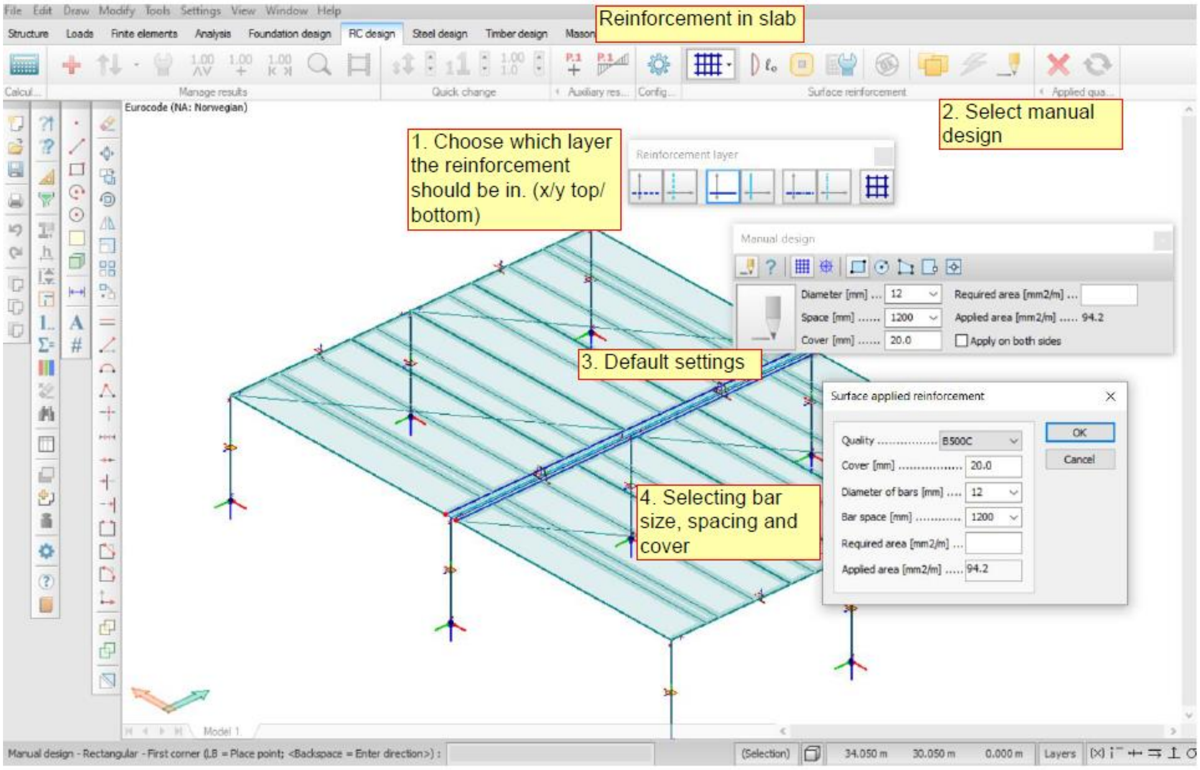
4. Analysis



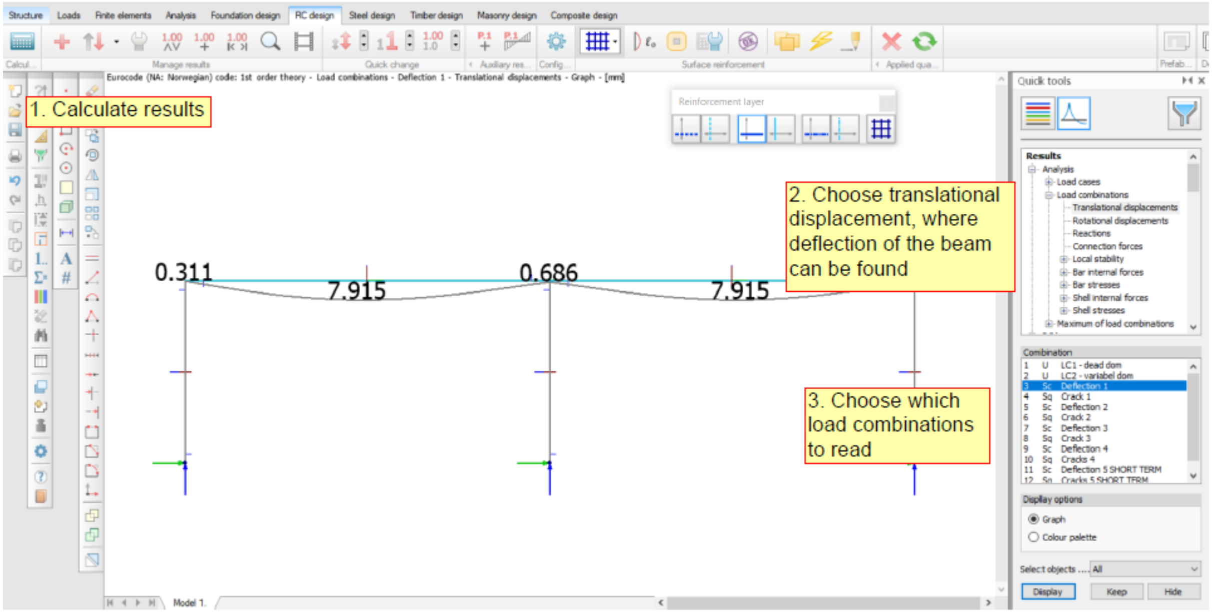
Under the analysis option, the model is calculated to see if the statics are correct. Here the deflections are calculated, and these can be used to compare to hand calculation, to see if the model can be trusted.

5. RC-design (Reinforced Concrete-design)

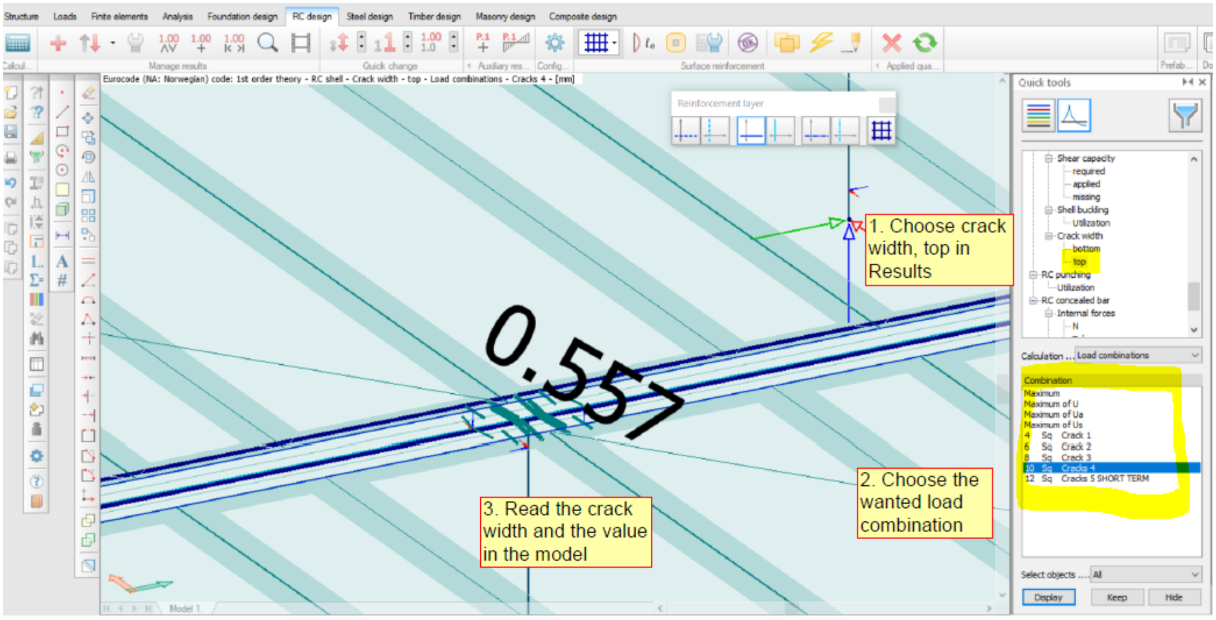
After the analysis is done, it is time to design the concrete. Reinforcement is laid in the concrete filling by clicking RC-design->Manual design->Default setting, and making sure it is put in the right layer of the concrete.



Finding the results for deflection for different load combinations are shown below.



To find the crack widths, see picture below.



List of all the beams and HCS used in the calculations:

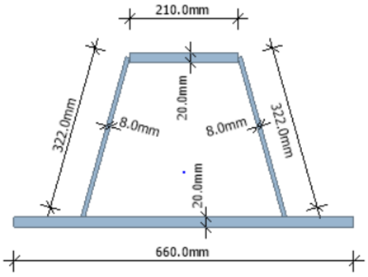


Figure 9-8 D32-400

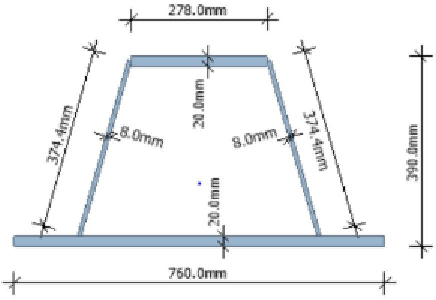


Figure 9-9 D37-500

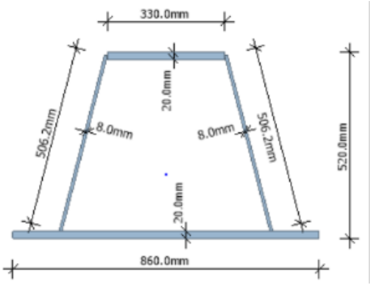


Figure 9-10 D50-600

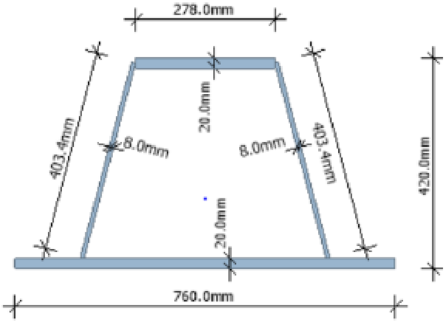


Figure 9-11 D40-500

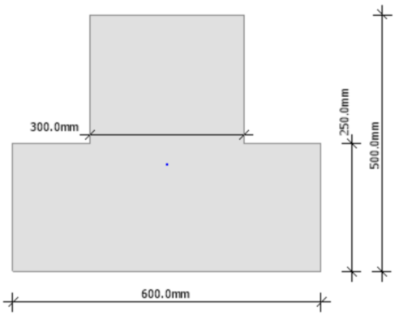


Figure 9-13 Concrete inverted T-beam (600x500)

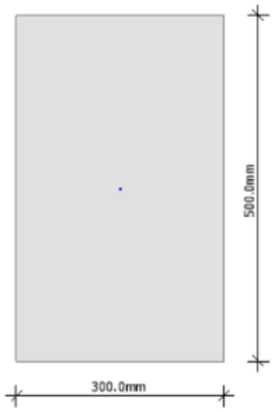


Figure 9-14 Concrete beam 300x500

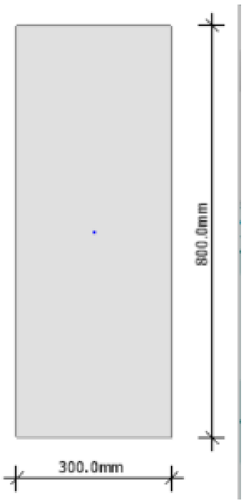


Figure 9-12 Concrete beam 300x800

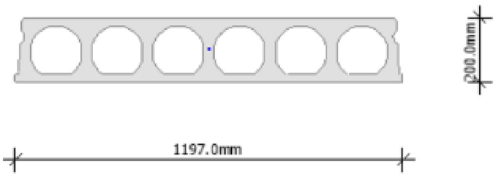


Figure 9-16 Hollow core slab 200

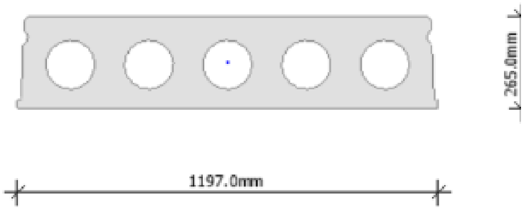
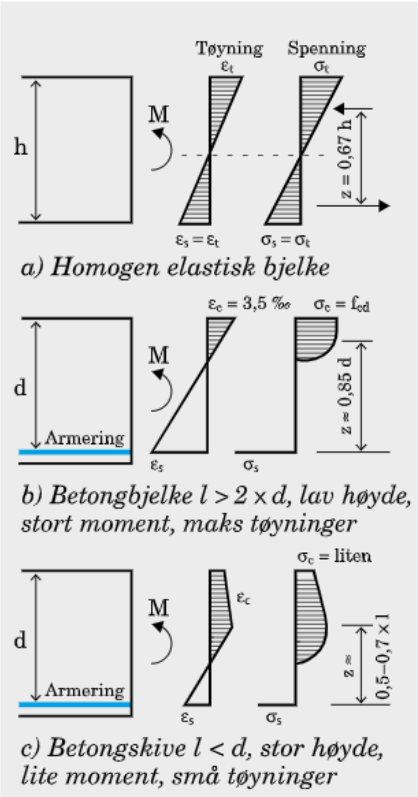
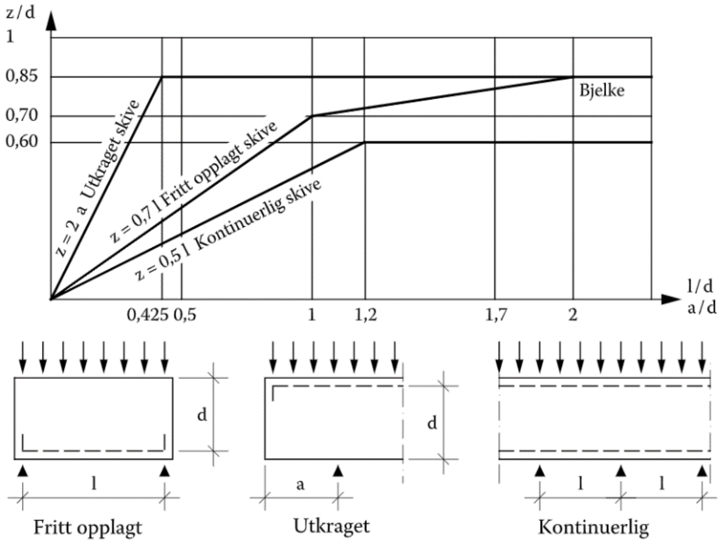


Figure 9-15 Hollow core slab 265

Appendix E – Seam Reinforcement Tables

To find the value of z, this table is used from BB-C.

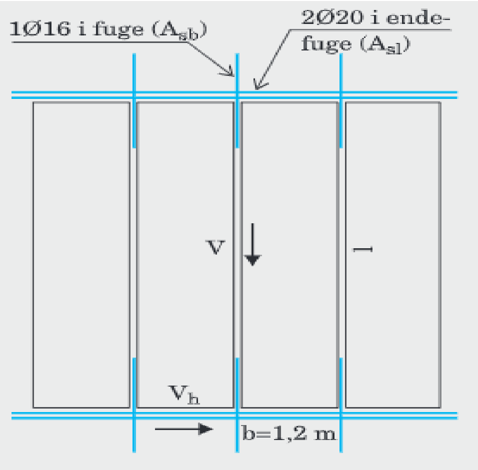




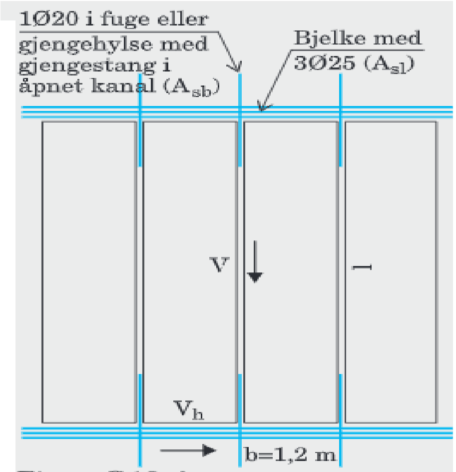
This table is used for finding the μ, which is normally chosen as 0.6 (risset, glatt fuge).

Overflate	Beskrivelse	Ruhet c		Friksjon μ
		Urisset fuge	Betydelig risset fuge	
Svært glatt	Støpt mot stål, plast, spesialform av tre	0,025 – 0,10	0	0,5
Glatt	Glideformet, ekstrudert eller ubehandlet etter vibrering	0,20	0	0,6
Ru	Minst 3 mm ujevnheter med en senteravstand omkring 40 mm utført med rive, eksponering av tilslag, eller andre tilsvarende metoder	0,40	0	0,7
Fortannet	Se figur B 16.13	0,50	0,50	0,9

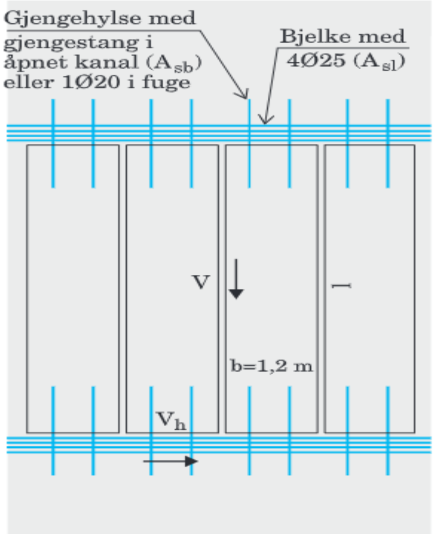
Below are figures of examples of reinforcement in the connections, due to horizontal forces:



Figur C 13.3.
Vanlig skivearmering.



Figur C 13.4.
Mye skivearmering.

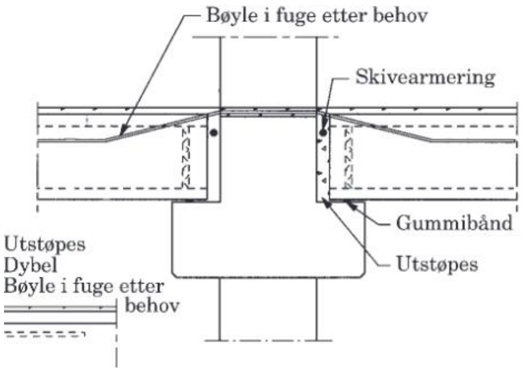


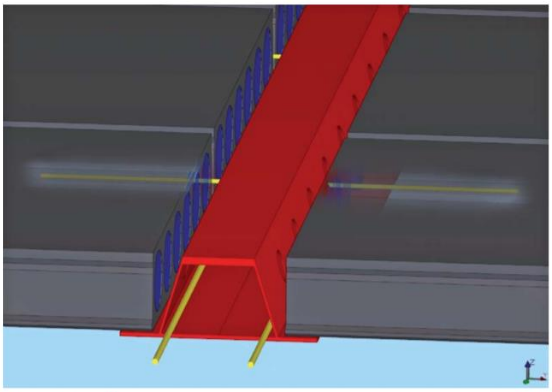
Figur C 13.5.
Ekstra stor skivearmering.

Tabell C 13.3. Skiveskjærkapasitet i utstøpte fuger.

Hulldekke-høyde (mm)	Fugehøyde h_f (mm)	$V_{Rd,c} = \tau \times h_f$ (kN / m)			
		B20 (τ = 0,142)	B25 (τ = 0,170)	B30 (τ = 0,188)	B35 (τ = 0,208)
200	170	24	29	32	35
265	235	33	40	44	49
320	290	41	49	54	60
400	370	52	63	69	77
500	470	66	80	88	97

Appendix F – Example of Reinforcement in HCS Connections.





Kommentarer: Relativt torsjonsstivt tverrsnitt. Mye utstøping på byggeplass. Brannbeskyttelse vanligvis unødvendig.