Master's thesis

NTNU
Norwegian University of Science and Technology
Faculty of Engineering
Department of Structural Engineering

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# Concrete design according to EC.2

## A comparison between the current and the revised version of EC.2

Master's thesis in Bygg- og miljøteknikk
Supervisor: Jan Arve Øverli

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# Abstract

Eurocode 2 covers the design of concrete structures and is undergoing a revision process. The dissertation's primary objective is to accentuate modifications and changes made in the revision of EC.2 within methods and models and compare them with the current EC.2. The dissertation approaches its primary objective by intensively studying the two versions of EC.2, which implies comparing the changes using graph representations and values gained by performed calculations of structural elements.

The revised version of EC.2 defines new factors within the expressions for design compressive and tensile strengths and young's modulus, resulting in different values relative to the current EC.2. The revised version defines $t_{ref}$, which is the time in days at which the characteristic concrete strength is determined. The concrete characteristic strength may be determined for $t_{ref}$ at 28-91 days. Moreover, the concrete compressive strength may be determined for times t that can be before or after $t_{ref}$. These changes will allow for more utilization of the concrete compressive strength and reduce unnecessary use of building materials, which positively impacts the carbon footprint of concrete structures.

Sections regarding shear were significantly revised. The expression for shear resistance without shear reinforcement in the revised version of EC.2 follows the critical shear crack theory and includes the size parameter $d_{dg}$. Furthermore, the effect of slenderness is taken into account by including the factor $\alpha_v$. The effect of slenderness is not considered in the current EC.2. Consequently, the new expression improves the correspondence between calculated and tested values of the shear resistance, providing a safer and more accurate design than the empirical expression defined in the current EC.2.

Crack width and creep calculations are more empirical, which means they are based on intensive laboratory experiments and observations. Thus, there are new empirical factors included, complicating the calculations procedure relative to the current EC.2.
However, the revision of EC.2 has enhanced the ease of use through considerable improvements within the content arrangement and the provided supplemental information/definitions within the clauses. Sections regarding anchorage length have been significantly improved as they are more straightforward and transparent than EN 1992-1-1.

# Sammendrag

Eurokode 2 dekker dimensjonering av betongkonstruksjoner og gjennomgår en revisjonsprosess. Hovedmålet med denne avhandlingen er å fremheve endringene og modifikasjonene gjort i metoder og modeller i revisjonen og sammenligne med gjeldende EK.2. Avhandlingen tilnærmer seg hovedmålet ved å intensivt studere de to utgavene av EK.2. Dette innbærer sammenlikning av endringene ved bruk av grafiske fremstillinger og oppnådde verdier fra utførte beregninger av konstruksjonselementer.

Den reviderte versjonen av Ek.2 definerer nye faktorer innenfor uttrykkene for trykk- og strekkfasthet og elastisitetsmodulen, noe som resulterer i forskjellige verdier ift. gjeldende EK.2. I den reviderte utgaven defineres $t_{ref}$, som er tiden i dager som den karakteristiske betongfasthet er bestemt for. Betongens karakteristiske fasthet kan bli spesifisert for tiden $t_{ref}$ for 28-91 dager. I tillegg kan betongens trykkfasthet bestemmes for tider t som kan være før eller etter $t_{ref}$. Disse endringene vil tillatte mer utnyttelse av betongens trykkfasthet og bidra til redusering av unødvendig bruk av byggematerialer. På denne måten vil karbonavtrykket kunne påvirkes i en positiv retning.

Avsnitt om skjær ble betydelig revidert. Uttrykket for skjærstresskapasitet uten skjærarmering i den reviderte utgaven av EK.2 følger den kritiske skjærriss teorien og inkluderer størrelses parameteren $d_{dg}$. Videre tas effekten av slankhet i betraktning ved å inkludere faktoren $\alpha_v$, en effekt som ikke er tatt i betraktning i gjeldende Ek.2. Som følge av disse endringene, forbedrer det nye uttrykket samsvaret mellom beregnede og testede verdier for skjærstresskapasitet uten skjærarmering. Dermed gir det nye uttrykket tryggere og mer nøyaktige skjær dimensjoneringer ift. det empriske uttrykket definert i gjeldende EK.2.

Rissvidde- og krypberegninger i den reviderte EK.2 er mer empiriske, dvs de er basert på intensive laboratorieforsøk og observasjoner. Som resultat av endringene kompliseres beregningene ift. gjeldene EK.2 ved at nye empiriske faktorer inkluderes.
Imidlertid har revisjonen forbedret brukervennligheten, gjennom betydelige forbedringer av rekkefølge av avsnittene, klausulene og tilleggsinformasjonen/definisjonene gitt i klausulene. Avsnitt om forankringslengde er betydelig forbedret da de er enklere og mer gjennomsiktig enn i EN 1992-1-1.

# Preface

This dissertation was written in a period equivalent to 20 weeks for my master's degree in Structural Engineering at the Norwegian University of Science and Technology.

My motivation to acquire more knowledge about concrete design has led me to choose the topic of this dissertation which is related to concrete design according to the revised version of Eurocode 2. It has been interesting to uncover changes made in the revised version and compare methods and values with the current Eurocode 2.

This dissertation has reached its goals, and I have acquired knowledge of the changes we will face in the concrete structure design through my investigation done in this thesis. It was challenging to limit the scope of this thesis; however, I have gained a thorough insight into the topics presented in this dissertation.

It has been an exciting journey with many ups and downs along the way; thankfully, I received a lot of support during my master's dissertation. I would like to use this opportunity to thank my supervisor Professor Jan Arve Overli, for his guidance, reviews, and recommendations. Last but not least, I want to thank my family for their endless support.

Mahmoud Abusharekh

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# List of Abbreviations

| | |
|---|---|
| EN 1992-1-1:2021-1 | The upcoming new version of Eurocode 2 part 1-1 |
| EN 1992-1-1 | The current Eurocode 2 part 1-1 |
| NA | National annex |
| ULS | Ultimate limit state |
| SLS | Serviceability limit state |
| CSCT | Critical shear crack theory |
| $A_c$ | Concrete cross sectional area |
| $A_s$ | Ordinary reinforcement cross sectional area |
| $A_{sw}$ | Shear reinforcement cross sectional area |
| $A_{sl}$ | Tensile reinforcement that extends beyond a section |
| $a_q$ | Distance between forces pushing against each other |
| $a_v$ | Mechanical shear span |
| $a_{cs}$ | Effective shear span with respect to the control section |
| $b_w$ | Minimum width between tension and compression chords\| |
| $b_0$ | Length of the control perimeter |
| $c_d$ | Nominal concrete cover (When considering anchorage) |
| $c_{nom}$ | Nominal concrete cover specified for a cross-section |
| $c_{min,dur}$ | Minimum concrete cover due durability |
| $c$ | Concrete cover to the surface of the bar |
| $c_s$ | Clear distance between parallel reinforcement |
| $D_{upper}$ | Largest value of the upper sieve in an aggregate for the coarsest fraction |
| $D_{lower}$ | Smallest value of the upper sieve in an aggregate for the coarsest fraction |
| $d$ | Effective depth of cross-section |
| $d_{dg}$ | Size parameter describing the crack and the failure zone roughness |
| $d_{eff}$ and $d_v$ | Shear resisting effective depth of the reinforcement |
| $E_{cm}$ | Concrete modulus of elasticity |

| | |
|---|---|
| $E_s$ | Ordinary steel modulus of elasticity |
| $e_b$ | The eccentricity of the shear forces resultant with respect to the control perimeter centroid. |
| $f_{ck}$ | Concrete cylinder compressive strength at age $t_{ref}$ |
| $f_{cm}$ | Mean cylinder compressive strength at age $t_{ref}$ |
| $f_{cd}$ | Design compressive strength |
| $f_{ctk,0.05}$ | Characteristic axial tensile strength by 5% fractile |
| $f_{ctm}$ | Mean axial tensile strength at age $t_{ref}$ |
| $f_{ct,eff}$ | The effective mean tensile strength of concrete, when first cracking occur |
| $f_{yd}$ and $f_{ywd}$ | Design yield strength of reinforcement and shear reinforcement, respectively. |
| $f_{bd}$ | Design value of ultimate bond stress |
| $I$ | Second moment of area of the concrete section |
| $k_{lb}$ | Factor for calculation anchorage length |
| $k_{cp}$ | Coefficient considering bond effect when calculation anchorage length |
| $K_I$ | Coefficient considering the effect of cracking for tension stiffening and creep deformation |
| $k_{pb}$ | Shear gradient enhancement coefficient for punching |
| $k_{\frac{1}{r}}$ | Coefficient accounting for the increase of crack width due to curvature |
| $l_{b,min}$ | Minimum design anchorage length |
| $l_{bd}$ | Design anchorage length |
| $l_{b,rqd}$ | Required anchorage length |
| $M$ | Bending moment in linear members |
| $M_{Ed}$ | Design moment |
| $M_{Rd}$ | Design moment resistance |
| $N_{Ed}$ | Design axial force |
| $N_{Rd}$ | Design value of axial resistance |
| $N_{vd}$ | Design value of the sum of the tension and compression chord's additional axial forces due to shear |
| $q_{Ed}$ | Design distributed load |
| $R_{ax}$ | Restrained factor (normally taken as 0.75) |
| $S_r$ | Spacing of shear links in radial direction |
| $S_{rm,cal}$ | Calculated mean crack spacing |
| $t_{ref}$ | Age of the concrete at which time (in days) the concrete strength is determined at. |

| | |
|---|---|
| $t$ | Time being considered, the age of concrete |
| $u_1$ | Length of the control perimeter |
| $V_{Ed}$ | Design shear force |
| $V_{Rd,max}$ | Maximum design shear force |
| $w_r$ | Required mechanical reinforcement ratio to resist the moment due to design load |
| $w_{lim,cal}$ | Limit of the crack width |
| $w_{k,cal}$ | Calculated crack width |
| $x$ | Depth of the concrete in compression |
| $Z$ | The internal lever arm |
| $\alpha$ | Angle |
| $\alpha_{cc}$ | Coefficient considering long-term loads (creep-effect) on concrete compressive strength |
| $\alpha_{ct}$ | Coefficient considering long-term loads (creep-effect) on concrete tensile strength |
| $\alpha_w$ | Angle between the shear reinforcement and the member axis perpendicular to the shear force. |
| $\alpha_e$ | Modular ratio $= E_s/E_c$ |
| $\beta_{cc}(t)$ | Coefficient for determining the compressive strength of concrete (determined at age $t_{ref}$) after times, t. |
| $\gamma_c$ | Concrete partial factor |
| $\gamma_v$ | Partial factor for shear and punching shear resistance without shear reinforcement |
| $\delta$ | Deflection |
| $\delta_{load}$ | Linear elastic deflection due to the relevant combination of action |
| $\delta_{\epsilon cs}$ | Linear elastic deflection due to shrinkage |
| $\epsilon_c$ | Compressive strain in the concrete |
| $\epsilon_{c1}$ | Compressive strain in the concrete at the peak stress $f_c$ |
| $\epsilon_{cu}$ | Ultimate compressive strain in the concrete |
| $\epsilon_{cm}$ | Mean strain at the concrete at the same level as mean strain in the reinforcement $\epsilon_{sm}$ |
| $\epsilon_{free}$ | Imposed strain |
| $\epsilon_s$ | Strain of the ordinary reinforcement |
| $\epsilon_{yd}$ | Design yield strain of reinforcement |
| $\Delta V_{Ed}$ and $\Delta q_{Ed}$ | The portion that may be reduced from $V_{Ed}$ and $q_{Ed}$ respectively. |
| $\eta$ | Ratio of strains used to define stress strain model |

| | |
|---|---|
| $\eta_{cc}$ | Brittleness factor |
| $\eta_c$ | Strength reduction factor for shear resistance $\tau_{Rd,c}$ |
| $\eta_{sys}$ | Coefficient accounting for performance of the punching shear reinforcement systems |
| $\phi$ | Angle between compression field and the member axis |
| $\phi_{min}$ | Minimum angle between compression field and the member axis |
| $\lambda$ | Slender ratio |
| $\nu$ and $\nu_1$ | Strength reduction factor for cracked concrete due to shear or other actions |
| $\rho$ | Reinforcement ratio |
| $\rho_l$ | Reinforcement ratio of the bonded longitudinal reinforcement in the tensile zone |
| $\rho_w$ | Shear reinforcement ratio |
| $\rho_p$ | Tensile reinforcement ratio accounting for bond properties of the bars |
| $\sigma_s$ | Steel stress determined with assumption of cracked cross-section |
| $\sigma_{cp}$ | Compressive stress in the concrete from axial load |
| $\sigma_c$ | Compressive stress in the concrete |
| $\tau_{Ed}$ | Design shear stress |
| $\tau_{Rdc,min}$ | Minimum shear resistance without shear reinforcement |
| $\tau_{Rd,c}$ | Shear resistance without shear reinforcement |
| $\tau_{Rd,sy}$ | Shear resistance with shear reinforcement |

# Chapter 1

# Introduction and methodology

## 1.1 Introduction

### 1.1.1 About the revision of EN 1992-1-1

EN 1992-1-1 covers the design of concrete structures. It contains general rules and rules for buildings, bridges, and civil engineering structures. The current EC.2 part 1-1 was published in 2004. Moreover, it is undergoing a revision process. The main objectives of the revision were mentioned in a presentation published on the official website of the British institution of structural engineering. The revision aims to enhance ease of use by improving clarity, simplifying routes through Eurocode, limiting the inclusion of alternative application rules, and avoiding or removing rules of little practical use in design. It also aims to reduce the number of nationally decided parameters (NBPs) in the national annexes and include more principles about design by non-linear FEM. As mentioned in the presentation, durability is also an issue that needs further development. The revision also aims to represent the size effect consistently and improve the early age thermo-mechanical design (early thermal cracking). [1]

The revised version of EC.2 (EN 1992-1-1:2021-1) has been submitted to CEN Enquiry. CEN Enquiry will allow for broader distribution and commentary. Even though the next generation of EC.2 is still not ready for public use, the content and form of the document are already relatively fixed. [2]

The new draft includes significant changes to reach the leading objectives mentioned above. Some of the significant changes regard shear and punching shear. The new methods build on the critical shear crack theory documented in the FIB model code. Crack controlling has been developed based on more information on early thermal and shrinkage restraint cracking.[2] EN 1992-1-1:2021-1 have new annexes regarding fibre-reinforced concrete, recycled aggregates, and assessment of concrete structures. Furthermore, it deals with bridges in an additional annexe to the main code.

### 1.1.2 Objective of thesis

The main objective of the thesis is to accentuate changes in methods and models made in the revised version of EC.2. A comparison between the two versions of Eurocode 2 will be conducted. Some selected topics within materials, ultimate limit state (ULS), serviceability limit state (SLS), and detailing of reinforcement will be covered in this thesis. The background theory on the selected topics is covered in chapters 2-5. Furthermore, a building is considered, and hand calculations of the most stressed structural parts are represented in chapter 6.

## 1.2 Methodology

### 1.2.1 Literature

This dissertation establishes itself on both quantitative and qualitative comparison methods. The dissertation intensely studied the two primary sources: the current Eurocode EN 1992-1-1:2004 and the upcoming new version EN 1992-1-1:2021-1. The two versions of EC.2 are compared by exploring differences in the provided expressions and procedures. Moreover, parametric comparisons of the defined expressions and factors are conducted where convenient.

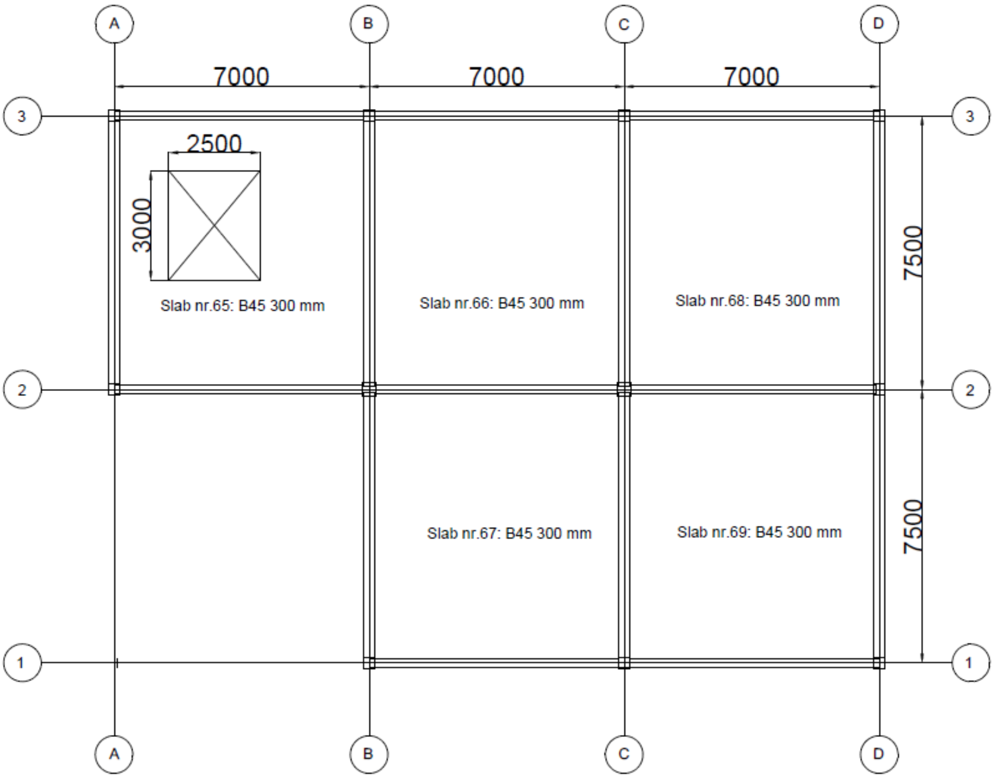
### 1.2.2 Case

A structural model is defined where the objective is performing calculations of the most stressed structural components according to the two versions of EC.2 and comparing gained values and calculations procedures.
The model's layout was developed using simple 2D drawings in AutoCAD, and secondly, 3D visualized using Revit. Robot Structural Analysis was then used to verify the structural components and extract necessary data (e.g. moment and shear diagrams). The most stressed parts were then hand calculated and checked in the ultimate limit state (ULS) and serviceability limit state (SLS) according to the two versions of EC.2. Hand calculations are done in MathCAD software.
The structural model consists of 3 stories with a gross heigh equal to 3.5 m. The spans are 7.5 m along the y-axis and 7 m along the x-axis. See figure 1.1.

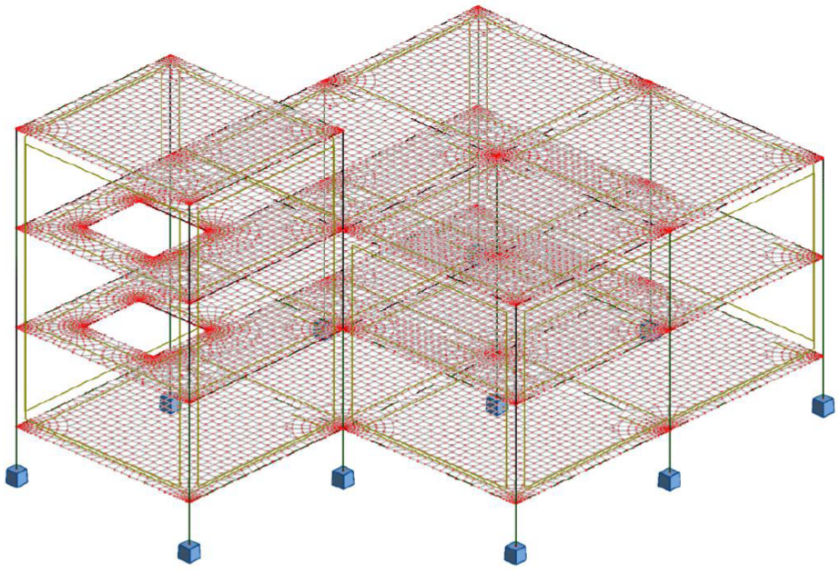
### 1.2.3 Assumptions

General assumptions are made in the model and hand calculations. The structural model is assumed to be used as office areas. Normal, cast-in-place concrete and steel reinforcement of type $B500C$ are presumed. Slabs are of a concrete class $B45$, while beams and columns are of concrete class $B35$. The slabs are modelled and calculated as two-way slabs. The building is presumed to be located in Trondheim, Norway.

The loads considered consist of snow, live, and wind loads. Live loads and snow loads are determined according to EN 1991-1-1:2002 [6.3.1.1(2)] and EN 1991-1-3:2003 Annex C, respectively. Wind loads are simulated using Robot Structural Analysis by setting wind speed to 26 m/s (applicable for Trondheim). The software Robot automatically determines and generates wind loads according to EN 1991-1-3/4:2003 + NA: 2008/2005.




**(a)** Plan view of the 3. floor



**(b)** 3D view in Robot structural analysis

**Figure 1.1:** Reference model

# Chapter 2

# Concrete mechanical properties

Chapter 3 in EN 1992-1-1 deals with materials. This corresponds to chapter 5 in the revised version of EC.2. The new version includes some changes regarding concrete compressive strength, tensile strength, and modulus of elasticity. In this chapter, these changes are highlighted.

## 2.1 Concrete compressive strength $f_{ck}$

Table [5.1] in EN 1992-1-1:2021, shows values for different characteristic concrete cylinder compressive strengths $f_{ck}$, obtained after $t_{ref} = 28$ days. $t_{ref}$ is a new addition to the code and is defined as the age of concrete, at which time (in days) the concrete strength is determined. According to clause [5.1.3 (2)] in EN 1992-1-1:2021-1, $t_{ref}$ could be taken as 28 days in general, or when specified, $t_{ref}$ may be taken between 28-91 days.
Furthermore, according to clause [5.1.3 (4)], concrete compressive strength, $f_{ck}(t)$ can be specified for times t (t is the time being considered) that can be before or after $t_{ref}$ for several stages (e.g. demoulding, removal of propping, transfer of prestressing).

The new regulations concerning the concrete age are substantial and less conservative than the current EC.2, which does not allow for determining the compressive strength $f_{ck}(t)$ for $t > 28$ days. Consequently, the new regulations will allow for more utilization of the concrete compressive strength relative to the current EC.2.

Mean cylinder compressive strength is defined as follows:

$$f_{cm} = f_{ck} + 8 \tag{2.1}$$

Mean cylinder compressive strength at age t is defined as follows:

$$f_{cm}(t) = \beta_{cc}(t) * f_{cm} \tag{2.2}$$

$\beta_{cc}(t)$ is determined according to annex B, as follows:

$$\beta_{cc}(t) = exp[s_c * (1 - \sqrt{\frac{t_{ref}}{t}}\sqrt{\frac{28}{t_{ref}}})] \tag{2.3}$$

$s_c$ is a coefficient that depends on the early strength development of the concrete and the concrete strength, which can be determined according to table B.2 in annexe B in EN 1992-1-1:2021-1. Generally, the values for $s_c$ coefficients are higher compared to the values for $s_c$

coefficients defined in clause [3.1.2 (6)] in EN 1992-1-1.

Cylinder compressive strength at time t:

$$f_{ck}(t) = f_{cm}(t) - 8 \tag{2.4}$$

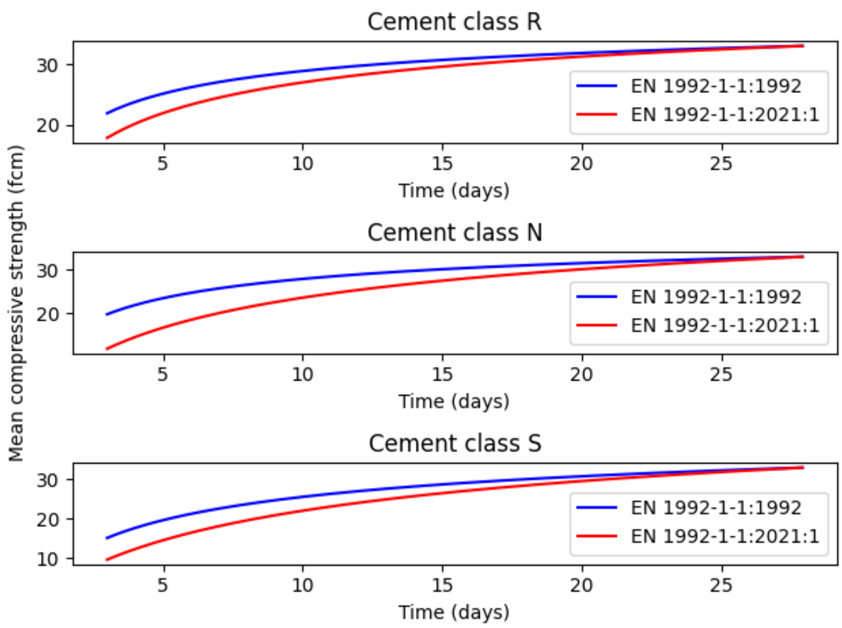



**Figure 2.1:** Comparison of mean compressive strength $f_{cm}$ for t less than 28 days

Figure 2.1 shows the development of concrete mean compressive strength with times up to 28 days. In the figure it is considered concrete class $B25$ and $t_{ref} = 28$. As seen in figure 2.1, the mean compressive strength, $f_{cm}$ is exact at 28 days in both versions of EC.2. However, the new expression for $\beta cc(t)$ provides lower values for mean compressive strength for all cement types for times t less than 28 days. The new expression causes slower mean compressive strength development than the expression defined in the current EC.2, hence the new expression is more conservative for the defined circumstances. However, as aforementioned, the revised version of EC.2 allows for determining compressive strength for times t even after $t_{ref}$, which will eventually induce higher compressive strength.

### 2.1.1 Design compressive strength $f_{cd}$

Design compressive strength is determined as follows:

| EN-1992-1-1:2021-1 | EN 1992-1-1 |
|---|---|
| $f_{cd} = \eta_{cc} * \kappa_{tc} \frac{f_{ck}}{\gamma_C}$ | $f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_C}$ |
| $\eta_{cc} = (\frac{40}{f_{ck}})^{\frac{1}{3}} \leq 1.0$ | $\alpha_{cc} = 1$ |

**Table 2.1:** Provided expressions for $f_{cd}$

In the same manner as $\alpha_{cc}$, $\kappa_{tc}$ is a factor considering long-term loads (creep-effect) on concrete strength. Clause [5.1.6 (1)] defines recommended values for $\kappa_{tc}$ based on the cement class and $t_{ref}$. The national annexe of Norway in EN 1992-1-1 defines a recommended value of 0.85 for $\alpha_{cc}$, while the main code EN 1992-1-1 recommends a value of 1.0. As Norway's national annexe is not yet provided in EN 1992-1-1:2021-1, the recommended values defined in the main code will be used.
$\eta_{cc}$ is a factor considering the difference between the undistributed cylinder compressive strength and the developed concrete effective compressive strength in a structure.

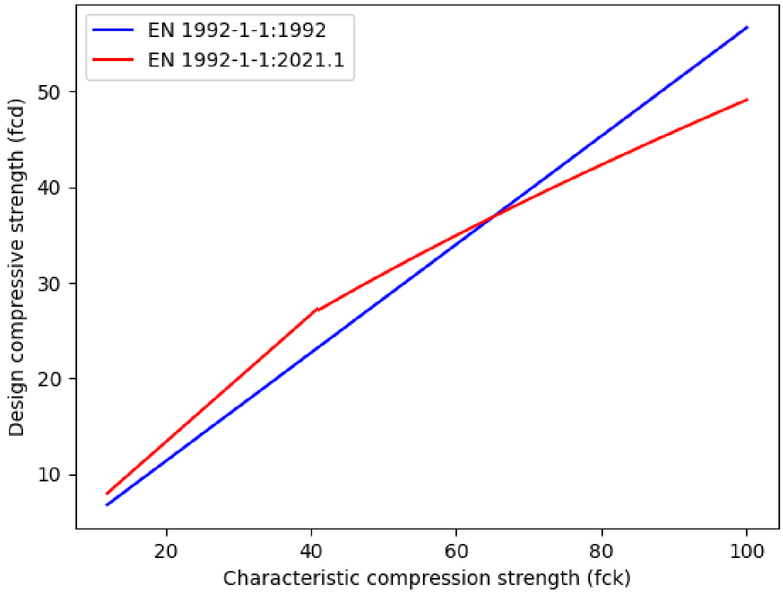



**Figure 2.2:** Comparison of compressive strength $f_{cd}$

Assuming concrete class CN and $t_{ref} \leq 28$ days, thus $\kappa_{tc} = 1.0$ and $\alpha_{cc} = 1.0$ as recommended in the revised and the current EC.2, respectively. By plotting the two expressions for $f_{cd}$, it is observed that the revised version of EC.2 provides higher $f_{cd}$ values for $f_{ck} < 60$ MPa, and lower $f_{cd}$ values for $f_{ck} > 60$ MPa. The pattern for the $f_{cd}$ values in the new version of EC.2 is caused by the factor $\eta_{cc}$, which has a limit of 1 for $f_{ck} \leq 40$ MPa, and descends below 1.0 for $f_{ck} > 40$ MPa. See figure 2.2.

### 2.1.2 Design tensile strength $f_{ctd}$

Design tensile strength, $f_{ctd}$ is determined as follows:

| EN 1992-1-1:2021-1 | EN 1992-1-1 |
|---|---|
| $f_{ctd} = \kappa_{tt} * \frac{f_{ctk,0,05}}{\gamma_c}$ | $f_{ctd} = \alpha_{ct} * \frac{f_{ctk,0,05}}{\gamma_c}$ |

**Table 2.2:** Provided expressions for $f_{ctd}$

$\kappa_{tt}$, corresponds to $\alpha_{ct}$, which is the factor considering long-term loads on concrete tensile strength. $\kappa_{tt}$ is determined according to [5.1.6(2)] based on cement type and $t_{ref}$. $\alpha_{ct}$ has a recommended value of 1.0 according to EN 1992-1-1.

The expression for $f_{ctk,0,05}$ is identical in both versions of EC.2, and it is expressed as follows:

$$f_{ctk,0,05} = 0.7 * f_{ctm}$$

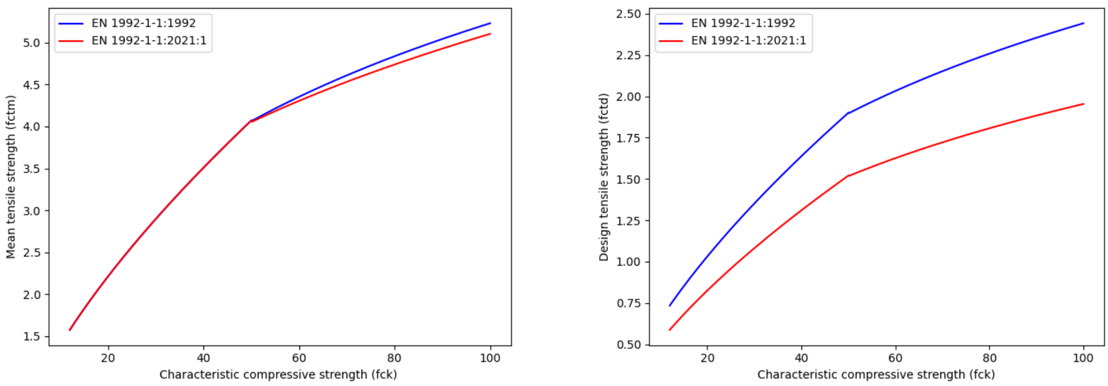
The expression for $f_{ctm}$ for ($f_{ck} \leq 50$) MPa is identical in both versions of EC.2, and it is expressed as follows:

$$f_{ctm} = 0.3 * f_{ck}^{\frac{2}{3}}$$

The expression for $f_{ctm}$ for $f_{ck} > 50$ MPa, is expressed differently in both versions of EC.2:

| **EN 1992-1-1:2021-1** | **EN 1992-1-1** |
|---|---|
| $f_{ctm} = 1,1 * f_{ck}^{\frac{1}{3}}$ | $f_{ctm} = 2.12 * ln(1 + \frac{f_{cm}}{10})$ |

**Table 2.3:** Provided expressions for $f_{ctm}$




**(a)** Comparison of mean tensile strength $f_{ctm}$

**(b)** Comparison of design tensile strength $f_{ctd}$

**Figure 2.3:** Comparison of $f_{ctm}$ and $f_{cd}$

Figure 2.3 (a) compares the mean axial concrete tensile strength expressions. The mean axial concrete tensile strength is exact for concrete classes B12 to B50. However, for concrete compressive strengths $f_{ck} > 50$ MPa, the new expression defined in the revised version of EC.2 provides lower values than the expression in the current EC.2.
The new expression for the design tensile strength is more conservative as it provides lower design tensile strength values for all concrete classes. The lower values can be explained by the lower recommended value for $\kappa_{tt} = 0.8$ (for $t_{ref} \leq 28$ days) than $\alpha_{ct} = 1.0$ as well as the lower $f_{ctm}$ values for $f_{ck} > 50$ MPa. See 2.3 (b). Hence, in the revision of EC.2, the utilization of concrete tensile strength is reduced relative to the current EC.2.

## 2.2 Stress-strain relation for non-linear design analysis

The clause [5.1.6 (3)] in EN 1992-1-1:2021-1 describes the stress-strain relation that may be used to model the response of concrete subjected to short-term uniaxial compression. The expression is identical to the one defined in clause [3.1.5 (1)] in EN 1992-1-1. The relation between $\sigma_c$ and $\epsilon_c$ is described as follows:

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta} \tag{2.5}$$

where:

$$\eta = \frac{\epsilon_c}{\epsilon_{c1}}$$

$$k = 1.05 E_{cm} * \frac{\epsilon_{c1}}{f_{cm}}$$

$\epsilon_{c1}$ is expressed as follows:

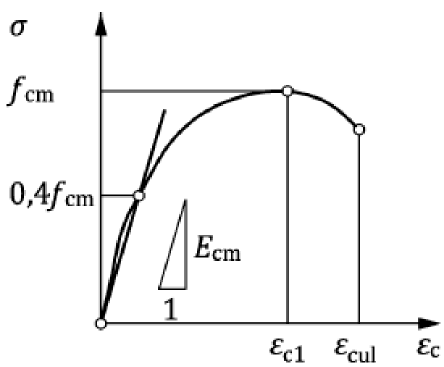
| **EN 1992-1-1:2021-1** | **EN 1992-1-1** |
|---|---|
| $\epsilon_{c1}[‰] = 0.7 f_{cm}^{\frac{1}{3}} \leq 2.8‰$ | $\epsilon_{c1}[‰] = 0.7 f_{cm}^{0.31} \leq 2.8‰$ |

**Table 2.4:** Provided expressions for $\epsilon_{c1}$

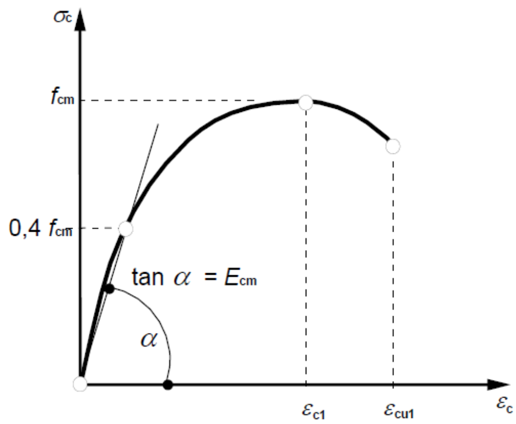
$\epsilon_c$ should be below $\epsilon_{cu1}$. Both versions have a different expression of $\epsilon_{cu1}$:

| **EN 1992-1-1:2021-1** | **EN 1992-1-1** |
|---|---|
| $\epsilon_{cu1}[‰] = 2.8 + 14(1 - \frac{f_{cm}}{108})^4 \leq 3.5[‰]$ | $\epsilon_{cu1}[‰] = 2.8 + 27(\frac{98 - f_{cm}}{100})^4$ |

**Table 2.5:** Provided expressions of $\epsilon_{cu1}$




**(a)** According to EN 1992-1-1:2021-1




**(b)** According to EN 1992-1-1 [3]

**Figure 2.4:** Stress-strain for concrete in compression

As shown in figure 2.4, both versions have the same approach for describing the stress-strain relation. However, the strains are determined differently, which will cause different values for stress-strain relation.

As seen in figure 2.5, that shows the stress-strain relation for $f_{ck} = 25$ MPa; the expression for stress-strain relation in EN 1992-1-1 have a steeper slope. The curve reaches maximum stress and then decreases with increasing strain. However, the expression in EN 1992-1-1:2021-1 have a less steep slope, reaches maximum stress more delayed, and slightly drops with increasing strain. Hence, the new version of EC.2 allows for more ductility relative to the current EC.2.

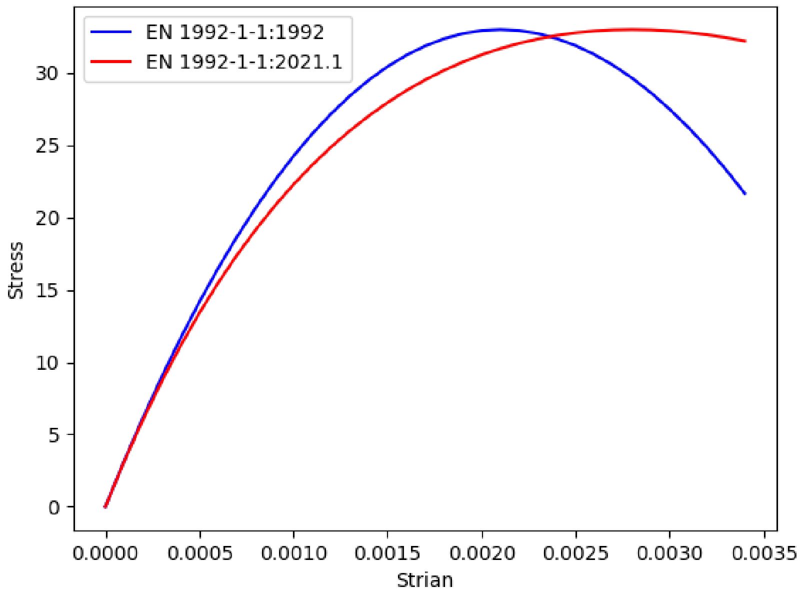



**Figure 2.5:** Stress strain relationship for non-linear analysis demonstrated for $\mathrm{f}_{ck} = 25MPa$

## 2.3 Young's modulus

EN 1992-1-1:2021-1 defines a new expression for modulus of elasticity, which intends to correct the expression defined in EN 1992-1-1. The expression for modulus of elasticity in EN 1992-1-1 was supposed to be a secant model; however, it presented tangent values. [4] Young's modulus is defined as:

| **EN 1992-1-1:2021-1** | **EN 1992-1-1** |
|---|---|
| $\mathrm{E}_{cm} = k_E * f_{cm}^{\frac{1}{3}}$ | $\mathrm{E}_{cm} = 22 * (\frac{f_{cm}}{10})^{0.3}$ |

**Table 2.6:** Provided expressions for young's modulus

In the new expression for young's modulus, the factor $k_E$ is introduced. $k_E$ considers and depends on the type of aggregate used in the concrete and can vary from 5 000-13 000. For quartzite aggregates, $k_E = 9500$ can be assumed. Clause [3.1.3(2)] in EN 1992-1-1 considers different aggregates and states that, $E_{cm}$ value should be reduced by 10% and 20% for aggregate of limestone and sandstone respectively. For basalt the value should be increased by 20%.

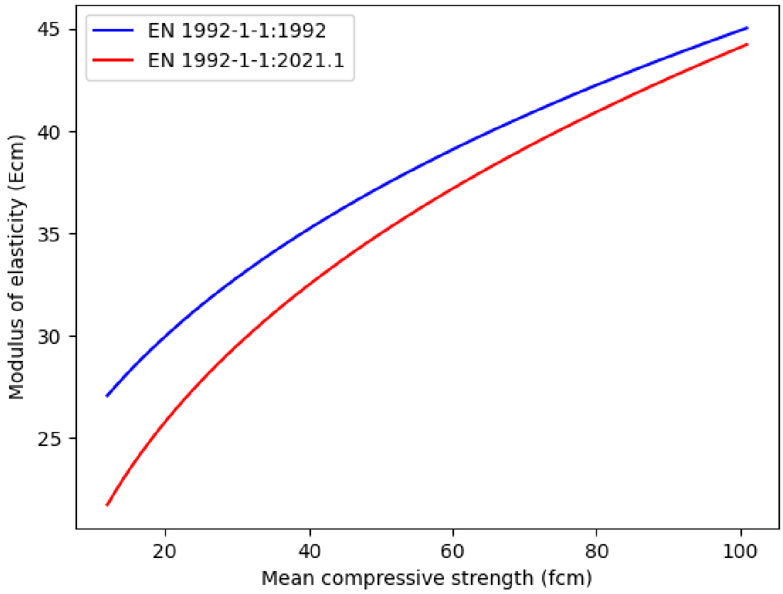



**Figure 2.6:** Comaprison of modulus of elasticity $E_{cm}$ at 28 days.

Figure 2.6 shows young's modulus values for different values of mean compressive strengths (at 28 days). By assuming quartzite aggregates, the new expression for $E_{cm}$ in EN 1992-1-1:2021 gives lower values than the expression in EN 1992-1-1, which implies, according to the new expression, concrete is less stiff (more elastic) than the expression defined in the current EC.2.

# Chapter 3

# Design in ultimate limit state (ULS)

## 3.1 Bending with or without axial force

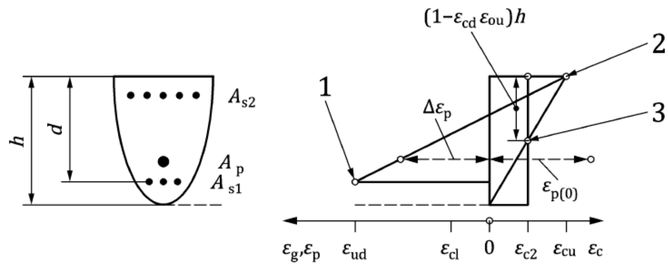
Eurocode assumptions, regulations, and design principles will be presented in this section and are only applicable for undisturbed regions of structural elements such as beams, slabs, and similar structural members for which sections stay approximately plane before and after loading. Discontinuous structural members or members where sections do not remain plane before and after loading will not be discussed here.

The following assumptions are made in the Eurocode when determining the ultimate moment resistance of reinforced concrete [5] [3]. These assumptions are identical in both EN 1992-1-1 and EN 1992-1-1:2021-1:

- Plane sections remain plane before and after loading.
- Bond strain in reinforcement is equal the bond strain in the surrounding concrete, whither in tension or compression.
- Tensile strength of concrete is ignored.
- Concrete compression stress is derived from the stress-strain relationship demonstrated in Eurocode. See section 3.1.3.
- Reinforcing steel or prestressing steel stresses are derived from the stress-strain relationship demonstrated in Eurocode.

### 3.1.1 Stress distribution

The possible strain distributions and the strain limits for respectively EN 1992-1-1 and EN 1992-1-1:2021-1 are shown in figure 3.1 and figure 3.2. EN 1992-1-1 and EN 1992-1-1:2021-1 have different approaches for defining the compression strain in concrete. EN 1992-1-1 states that the compression strain in the concrete should be limited to $\epsilon_{cu2}$ or $\epsilon_{cu3}$ depending on the used stress-strain diagram. However, EN 1992-1-1:2021-1 limits the compression strain to one value $\epsilon_{cu}$, unless the concrete is confined. The strains in the reinforcing steel and prestressed steel are identical in both versions of Eurocode 1992-1-1 and are limited to $\epsilon_{cd}$.

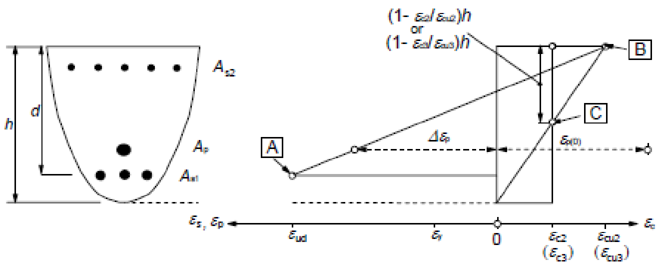



**Key**

1 Tensile strain limit of reinforcing steel

2 Compressive strain limit of concrete

**Figure 3.1:** Possible strain distributions in the ULS according to EN 1992-1-1:2021-1 [5]




A - reinforcing steel tension strain limit

B - concrete compression strain limit

C - concrete pure compression strain limit

**Figure 3.2:** Possible strain distributions in the ULS according to EN 1992-1-1:2004 [3]

### 3.1.2 Biaxial bending

The simplified criterion for the design of cross-sections exposed for biaxial bending when the design is not accurate is expressed in eq. [8.2] in EN 1992-1-1:2021-1 and in eq. [5.39] in EN 1992-1-1. The expressions are identical in both versions of EC.2.

$$\left(\frac{|M_{Edz}|}{M_{Rdz,N}}\right)^{a_N} + \left(\frac{|M_{Edy}|}{M_{Rdy,N}}\right)^{a_N} \leq 1.0 \tag{3.1}$$

$M_{Edz/y}$ is the design moment, including the 2. order moment, while $M_{Rdz/y,N}$ is the design resistance for the given axial forces. $a_N$ is an exponent. The values for $a_N$ are identical for both versions of EC.2. For circular and elliptical cross-sections, $a_N = 2$, while for rectangular cross-sections, $a_N$ is determined based on the utilization ratio between the axial design force $N_{Ed}$ and the design value for axial resistance under compression (without consideration of confinement) $N_{Rd,0}$. [5] $N_{Rd,0}$ corresponds to $N_{Rd}$ in EN 1992-1-1.

| $\frac{N_{Ed}}{N_{Rd,0}}$ | 0.1 | 0.7 | 1.0 |
|---|---|---|---|
| $a_N$ | 1.0 | 1.5 | 2 |

**Table 3.1:** Values for $a_N$ for rectangular cross sections [5]

$N_{Rd,0}$ is determined as follows:

$$N_{Rd,0} = A_c f_{cd} + A_s f_{yd} \tag{3.2}$$

### 3.1.3 Stress distributions in the compression zones

Stress distribution in the compression zones for the design of cross-sections is determined similarly in the two versions of EC.2. The only difference is that the exponent, n is a fixed value equal to 2 in EN 1992-1-1:2021-1, while in EN 1992-1-1, the exponent varies for different concrete classes and can be determined from table 3.1 in the code. The $\eta_{cc}$ factor, defined in the expression for $f_{cd}$, enables n to be a fixed value for all concrete classes.

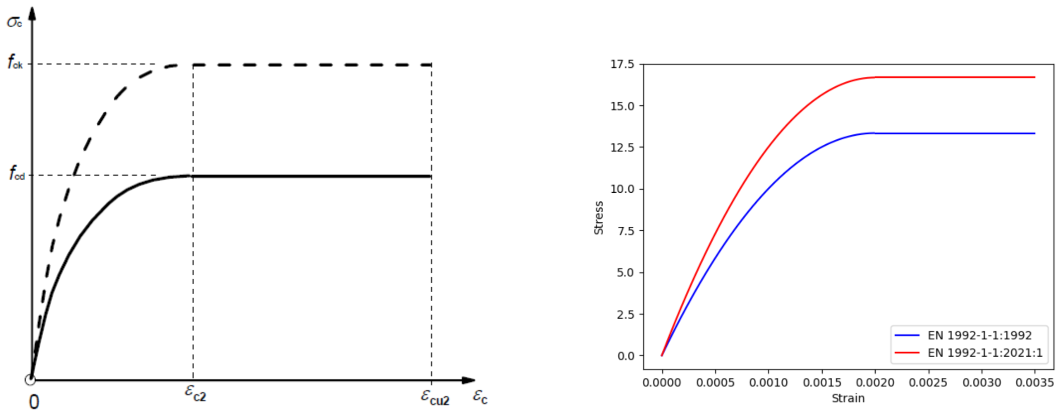
Stress-strain relation for $0 \leq \epsilon_c \leq \epsilon_{c2}$, is defined as follows:

$$\sigma_{cd} = f_{cd}[1 - (1 - \frac{\epsilon_c}{\epsilon_{c2}})^n] \tag{3.3}$$

While for $\epsilon_{c2} \leq \epsilon_c \leq \epsilon_{cu}$, the relation is defined as follows:

$$\sigma_{cd} = f_{cd} \tag{3.4}$$

$\epsilon_{c2}$ is the strain as the maximum stress is reached and $\epsilon_{cu}$ is the ultimate strain. EN 1992-1-1:2021 defines a fixed value for $\epsilon_{c2}$ and $\epsilon_{cu}$ as 0.002 and 0.0035, respectively. While in EN 1992-1-1 the values for $\epsilon_{c2}$ and $\epsilon_{cu}$ varies for different concrete classes, according to table 3.1 in the code [3].




**(a)** Parabola-rectangular stress distribution according to EN 1992-1-1 [3]

**(b)** Comparison of the stress-strain relation for the design of the cross-section (parabola-rectangle stress distribution)

**Figure 3.3:** Stress-strain for concrete in compression

Figure 3.3 (b) shows the stress-strain relation in the parabola-rectangle stress distribution model when $f_{ck} = 25$ MPa is assumed. The revised EC.2 provides higher $\sigma_c$ values relative to the current EC.2. The higher values is caused by the new expression for $f_{cd}$, which effects the stress-strain relationship. However, the pattern shown in 3.3 (b) will transform for other values of $f_{ck}$, and most significantly when $f_{ck} > 60$ MPa as EN 1992-1-1:2021-1 provides lower values for $f_{cd}$ than EN 1992-1-1. Consequently, the expression for stress-strain relation in EN

1992-1-1:2021 will induce lower $\sigma_c$ values than the current EC.2. See also figure 2.2.

Other stress-strain distributions could be used if equivalent to or more conservative than parabola-rectangle stress distribution, such as bi-linear or rectangular stress-strain distribution. [3]

In EN 1992-1-1, the following rectangular stress distribution is given:

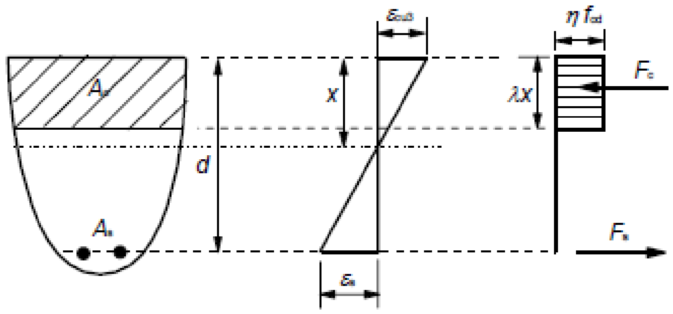



**Figure 3.4:** Rectangular stress distribution [3]

$\lambda$ is a factor defining the effective height in the compressive zone and is given as:

$$\lambda = 0.8 \qquad \text{for } f_{ck} \leq 50 \text{ MPa}$$
$$\lambda = 0.8 - \frac{f_{ck}-50}{400} \qquad \text{for } 50 < f_{ck} \leq 90$$

**Table 3.2:** $\lambda$ according to EN 1992-1-1:2004 [3]

$\eta$ is a factor defining the effective strength in the compression zone and is given as:

$$\eta = 1.0 \qquad \text{for } f_{ck} \leq 50 \text{ MPa}$$
$$\eta = 1,0 - \frac{f_{ck}-50}{200} \qquad \text{for } 50 < f_{ck} \leq 90$$

**Table 3.3:** $\eta$ according to EN 1992-1-1:2004 [3]

Figure 3.5 shows the rectangular and parabola rectangular stress distribution according to clause [8.1.2 (1)] in EN 1992-1-1:2021.

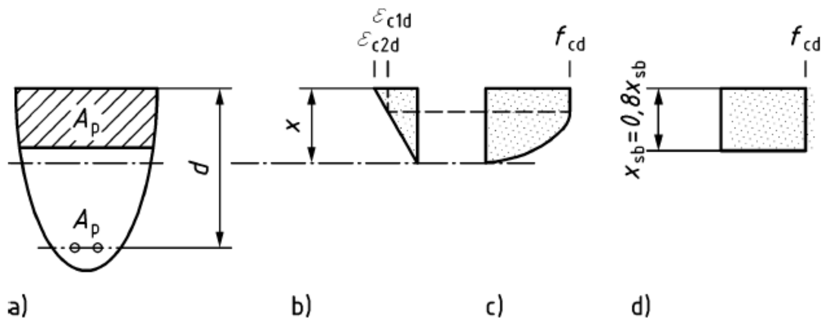
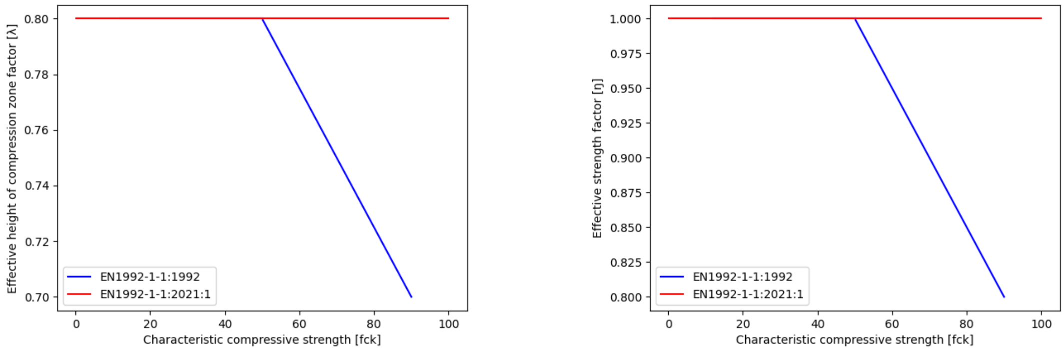



**Figure 3.5:** rectangular and parabola rectangular stress distribution [5]

(a) Is the cross-section.

(b) Is the assumed strain distribution.
(c) Is the parabola-rectangle stress distribution.
(d) Is the rectangular stress distribution.[5]

The rectangular stress distribution has been changed compared to EN 1992-1-1. The factor $\lambda$ is replaced with a fixed value equal to 0.8, while $\eta$ is removed, and the intensity of the distributed load in the compression zone simply equals $f_{cd}$.




**(a)** Comparison of the effective height factor of the compression zones $\lambda$

**(b)** Comparison of the effective strength factor of the compression zones $\eta$

**Figure 3.6:** Comparison of factors $\eta$, and $\lambda$ of the compression zones.

As shown in 3.6, the effective height and strength factors varies for $f_{ck} > 50MPa$ in EN 1992-1-1, while the factors are fixed in EN 1992-1-1:2021-1.

## 3.2 Shear

The shear section in EN 1992-1-1:2021-1 has been significantly revised. Several facets led to the significant revision done in EN 1992-1-1:2021. The size effect, k, defined in clause [6.2.2 (1)] in the current EC.2, underestimates such that the calculated shear capacity is larger than the tested capacity for larger effective depths ($\frac{\tau_{R,test}}{\tau_{EN1992-1-1:2004}}$) $< 1.0$. See figure 3.7 (a). [6]

Additionally, according to figure 3.7 (b), for cross-sections subjected to axial stress, the testing results give significantly higher shear capacity than the calculated capacity according to EN 1992-1-1. Which implies that the current expression for shear capacity for cross-sections subjected to axial tensile stresses ($\sigma_p > 0$) provides conservative values.

In EN 1992-1-1, the size parameter describing the crack and the failure zone roughness, $d_{dg}$, which takes concrete type and its aggregate properties into account, is neglected. The parameter $d_{dg}$ is taken into consideration in EN 1992-1-1:2021-1 and is consistently represented in the expressions for shear verification. [6]

Slenderness was not considered in the expressions for shear stress resistance in EN 1992-1-1, which can be critical for slender concrete elements, particularly beams. Thus, slenderness ought to be considered in the expressions for shear resistance. [6]
Furthermore, the expressions for shear capacity in EN 1992-1-1 are derived and calibrated based on testing results of a free-standing beam exposed to a concentrated load (the typical

way of testing shear capacity). This empirical method could be critical for other loading types and support conditions. [6]

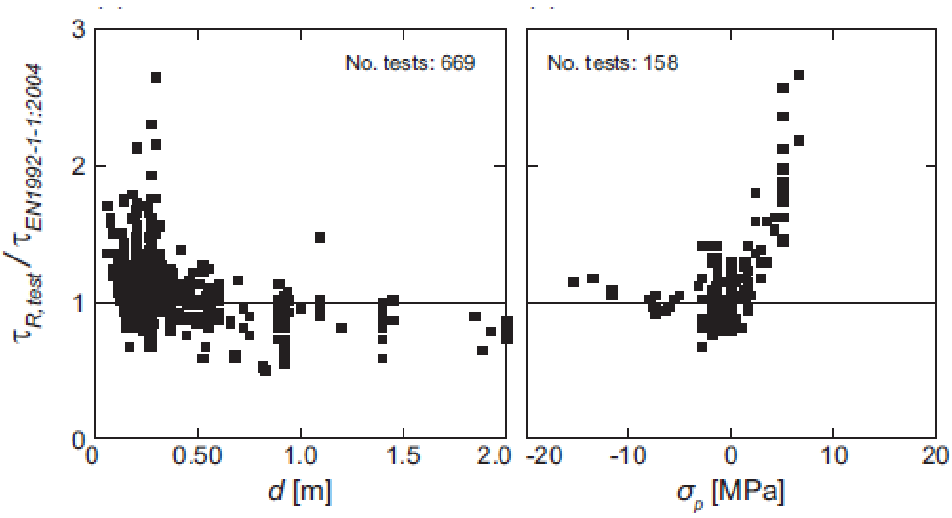



**Figure 3.7:** Results of $\frac{\tau_{R,test}}{\tau_{EN1992-1-1:2004}}$ as a function of effective depth (a) and axial stress (b) [6]

### 3.2.1 General rules and prerequisites for verification of shear

**General procedure for the verification of shear:**

EN 1992-1-1:2021-1 defines shear capacity as a stress value $\tau$ [MPa]. The general procedure for the verification of shear for linear members (such as beams and columns) and out-of-plane shear resistance of planar members are defined as follows: [5]

(a) Detailed verification of the shear capacity may be omitted when the design shear stress $\tau_{Ed}$ is less than or equal to the minimum shear stress resistance $\tau_{Rdc.min}$.

$$\tau_{Ed} \leq \tau_{Rdc,min} \tag{3.5}$$

(b) Shear reinforcement is not required in regions of the members; when shear stress resistance without reinforcement, $\tau_{Rdc}$ value is larger than or equal to the design shear stress $\tau_{Ed}$.

$$\tau_{Ed} \leq \tau_{Rdc} \tag{3.6}$$

(c) Otherwise, shear must be verified such that the design shear stress, $\tau_{Ed}$ value is less than or equal to the shear stress resistance with shear reinforcement $\tau_{Rd}$.

$$\tau_{Ed} \leq \tau_{Rd} \tag{3.7}$$

EN 1992-1-1 has the same procedure; however, it defines shear as a force $V$ [N]. The shear design force can be converted to shear stress as follows:

$$\tau_{Ed} = \frac{V_{Ed}}{b_w * z} \tag{3.8}$$

Z is lever arm and is defined according to clause [8.2 (3)] in EN 1992-1-1:2021 as 90% of the effective height, d. While $b_w$ is the width of the cross-section. For cross-sections with variable width or circular cross-sections, $b_w$ can be determined as shown in figure 3.8.

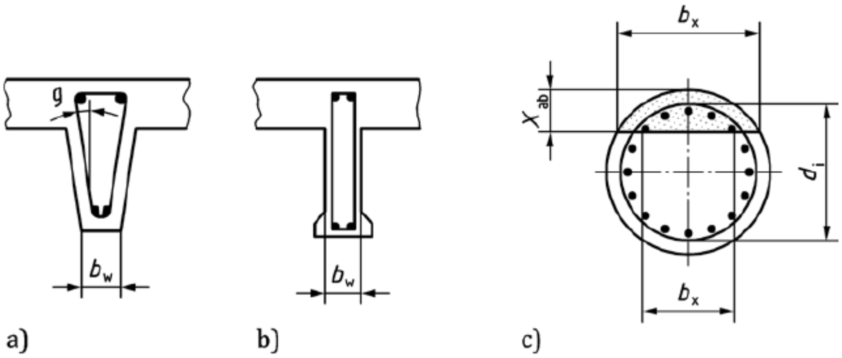



**Figure 3.8:** Cross section width $b_w$ for variable widths and circular cross-sections [5]

**Regions where detailed shear verification may be omitted:**

Clause [8.2.2 (1)] in EN 1992-1-1:2021-1 defines the regions where a detailed shear stress resistance verification may be omitted. See figure 3.9.


a) distributed loads b) concentrated loads

**Figure 3.9:** Regions where detailed verification of shear resistance may be omitted. [5]

As shown in figure 3.9, a detailed verification may be overlooked for cross-sections at a distance $\leq d$ from the face of the support or a concentrated load.
However, detailed verification of a control section at a distance equal to d should be executed when a concentrated load is applied at a distance $\leq 2d$ from the face of the support.[5]

**Definition for $v_{Ed}$ and effective depth d for planar member such as solid slabs and shells:**

Clause [8.2.1(5)] in EN 1992-1-1:2021 clarifies the definition of design shear $v_{Ed}$ and the effective depth d when out of plane shear forces $v_{Ed,x}$, and $v_{Ed,y}$ are acting on the cross-section perpendicular to the x- and y-direction, e.g. two-way slabs.
This clarification is a new edition compared to EN 1992-1-1. It clarifies the input value of the effective depth d that should be utilised in the expressions for shear verification, as well as the design shear $v_{Ed}$ that should be accordingly checked.

$v_{Ed}$ should be taken as:

$$v_{Ed} = \sqrt{v_{Ed,x}^2 + v_{Ed,y}^2} \tag{3.9}$$

The effective depth d is determined based on the relation $\frac{v_{Ed,y}}{v_{Ed,x}}$:

| | |
|---|---|
| for $\frac{v_{Ed,y}}{v_{Ed,x}} \leq 0.5$ | $\mathrm{d} = \mathrm{d}_x$ |
| for $0.5 < \frac{v_{Ed,y}}{v_{Ed,x}} < 2$ | $\mathrm{d} = 0.5 * (d_x + d_y)$ |
| for $\frac{v_{Ed,y}}{v_{Ed,x}} \geq 2$ | $\mathrm{d} = \mathrm{d}_y$ |

**Table 3.4:** Effective depth when $v_{Ed,x}$ and $v_{Ed,y}$ are acting on the cross-section

Alternatively, the effective depth may be determined from the following expression:

$$d = d_x * cos^2\alpha_v + d_y * sin^2\alpha_v \tag{3.10}$$

$\alpha_v$ is the angle between the principle shear force and the x-axis, and is determined as follows:

$$\alpha_v = arctan(\frac{V_{Ed,y}}{V_{Ed,x}}) \tag{3.11}$$

### 3.2.2 Design shear stress

**Reduced shear force near support:**

When the longitudinal reinforcement is fully anchored to the support, clause [6.2.2 (6)] in EN 1992-1-1 allows for reducing the shear force for loads acting at a distance $0.5d \leq \alpha_v \leq 2d$ from the edge of the support. When $\alpha_v$ is less than 0.5, $\alpha_v = 0.5$ should be used. See figure. 3.10

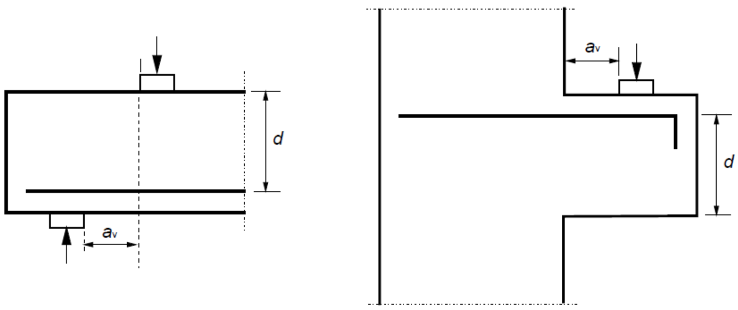



**Figure 3.10:** Loads near support for beams with direct support and corbels [3]

In order to determine the reduced force, the force $q_{Ed}$ should be multiplied by $\beta = \frac{\alpha_v}{2d}$. The reduced force may be expressed follows: [7]

$$\Delta q_{Ed} = (1 - \beta) q_{Ed} \tag{3.12}$$

Clause [8.2.2 (9)] in EN 1992-1-1:2021-1 allows for reducing the shear force. In case of concentrated forces pushing against each other (e.g. support force pushing against an applied load) within a distance $d \leq \alpha_q \leq 2d$, the force $q_{Ed}$ should be multiplied by $\frac{0.5*\alpha_q}{d}$.

The reduced force may be expressed as follows:

$$\Delta q_{Ed} = (1 - \frac{0.5\alpha_q}{d}) q_{Ed} \tag{3.13}$$

Thus, the maximum reduced shear for evenly distributed loads is achieved by applying the minimum allowed distance, which is $\alpha_v = 0.5d$ and $\alpha_q = d$ according to EN 1992-1-1 and EN 1992-1-1:2021-1, respectively.

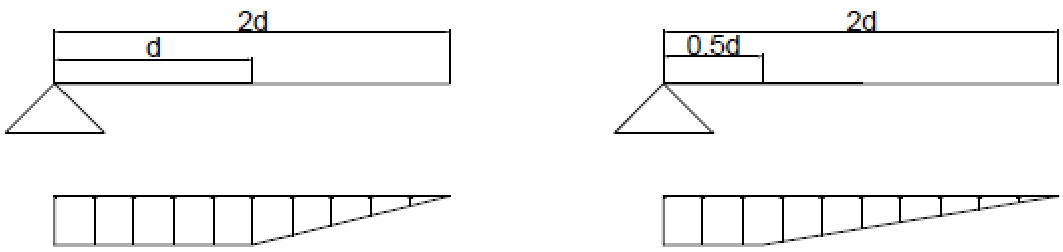



**Figure 3.11:** Illustration of the reduction of forces near support for distributed forces

| | |
|---|---|
| **EN 1992-1-1:** | $\Delta q_{Ed} = (1 - \frac{0.5d}{2d}) * q_{Ed} = 0.75 q_{Ed}$ |
| | $\Delta V_{Ed} = 0.75 * q_{Ed} * 0.5d + 0.75 * q_{Ed} * 1.5d * 0.5 = 0.94 q_{Ed} * d$ |
| **EN 1992-1-1:2021-1:** | $\Delta q_{Ed} = (1 - \frac{0.5*d}{d}) * q_{Ed} = 0.5 * q_{Ed}$ |
| | $\Delta V_{Ed} = 0.5 * q_{Ed} * d + 0.5 * q_{Ed} * d * 0.5 = 0.75 * q_{Ed} * d$ |

**Table 3.5:** Maximum allowed shear force

As seen in table 3.5, the reduction of shear forces near the support is more conservative in the revised version of EC.2 as it permits to reduce at a maximum of 75% of the shear forces near the support. Meanwhile, the current EC.2 permits for reducing at a maximum of 94% of the shear forces near the support.

**For distributed load (except high water- or gas pressure) on the tension side:**

Clause [8.2.2 (8)] in EN 1992-1-1:2021-1 describes the possible reduction of shear force at the control section in case of distributed loads (except for high water - or gas pressure) pushing against members on the tension side (e.g. gravity load on the top face of continuous members near intermediate support, or cantilever beams). See figure 3.12. [5] This is a new clause compared to EN 1992-1-1. The shear design force at a control section may be reduced by $\Delta V_{Ed} = q_{Ed} * d \leq \frac{1}{4} * V_{Ed}$. [5] The reduced shear design can then be defined as:

$$V_{Ed.red} = V_{Ed} - \Delta V_{Ed} = V_{Ed} - q_{Ed} * d \geq 0.75 * V_{Ed} \quad (3.14)$$

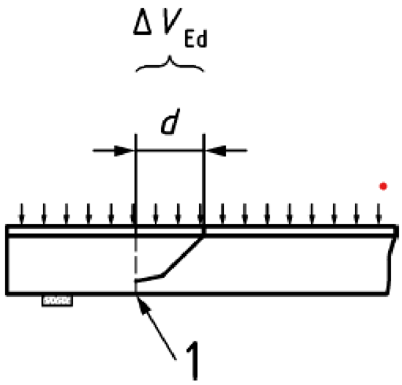



**Figure 3.12:** Distributed load pushing on the tension side of the member [5]

### 3.2.3 Shear resistance stress without shear reinforcement

The design expressions regarding shear and punching shear are based on critical shear crack theory (CSCT). CSCT is established on the assumption that shear capacity is governed by developing a critical shear crack that disturbs the shear stress transmission, thus limiting the shear capacity for a structural component. [6] The shear resistance stress without shear reinforcement is covered in this section.

The expressions regarding shear resistance in EN 1992-1-1:2021-1 are based on physical models rather than an empirical approach as in EN 1992-1-1. Consequently, the expressions are applicable for all load geometries and support conditions. [6]

Figure 3.13 indicates that the new expression for shear resistance without shear reinforcement in EN 1992-1-1:2021-1 provides more reasonable values for higher effective depths and cross-sections subjected to axial stresses relative to the current expression in EN 1992-1-1. See also figure 3.7.

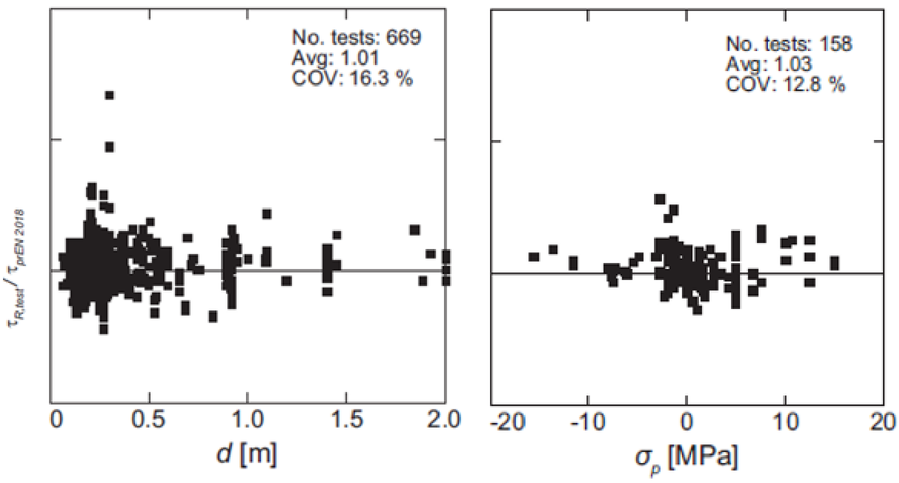



**Figure 3.13:** Results of $\frac{\tau_{R,test}}{\tau_{prEN2018}}$ as a function of effective depth (d) and axial stress $\sigma_p$ according to EN 1992-1-1:2021-1 [6]

The size effect $d_{dg}$ is considered and represented in the new expression for shear resistance without shear reinforcement. Moreover, the effect of slenderness is considered by the factor $\alpha_v$. $\alpha_v$ should replace the effective depth d in equation 3.17 in the case where the shear span $\alpha_{cs} < 4d$. Further $\alpha_v$ is defined according to clause [8.2.2 (3)] in EN 1992-1-1:2021-1 as follows:

$$\alpha_v = \sqrt{\frac{\alpha_{cs} * d}{4}} \tag{3.15}$$

The shear span $\alpha_{cs}$ is defined as a function of the internal forces $M_{Ed}$ and $V_{Ed}$ at the control section. Further, $\alpha_{cs}$ for sections without axial force is defined as follows:

$$\alpha_{cs} = \frac{|M_{Ed}|}{|V_{Ed}|} \geq d \tag{3.16}$$

Clause [8.2.2 (2)] in EN 1992-1-1:2021-1 defines the following expression for shear resistance stress:

$$\tau_{Rd,c} = \frac{0.66}{\gamma_v} * (100 * \rho_l * f_{ck} \frac{d_{dg}}{d})^{\frac{1}{3}} \geq \tau_{Rdc,min} \tag{3.17}$$

$A_{sl}$ is the effective area of tensile reinforcement at the distance d beyond the section considered. See figure 3.15.

$d_{dg}$, is the size parameter describing the failure zone roughness, which depends on the concrete type , and its aggregate properties. [5] See table 3.6.

$d_{lower}$ is defined as the smallest of the sieve size of the coarsest fraction of aggregates, specified in EN 206.

| | |
|---|---|
| $\rho_l$ | $= \frac{A_{sl}}{b_w * d}$ |
| $\gamma_v$ | $= 1.5$ |
| $d_{dg}$ for $f_{ck} \leq 60MPa$ | $= 16mm + D_{lower} \leq 40mm$ |
| $d_{dg}$ for $f_{ck} > 60MPa$ | $= 16mm + D_{lower} * (\frac{60}{f_{ck}})^4 \leq 40mm$ |

**Table 3.6:** Parameters defined in the expression for $\tau_{Rd,c}$ in EN 1992-1-1:2021-1

The expression for shear resistance stress, according to clause [6.2.2 (1)] in EN 1992-1-1 [6.2.2 (1)], is expressed as follows:

$$\tau_{Rd,c} = [C_{Rd,c} * k * (100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp}] \geq \tau_{Rdc,min} \tag{3.18}$$

$A_{sl}$ in EN 1992-1-1, is defined as the area of the tensile reinforcement, which extends $\geq (l_{bd} + d)$ beyond the considered section as shown in figure 3.14. [3] See table 3.7.

| | |
|---|---|
| k | $= 1 + \sqrt{\frac{200}{d}} \leq 2.0$ |
| $\rho_l$ | $= \frac{A_{sl}}{b_w * d} \leq 0.02$ |
| $C_{Rd,c}$ | $= \frac{k_2}{\gamma_c}$ |
| $\gamma_c$ | $= 1.5$ |
| $k_1$ | $= 0.15$ |
| $\sigma_{cp}$ | $= \frac{N_{Ed}}{A_c} < 0.2 f_{cd} [MPa]$ |

**Table 3.7:** Parameters defined in the expression for shear stress according according to, EN 1992-1-1

$k_2$ considers the largest aggregate, $D$, used in the concrete. $k_2 = 0.18$ for $D \geq 16$ mm and $k_2 = 0.15$ otherwise. [7]

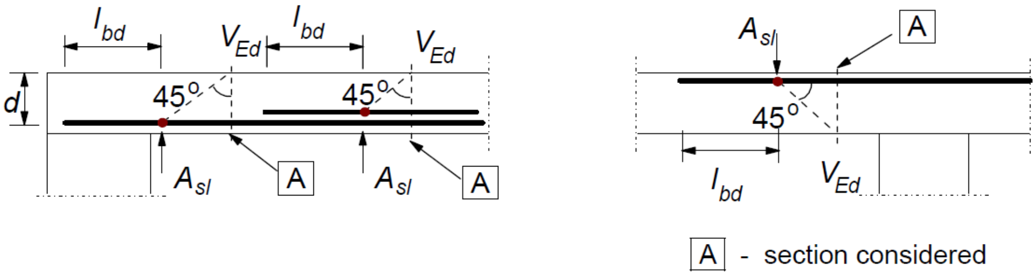



**Figure 3.14:** Definition of $A_{sl}$ in EN 1992-1-1 [3]

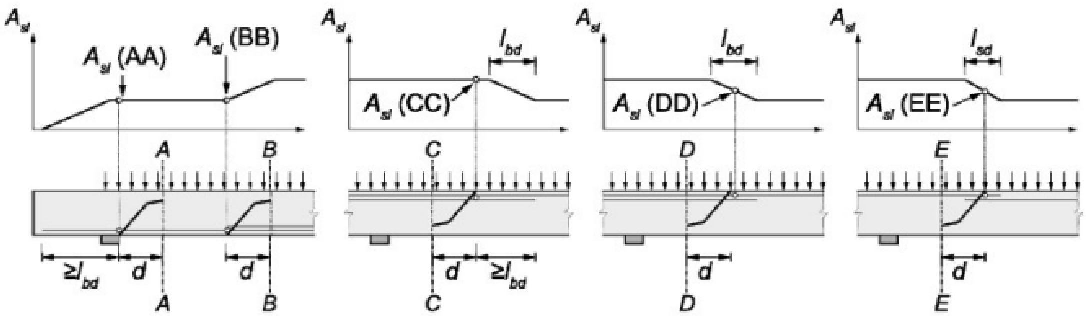


**Figure 3.15:** Definition of $A_{sl}$ in EN 1992-1-1:2021-1 [5]

In figure 3.15, sections A-A and C-C, anchored and curtailed reinforcement may be fully accounted for. For section B-B, curtailed reinforcement may not be accounted for, and for section D-D, curtailed and spliced reinforcement may be partially accounted for. [5]

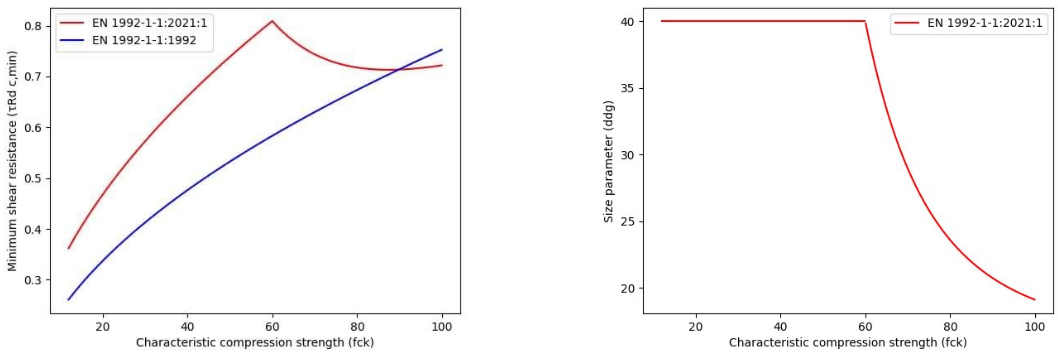
**Minimum shear resistance**

Minimum shear resistance according to clause [8.2.1(4)] in EN 1992-1-1:2021-1, is expressed as follows:

$$\tau_{Rdc,min} = \frac{11}{\gamma_v} * \sqrt{\frac{f_{ck}}{f_{yd}} * \frac{d_{dg}}{d}} \tag{3.19}$$

Minimum shear resistance, according to clause [6.2.2 (1)] in EN 1992-1-1, is expressed as follows:

$$\tau_{Rdc,min} = 0.035 * k^{\frac{3}{2}} * f_{ck}^{\frac{1}{2}} + k_1 * \sigma_{cp} \tag{3.20}$$




**(a)** $\tau_{Rdc,min}$ values for different $f_{ck}$ values

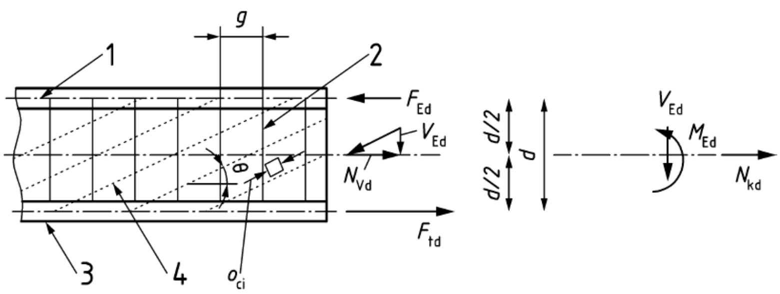
**(b)** Comparison of the effective strength factor of the compression zones $\eta$

**Figure 3.16:** Size parameter $d_{dg}$ for different values of $f_{ck}$

Figure 3.16 (a) is a comparison between the two expressions for the minimum shear resistance $\tau_{Rdc,min}$. The axial force is not considered, thus $k_1 + \sigma_{cp}$ is neglected, and $d_{lower} = 24$ mm is presumed. As seen in the figure, the new expression for minimum shear resistance in the revised version of EC.2 provides higher $\tau_{Rdc,min}$ values compared to the current expression in EN 1992-1-1.
The size parameter $d_{dg}$ decreases for $f_{ck} > 60$ MPa, and results the decreasing trend for $\tau_{Rdc,min}$ for $f_{ck} > 60$ MPa. See figure 3.16 (b).

### 3.2.4 Shear resistance with shear reinforcement

When shear capacity is insufficient $\tau_{Ed} > \tau_{Rd,c}$, shear reinforcement is required, and the criterion $\tau_{Ed} \leq \tau_{Rd}$, must be fulfilled. The approach for shear capacity with shear reinforcement should be based on a presumed truss model. See figure 3.17.




**Key**

1 Compression chord

2 Shear reinforcement

3 Tension chord

4 Struts (compression field)

**Figure 3.17:** Truss model, and keys for shear reinforced member [5]

**Stress resistance in the case of yielding shear reinforcement and shear reinforcement is perpendicular to the longitudinal reinforcement:**

According to clause [8.2.3 (4)] in EN 1992-1-1:2021-1, the expression for shear resistance stress in the case of yielding shear reinforcement and shear reinforcement are perpendicular to longitudinal reinforcement is expressed as follows:

$$\tau_{Rd,sy} = \rho_w * f_{ywd} * cot(\theta) \tag{3.21}$$

And the stress in the compression zone has to be controlled as follows:

$$\sigma_{cd} = \tau_{Ed}(cot\theta + tan\theta) \leq \nu * f_{cd} \tag{3.22}$$

$\rho_w$ is the shear reinforcement ratio and is defined as follows:

$$\rho_w = \frac{A_{sw}}{s * b_w} \tag{3.23}$$

$\theta$ is the angle between the concrete compression strut and the beam axis perpendicular to the shear force. The range of $cot(\theta)$ is defined as follows:

$$1 \leq cot\theta \leq cot\theta_{min} \tag{3.24}$$

cot($\theta_{min}$) can vary depending on the reinforcement ductility class and the forces acting on the member. See table 3.8.

| Case: for members | Class B or C | Class A |
|---|---|---|
| without axial force | =2.5 | Reduce by 20% |
| with significant axial compressive force * | = 3 | Reduce by 20% |
| with axial tension | $= 2.5 - 0.1 * \frac{N_{Ed}}{\lvert V_{Ed} \rvert} \geq 1$ | Reduce by 20% |

**Table 3.8:** Values of $\theta_{min}$ [5]

*When a significant axial compressive force is acting on the member, the following conditions are applicable: [5]

- The average axial compressive stress $\geq |3MPa|$.
- The depth of the compression cord, determined from the rectangular stress distribution and the strain distribution is $\leq 0.25d$.

In clause [6.2.3(3)] in EN 1992-1-1 , the expression for shear resistance in the case of yielding shear reinforcement, when vertical shear reinforcement is used, is defined as follows:

$$V_{Rd,s} = \frac{A_{sw}}{S} * Z * f_{ywd} * cot(\theta) \tag{3.25}$$

The shear resistance for vertical reinforcement has the upper limit $v_{Rd,max}$, defined as follows:

$$V_{Rd,max} = \alpha_{cw} * b_w * Z * v_1 \frac{f_{cd}}{cot\theta + tan\theta} \tag{3.26}$$

Where $A_{sw}$ is the cross-section area of the shear reinforcement, and $S$ is the spacing between stirrups. $v_1$ is the strength reduction factor and is determined as follows:

| | |
|---|---|
| For $f_{ck} \leq 60$ MPa | $v_1 = 0.6$ |
| For $f_{ck} \geq 60$ MPa | $v_1 = 0.9 - \frac{f_{ck}}{200} > 0.5$ |

**Table 3.9:** Strength reduction factor $v_1$

$\alpha_{cw}$ is the coefficient taking the state of stress in the compression chord into account. $\alpha_{cw}$ is determined, depending on the value of the mean compressive stress $\sigma_{cp}$ where $\sigma_{cp}$ is calculated at a distance $\leq 0.5 * d * cot\theta$ from support:

| | |
|---|---|
| For $0 < \sigma_{cp} \leq 0.25 * f_{cd}$ | $\alpha_{cw} = (1 + \frac{\sigma_{cp}}{f_{cd}})$ |
| For $0.25 f_{cd} < \sigma_{cp} \leq 0.5 * f_{cd}$ | $\alpha_{cw} = 1.25$ |
| For $0.5 f_{cd} < \sigma_{cp} < f_{cd}$ | $\alpha_{cw} = 2.5(1 - \frac{\sigma_{cp}}{f_{cd}})$ |

**Table 3.10:** Expressions for $\alpha_{cw}$ depending on mean compressive stress $\sigma_{cp}$

EN 1992-1-1, unlike EN 1992-1-1:2021-1, defines a fixed upper limit for $cot\theta$. According to clause [6.2.3 (2)], $cot\theta$ is limited as follows:

$$1 \leq cot\theta \leq 2.5 \tag{3.27}$$

**$\tau_{Rd}$, in the case where the shear reinforcement yields at the same time as the compression field fails according to EN 1992-1-1:2021:**

Eq. [8.30] in EN 1992-1-1:2021-1, is the expression for shear stress resistance in the case when the shear reinforcements yield $\epsilon_s = \epsilon_{yd}$, and the compression field fails, $\epsilon_c = \epsilon_{cu}$. The expression is defined as follows:

$$\tau_{Rd} = \rho_w * f_{ywd} * cot\theta \leq \frac{v * f_{cd}}{2} \tag{3.28}$$

Where $cot\theta$, is determined from the following expression:

$$cot\theta_{min} \geq cot\theta = \sqrt{\frac{\nu * f_{cd}}{\rho_w * f_{ywd}} - 1}$$

$\nu$ is the strength reduction factor for concrete cracked due to shear or other actions. The factor $\nu$ may be taken as $\nu = 0.5$ when using the $\theta_{min}$, as defined in table 3.8. When, $\theta < \theta_{min}$, v can be estimated from the following expression. [5]

$$\nu = \frac{1}{1.0 + 110 * (\epsilon_x + (\epsilon_x + 0.001) * cot^2\theta)} \leq 1.0$$

$\epsilon_x$ is the average strain of top and bottom chords calculated at a distance $\geq 0.5 * z * cot\theta$ from a support or a concentrated load. $\epsilon_x$ is determined as follows:

$$\epsilon_x = \frac{\epsilon_{xt} + \epsilon_{xc}}{2} \geq 0$$

Assuming elastic behaviour: $\epsilon_{xt} = \frac{F_{td}}{A_{st} * E_s}$

If $f_{cd} > 0$ (Flexural chord in compression): $\epsilon_{xc} = \frac{F_{cd}}{Acc * E_c}$

If $f_{cd} < 0$ (Flexural chord in tension): $\epsilon_{xs} = \frac{-F_{cd}}{A_{sc} * E_s}$

Chord force, $F_{td}$: $F_{td} = \frac{M_{Ed}}{z} + \frac{N_{vd} + N_{Ed}}{2} \leq \frac{M_{Ed,max}}{z} + \frac{N_{Ed}}{2}$

Chord force, $F_{cd}$: $F_{cd} = \frac{M_{Ed}}{z} - \frac{N_{vd} + N_{Ed}}{2}$

The additional tensile force, $N_{vd}$: $N_{vd} = |V_{Ed}| * cot\theta$

**Shear stress resistance when shear reinforcement is inclined**

Clause [8.2.3(13)] in EN 1992-1-1:2021-1, defines the following expression for inclined shear reinforcement, $45 \leq \alpha_W < 90$:

$$\tau_{Rd,sy} = \rho_w * f_{ywd} * (cot\theta + cot\alpha_W) * sin\alpha_w \quad (3.29)$$

The stress in the compression zone has to be controlled for inclined shear reinforcement as follows:

$$\sigma_{cd} = \tau_{Ed} * \frac{1 + cot^2\theta}{cot\theta + cot\alpha_w} \leq \nu * f_{cd} \quad (3.30)$$

Where $cot\theta$, is limited as follows:

$$tan\frac{\alpha_W}{2} \leq cot\theta \leq cot\theta_{min} \quad (3.31)$$

Clause [6.2.3] in EN 1992-1-1 defines the following expression for inclined shear reinforcement:

$$V_{Rd,s} = \frac{A_{sw}}{s} * Z * f_{ywd} * (cot\theta + cot\alpha) * sin\alpha \quad (3.32)$$

The shear resistance for inclined shear reinforcement has to be controlled such that $V_{Rd,s} \leq V_{Rd,max}$, where $V_{Rd,max}$ is defined as follows:

$$v_{Rd,max} = \alpha_{cw} * b_w * Z * v_1 * f_{cd} * \frac{cot\theta + cot\alpha}{1 + cot^2\theta} \tag{3.33}$$

$cot\theta$ is limited similarly to vertical shear reinforcement.

## 3.3 Punching shear

Punching shear occurs when a concentrated load acts on a small area, such as a concentrated load from columns against a slab. These areas have to be controlled for punching shear. Clause [8.4 (2)] in EN 1992-1-1:2021 defines the following procedure for punching shear verification:

(a) Detailed verification of punching shear is not required when the following condition is satisfied outside the control perimeter.

$$\tau_{Ed} \leq \tau_{Rdc.min}$$

$\tau_{Rdc.min}$ is determined as shown in chapter 4.2.2.

(b) Punching shear reinforcement is not required when:

$$\tau_{Ed} \leq \tau_{Rdc}$$

(c) If (b) is not satisfied, an upper limit for punching resistance at the control perimeter is defined and may not be exceeded.

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

(d) Further, the member has to be reinforced for punching shear and satisfy the following:

$$\tau_{Ed} \leq \tau_{Rd,cs}$$

(e) If punching shear is required, as a further control, a new control perimeter should be defined and checked for punching.

### 3.3.1 Control perimeters, the effective depth of slabs, and design shear stress

**Control perimeter**

According to clause [6.4.2(1)] in EN 1992-1-1, a critical control perimeter with a circumference $u_1$ and distance $2.0d$ from the loaded surface may be taken and should be constructed such that the length $u_1$ is minimized.

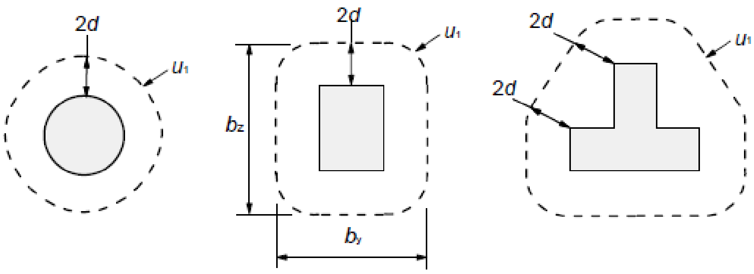



**Figure 3.18:** typical control perimeters [3]

Clause [8.4.2(2)] in EN 1992-1-1:2021 suggests that the control perimeter may be taken $0.5 * d_v$ and should be constructed such that the circumference $b_{0.5}$ is minimized. See figure 3.19.

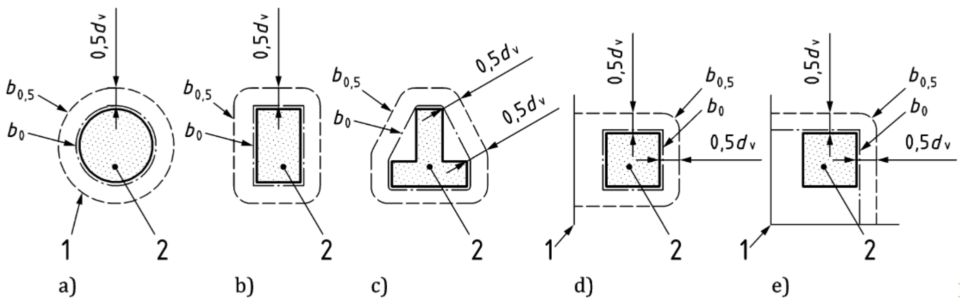



**Figure 3.19:** Typical control perimeters $b_{0.5}$ and $b_0$[5]

1) is the control perimeter, 2) is the supporting area

**Effective depth**

According to eq. [6.32] in EN 1992-1-1, the effective depth of the slab is presumed to be constant and is defined as follows:

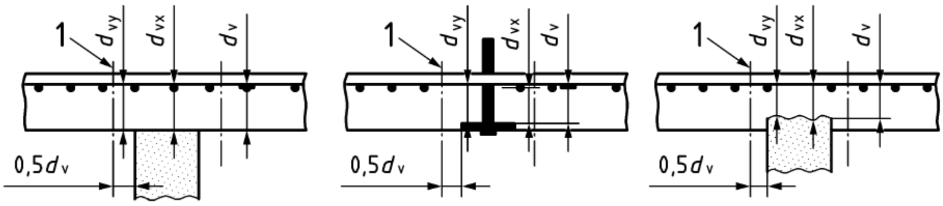
$$d_{eff} = \frac{d_y + d_z}{2} \tag{3.34}$$

Where $d_y$ and $d_z$ are the effective depth of the reinforcement in two orthogonal directions.

According to EN 1992-1-1:2021-1, the effective depth $d_v$ is defined as the distance from support to the average level of reinforcement layers and is expressed as follows:

$$d_v = \frac{d_{vx} + d_{vy}}{2} \tag{3.35}$$

The nominal effective depth $d_{nom}$, or the design effective depth $d_d$ can be used for $d_{vx}$ and $d_{vy}$. Figure 3.20, shows definitions for $d_{vx}$ and $d_{vy}$ for different supporting levels.




a) Direct support b) Hanging support with penetration c) Penetration by direct support

**Figure 3.20:** Shear resistance effective depth of slabs considering different supporting levels [5]

**Design punching shear stress**

The design punching shear stress is based on the shear distribution along the control perimeter. In EN 1992-1-1, the design shear stress when the support reaction is eccentric to the control perimeter is defined as follows:

$$v_{Ed} = \beta \frac{V_{Ed}}{u_1 * d_{eff}} \tag{3.36}$$

The design punching shear stress in EN 1992-1-1:2021 is defined as follows:

$$\tau_{Ed} = \beta_e \frac{V_{Ed}}{b_{0.5} * d_v} \tag{3.37}$$

**The $\beta$ factor**

The factor $\beta_e$ considers the concentrations of forces. The factor can be approximated if they satisfy the conditions listed in clause [8.4.2(6)]. Otherwise, they can be calculated for all cases as shown in figure 3.21. The table gives a good overview of $\beta_e$ values and expressions for several support locations.

| Support | Approximated | Refined[a] | |
|---|---|---|---|
| internal columns | $\beta_e = 1{,}15$ | $\beta_e = 1 + 1{,}1\frac{e_b}{b_b}$ | where $e_b = \sqrt{e_{b,x}^2 + e_{b,y}^2}$ |
| edge columns | $\beta_e = 1{,}4$ | | where $e_b = 0{,}5\lvert e_{b,x}\rvert + \lvert e_{b,y}\rvert$ |
| corner columns | $\beta_e = 1{,}5$ | | where $e_b = 0{,}27(\lvert e_{b,x}\rvert + \lvert e_{b,y}\rvert) \leq 0{,}45 \cdot b_b$ |
| ends of walls | $\beta_e = 1{,}4$ | | |
| corners of walls | $\beta_e = 1{,}2$ | | |

**Figure 3.21:** Values for $\beta_E$ according to EN 1992-1-1:2021-1 [5]

The factor $\beta$ in EN 1992-1-1 considers the increased distributed shear due to the unbalanced moment from the column. The factor is dependent on the geometry of the critical control perimeter, column dimensions, and the value of the moment that has to be balanced. [7] $\beta$ is defined as:

$$\beta = 1 + k * \frac{M_{Ed}}{V_{Ed}} * \frac{u_1}{W_1} \tag{3.38}$$

Where $W_1$ corresponds to the distribution of shear, for rectangular cross section $W_1$ is defined as:

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 4c_2 d + +16d^2 + 2\pi d c_1 \tag{3.39}$$

The factor k is determined based on the ratio between the column dimension parallel ($c_1$) and perpendicular ($c_2$) to the eccentricity of the load. See figure 3.22

| $c_1/c_2$ | ≤ 0,5 | 1,0 | 2,0 | ≥ 3,0 |
|---|---|---|---|---|
| $k$ | 0,45 | 0,60 | 0,70 | 0,80 |

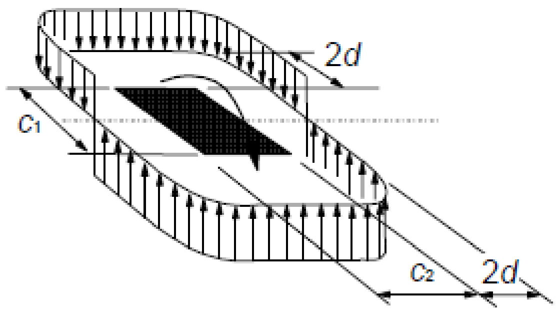



**Figure 3.22:** k-factors and shear distribution due to an unbalanced moment at a slab-internal column connection [3]

### 3.3.2 Punching shear resistance of slabs without shear reinforcement

Clause [8.4.3 (1)] in EN 1992-1-1:2021 defines the expression for shear resistance of slabs without shear reinforcement as follows:

$$\tau_{Rd,c} = \frac{0.6}{\gamma_V} * k_{pb} * (100 * \rho_l * f_{ck} * \frac{d_{dg}}{d_v})^{\frac{1}{3}} \leq \frac{0.6}{\gamma_v} * \sqrt{f_{ck}} \tag{3.40}$$

Where $k_{pb}$ is an enhancement coefficient for punching shear gradient, that takes the adjusted control perimeter $\frac{b_0}{b_{0.5}}$ into account. $k_{pb}$ is defined as follows:

$$1 \leq k_{pb} = 3.6\sqrt{1 - \frac{b_0}{b_{0.5}}} \leq 2.5$$

The expression for the reinforcement ratio $\rho_l$ is defined as follows:

$$\rho_l = \sqrt{\rho_{l,x} * \rho_{l,y}}$$

Where $\rho_{l,x}$ and $\rho_{l,y}$ are the mean values for the bonded flexural reinforcement ratios in the x and y-direction, respectively.

Clause [6.4.4(1)] in EN 1992-1-1 defines the shear resistance of slabs without shear reinforcement as follows:

$$V_{Rd,c} = C_{Rd,c} * k(100 * \rho_l * f_{ck})^{\frac{1}{3}} + k_1 * \sigma_{cp} \geq (v_{min} + k_1 * \sigma_{cp}) \tag{3.41}$$

### 3.3.3 Punching shear resistance with shear reinforcement

Where shear reinforcement is required, the reinforcement should be calculated according to clause [8.4.4 (1)] as follows:

$$\tau_{Rd,cs} = \eta_c * \tau_{Rd,c} + \eta_s * \rho_w * f_{ywd} \geq \rho_w * f_{ywd} \tag{3.42}$$

Where $\eta_c$ and $\eta_s$ are strength reduction coefficients for shear resistance $\tau_{Rdc}$ and the contribution of shear reinforcement, respectively.

$$\eta_c = \frac{\tau_{Rd,c}}{\tau_{Ed}}$$

$$\eta_s = \frac{d_v}{150\phi_v} + (15 * \frac{d_{dg}}{d_v})^{\frac{1}{3}} * (\frac{1}{\eta_c * k_{pb}})^{\frac{3}{2}} \leq 0.8$$

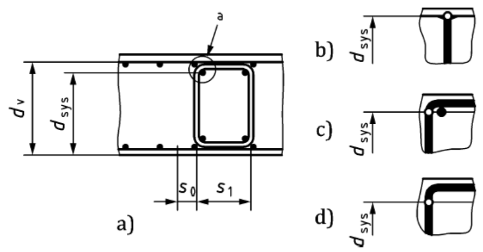
For inclined shear reinforcement, according to clause [8.4.4 (2)], the term $\rho_w$ should be multiplied by $(sin\alpha_w + cos\alpha_w)$.

The shear resistance is limited by $\tau_{Rd,max}$, which is defined as follows:

$$\tau_{Rd.max} = \eta_{sys} * \tau_{Rd,c} \tag{3.43}$$

Where the coefficient $\eta_{sys}$ considers the performance of punching shear reinforcement systems. $\eta_{sys}$ is defined as follows:

$$\eta_{sys} = 1.15 * \frac{d_{sys}}{d_v} + 0.63 * (\frac{b_0}{d_v})^{\frac{1}{4}} - 0.85 * \frac{s_0}{d_{sys}}$$




a) See details in b) to d) | b) Top of head | c) Level of axis of reinforcement inside the bend | d) Limit of bend

**Figure 3.23:** Definition of $d_{sys}$ and $s_0$ [5]

Clause [6.4.5 (1)] in EN 1992-1-1 defines the shear resistance with shear reinforcement as follows:

$$V_{Rd,cs} = 0.75 * V_{Rd,c} + 1.5(\frac{d}{s_r}) * A_{sw} * f_{ywd,eff}(\frac{1}{u_1 * d}) * sin\alpha \tag{3.44}$$

The shear resistance upper limit $V_{Rd,max} = 0.5 * \nu * f_{cd}$.

The punching shear resistance with shear reinforcement consists of the concrete and the reinforcement contributions to the shear resistance. The first term in the eq. 3.42 ($\eta_c * \tau_{Rd,c}$), and ($0.75 * V_{Rd.c}$) in eq. 3.44, describes the concrete contribution. By comparing the two terms, EN 1992-1-1 uses a fixed value of 75% of the concrete contribution. Meanwhile, EN 1992-1-1:2021-1 defines the contribution based on the utilization ratio of the shear resistance stress capacity without reinforcement and the design shear stress. This implies that where the design stress is approximately equal to the shear resistance capacity (without shear reinforcement) but still not sufficient, the contribution of concrete to the shear resistance is still fully accounted for. Hence, the new term for the concrete contribution will result in more accurate required amounts of reinforcement.

# Chapter 4

# Design in serviceability limit state, SLS

## 4.1 Deflection control

The deformation of a structural component or a structure should not affect the intended functioning or appearance. Thus, the deflection of a structure must be limited. The upper limit for deflection is $\frac{span}{250}$, in both versions of EC.2.

**Cases where deflection control may be omitted**

Both versions of EC.2 define a method for whether deflection control may or may not be omitted. The methods are based on limiting the span/depth ratio.

EN 1992-1-1 defines an expression for determining the span/depth ratio. The expression takes different structural systems into account through the factor $K$. The expression also considers the required tension reinforcement $\rho$ or compression reinforcement $\rho'$ to resist the design moment (at midspan for continuous or simply supported elements and at support for cantilevers).

When $\rho \leq \rho_0$ ( $\rho_0$ is the reference reinforcement ratio= $\sqrt{f_{ck}} * 10^{-3}$), the expression, according to clause [7.4.2 (2)], is defined as follows:

$$\frac{l}{d} = K * [11 + 1.5\sqrt{f_{ck}}\frac{\rho_0}{\rho} + 3.2\sqrt{f_{ck}}(\frac{\rho_0}{\rho} - 1)^{\frac{3}{2}}] \tag{4.1}$$

For $\rho > \rho_0$:

$$\frac{l}{d} = K * [11 + 1.5 * \sqrt{f_{ck}} * \frac{\rho_0}{\rho - \rho'} + \frac{1}{12} * \sqrt{f_{ck}} * \sqrt{\frac{\rho'}{\rho_0}}] \tag{4.2}$$

The required tension and compression reinforcement, respectively, may be defined as follows:

$$\rho = \frac{A_s}{bd} \tag{4.3}$$

$$\rho' = \frac{A_s'}{bd} \tag{4.4}$$

EN 1992-1-1:2021-1 have a similar approach as EN 1992-1-1. However, it provides tabulated values. Thus, fewer calculations are required to determine the span/depth ratio, simplifying the procedure relative to EN 1992-1-1. See figure 4.1.

| | Structural system | Required mechanical reinforcement ratio[a] | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | $\omega_r = 0{,}3$ | | | $\omega_r = 0{,}2$ | | | $\omega_r = 0{,}1$ | | |
| | | $LL/TL$[b] | | | $LL/TL$[b] | | | $LL/TL$[b] | | |
| | | 60 % | 45 % | 30 % | 60 % | 45 % | 30 % | 60 % | 45 % | 30 % |
| 1 | Simply supported beam, one-way spanning simply supported slab | 15 | 14 | 13 | 17 | 16 | 14 | 24 | 22 | 21 |
| 2 | End span of continuous beam or one-way spanning slab | 20 | 18 | 17 | 22 | 21 | 18 | 31 | 29 | 27 |
| 3 | Interior span of beam or one-way spanning slab | 23 | 21 | 20 | 26 | 24 | 21 | 36 | 33 | 32 |
| 4 | Cantilever | 6 | 5 | 5 | 6 | 6 | 5 | 9 | 8 | 8 |

**Figure 4.1:** Deflection control by limiting span/depth-ratio

Firstly, $w_r$ is determined. $w_r$ the required tension reinforcement to resist the moment due to design load (at midspan for simply supported elements and at support for cantilever elements). $w_r$ may be calculated as follows:

$$w_r = \frac{f_{yd}}{f_{cd}} * \frac{A_s}{A_c} \tag{4.5}$$

Secondly, the ratio $LL/TL$ is determined. LL is the live load, while TL is the total load on the structure. The span depth ratio may then be taken as shown in figure 4.1. Deflection control may be omitted if the following condition is satisfied:

$$\frac{l}{d}(actual) \leq \frac{l}{d}(tabulated) \tag{4.6}$$

For two-way slabs, the values in table 4.1 should be multiplied by the following:

$$\sqrt[4]{\frac{1}{1+(\frac{l_{min}}{l_{max}})^4}} \tag{4.7}$$

### Simplified calculation of deflection

Both versions of EC.2 define a general method for the deflection calculation. The general method is challenging and complex. Hence, EN 1992-1-1:2021-1 includes a simplified deflection calculation method in clause [9.3.3(2)]. The method is based on linear elastic analysis and assumes long-term properties $E_{c,eff}$.

The long-term deflection is determined as follows:

$$\delta = \frac{1}{K_I} * (\frac{h}{d})^{\frac{1}{3}} * [k_w * \delta_{load} + k_s * \delta_{\epsilon cs}] \tag{4.8}$$

The equation consists of the deflection contribution due to quasi-permanent combination of action $\delta_{load}$ (with $\psi_2 = 0.3$), and the deflection contribution due to differential shrinkage $\delta_{\epsilon cs}$. Both are determined for uncracked conditions. The cracking is then adjusted through the factor $k_I$ and $k_s$, while $k_w$ takes the over-reinforcement effect on the deflection into account.

## 4.2 Crack control

Cracking should be limited such that it does not affect structural elements or structural functioning, durability, and appearance.

Both version of EC.2 provide tables showing the allowed crack width $w_{max}$ based on the exposure class and the type of reinforcement. $k_c$ considers the effect of greater nominal cover $c_{nom}$ than the durability required cover $c_{min,dur}$. $k_{surf}$, considers the difference between increased crack width at the member surface and the required mean crack width. See figure 4.2, and 4.3.

$$k_c = \frac{c_{nom}}{c_{min,dur}} \leq 1.3 \tag{4.9}$$

$$1.0 \leq k_{surf} = \frac{c_{act}}{10mm + c_{min,dur}} \leq 1.5 \tag{4.10}$$

$c_{act}$ is the specified concrete cover $c_{act} \geq c_{nom}$.

<table>
<tr><th rowspan="2">Exposure Class</th><th colspan="2">Reinforced members and prestressed members with unbonded tendons</th><th colspan="2">Prestressed members with bonded tendons [c)]</th></tr>
<tr><th>Load combination</th><th>Limiting value</th><th>Load combination</th><th>Limiting value</th></tr>
<tr><td>X0</td><td>Quasi-permanent</td><td>0,40[a)]</td><td>Frequent</td><td>0,30 $k_c$</td></tr>
<tr><td>XC1, XC2, XC3, XC4</td><td>Quasi-permanent</td><td>0,30 $k_c$</td><td>Frequent</td><td>0,20 $k_c$</td></tr>
<tr><td rowspan="2">XD1, XD2, XS1, XS2</td><td rowspan="2">Quasi-permanent</td><td rowspan="2">0,30 $k_c$</td><td>Frequent</td><td>0,20 $k_c$</td></tr>
<tr><td>Quasi-permanent</td><td>Decompression[b)]</td></tr>
<tr><td>XD3, XS3</td><td>Frequent</td><td>0,30 $k_c$</td><td>Frequent</td><td>Decompression[b)]</td></tr>
<tr><td>XSA</td><td colspan="2">Given special consideration [d)]</td><td colspan="2">Given special consideration [d)]</td></tr>
</table>

**Figure 4.2:** Allowed crack width according to national annex in EN 1992-1-1 [3]

<table>
<tr><th rowspan="3">Exposure Class</th><th colspan="2">Reinforced members; prestressed members with unbonded tendons and with bonded tendons with Protection Levels 2 or 3 according to 5.4.1(3)</th><th colspan="3">Prestressed members with bonded tendons with Protection Level 1 according to 5.4.1(3) and pretensioned elements.</th></tr>
<tr><th colspan="2">combination of actions</th><th colspan="3">combination of actions</th></tr>
<tr><th>quasi-permanent</th><th>characteristic</th><th>quasi-permanent</th><th>frequent</th><th>characteristic</th></tr>
<tr><td>X0, XC1</td><td>–</td><td rowspan="2">–</td><td>–</td><td>$w_{lim,cal} =$ 0,2 mm · $k_{surf}$</td><td rowspan="2">–</td></tr>
<tr><td>XC2, XC3, XC4</td><td rowspan="2">$w_{lim,cal} =$ 0,3 mm · $k_{surf}$</td><td>Decompression [b]</td><td>$w_{lim,cal} =$ 0,2 mm · $k_{surf}$</td></tr>
<tr><td>XD1, XD2, XD3 XS1, XS2, XS3</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td><td rowspan="2">–</td><td rowspan="2">Decompression[b]</td><td rowspan="2">$\sigma_c \leq 0{,}6 f_{ck}$ [a,c]</td></tr>
<tr><td>XF1, XF3 XF2, XF4</td><td>–</td></tr>
</table>

**Figure 4.3:** Allowed crack width according to EN 1992-1-1:2021-1 [5]

Further, in EN 1992-1-1, the crack width may be controlled by the limitation of the reinforcement stress, which is related to the reinforcement diameter and the spacing. Crack width can also be controlled by direct crack width calculation. EN 1992-1-1:2021-1 have a similar approach. However, the way of determining the stress limit is different. EN 1992-1-1 provides values of the stress limits in a table, while in EN 1992-1-1:2021-1, the stress limit is calculated according to provided expression.

### 4.2.1 Simplified crack width control

According to the simplified method in EN 1992-1-1:2021-1, the crack width can be controlled by either limiting the bar diameter or the spacing between reinforcement bars, such that if the chosen bar diameter $\phi$ or spacing $S_l$ is within the calculated limit, the crack control is approved.

$$\phi \leq \frac{2.1 * \rho_p}{\frac{r}{d} * k_{fl,simpl} * k_{b,simpl}} * \left(\frac{w_{lim,cal}}{k_w * k_{\frac{1}{r},simpl} * 0.9 * \frac{\sigma_c}{E_s}} - 1.5c\right) \tag{4.11}$$

$$S_l \leq \frac{3.45 * \rho_p}{\frac{r^2}{d} * k^2_{fl,simpl} * k^2_{b,simpl}} * \left(\frac{w_{lim,cal}}{k_w * k_{\frac{1}{r},simpl} * 0.9 * \frac{\sigma_c}{E_s}} - 1.5c\right)^2 \tag{4.12}$$

Where $\rho_p$ is the reinforcement ration considering the tension face, for steel reinforcement,$\rho_p$ is determined as follows:

$$\rho_p = \frac{A_s}{bd} \tag{4.13}$$

$k_{fl,simpl}$ considers whether faces are subjected to tension or compression. For faces in tension $k_{fl,simpl} = 1.0$.

$k_{b,simpl}$ considers the bonding conditions. For good bonding condition $k_{b,simpl} = 0.9$, while for poor bonding condition $k_{b,simpl} = 1.2$.

r is the distance from concrete surface to the center of the first layer of bars.

As mentioned previously, EN 1992-1-1 have a similar approach for the simplified crack width control. However, the simplified crack width calculations in EN 1992-1-1 are based on the safe assumption that the concrete stress between two cracks can never be greater than its tensile strength. While EN 1992-1-1:2021-1 assumes that the crack result is directly related to the crack width at the surface of the member. [8].

Clause [7.3.3 (2)] in EN 1992-1-1 provides tabulated values for maximum bar diameters and bar spacing. The revisions aimed to replace the tabulated values with expressions to enable the application in spreadsheets or computer programs.

The maximum allowed steel stress $\sigma_{s,max}$ is determined for a specified maximum bar diameter and crack width $w_k$. The steel stress should be calculated and compared to the maximum allowed steel stress. See figure 4.4, and 4.5. Crack width control is approved if the following condition is satisfied:

$$\sigma_s \leq \sigma_{s,max} \tag{4.14}$$

Steel stress may be calculated as follows:

$$\sigma_s = E_s * \frac{M(1-\alpha)d}{EI} \tag{4.15}$$

| Steel stress[2] [MPa] | Maximum bar size [mm] $w_k$= 0,4 mm | $w_k$= 0,3 mm | $w_k$= 0,2 mm |
|---|---|---|---|
| 160 | 40 | 32 | 25 |
| 200 | 32 | 25 | 16 |
| 240 | 20 | 16 | 12 |
| 280 | 16 | 12 | 8 |
| 320 | 12 | 10 | 6 |
| 360 | 10 | 8 | 5 |
| 400 | 8 | 6 | 4 |
| 450 | 6 | 5 | - |

**Figure 4.4:** Maximum bar diameter for crack control [3]

| Steel stress[2] [MPa] | Maximum bar spacing [mm] $w_k$=0,4 mm | $w_k$=0,3 mm | $w_k$=0,2 mm |
|---|---|---|---|
| 160 | 300 | 300 | 200 |
| 200 | 300 | 250 | 150 |
| 240 | 250 | 200 | 100 |
| 280 | 200 | 150 | 50 |
| 320 | 150 | 100 | - |
| 360 | 100 | 50 | - |

**Figure 4.5:** Maximum bar spacing for crack control [3]

### 4.2.2 Detailed crack width control

Section [9.2.4] in EN 1992-1-1:2021-1 and [7.3.4] in EN 1992-1-1, concerns calculation of the crack width. The calculated crack width should be within the upper limit $w_k$ or $w_{lim,cal}$ as shown in section 4.2.

The equation is almost identical to the one in EN 1992-1-1, except that EN 1992-1-1:2021-1 defines the factor $k_w$, which converts the mean crack width into a calculated crack width. The crack width according to EN 1992-1-1:2021-1 may be calculated as follows:

$$w_{k,cal} = k_w * s_{rm,cal}(\epsilon_{sm} - \epsilon_{cm}) \tag{4.16}$$

$\epsilon_{sm}$ is the mean strain in the reinforcement closest to the most tensioned concrete surface when all loads are applied. $\epsilon_{cm}$ is the mean strain in the concrete at the same level as $\epsilon_{sm}$.The difference between the two strains, according to EN 1992-1-1:2021, is defined as follows:

$$\epsilon_{sm} - \epsilon_{cm} = k_{\frac{1}{r}} \frac{\sigma_s - k_t * \frac{f_{ct,eff}}{\rho_{p,eff}} * (1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6 \frac{\sigma_s}{E_s} \tag{4.17}$$

The expression for the difference between steel strain and concrete strain is almost identical in both versions of EC.2. However the factor $k_{\frac{1}{r}}$ defined in EN 1992-1-1:2021, which takes curvature into account is not defined in EN 1992-1-1. The new factor $k_{\frac{1}{r}}$ depends on the depth of the compressive zone $x$ and the concrete cover and is determined as follows:

$$k_{\frac{1}{r}} = \frac{h-x}{h-a_{y,i}-x} \tag{4.18}$$

When considering the least tensioned face, $k_{\frac{1}{r}}$ is defined as follows:

$$k_{\frac{1}{r}} = \frac{|x|}{a_{y,s}+|x|} \tag{4.19}$$

When considering the least tensioned face, the effect of curvature becomes favourable; thus, the factor $k_{\frac{1}{r}}$ becomes less than 1.0.

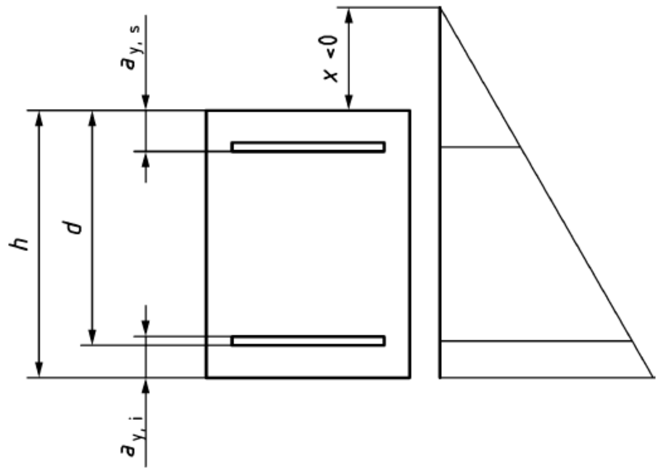



**Figure 4.6:** Notation for definition of $k_{\frac{1}{r}}$ [5]

EN 1992-1-1:2021-1 takes into account elements with restrained edges that will cause imposed strains, see figure 4.7. This matter is not considered in EN 1992-1-1. The expression for $\epsilon_{sm}-\epsilon_{cm}$ is then defined as follows:

$$\epsilon_{sm}-\epsilon_{cm} = R_{ax}\epsilon_{free} - k_t \frac{f_{ct,eff}}{E_{cm}} \geq 0 \tag{4.20}$$

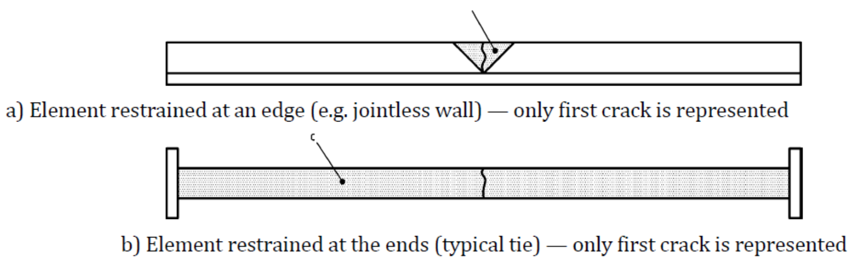



**Figure 4.7:** Elements subjected to imposed strains [5]

The mean cracking spacing $s_{r,m,cal}$ and the maximum crack spacing $s_{r,max}$ according to EN 1992-1-1:2021-1 and EN 1992-1-1, respectively, are determined as follows:

| | EN 1992-1-1 | EN 1992-1-1:2021-1 |
|---|---|---|
| $s \leq 10\phi$: | $s_{r,m,cal} = 1.5 * c + \frac{k_{fl} * k_b}{7.2} * \frac{\phi}{\rho_{p,eff}} \leq 1.3 * (h - x)$ | $s_{r,max} = k_3 c + k_1 * k_3 * k_4 \phi / \rho_{p,eff}$ |
| $s > 10\phi$: | $s_{r,m,cal} = 1.3 * (h - x)$ | $s_{r,max} = 1.3 * (h - x)$ |

The expression for cracking spacing in EN 1992-1-1:2021-1 takes casting condition (poor or good casting position) into account through the factor $k_b$. $k_1$ takes the bond properties into account. $k_{fl}$ corresponds to the factor $k_2$, and considers whether the cross-section is subjected to pure bending or tension. $k_3$, and $k_4$ are national decided reduction coefficients.

A detailed study compares the new proposal for crack width calculations with 2D FEM simulations. It shows that the new approach for calculating crack width is more empirical and is based on intensive laboratory experiments and analysis. The new approach takes less account of the mechanical relationship of reinforcement and prestressed reinforcement after cracking. However, accuracy improvement is not achieved compared to EN 1992-1-1. [8]

# Chapter 5

# Reinforcement detailing

Some of the main objectives of the revised EC.2 were to improve the durability of concrete and enhance ease of use. Thus, significant changes have been made in the revised versions of EC.2 regarding reinforcement detailing, especially the sections concerning anchorage length, which will be covered in this chapter.

## 5.1 Spacing of bars

A clear distance between bars should be chosen such that the concrete can be properly compacted and ensure good bonding of the concrete. EC.2 defines a minimum clear distance between horizontal or vertical bars.

| **EN 1992-1-1:2021** | **EN 1992-1-1** |
|---|---|
| $c_s = max[\phi; D_{upper} + 5mm; 20mm]$ | $c_s = max[k_1\phi; d_g + k_2; 20mm]$ <br> $k_1 = 1,\ k_2 = 5mm$ |

Both versions have the exact definition of minimum clear distance. $D_{upper}$ corresponds to $d_g$, and is the maximum size of aggregate used in the concrete.

## 5.2 Anchorage of reinforcing steel in tension and compression

The reinforcement should be anchored to assure safe transmission of forces to the concrete in compression or another reinforcement. The clauses [11.4.1 (2), (3), and (4)] in EN 1992-1-1:2021-1 defines potential delamination and longitudinal cracking risks that could occur and provides instructions for how to control these risks. According to [11.3.1 (3)], stirrups enclosing the bars should be provided as shown in 5.1 a) and b) to control longitudinal cracking and delamination. Such instructions are not specified in EN 1992-1-1.

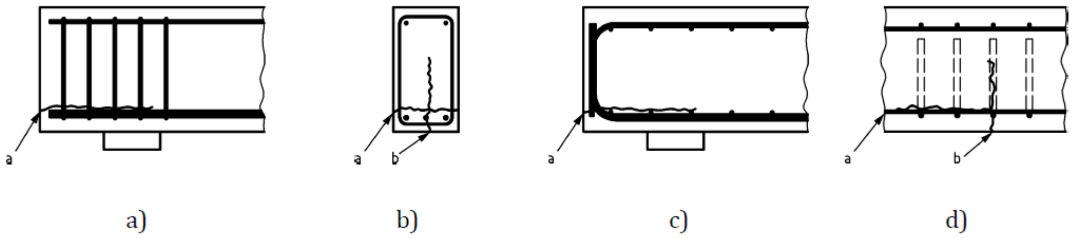



**Figure 5.1:** Examples of reinforcement controlling delamination and longitudinal cracks

a) Potential delamination crack
b) Potential longitudinal crack

According to EN 1992-1-1, the anchorage length is calculated by determining, the design value of the ultimate bond stress $f_{bd}$, basic anchorage length $l_{b,rgd}$, and lastly the design anchorage length $l_{bd}$.
There are many factors to consider when calculating the design anchorage length according to the current EC.2. EN 1992-1-1 provides the general expressions, and the factors have to be determined for the used anchorage method with poor further explanation. Consequently, anchorage length calculations are time-consuming, and the usability is poor.

The revised version has rearranged the method for determining the anchorage length to enhance ease of use. The design anchorage length $l_{bd}$ is directly calculated based on the used anchorage method. The included methods in EN 1992-1-1:2021 are shown in figure 5.2.

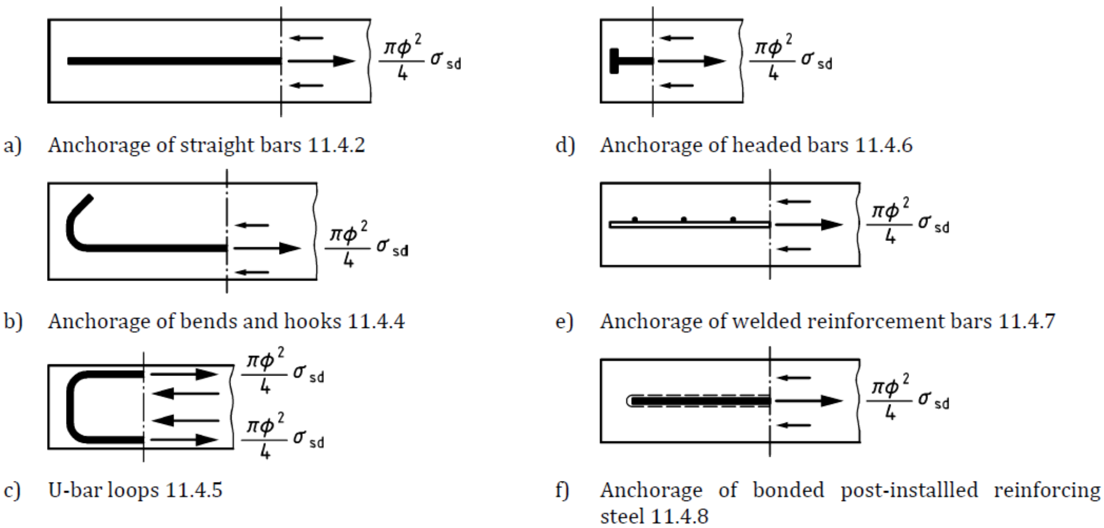



**Figure 5.2:** Methods of anchorage [5]

### 5.2.1 Anchorage according to EN 1992-1-1:2021

**For straight bars**

For straight bars, EN 1992-1-1:2021-1 provides a table at which the anchorage length is determined directly, based on the compressive concrete strength $f_{ck}$, and the utilised bar diameter $\phi$. See figure 5.3.

The values provided in the table are applicable for $\sigma_{sd} = 435$ MPa, $c_s \geq 1.5$, and good bond conditions. However, if $\sigma_s < 435MPa$, the table values should be multiplied by $\frac{\sigma_s}{435}$, and in order to account for poor bond conditions, the values should be multiplied by 1.2. Furthermore, the minimum anchorage length according to the revised EC.2, is defined as follows:

$$l_{bd}/\phi \geq 10 \tag{5.1}$$

| $\phi$ [mm] | Anchorage length $l_{bd}/\phi$ | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | $f_{ck}$ | | | | | | | |
| | **20** | **25** | **30** | **35** | **40** | **45** | **50** | **60** |
| ≤ 12 | 47 | 42 | 38 | 36 | 33 | 31 | 30 | 27 |
| 14 | 50 | 44 | 41 | 38 | 35 | 33 | 31 | 29 |
| 16 | 52 | 46 | 42 | 39 | 37 | 35 | 33 | 30 |
| 20 | 56 | 50 | 46 | 42 | 40 | 37 | 35 | 32 |
| 25 | 60 | 54 | 49 | 46 | 43 | 40 | 38 | 35 |
| 28 | 63 | 56 | 51 | 47 | 44 | 42 | 40 | 36 |
| 32 | 65 | 58 | 53 | 49 | 46 | 44 | 41 | 38 |

**Figure 5.3:** Anchorage length for straight bars [5]

The design anchorage length for straight bars, according to clause [11.4.2 (3)] in EN 1992-1-1:2021-1, is expressed as follows:

$$l_{bd} = k_{lb} * k_{cp} * \phi * (\frac{\sigma_{sd}}{435})^{n_\sigma} * (\frac{25}{f_{ck}})^{\frac{1}{2}} * (\frac{\phi}{20})^{\frac{1}{3}} * (\frac{1.5\phi}{c_d})^{\frac{1}{2}} \geq 10\phi \tag{5.2}$$

**EN 1992-1-1:2021-1**

$$k_{cp} \begin{cases} 1.0 & \text{for good bond} \\ 1.2 & \text{for poor bond} \\ 1.4 & \text{(slurry or bentonite)} \end{cases}$$

$$n_\sigma \begin{cases} 1.0 & \text{for } \sigma_{sd} \leq 435MPa \\ 1.5 & \text{for } \sigma_{sd} > 435MPa \end{cases}$$

$$\mathrm{k}_{lb} \begin{cases} 50 & \text{for persistent design situation} \\ 39 & \text{for accidental design situation} \end{cases}$$

EN 1992-1-1:2021 has a definition for good bonding. A good bonding is assumed when bars have 45° to 90° inclination to the horizontal and bars with inclination < 45° to the horizontal, which is more than 300 mm from the bottom of the formwork. In figure 5.4, index 1 shows the surface during concreting, index 2 is the zone with a poor bond condition, and index 3 is the zone with a good bonding condition.

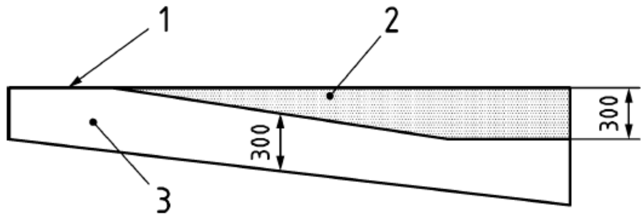



**Figure 5.4:** Description of bonding conditions

### 5.2.2 Anchorage according to EN 1992-1-1

The design anchorage length according to clause [8.4.4(1)] in EN 1992-1-1, is determined as follows:

$$l_{bd} = \alpha_1 * \alpha_2 * \alpha_3 * \alpha_4 * \alpha_5 * l_{b,rqd} \geq l_{b,min} \tag{5.3}$$

The required anchorage length is determined as follows:

$$l_{b,rqd} = \frac{\phi}{4} * \frac{\sigma_{sd}}{f_{bd}}$$

$$f_{bd} = 2.25 * \eta_1 * \eta_2 * f_{ctd}$$

$$\eta_1 \begin{cases} 1.0 & \text{for good bond} \\ 0.7 & \text{for all othe cases} \end{cases} ; \eta_2 \begin{cases} 1.0 & \text{for } \phi \leq 32mm \\ \frac{132-\phi}{100} & \text{for } \phi > 32mm \end{cases}$$

The factors $\alpha_1 - \alpha_5$ consider several effects on the anchorage length, such as the form of the bars $\alpha_1$, the concrete minimum cover $\alpha_2$, confinement by transverse reinforcement $\alpha_3$, welded bars $\alpha_4$, and pressure transverse to the plane of splitting along the design anchorage length $\alpha_5$. $\alpha_1 - \alpha_5$ are determined according to the table [8.2] in EN 1992-1-1. The factors are determined based on the type of anchorage and whether the bar is in tension or compression. The product of $(\alpha_2 * \alpha_3 * \alpha_5)$ must be at minimum $= 0.7$.

The minimum anchorage length is determined as follows:

$$l_{b.min} = max\{n * l_{b,rgd}; 10 * \phi; 100mm] \tag{5.4}$$

Where $n = 0.3$ for anchorages in tension and $n = 0.6$ for anchorages in compression.

**Comparison of the two approaches for the design anchorage length**

The method in EN 1992-1-1:2021-1 is more convenient as it is more straightforward and transparent than EN 1992-1-1. Several matters are cleared up in the revised version, with more detailed definitions, such as the definition of good and bad bonding. Furthermore, EN 1992-1-1:2021-1 includes a section for every anchorage method, where detailed information is provided along with the factor that should be replaced in the central equation 5.2 to account for the method. Thus, the calculation of the design anchorage length in EN 1992-1-1:2021 is less time-consuming.

# Chapter 6

# Design of concrete slabs and beams

This chapter will present hand calculations according to the two versions of EC.2. The objective of performing hand calculations is to compare procedures and values using practical examples. Moment and shear diagrams are extracted from Robot. See appendices A.

## 6.1 Detailed calculation of slab nu.69

According to Robot calculations, slab nu.69 is the most stressed slab. Hence detailed calculations of the slab are shown in this section. The longitudinal reinforcement in the slab is determined according to publication nu.33, which concerns flat slabs [9].

### 7.1.1 Geometry, applied loads, exposure class, and materials:

| | | | |
|---|---|---|---|
| Thickness: | h=300 mm ; | Live load: | p=3$\frac{kN}{m^2}$ |
| Span x-direction: | $l_x = 6.85m$; | Snow load: | S= 2.8$\frac{kN}{m^2}$ |
| span y-direction: | $l_y = 7.35$; | Self weight: | g=25$\frac{kN}{m^3} * h = 7.5\frac{kN}{m^2}$ |
| Design life time: | 50 years | | |
| Exposure class: | XC2 | | |
| Concrete strength | $f_{ck} = 45 * \frac{N}{mm^2}$ | | |
| Yield strength of reinforcement | $f_{yk} = 500 * \frac{N}{mm^2}$ | | |
| Assumed bar diameter | $\phi = 12mm$ | | |
| Nominal cover | $C_{nom} = 30mm$ | | |
| Effective depth x-direction: | $d_x = h - c_{nom} - \phi = 264mm$ | | |
| Effective depth y-direction: | $d_y = h - c_{nom} - \phi - \frac{\phi}{2} = 252mm$ | | |

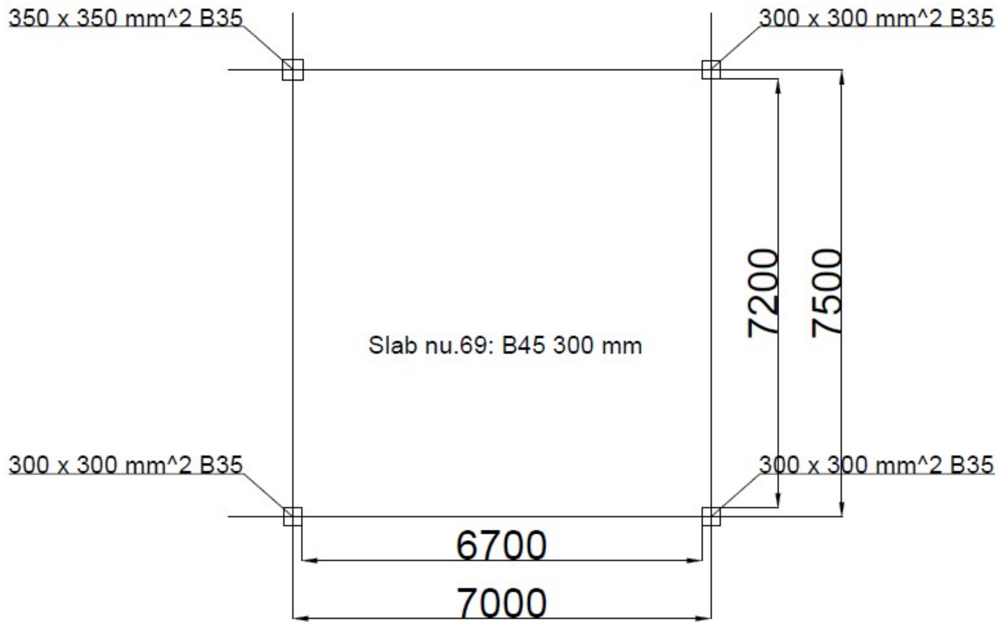
**Table 6.1:** Data regarding the design of slab nu.69

### 7.2.2 Summary of the design of slab nu.69

| **Summary of the design of slab nu.69** | | |
|---|---|---|
| | **EN 1992-1-1** | **EN 1992-1-1:2021-1** |
| **Moment capacity** | **[kN*m/m]** | **[kN*m/m]** |
| $M_{Rd,x}$ | 488.70 | 552.85 |
| $M_{Rd,y}$ | 445.30 | 503.70 |
| **Minimum reinforcement** | **[mm$^2/m$]** | $[mm^2/m]$ |
| $A_{sx.min}$ | 521.60 | 479.80 |
| $A_{sy.min}$ | 498.00 | 502.60 |
| **Reinforcement in the tension field** | **[mm$^2/m$]** | $[mm^2/m]$ |
| $A_{sx.is}$ | 1835.00 | 1819.00 |
| $A_{sx,ys}$ | 1200.00 | 1200.00 |
| $A_{sx,ms}$ | 521.60 | 499.80 |
| $A_{sy.is}$ | 2263.00 | 2238.00 |
| $A_{sy.ys}$ | 1463.00 | 1452.00 |
| $A_{sy,ms}$ | 605.00 | 604.50 |
| **Reinforcement in the compression field** | **[mm$^2/m$]** | $[mm^2/m]$ |
| $A_{sx,if}$ | 786.00 | 786.00 |
| $A_{sx,mf}$ | 524.00 | 524.00 |
| $A_{sy,if}$ | 947.70 | 947.70 |
| $A_{sy,mf}$ | 631.80 | 631.80 |
| **Utilization of shear capacity without shear reinforcement** | [-] | [-] |
| $V_{Ed}/V_{Rd,c}$ | 0.556 | 0.498 |
| **Utilization of punching shear capacity without shear reinforcement** | [-] | [-] |
| $V_{Ed}/V_{Rd,c}$ | 0.88 | 0.96 |
| **Deflection at mid-span** | **[mm]** | **[mm]** |
| $\delta_{midspan}$ | 44.01 | 44.91 |
| **Crack width** | **[mm]** | **[mm]** |
| Allowed crack width | 0.39 | 0.30 |
| Calculated crack width | 0.143 | 0.124 |

**Table 6.2:** Summary of the design of slab nu.69

**<u>The layout of the slab:</u>**




**<u>Max moments from Robot:</u>**

| $m_{xs}$ $\left(\frac{kN \cdot m}{m}\right)$ | $m_{xf}$ $\left(\frac{kN \cdot m}{m}\right)$ | $m_{ys}$ $\left(\frac{kN \cdot m}{m}\right)$ | $m_{yf}$ $\left(\frac{kN \cdot m}{m}\right)$ |
|---|---|---|---|
| 109 | 71.4 | 125.84 | 82.2 |

**7.1.3 Longitudinal reinforcement according to EC.2:2004, and NB33**

**<u>Moment capacity</u>**

$\alpha_{cc} := 0.85$ $\gamma_c := 1.5$

$$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 25.5\ \frac{N}{mm^2}$$

$$M_{Rd.x} := 0.275 \cdot f_{cd} \cdot b \cdot d_x^{\ 2} = 488.743\ kN \cdot \frac{m}{m}$$

$$M_{Rd.y} := 0.275 \cdot f_{cd} \cdot b \cdot d_y^{\ 2} = 445.322\ kN \cdot \frac{m}{m}$$

**Minimum reinforcement area:** ***[EC.2:2004 N.A.9.2.1.11(1)]***

$$A_{s.minx} := 0.13\% \cdot b \cdot d_x = 343.2 \ \frac{mm^2}{m}$$

$$A_{s.miny} := 0.13\% \cdot b \cdot d_y = 327.6 \ \frac{mm^2}{m}$$

$$A_{s.minx1} := 0.26 \cdot \left(\frac{f_{ctm}}{f_{yk}}\right) \cdot b \cdot d_x = 521.664 \ \frac{mm^2}{m}$$

$$A_{s.miny1} := 0.26 \cdot \left(\frac{f_{ctm}}{f_{yk}}\right) \cdot b \cdot d_y = 497.952 \ \frac{mm^2}{m}$$

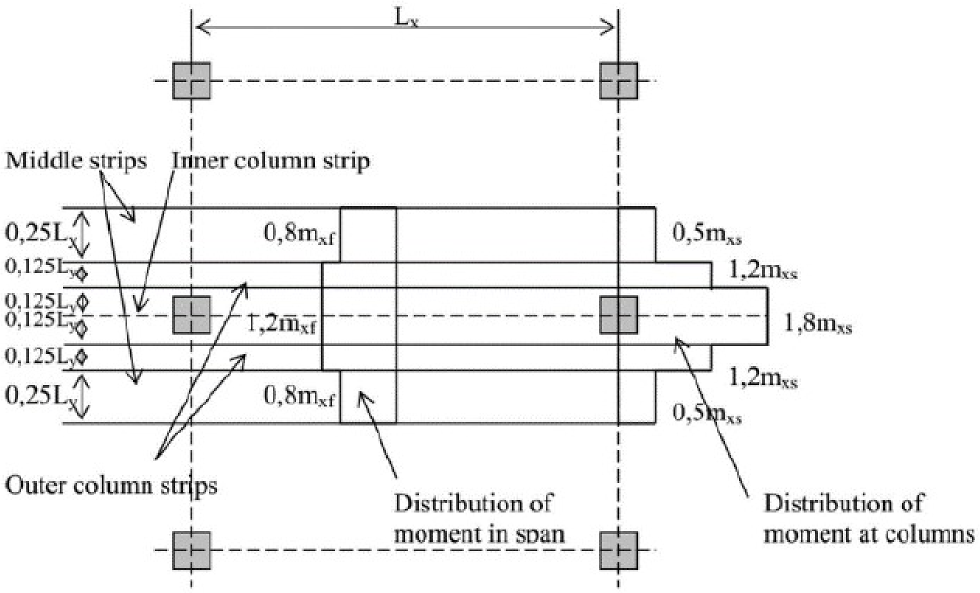
$$A_{s.minx} := \max\left(A_{s.minx}, A_{s.minx1}\right) = 521.664 \ \frac{mm^2}{m}$$

$$A_{s.miny} := \max\left(A_{s.miny}, A_{s.miny1}\right) = 497.952 \ \frac{mm^2}{m}$$

$$A_{s.max} := 0.04 \cdot b \cdot h = \left(1.2 \cdot 10^4\right) \ \frac{mm^2}{m}$$

According to publication nu.33 (published by the Norwegian concrete association), the moment distribution shown below could be used for the reinforcement calculation.

**MOMENT DISTRIBUTION IN FLAT SLABS ACCORDING TO NB Publikasjon 33 :**



**Reinforcement in x-direction:**

**Tension side**

$$z_x := 0.95 \cdot d_x = 250.8\ mm$$

**Inner column strips:**

$$z_{x1} := \left(1 - \left(0.17 \cdot 1.8 \cdot \frac{m_{xs}}{M_{Rd.x}}\right)\right) \cdot d_x = 245.983\ mm$$

$$f_{yd} := \frac{f_{yk}}{1.15} = 434.783\ \frac{N}{mm^2}$$

$$A_{sx.is} := 1.8 \cdot \frac{m_{xs}}{z_{x1} \cdot f_{yd}} = \left(1.835 \cdot 10^3\right)\ \frac{mm^2}{m}$$

**Outer column strips**

$$z_{x2} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{xs}}{M_{Rd.x}}\right)\right) \cdot d_x = 251.989\ mm$$

$$A_{sx.ys} := 1.2 \cdot \frac{m_{xs}}{z_x \cdot f_{yd}} = \left(1.2 \cdot 10^3\right)\ \frac{mm^2}{m}$$

$A_{sx.is} > 0.5 \cdot A_{s.ys}$ **ok!** ***[EC.2:2004 9.4.1]***

**Outer column strips:**

$$z_{x3} := \left(1 - \left(0.17 \cdot 0.5 \cdot \frac{m_{xs}}{M_{Rd.x}}\right)\right) \cdot d_x = 258.995\ mm$$

$$A_{sx.ms} := 0.5 \cdot \frac{m_{xs}}{z_x \cdot f_{yd}} = 499.801\ \frac{mm^2}{m}$$

$$A_{sx.ms} := A_{s.minx} = 521.664\ \frac{mm^2}{m}$$

**Compression side**

**Indre column strips**

$$z_{x4} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{xf}}{M_{Rd.x}}\right)\right) \cdot d_x = 256.132\ mm$$

$$A_{sx.if} := 1.2 \cdot \left(\frac{m_{xf}}{z_x \cdot f_{yd}}\right) = 785.742\ \frac{mm^2}{m}$$

**Middle column strip**

$$z_{x5} := \left(1 - \left(0.17 \cdot 0.8 \cdot \frac{m_{xf}}{M_{Rd.x}}\right)\right) \cdot d_x = 258.755\ mm$$

$$A_{sx.mf} := 0.8 \cdot \left(\frac{m_{xf}}{z_x \cdot f_{yd}}\right) = 523.828\ \frac{mm^2}{m}$$

**Reinforcement in y-direction:**

**Tension side**

$$z_y := 0.95 \cdot d_y = 239.4\ mm$$

**Inner column strip:**

$$z_{y1} := \left(1 - \left(0.17 \cdot 1.8 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 230.21\ mm$$

$$A_{sy.is} := 1.8 \cdot \frac{m_{ys}}{z_{y1} \cdot f_{yd}} = \left(2.263 \cdot 10^3\right) \frac{mm^2}{m}$$

**Outer column strip:**

$$z_{y2} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 237.473\ mm$$

$$A_{sy.ys} := 1.2 \cdot \frac{m_{ys}}{z_{y2} \cdot f_{yd}} = \left(1.463 \cdot 10^3\right) \frac{mm^2}{m}$$

$A_{sy.is} > 0.5 \cdot A_{sy.ys}$ **ok! *[EC.2:2004 9.4.1]***

**Middle column strip:**

$$z_{y3} := \left(1 - \left(0.17 \cdot 0.5 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 245.947\ mm$$

$$A_{sy.ms} := 0.5 \cdot \frac{m_{ys}}{z_y \cdot f_{yd}} = 604.495\ \frac{mm^2}{m}$$

**Compression side**

**Inner column strip:**

$$z_{y4} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{yf}}{M_{Rd.y}}\right)\right) \cdot d_y = 242.511\ mm$$

$$A_{sy.if} := 1.2 \cdot \frac{m_{yf}}{z_y \cdot f_{yd}} = 947.669\ \frac{mm^2}{m}$$

**Middle column strip:**

$$z_{y5} := \left(1 - \left(0.17 \cdot 0.8 \cdot \frac{m_{yf}}{M_{Rd.y}}\right)\right) \cdot d_y = 245.674\ mm$$

$$A_{sy.mf} := 0.8 \cdot \frac{m_{yf}}{z_y \cdot f_{yd}} = 631.779\ \frac{mm^2}{m}$$

## Summary of the chosen reinforcement according to EC.2:2004, and NB.33

| Tension. side | x-direction | y-direction |
|---|---|---|
| $A_{sx.is}=\left(1.835\cdot10^{3}\right)\dfrac{mm^2}{m}$ | **Ø12 C.C. 60 mm** | |
| $A_{sx.ys}=\left(1.2\cdot10^{3}\right)\dfrac{mm^2}{m}$ | **Ø12 C.C. 90 mm** | |
| $A_{sx.ms}=521.664\dfrac{mm^2}{m}$ | **Ø12 C.C. 215 mm** | |
| $A_{sy.is}=\left(2.263\cdot10^{3}\right)\dfrac{mm^2}{m}$ | | **Ø12 C.C. 45 mm** |
| $A_{sy.ys}=\left(1.463\cdot10^{3}\right)\dfrac{mm^2}{m}$ | | **Ø12 C.C. 75 mm** |
| $A_{sy.ms}=604.495\dfrac{mm^2}{m}$ | | **Ø12 C.C. 185 mm** |
| **Compression side** | | |
| $A_{sx.if}=785.742\dfrac{mm^2}{m}$ | **Ø12 C.C. 140 mm** | |
| $A_{sx.mf}=523.828\dfrac{mm^2}{m}$ | **Ø12 C.C. 215 mm** | |
| $A_{sy.if}=947.669\dfrac{mm^2}{m}$ | | **Ø12 C.C. 115 mm** |
| $A_{sy.mf}=631.779\dfrac{mm^2}{m}$ | | **Ø12 C.C. 175 mm** |

## Spacing between bars for regions with high moment stress according to EC.2:2004 NA.9.3.1.1

Main reinforcement

$$S_{max.slab.H}:=2\cdot h=600\ mm$$

$$S_{max.slab.H}>250\ mm$$

$$S_{max.slab.H}:=250\ mm$$

$ok!$

Distribution reinforcement

$$S_{max.slab.F}:=3\cdot h=900\ mm$$

$$S_{max.slab.F}>400\ mm$$

$$S_{max.slab.F}:=400\ mm$$

$ok!$

### 7.1.4 Longitudinal reinforcement according to the revised version EC.2:21.5.1.6 (1)

$$\eta_{cc} := \left( \frac{40 \frac{N}{mm^2}}{f_{ck}} \right)^{\frac{1}{3}} = 0.961 \qquad \kappa_{tc} := 1 \qquad \gamma_c := 1.5$$

$$f_{cd} := \eta_{cc} \cdot \kappa_{tc} \cdot \frac{f_{ck}}{\gamma_c} = 28.845 \frac{N}{mm^2}$$

**Moment capacity:**

$$\xi_{cu} := 0.0035 \qquad \xi_{yk} := 0.0025 \qquad \alpha := \frac{\xi_{cu}}{\xi_{cu} + 2 \cdot \xi_{yk}} = 0.412$$

$$0.8 \cdot \alpha \cdot (1 - 0.4\ \alpha) = 0.275$$

$$M_{Rd.x} := 0.275 \cdot f_{cd} \cdot {d_x}^2 \cdot b = 552.855 \frac{kN \cdot m}{m}$$

$$M_{Rd.y} := 0.275 \cdot f_{cd} \cdot {d_y}^2 \cdot b = 503.737 \frac{kN \cdot m}{m}$$

**Minimum reinforcement:**

According to [EC.2:21 12.2.2 (a)]: the minimum reinforcement area is calculated, such that, the cross-section resists the effect of cracks.

$$M_{R.min}(N_{Ed}) \geq k \cdot M_{cr}(N_{Ed})$$

$$A_{s.minx} := \frac{1}{6} \cdot b \cdot h^2 \cdot \frac{f_{ctm}}{f_{yk} \cdot 0.9\ d_x} = 479.798 \frac{mm^2}{m}$$

$$A_{s.miny} := \frac{1}{6} \cdot b \cdot h^2 \cdot \frac{f_{ctm}}{f_{yk} \cdot 0.9 \cdot d_y} = 502.646 \frac{mm^2}{m}$$

**Reinforcement in x-direction:**

$$z_x = 250.8\ mm$$

**Tension side:**

**Inner column strip:**

$$z_{x1} := \left( 1 - \left( 0.17 \cdot 1.8 \cdot \frac{m_{xs}}{M_{Rd.x}} \right) \right) \cdot d_x = 248.073\ mm$$

$$A_{sx.is} := 1.8 \cdot \frac{m_{xs}}{z_{x1} \cdot f_{yd}} = \left( 1.819 \cdot 10^3 \right) \frac{mm^2}{m}$$

**Outer column strip:**

$$z_{x2} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{xs}}{M_{Rd.x}}\right)\right) \cdot d_x = 253.382\ mm$$

$$A_{sx.ys} := 1.2 \cdot \frac{m_{xs}}{z_x \cdot f_{yd}} = \left(1.2 \cdot 10^3\right)\ \frac{mm^2}{m}$$

**Middle column strip:**

$$z_{x3} := \left(1 - \left(0.17 \cdot 0.5 \cdot \frac{m_{xs}}{M_{Rd.x}}\right)\right) \cdot d_x = 259.576\ mm$$

$$A_{sx.ms} := 0.5 \cdot \frac{m_{xs}}{z_x \cdot f_{yd}} = 499.801\ \frac{mm^2}{m}$$

**<u>Field reinforcement:</u>**

**Inner column strip:**

$$z_{x4} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{xf}}{M_{Rd.x}}\right)\right) \cdot d_x = 257.045\ mm$$

$$A_{sx.if} := 1.2 \cdot \frac{m_{xf}}{z_x \cdot f_{yd}} = 785.742\ \frac{mm^2}{m}$$

**Middle column strip:**

$$z_{x5} := \left(1 - \left(0.17 \cdot 0.8 \cdot \frac{m_{xf}}{M_{Rd.x}}\right)\right) \cdot d_x = 259.363\ mm$$

$$A_{sx.mf} := 0.8 \cdot \frac{m_{xf}}{z_x \cdot f_{yd}} = 523.828\ \frac{mm^2}{m}$$

**<u>Reinforcement in y-direction:</u>**
**<u>Tension side</u>**

$$z_y = 239.4\ mm$$

**Inner column strip:**

$$z_{y1} := \left(1 - \left(0.17 \cdot 1.8 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 232.736\ mm$$

$$A_{sy.is} := 1.8 \cdot \frac{m_{ys}}{z_{y1} \cdot f_{yd}} = \left(2.238 \cdot 10^3\right)\ \frac{mm^2}{m}$$

**Outer column strip:**

$$z_{y2} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 239.158\ mm$$

$$A_{sy.ys} := 1.2 \cdot \frac{m_{ys}}{z_{y2} \cdot f_{yd}} = \left(1.452 \cdot 10^3\right)\ \frac{mm^2}{m}$$

**Middle column strip:**

$$z_{y3} := \left(1 - \left(0.17 \cdot 0.5 \cdot \frac{m_{ys}}{M_{Rd.y}}\right)\right) \cdot d_y = 246.649\ mm$$

$$A_{sy.ms} := 0.5 \cdot \frac{m_{ys}}{z_y \cdot f_{yd}} = 604.495 \ \frac{mm^2}{m}$$

**Field reinforcement:**

**Inner column strip:**

$$z_{y4} := \left(1 - \left(0.17 \cdot 1.2 \cdot \frac{m_{yf}}{M_{Rd.y}}\right)\right) \cdot d_y = 243.611 \ mm$$

$$A_{sy.if} := 1.2 \cdot \frac{m_{yf}}{z_y \cdot f_{yd}} = 947.669 \ \frac{mm^2}{m}$$

**Middle column strip:**

$$z_{y5} := \left(1 - \left(0.17 \cdot 0.8 \cdot \frac{m_{yf}}{M_{Rd.y}}\right)\right) \cdot d_y = 246.407 \ mm$$

$$A_{sy.mf} := 0.8 \cdot \frac{m_{yf}}{z_y \cdot f_{yd}} = 631.779 \ \frac{mm^2}{m}$$

**Summary of the chosen reinforcement according to EN 1992-1-1:2021, and NB.33**

| **Tension side** | **x-direction** | **y-direction** |
|---|---|---|
| $A_{sx.is} = \left(1.819 \cdot 10^3\right) \frac{mm^2}{m}$ | **Ø12 C.C. 60 mm** | |
| $A_{sx.ys} = \left(1.2 \cdot 10^3\right) \frac{mm^2}{m}$ | **Ø12 C.C. 90 mm** | |
| $A_{sx.ms} = 499.801 \frac{mm^2}{m}$ | **Ø12 C.C. 225 mm** | |
| $A_{sy.is} = \left(2.238 \cdot 10^3\right) \frac{mm^2}{m}$ | | **Ø12 C.C. 50 mm** |
| $A_{sy.ys} = \left(1.452 \cdot 10^3\right) \frac{mm^2}{m}$ | | **Ø12 C.C. 75 mm** |
| $A_{sy.ms} = 604.495 \frac{mm^2}{m}$ | | **Ø12 C.C. 185 mm** |

**Field reinforcement**

| | | |
|---|---|---|
| $A_{sx.if} = 785.742 \frac{mm^2}{m}$ | **Ø12 C.C. 140 mm** | |
| $A_{sx.mf} = 523.828 \frac{mm^2}{m}$ | **Ø12 C.C. 215 mm** | |
| $A_{sy.if} = 947.669 \frac{mm^2}{m}$ | | **Ø12 C.C. 115 mm** |
| $A_{sy.mf} = 631.779 \frac{mm^2}{m}$ | | **Ø12 C.C. 175 mm** |

**7.1.5 Shear verification according to EC.2:2004 [EC.2 6.2.2 (1)]**

$$k := 1 + \sqrt[2]{\left(\frac{200}{d_y}\right)} = 1.891 \qquad k < 2$$

$$\rho_l := \frac{522\ mm^2}{10^3\ mm \cdot 249\ mm} = 0.002 \qquad 0.002 < 0.02$$

$$k_2 := 0.18 \qquad C_{Rd.c} := \frac{k_2}{1.5} = 0.12$$

<u>**Shear capacity without shear reinforcement:**</u>

$$V_{Rd.c} := C_{Rd.c} \cdot k \cdot \left(100 \cdot \rho_l \cdot f_{ck}\right)^{\frac{1}{3}} \cdot b_w \cdot d_y \cdot 10^{-3}\ \frac{kN}{m} = 120.82\ \frac{kN}{m}$$

$$V_{min} := 0.035 \cdot \left(k^{\frac{3}{2}}\right) \cdot \left(f_{ck}\right)^{\frac{1}{2}} \cdot 10^3 \cdot d_y \cdot 10^{-3}\ \frac{kN}{m} = 153.839\ \frac{kN}{m}$$

$$V_{min} > V_{Rd.c} \qquad V_{Rd.c} := V_{min} = 153.839\ \frac{kN}{m}$$

<u>**Largest design shear value from Robot:**</u>

$$V_{Ed} := 85.6\ \frac{kN}{m} \qquad \frac{V_{Ed}}{V_{Rd.c}} = 0.556\ \left(0.556 < 1\right)$$

Shear capacity is sufficient, no further check is needed!

**7.1.6 Shear capacity according to EN 1992-1-1:2021**

Assume: $d_{lower} := 16\ mm$

$f_{ck} < 60$, thus: $d_{dg} := 16\ mm + d_{lower} = 32\ mm$

$$V_{Ed.x} := 79.8\ \frac{kN}{m} \qquad V_{Ed.y} := 85.6\ \frac{kN}{m}$$

$$V_{Ed} := \sqrt[2]{V_{Ed.x}^{\ 2} + V_{Ed.y}^{\ 2}} = 106.403\ \frac{kN}{m}$$ ***[EC.2:21 8.2.1 (3)]***

$$\frac{V_{Ed.y}}{V_{Ed.x}} = 1.354 \qquad 0.5 < \frac{V_{Ed.y}}{V_{Ed.x}} < 2$$

Thus: $d := 0.5 \cdot \left(d_x + d_y\right) = 256.5\ mm$ ***[EC.2:21 8.2.1 (3) [8.13b]]***

$$\tau_{Ed} := \frac{V_{Ed} \cdot 1\ m}{b_w \cdot d} = 0.415\ \frac{N}{mm^2}$$ ***[EC.2:21 8.2.1 (3)]***

**Minimum shear stress capacity:**

$\gamma_v := 1.5$ $\qquad\qquad f_{ck} := 45\ \frac{N}{mm^2}$

$$\tau_{Rd.c.min} := \frac{11}{\gamma_v} \cdot \sqrt[2]{\left(\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}\right)}\ \frac{N}{mm^2} = 0.833\ \frac{N}{mm^2}$$ ***[EC.2:21 8.2.1 (4)]***

**Shear stress capacity:**

$$\rho_l := \frac{A_{sx.mf} \cdot 1\ m}{b_w \cdot d} = 0.002$$

$$\tau_{Rd.c} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d}\right)^{\left(\frac{1}{3}\right)} \cdot \frac{N}{mm^2} = 0.461\ \frac{N}{mm^2}$$

$\tau_{Rd.c} := \tau_{Rd.c.min} = 0.833\ \frac{N}{mm^2}$ $\qquad\qquad \frac{\tau_{Ed}}{\tau_{Rd.c}} = 0.498\ (0.498 < 1)$

Shear capacity is sufficient; no further check is needed!

### 7.1.7 Punching for inner column according to *[6.4 EN 1992-1-1:2004 ]*

Punching shear verification for the inner column with dimensions 350x350

**Design shear force (the axial force in inner column)**

$$V_{Ed} := 676.46\ kN$$

**Mean effective height and reinf. ratio**

$$d_{eff} := \frac{(d_x + d_y)}{2} = 256.5\ mm$$ ***[Eq. 6.32]***

Reinforcement ratio (Reinforcement in the slab above the column) in y and x-direction (When working in two dimensional >> x and y corresponds to y and z)

$$A_{sx.is} := 1835\ \frac{mm^2}{m} \qquad \rho_{ly} := \frac{A_{sx.is}}{b \cdot d_{eff}} = 0.007$$

$$A_{sy.is} := 2263\ \frac{mm^2}{m} \qquad \rho_{lz} := \frac{A_{sy.is}}{b \cdot d_{eff}} = 0.009$$

**Length of the control perimeter:**

$$c_1 := 350\ mm \qquad c_2 := 350\ mm$$

$$u_1 := 2 \cdot (c_1 + c_2) + 2 \cdot \pi \cdot 2 \cdot d_{eff} = (4.623 \cdot 10^3)\ mm$$

**Shear stress at the control perimeter (distance 2d from the column edge)**

$\beta := 1.15$ The recommended value for an inner column according to ***6.4.3 (6)***

$$v_{Ed} := \beta \cdot \frac{V_{Ed}}{u_1 \cdot d_{eff}} = 0.656\ \frac{N}{mm^2}$$ ***[Eq. 6.48]***

**Shear capacity without reinforcement*: [6.4.4(1)]***

$$f_{ck} := 45\ \frac{N}{mm^2} \qquad k := 1 + \sqrt{\frac{200\ mm}{d_{eff}}} = 1.883$$

$$\rho_l := (\rho_{ly} \cdot \rho_{lz})^{0.5} = 0.008$$

$$V_{Rd.c} := \frac{0.18}{\gamma_c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{\frac{1}{3}}\ \frac{N}{mm^2} = 0.744\ \frac{N}{mm^2}$$

$$v_{min} := 0.035 \cdot k^{1.5} \cdot \sqrt{f_{ck}}\ \frac{N}{mm^2} = 0.607\ \frac{N}{mm^2}$$

$$V_{Rd.c} > v_{Ed} \qquad \frac{v_{Ed}}{V_{Rd.c}} = 0.881$$

Punching for slab 69 by inner column ok; no further check is needed!

### 7.1.8 Punching for inner column according to EN 1992-1-1:2021-1

**Shear-resisting effective depth of the slab and reinf. ratio:**

According to note 1 in 8.4.2, it is recommended to use nominal effective depths, thus:

$$d_v := d_{eff} = 256.5\ mm$$

According to 8.4.3(1) The values of $\rho_{l,x}$ and $\rho_{l,y}$ should be calculated as mean values over the width $b_s$

$$A_{sx.is} := 1819\ \frac{mm^2}{m} \qquad \rho_{ly} := \frac{A_{sx.is}}{d_v \cdot b} = 0.007$$

$$A_{sy.is} := 2238\ \frac{mm^2}{m} \qquad \rho_{lz} := \frac{A_{sy.is}}{d_v \cdot b} = 0.009$$

**Length of the control perimeter distance 0.5d from the column edge:**

$$b_{0.5} := 2 \cdot (c_1 + c_2) + 2 \cdot \pi \cdot 0.5 \cdot d_v = (2.206 \cdot 10^3)\ mm$$

**Shear stress at the control perimeter (distance 0.5d from the column edge):**

Assuming the conditions defined in [8.4.2 (6)] for the approximated $\beta_e$.
The approximated value for an internal column:

$$\beta_e := 1.15$$

$$v_{Ed} := \beta_e \cdot \frac{V_{Ed}}{b_{0.5} \cdot d_{eff}} = 1.375\ \frac{N}{mm^2}$$

**Punching shear stress resistance without reinforcement:**

$$d_{dg} := 32\ mm$$

$$\rho_l := \sqrt{\rho_{lz} \cdot \rho_{ly}} = 0.008$$

$$b_0 := c_1 \cdot 2 + c_2 \cdot 2 = (1.4 \cdot 10^3)\ mm$$

$$k_{pb} := 3.6 \cdot \sqrt{1 - \frac{b_0}{b_{0.5}}} = 2.176$$

$$\tau_{Rd.c} := \frac{0.6}{\gamma_v} \cdot k_{pb} \cdot \left(100 \cdot \rho_l \cdot f_{ck} \cdot \frac{d_{dg}}{d_v}\right)^{\frac{1}{3}} \frac{N}{mm^2} = 1.428\ \frac{N}{mm^2}$$

$$\frac{v_{Ed}}{\tau_{Rd.c}} = 0.963$$

Punching for slab 69 by inner column ok, no further check is needed!

**7.1.9 Deflection control according to** ***[EC.2:2004 and Annex B]***
**Following are presumed:**

30% of the live loads, is presumed to be permanent.
Live loads are applied, 90 days after casting.
Self-weight are applied, 7 days after casting.

**Creep coefficient [EC.2:2004 annex B]:**

$$f_{cm} := 53 \frac{N}{mm^2}$$

$$\alpha_1 := \left(\frac{35}{f_{cm}}\right)^{0.7} = 0.748 \qquad \alpha_2 := \left(\frac{35}{f_{cm}}\right)^{0.2} = 0.92 \qquad \alpha_3 := \left(\frac{35}{f_{cm}}\right)^{0.5} = 0.813$$

$$A_c := 1000 \cdot 300 = 3 \cdot 10^5 \quad mm^2$$

$$u := 2 \cdot 1000 = 2 \cdot 10^3 \ mm \qquad h_0 := 2 \cdot \frac{A_c}{u} = 300 \ mm$$

Assume: $RH := 50\%$

$$\varphi_{RH} := \left(1 + \frac{1 - RH}{0.1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right) \cdot \alpha_2 = 1.434$$

$$\beta_f(f_{cm}) := \frac{16.8}{\sqrt{f_{cm}}} \qquad \beta_f(53) = 2.308$$

**At: t=7 days:**

$$t := 50 \cdot 365 = 1.825 \cdot 10^4 \qquad days$$

$$\beta_t(t_0) := \frac{1}{\left(1 + {t_0}^{0.2}\right)} \qquad \beta_t(7) = 0.404$$

$$\varphi_0 := \varphi_{RH} \cdot \beta_f(53) \cdot \beta_t(7) = 1.337$$

$$\beta_H := 1.5 \cdot \left(1 + (0.012 \cdot RH)^{18}\right) \cdot h_0 + 250 \cdot \alpha_3 = 653.159$$

$$\beta(t, t_0) := \left(\frac{(t - t_0)}{\beta_H + t - t_0}\right) \qquad \beta\left(1.825 \cdot 10^4, 7\right) = 0.965$$

$$\varphi(t, t_0) := \varphi_0 \cdot \beta(1.825 \cdot 10^4, 7) = 1.291$$

**At t=90 days:**

$$\beta_t(90) = 0.289$$

$$\varphi_0 := \varphi_{RH} \cdot \beta_f(53) \cdot \beta_t(90) = 0.957$$

$$\beta(1.825 \cdot 10^4, 90) = 0.965$$

$$\varphi_{90}(t, t_0) := \varphi_0 \cdot \beta(1.825 \cdot 10^4, 90) = 0.924$$

**Effective E-modulus [EC.2:2004 7.4.3(5)]:**

$$E_{cm} := 36 \cdot 10^3 \ MPa$$

$$E_{cm.7} := \frac{E_{cm}}{1 + 1.291} = (1.571 \cdot 10^4) \ MPa$$

$$E_{cm.90} := \frac{E_{cm}}{1 + 0.924} = (1.871 \cdot 10^4) \ MPa$$

$$M_g := 7.5 \cdot \frac{l_y^{\ 2}}{8} \frac{kN}{m} = 50.646 \ kN \cdot m$$

$$M_{0.3p} := 0.3 \cdot \frac{3 \cdot l_y^{\ 2}}{8} \frac{kN}{m} = 6.078 \ kN \cdot m$$

$$M_{0.7p} := 0.7 \cdot \frac{3 \cdot l_y^{\ 2}}{8} \frac{kN}{m} = 14.181 \ kN \cdot m$$

$$M_{snow} := 2.8 \cdot \frac{l_y^{\ 2}}{8} \frac{kN}{m} = 18.908 \ kN \cdot m$$

$$E_{c.middel} := \frac{M_g + M_{0.3p} + M_{0.7p} + M_{snow}}{\frac{M_g}{E_{cm.7}} + \frac{M_{0.3p}}{E_{cm.90}} + \frac{M_{0.7p}}{E_{cm}} + \frac{M_{snow}}{E_{cm}}} = (2.011 \cdot 10^4) \ MPa$$

**Bending stiffness in x-direction**

**Above support:**

$A_{sx.is} := 1885 \ mm^2$ "ø12 cc. 60"

$$E_s := 2 \cdot 10^5 \ MPa$$

$$\rho_{sx} := \frac{A_{sx.is}}{b \cdot d_x} = 0.007$$

$$\eta := \frac{E_s}{E_{c.middel}} = 9.947$$

$$\alpha_{sx} := \sqrt{\left(\left(\eta \cdot \rho_{sx}\right)^2 + \left(2 \cdot \eta \cdot \rho_{sx}\right)\right)} - \eta \cdot \rho_{sx} = 0.313$$

$$I_{sx} := 0.5 \cdot \alpha_{sx}^{\ 2} \cdot \left(1 - \frac{\alpha_{sx}}{3}\right) \cdot b \cdot d_x^{\ 3} = \left(8.049 \cdot 10^8\right)\ mm^4$$

**At the middle of the span:**

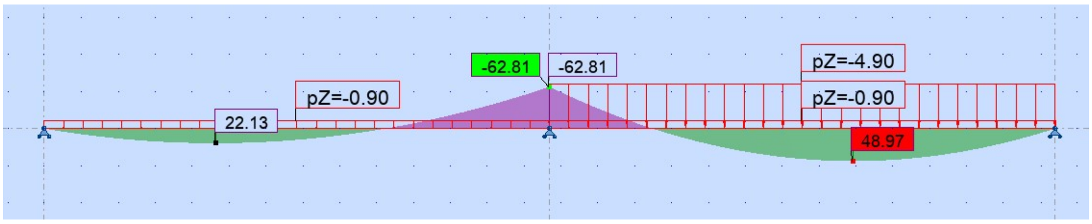
$A_{sx.if} := 808\ mm^2$ ***Ø12 C.C 140mm*** $\rho_{fx} := \frac{A_{sx.if}}{d_x \cdot b} = 0.003$

$$\alpha_{fx} := \sqrt{\left(\left(\eta \cdot \rho_{fx}\right)^2 + \left(2 \cdot \eta \cdot \rho_{fx}\right)\right)} - \eta \cdot \rho_{fx} = 0.218$$

$$I_{fx} := 0.5 \cdot \alpha_{fx}^{\ 2} \cdot \left(1 - \frac{\alpha_{fx}}{3}\right) \cdot b \cdot d_x^{\ 3} = \left(4.061 \cdot 10^8\right)\ mm^4$$

**Moment diagram in x-direction (SLS):**

**Checking for the left field.**




**Deflection in i x-direction:**

$$I_{xm} := 0.85 \cdot I_{fx} + 0.15 \cdot I_{sx} = \left(4.659 \cdot 10^8\right)\ mm^4$$

$E_{c.middel} \cdot I_{xm} = \left(9.368 \cdot 10^{12}\right)\ N \cdot mm^2$ $l_x = 6.85\ m$

The deflection is now calculated by using the virtual unit load method (converting the moment diagram at the left-field to a sum of a rectangle and parabola-shaped moment diagrams):

$m_f := 74.09\ kN \cdot m$ $m_s := -62.8\ kN \cdot m$

$$\delta_{xs} := \frac{l_x}{E_{c.middel} \cdot I_{xm}} \cdot \left( \left( \frac{5}{12} \cdot m_f \cdot 0.24 \cdot l_x \cdot (0.4 + 0.6) \right) + \left( \frac{1}{6} \cdot \left( 1 + \frac{0.4\ l_x}{l_x} \right) \cdot m_s \cdot 0.24 \cdot l_x \right) \right) = 19.496\ mm$$

**Bending stiffness in i y-direction:**

**Above support:**

$A_{sy.is} := 2513\ mm^2$ **Ø 12 C.C 45 mm** $\rho_{sy} := \frac{A_{sy.is}}{b \cdot d_y} = 0.01$

$$\alpha_{sy} := \sqrt{\left( (\eta \cdot \rho_{sy})^2 + (2 \cdot \eta \cdot \rho_{sy}) \right)} - \eta \cdot \rho_{sy} = 0.359$$

$$I_{sy} := 0.5 \cdot \alpha_{sy}^2 \cdot \left( 1 - \frac{\alpha_{sy}}{3} \right) \cdot b \cdot d_y^3 = (8.749 \cdot 10^8)\ mm^4$$

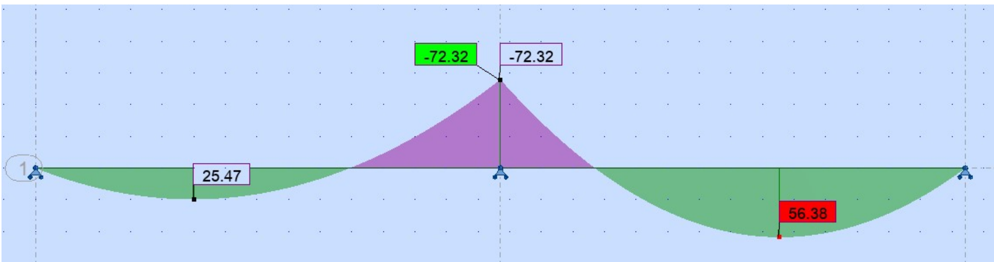
**In the middle of the span:**

$A_{sy.if} := 983\ mm^2$ **(Ø 12 C.C 115 mm)** $\rho_{fy} := \frac{A_{sy.if}}{b \cdot d_y} = 0.004$

$$\alpha_{fy} := \sqrt{\left( (\eta \cdot \rho_{fy})^2 + (2 \cdot \eta \cdot \rho_{fy}) \right)} - \eta \cdot \rho_{fy} = 0.244$$

$$I_{fy} := 0.5 \cdot \alpha_{fy}^2 \cdot \left( 1 - \frac{\alpha_{fy}}{3} \right) \cdot b \cdot d_y^3 = (4.213 \cdot 10^8)\ mm^4$$

**Moment diagram in y-direction from Robot (SLS)**




**Deflection in y-direction**

$$I_{ym} := 0.85 \cdot I_{fy} + 0.15 \cdot I_{sy} = (4.893 \cdot 10^8)\ mm^4$$

$$E_{c.middel} \cdot I_{ym} = (9.838 \cdot 10^{12})\ N \cdot mm^2$$

$$m_{f1} := 85.3\ kN \cdot m \quad m_{s1} := -72.32\ kN \cdot m$$

$$\delta_{ys} := \frac{l_y}{E_{c.middel} \cdot I_{ym}} \cdot \left( \left( \frac{5}{12} \cdot m_{f1} \cdot 0.24 \cdot l_y \cdot (0.4 + 0.6) \right) + \left( \frac{1}{6} \cdot \left( 1 + \frac{0.4\ l_y}{l_y} \right) \cdot m_{s1} \cdot 0.24 \cdot l_y \right) \right) = 24.601\ mm$$

**Total deflection:**

$$\delta := \delta_{xs} + \delta_{ys} = 44.097\ mm \qquad \frac{l_x}{250} = 27.4\ mm \qquad \delta > 27.4\ mm$$

Too high deflection, not ok!

### 7.1.10 Deflection control according to EN 1992-1-1:2021-1 and B annex:

**Creep coefficient at 7 days:**

Assume CEMII/B: $\alpha_{sc} := -1$

Assume: $T := 20$ $\quad t_{0.T} := 7$

$$t_{0.adj} := t_{0.T} \cdot \left( \frac{9}{2 + {t_{0.T}}^{1.2}} + 1 \right)^{\alpha_{sc}} = 4.046$$ ***Eq. [B.17]***

$$\alpha_{fcm} := \left( \frac{35}{f_{cm}} \right)^{0.5} = 0.813$$ ***Eq. [B.16]***

$$h_n := \frac{2 \cdot A_c}{u} = 300$$

$$\alpha_{fcm} \cdot 1500 = 1.219 \cdot 10^3$$

$$\beta_h := 1.5 \cdot h_n + 250 \cdot \alpha_{fcm} = 653.159 \quad \left(653.159 < 1219 \cdot 10^3\right)$$

$$\gamma\left(t_{0.adj}\right) := \frac{1}{2.3 + \frac{3.5}{\sqrt{t_{0.adj}}}} \qquad \gamma\left(t_{0.adj}\right) = 0.248$$

$$\beta_{bc.fcm} := \frac{1.8}{{f_{cm}}^{0.7}} = 0.112$$

$$t := 50 \cdot 365 = 1.825 \cdot 10^4 \qquad t_0 := t_{0.T} = 7$$

$$\beta_{bc.t\_t0} := \ln\left( \left( \frac{30}{t_{0.adj}} + 0.035 \right)^2 \cdot \left(t - t_0\right) + 1 \right) = 13.828$$

$$\varphi_{bc}\left(t, t_0\right) := \beta_{bc.fcm} \cdot \beta_{bc.t\_t0} = 1.545$$

$$\beta_{dc.fcm} := \frac{412}{{f_{cm}}^{1.4}} = 1.588$$

$$\beta_{dc.RH} := \frac{1 - RH}{\sqrt[3]{0.1 \cdot \frac{h_n}{100}}} = 0.747$$

$$\beta_{dc.t0} := \frac{1}{0.1 + t_{0.adj}^{\ 0.2}} = 0.703$$

$$\beta_{dc.t\_t0} := \left(\frac{t - t_0}{\beta_h + \left(t - t_0\right)}\right)^{\gamma\left(t_{0.adj}\right)} = 0.991$$

$$\beta_{dc}\left(t, t_0\right) := \beta_{dc.fcm} \cdot \beta_{dc.RH} \cdot \beta_{dc.t0} \cdot \beta_{dc.t\_t0} = 0.827$$

$$\beta\left(t.t_0\right) := \varphi_{bc}\left(t, t_0\right) \cdot \beta_{dc}\left(t, t_0\right) = 1.277$$

**Creep coefficient at 90 days:**

$$t_0 := 90 \qquad t_{0.T} := t_0 = 90$$

$$t_{0.adj} := t_{0.T} \cdot \left(\frac{9}{2 + t_{0.T}^{\ 1.2}} + 1\right)^{\alpha_{sc}} = 86.514$$

$$\gamma(90) = 0.375$$

$$\beta_{bc.t\_t0} := \ln\left(\left(\frac{30}{t_{0.adj}} + 0.035\right)^2 \cdot \left(t - t_0\right) + 1\right) = 7.881$$

$$\varphi_{bc}\left(t, t_0\right) := \beta_{bc.fcm} \cdot \beta_{bc.t\_t0} = 0.881$$

$$\beta_{dc.t0} := \frac{1}{0.1 + t_{0.adj}^{\ 0.2}} = 0.394$$

$$\beta_{dc.t\_t0} := \left(\frac{t - t_0}{\beta_h + \left(t - t_0\right)}\right)^{\gamma\left(t_{0.adj}\right)} = 0.987$$

$$\beta_{dc}\left(t, t_0\right) := \beta_{dc.fcm} \cdot \beta_{dc.RH} \cdot \beta_{dc.t0} \cdot \beta_{dc.t\_t0} = 0.461$$

$$\beta\left(t.t_0\right) := \varphi_{bc}\left(t, t_0\right) \cdot \beta_{dc}\left(t, t_0\right) = 0.406$$

**Effective E-modulus:**

Assume: $\kappa_E := 9500$

$$E_{cm} := \kappa_E \cdot f_{cm}^{\ \frac{1}{3}}\ MPa = \left(3.568 \cdot 10^4\right)\ MPa$$

$$E_{c7.eff} := \frac{1.05 \cdot E_{cm}}{1 + 1.277} = \left(1.646 \cdot 10^{4}\right)\ MPa$$ ***[9.1(3) eq.(3)]***

$$E_{c90.eff} := \frac{1.05 \cdot E_{cm}}{1 + 0.406} = \left(2.665 \cdot 10^{10}\right)\ Pa$$

$$E_{c.middel} := \frac{M_g + M_{0.3p} + M_{0.7p} + M_{snow}}{\frac{M_g}{E_{c7.eff}} + \frac{M_{0.3p}}{E_{c90.eff}} + \frac{M_{0.7p}}{E_{cm}} + \frac{M_{snow}}{E_{cm}}} = \left(2.122 \cdot 10^{4}\right)\ MPa$$

**<u>Bending stiffness in x-direction:</u>**

**Above support:**

$$A_{sx.is} := 1885\ mm^2 \qquad \rho_{sx} := \frac{A_{sx.is}}{b \cdot d_x} = 0.007$$

$$\eta := \frac{E_s}{E_{c.middel}} = 9.427$$

$$\alpha_{sx} := \sqrt{\left(\left(\eta \cdot \rho_{sx}\right)^2 + \left(2 \cdot \eta \cdot \rho_{sx}\right)\right)} - \eta \cdot \rho_{sx} = 0.306$$

$$I_{sx} := 0.5 \cdot \alpha_{sx}^{\ 2} \cdot \left(1 - \frac{\alpha_{sx}}{3}\right) \cdot b \cdot d_x^{\ 3} = \left(7.722 \cdot 10^{8}\right)\ mm^4$$

**Middle of the span:**

$$A_{sx.if} := 808\ mm^2 \qquad \rho_{fx} := \frac{A_{sx.if}}{b \cdot d_x} = 0.003$$

$$\alpha_{fx} := \sqrt{\left(\left(\eta \cdot \rho_{fx}\right)^2 + \left(2 \cdot \eta \cdot \rho_{fx}\right)\right)} - \eta \cdot \rho_{fx} = 0.213$$

$$I_{fx} := 0.5 \cdot \alpha_{fx}^{\ 2} \cdot \left(1 - \frac{\alpha_{fx}}{3}\right) \cdot b \cdot d_x^{\ 3} = \left(3.881 \cdot 10^{8}\right)\ mm^4$$

**<u>Deflection in x-direction:</u>**

$$I_{xm} := 0.85 \cdot I_{fx} + 0.15 \cdot I_{sx} = \left(4.457 \cdot 10^{8}\right)\ mm^4$$

$$E_{c.middel} \cdot I_{xm} = \left(9.456 \cdot 10^{12}\right)\ N \cdot mm^2$$

$$\delta_{xs} := \frac{l_x}{E_{c.middel} \cdot I_{xm}} \cdot \left(\left(\frac{5}{12} \cdot m_f \cdot 0.24 \cdot l_x \cdot (0.4 + 0.6)\right) + \left(\frac{1}{6} \cdot \left(1 + \frac{0.4\ l_x}{l_x}\right) \cdot m_s \cdot 0.24 \cdot l_x\right)\right) = 19.314\ mm$$

**Bending stiffness:**

**Above support:**

$$A_{sy.is} := 2261 \ mm^2$$

$$\rho_{sy} := \frac{A_{sy.is}}{b \cdot d_y} = 0.009$$

$$\alpha_{sy} := \sqrt{\left(\left(\eta \cdot \rho_{sy}\right)^2 + \left(2 \cdot \eta \cdot \rho_{sy}\right)\right)} - \eta \cdot \rho_{sy} = 0.337$$

$$I_{sy} := 0.5 \cdot \alpha_{sy}{}^2 \cdot \left(1 - \frac{\alpha_{sy}}{3}\right) \cdot b \cdot d_y{}^3 = \left(7.778 \cdot 10^8\right) \ mm^4$$

**Middle of the span:**

$$A_{sy.if} := 983 \ mm^2$$

$$\rho_{fy} := \frac{A_{sy.if}}{b \cdot d_y} = 0.004$$

$$\alpha_{fy} := \sqrt{\left(\left(\eta \cdot \rho_{fy}\right)^2 + \left(2 \cdot \eta \cdot \rho_{fy}\right)\right)} - \eta \cdot \rho_{fy} = 0.238$$

$$I_{fy} := 0.5 \cdot \alpha_{fy}{}^2 \cdot \left(1 - \frac{\alpha_{fy}}{3}\right) \cdot b \cdot d_y{}^3 = \left(4.03 \cdot 10^8\right) \ mm^4$$

**Deflection in y-direction:**

$$I_{ym} := 0.85 \cdot I_{fy} + 0.15 \cdot I_{sy} = \left(4.592 \cdot 10^8\right) \ mm^4$$

$$E_{c.middel} \cdot I_{ym} = \left(9.743 \cdot 10^{12}\right) \ N \cdot mm^2$$

$$\delta_{ys} := \frac{l_y}{E_{c.middel} \cdot I_{xm}} \cdot \left(\left(\frac{5}{12} \cdot m_{f1} \cdot 0.24 \cdot l_y \cdot (0.4 + 0.6)\right) + \left(\frac{1}{6} \cdot \left(1 + \frac{0.4 \ l_y}{l_y}\right) \cdot m_{s1} \cdot 0.24 \cdot l_y\right)\right) = 25.595 \ mm$$

**Total deflection:**

$$\delta := \delta_{xs} + \delta_{ys} = 44.909 \ mm$$

$$\frac{l_x}{250} = 27.4 \ mm \qquad \delta > 27.4 \ mm$$

Too high deflection, not ok!

### 7.1.11 Crack control according to *[7.3.4 EC.2:2004]*

$c_{min.dur} := 15\ mm$

**Maximum allows crack width:**

$k_c := \frac{c_{nom}}{c_{min.dur}} = 2 \qquad k_c := 1.3$

***[EC.2:2004 NA.7.3.1 table NA.7.1]***

$w_{max} := k_c \cdot 0.3 = 0.39$

**Crack width in y-direction:**

**Reinforcement stress** - Checking for the tension zone in the y-direction at the outer column strip.

$M_{Ed.yf} := 72.3\ kN \cdot m$

$A_{sy.ys} := 1508\ mm^2$ "ø12 cc. 75 mm"

$\rho := \frac{A_{sy.ys}}{b \cdot d_y} = 0.006 \qquad \alpha := \sqrt{(\eta \cdot \rho)^2 + 2 \cdot \eta \cdot \rho} - \eta \cdot \rho = 0.286$

$I_y := 0.5 \cdot \alpha^2 \cdot \left(1 - \frac{\alpha}{3}\right) \cdot b \cdot d_y^{\ 3} = \left(5.697 \cdot 10^8\right)\ mm^4$

$E_{c.middel} := \left(2.011 \cdot 10^4\right)\ MPa$

$\sigma_s := E_s \cdot \frac{M_{Ed.yf} \cdot (1 - \alpha) \cdot d_y}{E_{c.middel} \cdot I_y} = 224.519\ MPa$

**Crack width calculation:**

$h := 300\ mm$

$h_{c.eff} := min\left(2.5 \cdot \left(h - d_y\right), \frac{1}{3} \cdot \left(h - \alpha \cdot d_y\right), 0.5 \cdot h\right) = 76.295\ mm$

$A_{c.eff} := b \cdot h_{c.eff} = \left(7.63 \cdot 10^4\right)\ mm^2$

$\rho_{p.eff} := \frac{A_{sy.ys}}{A_{c.eff}} = 0.02$

$k_t := 0.4 \qquad f_{ct.eff} := f_{ctm} = 3.8\ \frac{N}{mm^2}$

$$E_{cm} := 36 \cdot 10^3 \ MPa$$

$$\alpha_e := \frac{E_s}{E_{cm}} = 5.556$$

$$\varepsilon_{sm\_cm} := \frac{\sigma_s - k_t \cdot \frac{f_{ct.eff}}{\rho_{p.eff}} \cdot \left(1 + \alpha_e \cdot \rho_{p.eff}\right)}{E_s} = 6.959 \cdot 10^{-4}$$ ***[EC.2:2004 7.3.4(2)]***

$$0.6 \cdot \frac{\sigma_s}{E_s} = 6.736 \cdot 10^{-4} \quad 6.424 \cdot 10^{-4} > 6.398 \cdot 10^{-4} \qquad ok!$$

$$5 \cdot \left(c_{nom} + \frac{\varnothing}{2}\right) = 180 \ mm \qquad (180 \ mm > 75 \ mm)$$ ***[EC.2 2.7.3.4(eq.7.12)]***

Assume good bond conditions: $k_1 := 0.8$

Pure bending: $k_2 := 0.5$

Recommended values: $k_3 := 3.4$ $k_4 := 0.425$

$$S_{r.max} := k_3 \cdot c_{nom} + k_1 \cdot k_2 \cdot k_4 \cdot \frac{\varnothing}{\rho_{p.eff}} = 205.211 \ mm$$

$$\omega_h := S_{r.max} \cdot \left(\varepsilon_{sm\_cm}\right) = 0.143 \ mm \qquad 0.143 \ mm < 0.39 \ mm$$

Crack width is under the limit ok!

### 7.1.12 Crack width control according to *[9.2.4 EN 1992-1-1:2021-1]*

$$k_w := 1.7$$

$$E_{c.middel} := \left(2.122 \cdot 10^4\right) \ MPa$$

$A_{sy.ys} := 1508 \ mm^2$ **Ø12 C.C 75 mm**

$$\rho := \frac{A_{sy.ys}}{b \cdot d_y} = 0.006 \qquad \eta := \frac{E_s}{E_{c.middel}} = 9.425$$

$$\alpha := \sqrt{(\eta \cdot \rho)^2 + 2 \cdot \eta \cdot \rho} - \eta \cdot \rho = 0.286$$

$$\alpha_{y.i} := \varnothing = 12 \ mm$$

$$k_{1\_r} := \frac{\left(h - \alpha \cdot d_y\right)}{h - \alpha_{y.i} - \alpha \cdot d_y} = 1.055$$

$$I_y := 0.5 \cdot \alpha^2 \cdot \left(1 - \frac{\alpha}{3}\right) \cdot b \cdot {d_y}^3 = \left(5.696 \cdot 10^8\right)\ mm^4$$

$$\sigma_s := E_s \cdot \frac{M_{Ed.yf} \cdot (1-\alpha) \cdot d_y}{E_{c.middel} \cdot I_y} = 212.805\ MPa$$

$$f_{ct.eff} = 3.8\ MPa$$

$$h_{c.eff} := min\left(\left(h - d_y\right) + 5 \cdot ø , 10 \cdot ø , 3.5 \cdot \left(h - d_y\right)\right) = 111\ mm$$

$$h - \alpha \cdot d_y = 228.89\ mm \qquad 228.89 > h_{c.eff}$$

$$\rho_{p.eff} := \frac{A_{sy.ys}}{b \cdot h_{c.eff}} = 0.014$$

Long term loading: $k_t := 0.4$

$$E_{cm} := \left(3.568 \cdot 10^4\right)\ MPa$$

$$\alpha_e := \frac{E_s}{E_{cm}} = 5.605$$

$$\varepsilon_{sm\_\xi cm} := k_{1\_r}\ \frac{\sigma_s - k_t \cdot \dfrac{f_{ct.eff}}{\rho_{p.eff}} \cdot \left(1 + \alpha_e \cdot \rho_{p.eff}\right)}{E_s} = 4.876 \cdot 10^{-4}$$ ***[9.2.4(3) eq.9.13]***

$$\varepsilon_{sm\_\xi cm.min} := 0.6 \cdot \frac{\sigma_s}{E_s} = 6.384 \cdot 10^{-4}$$

$$\varepsilon_{sm\_\xi cm} := \varepsilon_{sm\_\xi cm.min} = 6.384 \cdot 10^{-4}$$

Pure bending: $k_{fl} := \dfrac{\left(h - h_{c.eff}\right)}{h} = 0.63$

Assume good casting position: $k_b := 0.9$

$$s_{rm.cal} := 1.5 \cdot c_{nom} + \frac{k_{fl} \cdot k_b}{7.2} \cdot \frac{ø}{\rho_{p.eff}} = 114.559\ mm$$

$$w_{k.cal} := k_w \cdot s_{rm.cal} \cdot \left(\varepsilon_{sm\_\xi cm}\right) = 0.124\ mm$$ ***[9.2.4(2) Eq.9.12]***

**Maximum allowed crack width:**

Assume: $k_{surf} := 1$

$$w_{lim.cal} := 0.3 \cdot k_{surf} \cdot mm = 0.3\ mm$$ ***For XC1 [table 9.2, EN 1992-1-1:2021]***

Crack width is under the limit ok!

## 6.2 Detailed calculation of beam nu.61

Design of beam nu. 61 according to the two versions of EC.2 is presented in this section. In the presented calculations, longitudinal reinforcement, shear reinforcement, and anchorage length are determined, and some assumptions are made.

### 7.2.1 Geometry, applied loads, exposure class, and materials

| | | | |
|---|---|---|---|
| Thickness: | h=300 mm ; | Live load: | p=3$\frac{kN}{m^2}$ |
| Width: | b= 450 mm ; | Snow load: | S= 2.8$\frac{kN}{m^2}$ |
| span: | l= 7.18 m ; | Self weight: | g=25$\frac{kN}{m^3} * h = 7.5\frac{kN}{m^2}$ |
| Design life time: | 50 years | | |
| Exposure class: | XC2 | | |
| Concrete strength | $f_{ck} = 35 * \frac{N}{mm^2}$ | | |
| Yield strength of reinforcement | $f_{yk} = 500 * \frac{N}{mm^2}$ | | |
| Assumed main bar diameter | $\phi_{main} = 14mm$ | | |
| Assumed stirrup diameter | $\phi_{stirrups} = 14mm$ | | |
| Nominal cover | $C_{nom} = 40mm$ | | |
| Effective depth: | d= h-$c_{nom} - \phi_{stirrup} - \frac{\phi_{main}}{2} = 245mm$ | | |

**Table 6.3:** Data regarding the design of beam nu.61

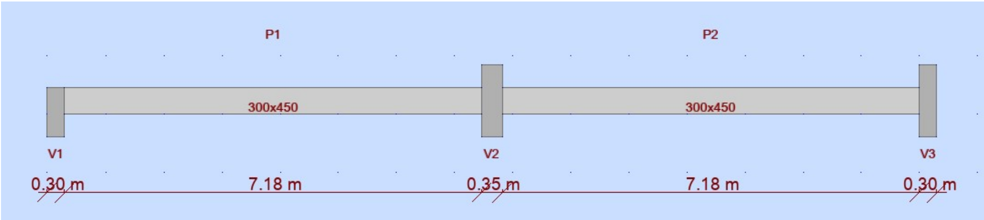
### 7.2.3 Summary of the design of beam nu.61

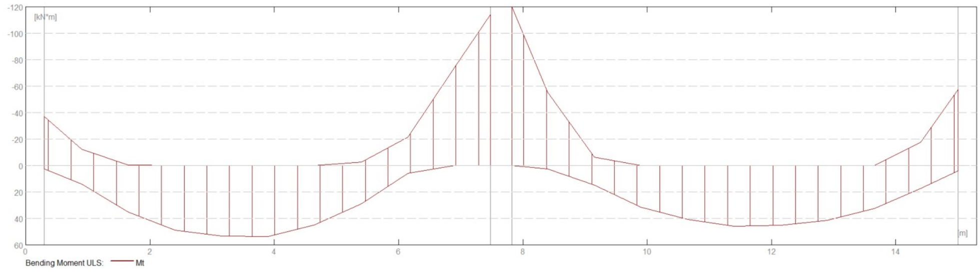
| **Summary of the design of beam nu.61** | | |
|---|---|---|
| | **EN 1992-1-1** | **EN 1992-1-1:2021-1** |
| **Moment capacity** | **[kN*m]** | **[kN*m]** |
| $M_{Rd}$ | 147.30 | 173.30 |
| **Minimum and maximum reinforcement** | **[mm$^2$]** | [$mm^2$] |
| $A_{s,min}$ | 183.50 | 195.90 |
| $A_{s,max}$ | 5400.00 | 5400.00 |
| **Required tension and compression reinforcement** | **[mm$^2$]** | [$mm^2$] |
| $A_{s,support,tension}$ | 1308.00 | 1277.00 |
| $A_{s,field,tension}$ | 539.10 | 533.80 |
| $A_{s,field,compression}$ | 183.50 | 196.00 |
| **Shear capacity without shear reinforcement** | **[N/mm$^2$]** | [$N/mm^2$] |
| $\tau_{Rd,c}$ | 0.791 | 0.697 |
| **Provided shear stirrups** | **[stirrup/mm]** | **[stirrup/mm]** |
| $A_{sw,H1}$ | Ø8 c.c. 90 mm | Ø8 c.c. 100 mm |
| $A_{sw,H2}$ | Ø8 c.c. 125 mm | Ø8 c.c. 135 mm |
| $A_{sw,V1}$ | Ø8 c.c. 160 mm | Ø8 c.c. 180 mm |
| $A_{sw,V2}$ | Ø8 c.c. 160 mm | Ø8 c.c. 180 mm |
| **Anchorage length $l_{bd}$** | **[mm]** | **[mm]** |
| Beyond the left support | 140.00 | 140.00 |
| Beyond the right support | 227.35 | 228.20 |

**Table 6.4:** Summary of the design of beam nu.61.

### 7.2.3 Design of beam nu.61 (located at 3. floor at B-axes) according to EC.2:2004

**Layout and moment diagram:**



$M_{Ed.field} := 53.86 \ kN \cdot m$ $\qquad M_{Ed.support} := 120 \ kN \cdot m$

**Moment capacity:**

$$f_{cd} := 0.85 \cdot \frac{f_{ck}}{1.5} = 19.833 \ MPa$$

$$M_{Rd} := 0.275 \cdot f_{cd} \cdot b \cdot d^2 = 147.324 \ kN \cdot m$$

**Minimum and maximum reinforcement areas:**

$$A_{s.min} := 0.13\% \cdot b \cdot d = 143.325 \ mm^2$$

$$A_{s.min1} := 0.26 \cdot \left(\frac{f_{ctm}}{f_{yk}}\right) \cdot b \cdot d = 183.456 \ mm^2$$

$$A_{s.min} := \max\left(A_{s.min}, A_{s.min1}\right) = 183.456 \ mm^2$$

$$A_{s.max} := 0.04 \cdot b \cdot h = \left(5.4 \cdot 10^3\right) \ mm^2$$

**Longitudinal reinforcement:**

$$z_{max} := 0.95 \cdot d = 232.75 \ mm$$

$$z := \left(1 - 0.17 \cdot \frac{M_{Ed.field}}{M_{Rd}}\right) \cdot d = 229.773 \ mm$$

$$A_{s.field.tension} := \frac{M_{Ed.field}}{f_{yd} \cdot z} = 539.132 \ mm^2$$

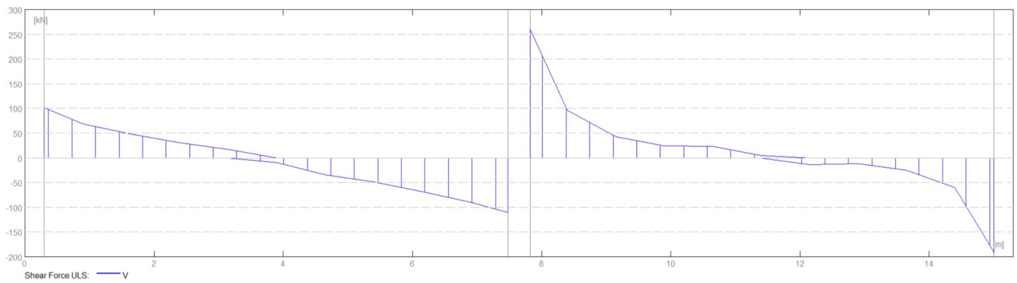
$$z := \left(1 - 0.17 \cdot \frac{M_{Ed.support}}{M_{Rd}}\right) \cdot d = 211.075\ mm$$

$$A_{s.support.tension} := \frac{M_{Ed.support}}{f_{yd} \cdot z} = \left(1.308 \cdot 10^3\right)\ mm^2$$

**Armering oversikt:**

| | |
|---|---|
| $A_{s.support.tension} = \left(1.308 \cdot 10^3\right)\ mm^2$ | **9ø14** |
| $A_{s.field.tension} = 539.132\ mm^2$ | **4ø14** |
| $A_{s.field.comp.} := A_{s.min} = 183.456\ mm^2$ | **2ø14** |

**Shear control:**




Design shear forces at the left beam:

$$V_{Ed.V1} := 110.5\ kN$$

$$V_{Ed.V2} := 101\ kN$$

Design shear forces at the right beam:

$$V_{Ed.H1} := 255.5\ kN$$

$$V_{Ed.H2} := 190.6\ kN$$

**Shear capacity without reinforcement:**

$$C_{Rd.c} := \frac{0.18}{1.5} = 0.12$$

$$k := 1 + \sqrt{\frac{200\ mm}{d}} = 1.904\ (1.909 < 2)$$

$$\rho_L := \frac{A_{s.support.tension}}{b \cdot d} = 0.012$$

$$V_{Rd.c} := C_{Rd.c} \cdot k \cdot \left(100 \cdot \rho_L \cdot 35\right)^{\frac{1}{3}} \cdot b \cdot d \cdot \frac{N}{mm^2} = 87.197\ kN$$

$$\tau_{Rd.c} := \frac{V_{Rd.c}}{b \cdot d} = 0.791 \ \frac{N}{mm^2}$$

$$V_{min} := 0.035 \cdot k^{\frac{3}{2}} \cdot 35^{\frac{1}{2}} \cdot b \cdot d \cdot \frac{N}{mm^2} = 59.953 \ kN \qquad ok!$$

**Tensile shear capacity:**

$$V_{Rd.s} := \frac{A_{sw}}{s} \cdot z \cdot f_{ywd} \cdot cot(\o) \geq V_{Ed}$$

$$z := 0.9 \cdot d = 220.5 \ mm$$

$$cot(\o) := 2.5$$

$$A_{sw.H1.perS} := \frac{V_{Ed.H1} \cdot mm}{f_{yd} \cdot z \cdot cot(\o) \cdot m} = 1.066 \ \frac{mm^2}{m}$$

$$A_{sw.H2.perS} := \frac{V_{Ed.H2} \cdot mm}{f_{yd} \cdot z \cdot cot(\o) \cdot m} = 0.795 \ \frac{mm^2}{m}$$

$$A_{sw.V1.perS} := \frac{V_{Ed.V1} \cdot mm}{f_{yd} \cdot z \cdot cot(\o) \cdot m} = 0.461 \ \frac{mm^2}{m}$$

$$A_{sw.V2.perS} := \frac{V_{Ed.V2} \cdot mm}{f_{yd} \cdot z \cdot cot(\o) \cdot m} = 0.421 \ \frac{mm^2}{m}$$

**Minimum shear reinforcement area:**

$$\rho_{w.min} := 0.1 \cdot \frac{\sqrt{35}}{f_{yk}} \cdot \frac{N}{mm^2} \cdot \frac{b \cdot mm}{m} = 0.532 \ \frac{mm^2}{m}$$

**Maximum spacing between shear reinforcement:**

$$h' := 0.9 \cdot h = 270 \ mm \qquad S_{I.max} := 0.6 \cdot h' = 162 \ mm$$

**Provided stirrups:**

$$A_{sw} := 2 \cdot \pi \cdot 4^2 \cdot mm^2 = 100.531 \ mm^2$$

$$A_{sw.H1} := \frac{A_{sw}}{A_{sw.H1.perS} \cdot 10^3} = 94.304 \ mm$$ **ø8 c.c. 90 mm**

$$A_{sw.H2} := \frac{A_{sw}}{A_{sw.H2.perS} \cdot 10^3} = 126.415 \ mm$$ **ø8 c.c. 125 mm**

$$A_{sw.V1} := \frac{A_{sw}}{\rho_{w.min} \cdot 10^3} = 188.809 \ mm$$ **ø8 c.c. 160 mm**

$$A_{sw.V1} := \frac{A_{sw}}{\rho_{w.min} \cdot 10^3} = 188.809 \ mm$$ **ø8 c.c. 160 mm**

**Maximum allowed shear stress in member:**

$v_1 := 0.6$ $\quad tan(ø) := 0.4$

$$V_{Rd.max} := v_1 \cdot f_{cd} \cdot b \cdot z \cdot \frac{1}{cot(ø) + tan(ø)} = 407.165 \ kN$$ $ok!$

**Anchorage length beyond the theoretical left support:**

$a$

$l_{b.min} := 10 \cdot ø_{main} = 140 \ mm$

Assume a good bond: $\eta_1 := 1$

ø<32 mm $\eta_2 := 1$

$\alpha_{ct} := 0.85$ $\quad f_{ctk.0.05} := 2.2 \ \frac{N}{mm^2}$ $\quad f_{ctd} := \alpha_{ct} \cdot \frac{f_{ctk.0.05}}{\gamma_c} = 1.247 \ \frac{N}{mm^2}$

$$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.805 \ \frac{N}{mm^2}$$

$$\Delta F_{td.V1} := 0.5 \cdot V_{Ed.V1} \cdot cot(ø) = 138.125 \ kN$$

$$\sigma_{sd.V1} := \frac{\Delta F_{td.V1}}{A_{s.support.tension}} = 105.633 \ \frac{N}{mm^2}$$

$$l_{b.rqd} := \frac{ø_{main}}{4} \cdot \frac{\sigma_{sd.V1}}{f_{bd}} = 131.806 \ mm$$

$$l_{b.rqd} := l_{b.min} = 140 \ mm$$

Bar in tension: $\alpha_1 := 1$ $\quad \alpha_2 := 1$ $\quad \alpha_3 := 1$ $\quad \alpha_4 := 1$ $\quad \alpha_5 := 1$

$$l_{bd} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 140 \ mm$$

**Anchorage length beyond the theoretical right support:**

$$\Delta F_{td.H2} := 0.5 \cdot V_{Ed.H2} \cdot cot(\phi) = 238.25\ kN$$

$$\sigma_{sd.H2} := \frac{\Delta F_{td.H2}}{A_{s.support.tension}} = 182.205\ \frac{N}{mm^2}$$

$$l_{b.rqd} := \frac{\phi_{main}}{4} \cdot \frac{\sigma_{sd.H2}}{f_{bd}} = 227.35\ mm$$

$$l_{bd} := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b.rqd} = 227.35\ mm$$

### 7.2.4 Design of beam nu.61 (located at the 3. floor at B-axes) according to EC.2:2021:

**Moment capacity**

$$\eta_{cc} := \left(\frac{40}{35}\right)^{\frac{1}{3}} = 1.046 \qquad \eta_{cc} := 1 \qquad \kappa_{tc} := 1 \qquad \gamma_c := 1.5$$

$$f_{cd} := \eta_{cc} \cdot \kappa_{tc} \cdot \frac{f_{ck}}{\gamma_c} = 23.333\ \frac{N}{mm^2}$$

$$M_{Rd} := 0.275 \cdot f_{cd} \cdot b \cdot d^2 = 173.322\ kN \cdot m$$

**Minimum and maximum reinforcement area**

$$f_{ctm} := 3.2\ \frac{N}{mm^2}$$

$$M_{cr} \le M$$

$$M_{cr} := \frac{1}{6} \cdot b \cdot h^2 \cdot f_{ctm} = 21.6\ kN \cdot m$$

$$M := A_{s.min} \cdot f_{yk} \cdot z$$

$$z := 0.9 \cdot d = 220.5\ mm$$

$$A_{s.min} := \frac{1}{6} \cdot b \cdot h^2 \cdot \frac{f_{ctm}}{f_{yk} \cdot z} = 195.918\ mm^2$$

**Required longitudinal reinforcement:**

$$z := \left(1 - 0.17 \cdot \frac{M_{Ed.field}}{M_{Rd}}\right) \cdot d = 232.057\ mm$$

$$A_{s.field.tension} := \frac{M_{Ed.field}}{f_{yd} \cdot z} = 533.825\ mm^2$$

$$z := \left(1 - 0.17 \cdot \frac{M_{Ed.support}}{M_{Rd}}\right) \cdot d = 216.164\ mm$$

$$A_{s.support.tension} := \frac{M_{Ed.support}}{f_{yd} \cdot z} = \left(1.277 \cdot 10^3\right)\ mm^2$$

**Provided longitudinal reinforcement:**

$A_{s.support.tension} = \left(1.277 \cdot 10^3\right)\ mm^2$ **9ø14**

$A_{s.field.tension} = 533.825\ mm^2$ **4ø14**

$A_{s.field.comp.} := A_{s.min} = 195.918\ mm^2$ **2ø14**

**Design shear stress:**

$$\tau_{Ed.H1} := \frac{V_{Ed.H1}}{b \cdot d} = 2.317\ \frac{N}{mm^2} \qquad \tau_{Ed.V1} := \frac{V_{Ed.V1}}{b \cdot d} = 1.002\ \frac{N}{mm^2}$$

$$\tau_{Ed.H2} := \frac{V_{Ed.H2}}{b \cdot d} = 1.729\ \frac{N}{mm^2} \qquad \tau_{Ed.V2} := \frac{V_{Ed.V2}}{b \cdot d} = 0.916\ \frac{N}{mm^2}$$

**Minimum shear stress:**

$$d_{lower} := 8\ mm$$

$$d_{dg} := 16\ mm + d_{lower} = 24\ mm$$

$$\gamma_v := 1.5$$

$$\tau_{Rd.c.min} := \frac{11}{\gamma_v} \cdot \sqrt[2]{\left(\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}\right)}\ \frac{N}{mm^2} = 0.651\ \frac{N}{mm^2}$$

**Shear stress without shear reinforcement:**

$$\rho_L := \frac{A_{s.support.tension}}{b \cdot d} = 0.012$$

$$\tau_{Rd.c} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho_L \cdot 35 \cdot \frac{d_{dg}}{d}\right)^{\left(\frac{1}{3}\right)} \cdot \frac{N}{mm^2} = 0.697\ \frac{N}{mm^2}$$

Shear reinforcement should be provided!

**Required shear reinforcement:**

$$\tau_{Rd.sy} := \rho_w \cdot f_{ywd} \cdot cot(\phi) \geq \tau_{Ed}$$ ***[1992-1-1:2020 (8.27)]***

$$\rho_{w.V1} := \frac{\tau_{Ed.V1}}{f_{yd} \cdot cot(\phi)} = 0.00092$$

$$A_{sw.V1.perS} := \rho_{w.V1} \cdot b \cdot \frac{mm}{1 \cdot m} = 0.415\ \frac{mm^2}{m}$$

$$\rho_{w.V2} := \frac{\tau_{Ed.V2}}{f_{yd} \cdot cot(\phi)} = 0.00084$$

$$A_{sw.V2.perS} := \rho_{w.V2} \cdot b \cdot \frac{mm}{1 \cdot m} = 0.379\ \frac{mm^2}{m}$$

$$\rho_{w.H1} := \frac{\tau_{Ed.H1}}{f_{yd} \cdot cot(\phi)} = 0.00213$$

$$A_{sw.H1.perS} := \rho_{w.H1} \cdot b \cdot \frac{mm}{1 \cdot m} = 0.959\ \frac{mm^2}{m}$$

$$\rho_{w.H2} := \frac{\tau_{Ed.H2}}{f_{yd} \cdot cot(\phi)} = 0.00159$$

$$A_{sw.H2.perS} := \rho_{w.H2} \cdot b \cdot \frac{mm}{1 \cdot m} = 0.716\ \frac{mm^2}{m}$$

**Minimum shear reinforcement area:**

$$A_{s.min.perS} := 0.08 \cdot \frac{\sqrt{35} \cdot \frac{N}{mm^2}}{f_{yk}} \cdot b \cdot \sin(90) \cdot \frac{mm}{m} = 0.381\ \frac{mm^2}{m}$$

**Maximum spacing between shear reinforcement:**

$$S_{l.max} := 0.75 \cdot d = 183.75\ mm$$

**Provided shear reinforcement (stirrups)**

$$A_{sw.H1} := \frac{A_{sw}}{A_{sw.H1.perS} \cdot 10^3} = 104.782\ mm$$ **ø8 c.c. 100 mm**

$$A_{sw.H2} := \frac{A_{sw}}{A_{sw.H2.perS} \cdot 10^3} = 140.461\ mm$$ **ø8 c.c. 135 mm**

$$A_{sw.V1} := \frac{A_{sw}}{A_{sw.V1.perS} \cdot 10^3} = 242.279\ mm$$ **ø8 c.c. 180 mm**

$$A_{sw.V2} := \frac{A_{sw}}{A_{sw.V2.perS} \cdot 10^3} = 265.068\ mm$$ **ø8 c.c. 180 mm**

**Anchorage length beyond the theoretical left support**
**Value according to [table 11.1] $l_{bd}/ø = 38$**

$$l_{bd.min} := 10 \cdot ø_{main} = 140\ mm$$

$$\Delta F_{td} := 0.5 \cdot V_{Ed.V1} \cdot cot(ø) = 138.125\ kN$$

$$\sigma_{cd} := \frac{\Delta F_{td}}{A_{s.support.tension}} = 108.18\ \frac{N}{mm^2}$$

$$l_{bd} := \frac{\sigma_{cd}}{435\ \frac{N}{mm^2}} \cdot 38 \cdot ø_{main} = 132.302\ mm$$

$$l_{bd.V1} := l_{bd.min} = 140\ mm$$

**Anchorage length beyond the theoretical  left support**

$$\Delta F_{td} := 0.5 \cdot V_{Ed.H2} \cdot cot(ø) = 238.25\ kN$$

$$\sigma_{cd} := \frac{\Delta F_{td}}{A_{s.support.tension}} = 186.598\ \frac{N}{mm^2}$$

$$l_{bd.H2} := \frac{\sigma_{cd}}{435\ \frac{N}{mm^2}} \cdot 38 \cdot ø_{main} = 228.207\ mm$$

# Chapter 7

# Discussion

From a user's perspective, the design according to the applicable EC.2 is challenging, particularly for users with short experience with the standard. The main challenge origins from poor usability: clauses and section locations seem inefficient. Some definitions might be vague and require high expertise and further made assumptions, e.g. the definition of good bonding, which is undefined in EN 1992-1-1. However, the revision has achieved its objective of enhancing the ease of use throughout the standard. The revised version includes more additional clauses providing precise information and definitions. The sections and clauses were placed with consideration to the usual design order. Furthermore, a more detailed list of abbreviations, terms, and definitions is provided at the document's beginning. As a consequence of the development within usability, the concrete design should be less time-consuming as it is less effort and experience are needed to locate sections, and clauses are more transparent than in EN 1992-1-1.

Ongoing studies on concrete behaviours have reached a milestone since the last edition of EC.2 in 2004. Thus, an update of the EC.2 was required. The new factor $\eta_{cc}$ defined in the expression for the compressive design strength $f_{cd}$ in the revised version of EC.2 accounts for the difference between the undistributed cylinder compressive strength and the developed compressive strength in a structure. Thus the new factor will ensure a more precise and safe concrete design.
As seen in table 6.2 and 6.4, where $f_{ck} = 45$ and $f_{ck} = 35$ MPa are used in the design, the moment capacity $M_{Rd}$ has increased according to EN 1992-1-1:2021. The increased moment capacity is induced by the new expression for design compressive strength and the factor $\eta_{cc}$. The new expression gives higher values for $f_{ck} < 60$ MPa compared to EN 1992-1-1. Thus, the required longitudinal reinforcement has decreased for cases where the lever arm Z is less than $0.95 * d$ (the upper limit for Z). However, for $f_{ck} > 60$ MPa, the contrary will apply since the design compressive strength according to EN 1992-1-1:2021 is lower than in EN 1992-1-1 and will provide lower moment capacity as well. See figure 2.2.

The section regarding shear was significantly revised. From the calculations of slab nu.69 and beam nu.61, it is observed that shear resistance without shear reinforcement is less conservative for the given cases, and thus the utilization rate $\frac{\tau_{Ed}}{\tau_{Rdc}}$ is less than in EN 1992-1-1. The leading cause of these utilization rates is that expression in EN 1992-1-1:2021-1 for shear resistance without reinforcement follows the critical shear crack theory (CSCT). In contrast, the expression in EN 1992-1-1 is empirical and derived and calibrated according to experiments. The difference is that the effective depth directly influences the shear resistance according to

CSCT, while in EN 1992-1-1, the effective depth is included in the equations and increases the shear resistance proportionally, which is critical for small and large effective depths. See figure 3.7. Another cause is that EN 1992-1-1:2021-1 accounts for the size parameter $d_{dg}$ and provides expressions for it, which are a function of the characteristic compressive strength and the smallest aggregate size $D_{lower}$. Although, the largest aggregate size is also taken into account in EN 1992-1-1 by replacing the factor $k_2$ in $C_{Rdc}$, such that; $k_2 = 0.18$ for $D \geq 16$ mm and $k_2 = 0.15$ for other cases, the size factor is not described as precisely as in EN 1992-1-1:2021-1.
The minimum shear resistance expression is also less conservative in EN 1992-1-1:2021-1 as it gives higher minimum resistance than EN 1992-1-1. See figure 3.16. Nevertheless, the utilization's ratio will be larger when $f_{ck} > 60MPa$ as a result of the provided expression for $d_{dg}$ for $f_{ck} > 60$ MPa. See figure 3.16.

From the calculation of punching, it is observed that EN 1992-1-1:2021 provides lower punching shear resistance without reinforcement; thus, a higher utilization rate compared to EN 1992-1-1. The control perimeter length is determined at a distance of 0.5d in EN 1992-1-1:2021-1 and 2d in EN 1992-1-1. Thus the shear stress $v_{Ed}$ is higher in EN 1992-1-1:2021 as it is distributed into a smaller area, which is one of the reasons for the higher utilization ratio. The size parameter is represented in the expression for punching resistance and has the same previously mentioned effect on punching resistance.
In general, punching shear is improved as the size effect and enhancement coefficient $k_{pb}$ are included. Furthermore, the expression for punching shear resistance with shear reinforcement account for the utilization rate $\frac{\tau_{Ed}}{\tau_{Rd,c}}$ through the defined reduction factor $\eta_c$ which will ensure the precise required amount of reinforcement, providing a more safe design. For some cases where the utilization rate is a little higher than 1.0, the required amount of reinforcement will be less than in EN 1992-1-1, which has a fixed value of 75%.

The calculated deflection at the midspan of the slab is almost equivalent in both versions of EC.2. In the deflection calculations, the unit load method is used, and the creep coefficients at 7 and 9 days were determined according to Annex B. The revised version defines more parameters in the approach to determining creep coefficient compared to EN 1992-1-1. The new formula for determining creep coefficient consists of the sum of basic and drying creep coefficients. In the new expressions, many effects are considered, such as the effect of concrete strength on basic and drying creep coefficients, time development of basic and drying creep, and the effect of relative humidity and adjusted concrete age on drying creep. Each of these effects has its own empirically derived expression; thus, the calculations are more complex than EN 1992-1-1. Furthermore, the determined creep coefficients according to EN 1992-1-1:2021-1 were lower than in EN 1992-1-1. The calculated creep coefficients at 90 days according to annex B in EN 1992-1-1:2021-1 and EN 1992-1-1 were 0.406 and 0.924, respectively. There is a significant disparity between the two values. Creep calculations have been significantly revised based on the developments within material studies, providing more accurate values.

The calculated crack width in slab nu.69 has decreased by 13% according to EN 1992-1-1:2021-1. The new expression for calculated crack width take curvature into account through the factor $k_{\frac{1}{r}}$. The maximum allowed crack width according to EN 1992-1-1:2021-1 is more conservative than EN 1992-1-1. For the defined case in slab calculations, the maximum allowed crack width = 0.39 in EN 1992-1-1 and 0.3 in EN 1992-1-1:2021-1. The new calculation method of the crack width is more empirical and is based on observations and laboratory tests, hence more factors are introduced which complicates the calculation procedure. However, a study shows that accuracy improvement is not achieved compared to EN 1992-1-1.

The calculated anchorage length is almost identical in both versions for the given case. However, as mentioned previously, the method for determining anchorage length according to EN 1992-1-1:2021-1 is more straightforward and transparent. The sections regarding anchorage length are well structured such that it is easy to understand and follow when designing.

The substantial changes made in the revision of EC.2 will significantly influence the future of the concrete design. The development within materials and their behaviours are represented consistently in the expressions and factors in the standard. The new improved expressions and procedures will give societal advantages. These advantages include the safety of structures and environmental benefits. EN 1992-1-1:2021-1 provides precise calculations of the required amounts of reinforcement, which will lead to a safer design and, in some cases, limit more reinforcement amounts and thus the $CO_2$ emission compared to the design according to EN 1992-1-1. Moreover, EN 1992-1-1:2021-1 permits determining characteristic compressive strength for $t_{ref}$ between 28-91 days and for $t$ before or after $t_{ref}$, which will reduce the unnecessary use of building materials; thus, the carbon footprint as well. Although these changes might be minor concerning the current need to reduce buildings' carbon footprint, it is promising to see the developments goes in the right direction.

# Chapter 8

# Conclusion

The revision of EC.2 has enhanced the ease of use through considerable improvements within the content arrangement and the provided supplemental information/definitions within the clauses. Sections regarding anchorage length have been significantly improved as they are more straightforward and transparent than EN 1992-1-1; nonetheless, calculated results show almost identical values of the anchorage length determined according to the two versions of EC.2.

As a result of the increased moment capacity due to the new defined expression for design compressive strength, the performed calculations according to the revised EC.2 show a reduction 0.1% in the tensile reinforcement area for the slab nu.69.

Sections regarding shear reinforcement closely follow the CSCT, which improves the correspondence between calculated and tested values, providing a safer and more accurate design. The size factor describing the crack and the failure zone roughness $d_{dg}$ is represented in the expression for shear without shear reinforcement. As a result, shear resistance without shear reinforcement has increased by 11% and 12% for the calculated beam and slab, with effective depths of 245 mm and 252 mm, respectively. Hence, the required shear reinforcement in the beam nu.61 has decreased by approximately 9.5 % according to EN 1992-1-1:2021-1. These results comply with the theory that EN 1992-1-1 underestimates shear capacity for smaller effective depths.
Punching shear resistance without shear reinforcement has decreased according to EN 1992-1-1:2021-1 for the given cases. The new, more restricted definition for control perimeter length causes the reduction of the punching shear resistance, which is 0.5d from the edge of the support, compared to a distance of 2d according to EN 1992-1-1.

The calculation procedures of crack width and creep coefficients according to revised EC.2 are more empirical and are based on improved observations and laboratory experiments; hence, more additional empirical factors are defined, complicating the calculations. However, a study shows a lack of accuracy improvement in the new approach of calculating crack width in EN 1992-1-1:2021-1. The performed crack width calculations of slab nu.69 show a reduction of 13% according to EN 1992-1-1:2021-1 compared to EN 1992-1-1.

## Future work

The revised EC.2 defines the factor $\alpha_v$, which considers the slenderness of RC elements. It would be interesting to investigate the effect of slenderness on the shear capacity for future work. More precisely, comparing shear capacities of several beams with shorter shear spans than 4*d (effective depth) by testing in a laboratory and correspondingly estimating shear capacities according to the expressions provided in the revised version of EC.2.

The dissertation was limited to ordinary reinforced concrete. EN 1992-1-1:2021-1 have new clauses and expressions regarding prestressed concrete and a new annexe concerning fibre reinforced concrete. These topics could be explored in depth. Moreover, the sections regarding anchorage length could be explored in-depth by investigating other types of anchorages (e.g. shear links) and conducting a parametric comparison of the factors and expressions defined in the two versions of EC.2.

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# Appendix A

# Robot calculations

Robot calculations of beam 61, column 33 and slab 69 are attached below. Data used in the hand calculations are extracted from these files.

# 1 Level:

- Name :
- Reference level : 8,50 (m)
- Maximum cracking : 0,30 (mm)
- Exposure : XC2
- Concrete creep coefficient : $\varphi_{\pi} = 2,11$
- Cement class : N
- Concrete age (loading moment) : 28 (days)
- Concrete age : 100 (years)
- Concrete age after erecting a structure : 365 (years)
- Durability class: : M60
- Fire resistance class : no requirements
- FFB Recommendations 7.4.3(7) : 0,00

# 2 Beam: Beam61 Number of identical elements: 1

## 2.1 Material properties:

- Concrete : B35 $f_{ck}$ = 35,00 (MPa)
  Rectangular stress distribution [3.1.7(3)]
  Density : 2501,36 (kG/m3)
  Aggregate size : 20,0 (mm)
- Longitudinal reinforcement: : B500C $f_{yk}$ = 500,00 (MPa)
  Horizontal branch of the stress-strain diagram
  Ductility class : C
- Transversal reinforcement: : B500C $f_{yk}$ = 500,00 (MPa)
  Horizontal branch of the stress-strain diagram
  Ductility class : C
- Additional reinforcement: : B500C $f_{yk}$ = 500,00 (MPa)
  Horizontal branch of the stress-strain diagram

## 2.2 Geometry:

### 2.2.1

| Span | Position | L.supp. (m) | L (m) | R.supp. (m) |
|---|---|---|---|---|
| **P1** | **Span** | **0,30** | **7,18** | **0,35** |

Span length: $L_o$ = 7,50 (m)
Section from 0,00 to 7,18 (m)
300 x 450 (mm)
without left slab
without right slab

### 2.2.2

| Span | Position | L.supp. (m) | L (m) | R.supp. (m) |
|---|---|---|---|---|
| **P2** | **Span** | **0,35** | **7,18** | **0,30** |

Span length: $L_o$ = 7,50 (m)
Section from 0,00 to 7,18 (m)
300 x 450 (mm)
without left slab
without right slab

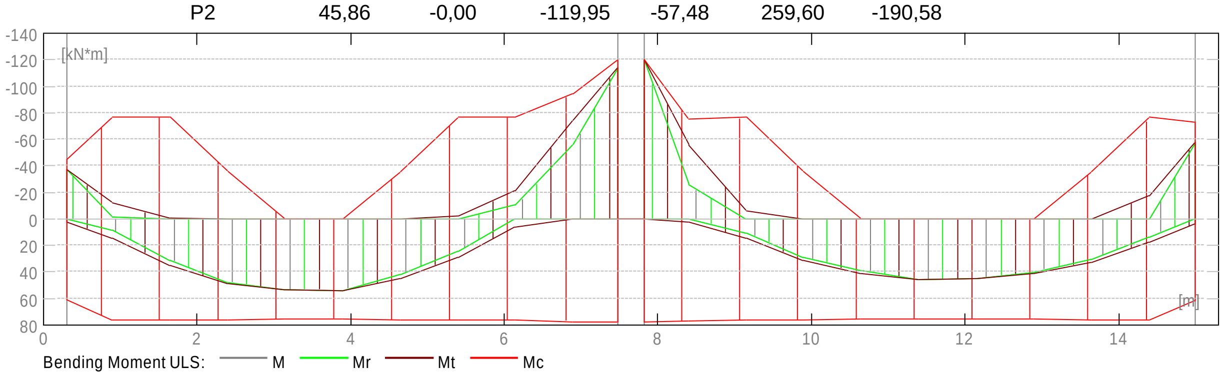
## 2.3 Calculation options:

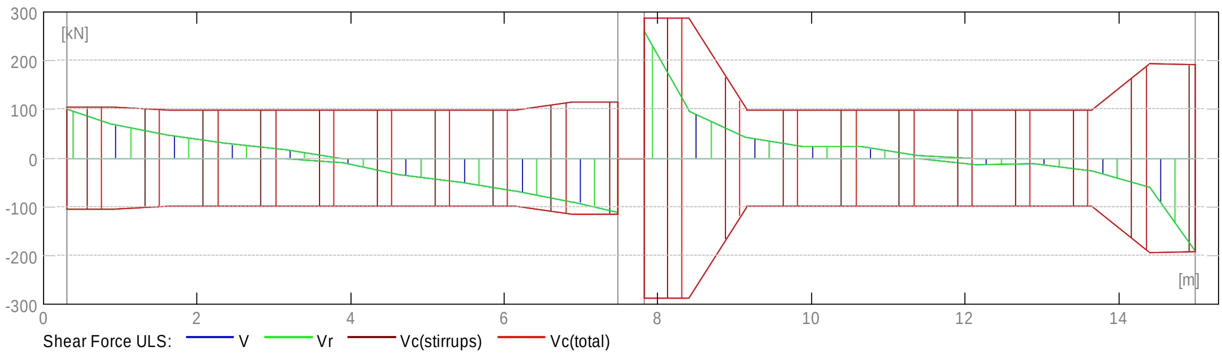
- Regulation of combinations : NS-EN 1990:2002/NA:2016
- Calculations according to : NS-EN 1992-1-1:2004/A1:2014/NA:2018
- Seismic dispositions : No requirements
- Precast beam : no
- Cover : bottom c = 40 (mm)
  : side c1= 40 (mm)
  : top c2= 40 (mm)
- Cover deviations : Cdev = 10(mm), Cdur = 0(mm)
- Coefficient $\beta_2$ =0.50 : long-term or cyclic load
- Method of shear calculations : strut inclination

## 2.4 Calculation results:

### 2.4.1 Internal forces in ULS

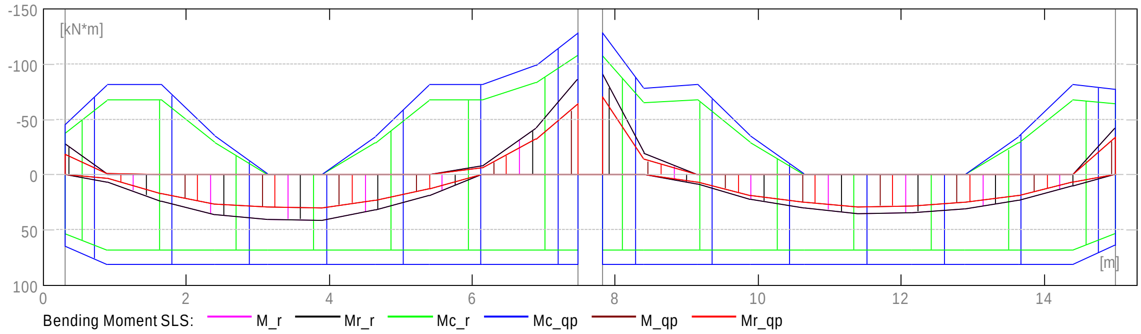
| Span | Mt max. (kN*m) | Mt min. (kN*m) | Ml (kN*m) | Mr (kN*m) | Ql (kN) | Qr (kN) |
|---|---|---|---|---|---|---|
| P1 | 53,86 | -2,50 | -37,03 | -113,96 | 101,54 | -110,94 |
| P2 | 45,86 | -0,00 | -119,95 | -57,48 | 259,60 | -190,58 |

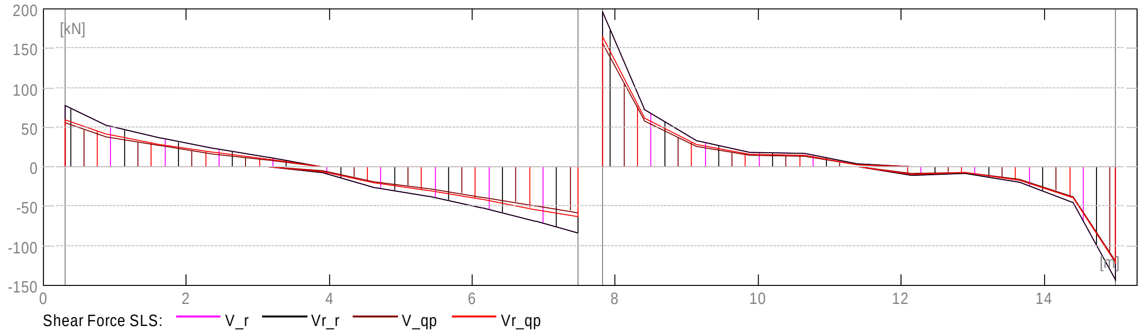


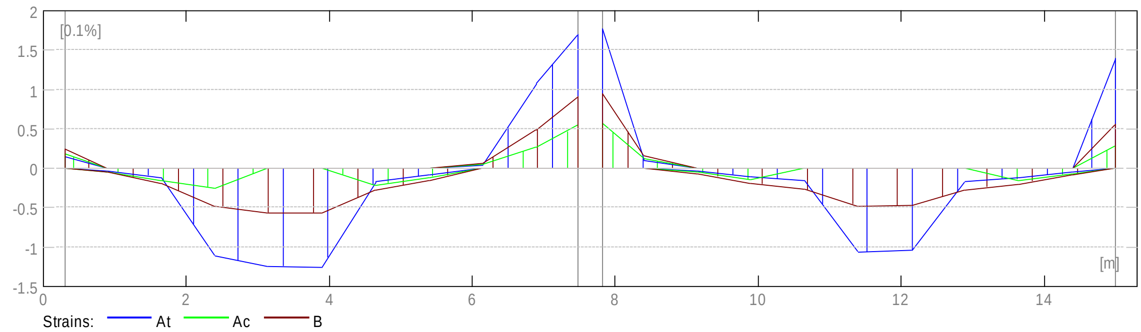



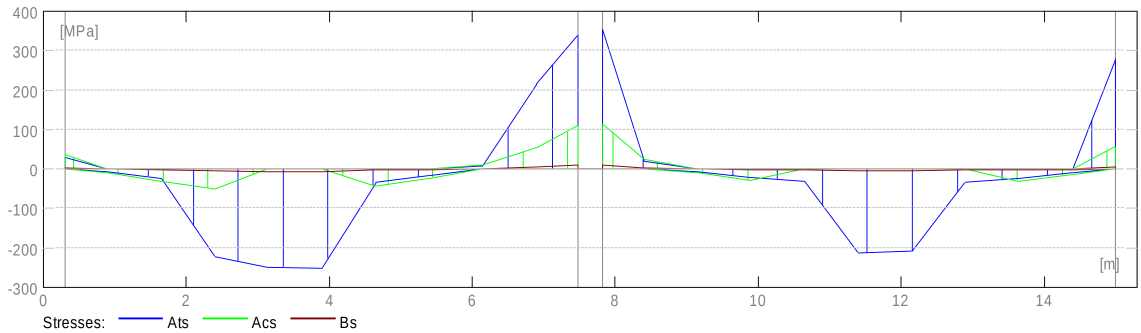

### 2.4.2 Internal forces in SLS

| Span | Mt max. (kN*m) | Mt min. (kN*m) | Ml (kN*m) | Mr (kN*m) | Ql (kN) | Qr (kN) |
|---|---|---|---|---|---|---|
| P1 | 40,88 | 0,00 | -27,81 | -86,84 | 77,23 | -83,44 |
| P2 | 34,69 | 0,00 | -90,78 | -43,04 | 196,22 | -143,71 |



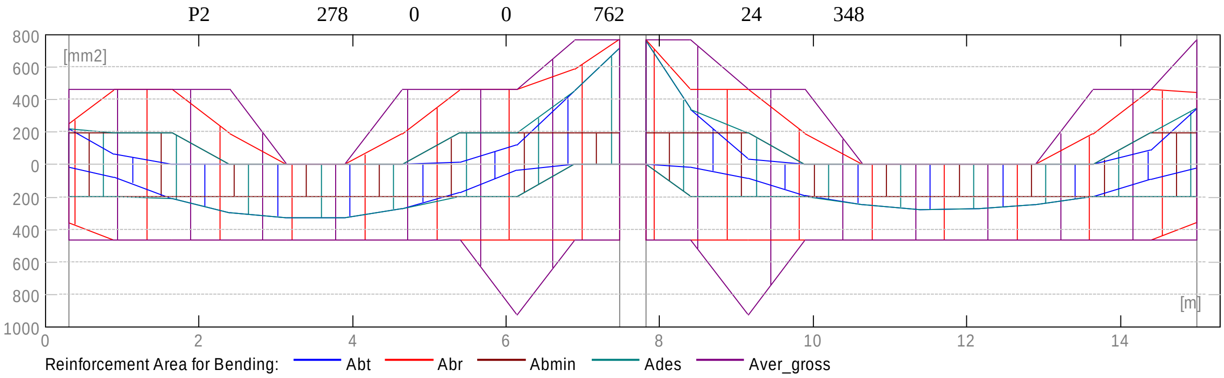



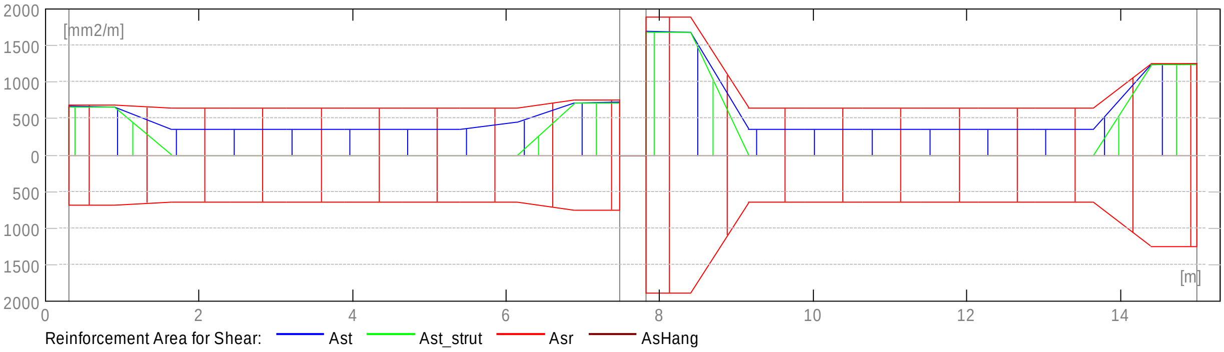




### 2.4.3 Required reinforcement area

| Span | Span (mm2) | | Left support (mm2) | | Right support (mm2) | |
|---|---|---|---|---|---|---|
| | bottom | top | bottom | top | bottom | top |
| P1 | 328 | 0 | 16 | 221 | 0 | 721 |




### 2.4.4 Deflection and cracking

wt(QP) Total due to quasi-permanent combination
wt(QP)dop Allowable due to quasi-permanent combination
Dwt(QP) Deflection increment from the quasi-permanent load combination after erecting a structure.
Dwt(QP)dop Admissible deflection increment from the quasi-permanent load combination after erecting a structure.

wk - width of perpendicular cracks

| Span | wt(QP) (mm) | wt(QP)dop (mm) | Dwt(QP) (mm) | Dwt(QP)dop (mm) | wk (mm) |
|---|---|---|---|---|---|
| P1 | 4 | 30 | 1 | 15 | 0,2 |
| P2 | 5 | 30 | 1 | 15 | 0,0 |

## 2.5 Theoretical results - detailed results:

### 2.5.1 P1 : Span from 0,30 to 7,48 (m)

| | ULS | | SLS | | | |
|---|---|---|---|---|---|---|
| Abscissa (m) | M max. (kN*m) | M min. (kN*m) | M max. (kN*m) | M min. (kN*m) | A bottom (mm2) | A top (mm2) |
| 0,30 | 2,65 | -37,03 | 0,00 | -27,81 | 16 | 221 |
| 0,90 | 14,20 | -12,38 | 6,35 | -0,21 | 76 | 66 |
| 1,65 | 35,23 | -0,39 | 23,70 | 0,00 | 212 | 2 |
| 2,40 | 49,03 | -0,00 | 36,25 | 0,00 | 297 | 0 |
| 3,15 | 53,42 | -0,00 | 40,47 | 0,00 | 325 | 0 |
| 3,90 | 53,86 | -0,00 | 40,88 | 0,00 | 328 | 0 |
| 4,65 | 44,86 | -0,00 | 31,77 | 0,00 | 271 | 0 |
| 5,40 | 28,91 | -2,50 | 18,52 | 0,00 | 172 | 15 |
| 6,15 | 6,01 | -21,39 | 0,00 | -7,86 | 34 | 123 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 6,90 | 0,00 | -74,36 | 0,00 | -42,69 | 0 | 458 |
| 7,48 | 0,00 | -113,96 | 0,00 | -86,84 | 0 | 721 |

| | ULS | SLS | |
|---|---|---|---|
| Abscissa | V max. | V max. | afp |
| (m) | (kN) | (kN) | (mm) |
| 0,30 | 101,54 | 77,23 | 0,0 |
| 0,90 | 68,63 | 52,23 | 0,0 |
| 1,65 | 47,61 | 36,21 | 0,0 |
| 2,40 | 30,02 | 22,78 | 0,0 |
| 3,15 | 16,84 | 12,73 | 0,0 |
| 3,90 | -9,68 | -7,32 | 0,0 |
| 4,65 | -34,53 | -26,26 | 0,0 |
| 5,40 | -48,97 | -37,24 | 0,0 |
| 6,15 | -68,25 | -51,92 | 0,0 |
| 6,90 | -90,08 | -68,35 | 0,0 |
| 7,48 | -110,94 | -83,44 | 0,2 |

### 2.5.2 P2 : Span from 7,83 to 15,00 (m)

| | ULS | | SLS | | | |
|---|---|---|---|---|---|---|
| Abscissa | M max. | M min. | M max. | M min. | A bottom | A top |
| (m) | (kN*m) | (kN*m) | (kN*m) | (kN*m) | (mm2) | (mm2) |
| 7,83 | 0,00 | -119,95 | 0,00 | -90,78 | 0 | 762 |
| 8,40 | 2,53 | -55,42 | 0,00 | -19,36 | 15 | 336 |
| 9,15 | 14,90 | -6,19 | 7,99 | 0,00 | 84 | 35 |
| 9,90 | 31,35 | -0,00 | 21,74 | 0,00 | 188 | 0 |
| 10,65 | 40,82 | -0,00 | 29,63 | 0,00 | 246 | 0 |
| 11,40 | 45,86 | -0,00 | 34,69 | 0,00 | 278 | 0 |
| 12,15 | 45,18 | -0,00 | 34,02 | 0,00 | 273 | 0 |
| 12,90 | 41,30 | -0,00 | 30,37 | 0,00 | 249 | 0 |
| 13,65 | 32,62 | -0,00 | 22,84 | 0,00 | 196 | 0 |
| 14,40 | 17,48 | -17,46 | 10,13 | 0,00 | 92 | 92 |
| 15,00 | 4,07 | -57,48 | 0,00 | -43,04 | 24 | 348 |

| | ULS | SLS | |
|---|---|---|---|
| Abscissa | V max. | V max. | afp |
| (m) | (kN) | (kN) | (mm) |
| 7,83 | 259,60 | 196,22 | 0,3 |
| 8,40 | 96,73 | 73,26 | 0,0 |
| 9,15 | 42,90 | 32,53 | 0,0 |
| 9,90 | 24,09 | 18,28 | 0,0 |
| 10,65 | 22,97 | 17,43 | 0,0 |
| 11,40 | 4,86 | 3,69 | 0,0 |
| 12,15 | -13,70 | -10,37 | 0,0 |
| 12,90 | -11,56 | -8,73 | 0,0 |
| 13,65 | -25,00 | -18,90 | 0,0 |
| 14,40 | -59,62 | -45,00 | 0,0 |
| 15,00 | -190,58 | -143,71 | 0,2 |

## 2.6 Reinforcement:

### 2.6.1 P1 : Span from 0,30 to 7,48 (m)

**Longitudinal reinforcement:**

- bottom (B500C)

| | | | | | |
|---|---|---|---|---|---|
| 3 | ϕ14 | l = 6,23 | from 0,04 | to | 6,27 |
| 3 | ϕ14 | l = 3,68 | from 5,81 | to | 9,49 |

- assembling (top) (B500C)

| | | | | | |
|---|---|---|---|---|---|
| 3 | ϕ14 | l = 3,57 | from 1,74 | to | 5,31 |

- support (B500C)

| | | | | | |
|---|---|---|---|---|---|
| 3 | ϕ14 | l = 2,56 | from 0,04 | to | 2,60 |
| 3 | ϕ14 | l = 5,65 | from 4,45 | to | 10,10 |
| 2 | ϕ14 | l = 1,78 | from 6,70 | to | 8,48 |

**Transversal reinforcement:**

- main (B500C)
  stirrups 31 $\phi$8 l = 1,33
  e = 1*0,18 + 3*0,22 + 24*0,23 + 3*0,20 (m)

  pins 31 $\phi$8 l = 0,58
  e = 1*0,18 + 3*0,22 + 24*0,23 + 3*0,20 (m)

### 2.6.2 P2 : Span from 7,83 to 15,00 (m)

**Longitudinal reinforcement:**

- bottom (B500C)
  3 $\phi$14 l = 6,23 from 9,03 to 15,26
- assembling (top) (B500C)
  3 $\phi$14 l = 5,07 from 9,24 to 14,31
- support (B500C)
  3 $\phi$14 l = 1,81 from 13,45 to 15,26
  1 $\phi$14 l = 1,51 from 15,25 to 15,25

**Transversal reinforcement:**

- main (B500C)
  stirrups 40 $\phi$8 l = 1,33
  e = 1*0,08 + 8*0,08 + 24*0,23 + 7*0,12 (m)

  pins 40 $\phi$8 l = 0,58
  e = 1*0,08 + 8*0,08 + 24*0,23 + 7*0,12 (m)

# 3 Material survey:

- Concrete volume = 2,07 (m3)
- Formwork = 18,35 (m2)

- Steel B500C
  - Total weight = 185,82 (kG)
  - Density = 89,96 (kG/m3)
  - Average diameter = 10,7 (mm)
  - Survey according to diameters:

| Diameter (mm) | Length (m) | Weight (kG) | Number (No.) | Total weight (kG) |
|---|---|---|---|---|
| 8 | 0,58 | 0,23 | 71 | 16,31 |
| 8 | 1,33 | 0,52 | 71 | 37,22 |
| 14 | 1,51 | 1,83 | 1 | 1,83 |
| 14 | 1,78 | 2,15 | 2 | 4,30 |
| 14 | 1,81 | 2,19 | 3 | 6,56 |
| 14 | 2,56 | 3,09 | 3 | 9,28 |
| 14 | 3,57 | 4,31 | 3 | 12,94 |
| 14 | 3,68 | 4,45 | 3 | 13,35 |
| 14 | 5,07 | 6,13 | 3 | 18,38 |
| 14 | 5,65 | 6,82 | 3 | 20,47 |
| 14 | 6,23 | 7,53 | 6 | 45,19 |

# 1 Level:

- Name :
- Reference level : 1,50 (m)
- Concrete creep coefficient : $\varphi_p$ = 2,13
- Cement class : N
- Environment class : XC2
- Durability class: : M60

# 2 Column: Column33

Number of identical elements: 1

## 2.1 Material properties:

- Concrete : B35 $f_{ck}$ = 35,00 (MPa)
  Unit weight : 2501,36 (kG/m3)
  Aggregate size : 20,0 (mm)
- Longitudinal reinforcement: : B500C $f_{yk}$ = 500,00 (MPa)
  Ductility class : C
- Transversal reinforcement: : B500C $f_{yk}$ = 500,00 (MPa)

## 2.2 Geometry:

2.2.1 Rectangular 350 x 350 (mm)
2.2.2 Height: L = 3,50 (m)
2.2.3 Slab thickness = 0,30 (m)
2.2.4 Beam height = 0,45 (m)
2.2.5 Cover = 40 (mm)

## 2.3 Calculation options:

- Calculations according to : NS-EN 1992-1-1:2004/A1:2014/NA:2018
- Seismic dispositions : No requirements
- Precast column : no
- Pre-design : no
- Slenderness taken into account : yes
- Compression : with bending
- Ties : to slab
- Fire resistance class : No requirements

## 2.4 Loads:

| Case | Nature | Group | $\gamma$ | N (kN) | MyA (kN*m) | MyB (kN*m) | MyC (kN*m) | MzA (kN*m) | MzB (kN*m) | MzC (kN*m) |
|---|---|---|---|---|---|---|---|---|---|---|
| DL1 | dead load(Structural) | 33 | 1,35 | 1034,76 | 1,38 | -1,57 | -0,63 | -0,31 | 0,44 | 0,18 |
| DL2 | live load(Category A) | 33 | 1,50 | -3,13 | 0,12 | -0,56 | -0,29 | -0,06 | -0,05 | -0,06 |
| DL21 | live load(Category A) | 33 | 1,50 | 184,75 | 0,71 | -0,20 | 0,34 | 0,05 | 0,10 | 0,08 |
| DL21112 | live load(Category A) | 33 | 1,50 | 186,13 | -0,15 | 0,13 | -0,06 | 0,05 | -0,03 | 0,02 |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| DL2111 | snow | 33 | 1,50 | 173,72 | -0,14 | 0,13 | -0,06 | 0,05 | -0,03 | 0,02 |
| SN2 | snow | 33 | 1,50 | 1,68 | -0,06 | 0,01 | -0,03 | -0,01 | 0,02 | 0,01 |
| WIND1 | wind | 33 | 1,50 | -4,25 | -13,90 | 13,86 | -5,56 | -2,26 | 2,26 | 0,91 |
| WIND2 | wind | 33 | 1,50 | -20,05 | -11,52 | 11,49 | -4,61 | -16,18 | 16,11 | -6,47 |
| WIND3 | wind | 33 | 1,50 | -14,84 | -4,21 | 4,13 | -1,68 | -18,19 | 18,07 | -7,27 |
| WIND4 | wind | 33 | 1,50 | -1,75 | 13,24 | -13,10 | 5,30 | 1,67 | -1,65 | 0,67 |
| WIND5 | wind | 33 | 1,50 | -17,13 | -3,12 | 3,08 | -1,25 | 22,41 | -22,29 | 8,96 |

$\gamma$- load factor

## 2.5 Calculation results:

Safety factors Rd/Ed = 1,33 > 1.0

### 2.5.1 ULS/ALS Analysis

Design combination: 1.20DL1+1.50DL21+1.50DL21112+1.05DL2111 (A)
Combination type: ULS
Internal forces:
Nsd = 1982,00 (kN) Msdy = 2,33 (kN*m) Msdz = -0,16 (kN*m)
Design forces:
Upper node
N = 1982,00 (kN) N*etotz = 39,64 (kN*m) N*etoty= -39,64 (kN*m)

| Eccentricity: | | ez (My/N) (mm) | ey (Mz/N) (mm) |
|---|---|---|---|
| Initial | e0: | 1 | -0 |
| Imperfection | ei: | 9 | 9 |
| I order (e0 + ei) | e0Ed: | 10 | 9 |
| Minimal | eEdmin: | 20 | 20 |
| Total | eEd: | 20 | -20 |

#### 2.5.1.1. Detailed analysis-Direction Y:

##### 2.5.1.1.1 Slenderness analysis

Non-sway structure

| L (m) | Lo (m) | $\lambda$ | $\lambda$N | $\lambda$Nlim | |
|---|---|---|---|---|---|
| 3,50 | 3,50 | 34,64 | 23,98 | 11,39 | Slender column |

##### 2.5.1.1.2 Buckling analysis

MA = 2,33 (kN*m) MB = -1,85 (kN*m)
Case: Cross-section at the column end (Upper node), Slenderness not taken into account
M0 = 2,33 (kN*m)
ei = $\theta_1$*lo/2 = 9 (mm)
$\theta_1 = \theta_o * \alpha_\eta * \alpha_m = 0,01$
$\theta_o = 0,01$
$\alpha_h = 1,00$
$\alpha_m = (0,5(1+1/m))$^0.5 = 1,00
m = 1,00
Ma = N*ei = 17,34 (kN*m)
MEdmin = 39,64 (kN*m)
M0Ed = max(MEdmin,M0 + Ma) = 39,64 (kN*m)

### 2.5.1.2. Detailed analysis-Direction Z:

MA = -0,16 (kN*m)  MB = 0,61 (kN*m)
Case: Cross-section at the column end (Upper node), Slenderness not taken into account
M0 = -0,16 (kN*m)
ei = $\theta 1$* lo/2 = 9 (mm)
$\theta 1 = \theta o * \alpha h * \alpha m$ = 0,01
$\theta o$ = 0,01
$\alpha h$ = 1,00
$\alpha m$ = (0,5(1+1/m))^0.5 = 1,00
m = 1,00
Ma = N*ei = 17,34 (kN*m)
MEdmin = 39,64 (kN*m)
M0Ed = max(MEdmin,M0 + Ma) = -39,64 (kN*m)

### 2.5.2 Reinforcement:

Real (provided) area  Asr = 1963 (mm2)
Ratio:  $\rho$ = 1,60 %

## 2.6 Reinforcement:

**Main bars (B500C):**

- 4 $\phi$25  l = 3,46 (m)

**Transversal reinforcement: (B500C):**

stirrups:  12 $\phi$8  l = 1,23 (m)

# 3 Material survey:

- Concrete volume  = 0,37 (m3)
- Formwork  = 4,27 (m2)

- Steel B500C
  - Total weight  = 59,18 (kG)
  - Density  = 158,40 (kG/m3)
  - Average diameter = 16,2 (mm)
  - Reinforcement survey:

| Diameter | Length (m) | Weight (kG) | Number (No.) | Total weight (kG) |
|---|---|---|---|---|
| 8 | 1,23 | 0,49 | 12 | 5,83 |
| 25 | 3,46 | 13,34 | 4 | 53,35 |

# 5. Slab: Slab65...69 - Panel no. 69

## 5.1. Reinforcement:

| | |
|---|---|
| • Type | : RC floor |
| • Main reinforcement direction | : 0° |
| Main reinforcement grade | : B500C; Characteristic strength = 500,00 MPa<br>Horizontal branch of the stress-strain diagram |
| • Ductility class | : C |
| • Bar diameters | bottom d1 = 12 (mm) d2 = 12 (mm)<br>top d1 = 12 (mm) d2 = 12 (mm) |
| • Cover | bottom c1 = 30 (mm)<br>top c2 = 30 (mm) |
| • Cover deviations | Cdev = 10(mm), Cdur = 0(mm) |

## 5.2. Concrete

| | |
|---|---|
| Class | : B45; Characteristic strength = 45,00 MPa<br>Rectangular stress distribution [3.1.7(3)] |
| • Density | : 2501,36 (kG/m3) |
| • Concrete creep coefficient | : 1,02 |
| • Cement class | : N |

## 5.3. Hypothesis

| | |
|---|---|
| • Calculations according to 1992-1-1:2004/A1:2014/NA:2018 | : NS-EN |
| • Method of reinforcement area calculations | : analytical |
| • Allowable cracking width | |
| - upper layer | : 0,40 (mm) |
| - lower layer | : 0,40 (mm) |
| • Allowable deflection | : 30 (mm) |
| • Verification of punching | : yes |
| • Exposure | |
| - upper layer | : X0 |
| - lower layer | : X0 |
| • Calculation type | : simple bending |
| • Durability class: | : M90 |

## 5.4. Slab geometry

Thickness 0,30 (m)

**Contour:**

| **edge** | **beginning** | | **end** | | **length** |
|---|---|---|---|---|---|
| | **x1** | **y1** | **x2** | **y2** | **(m)** |
| 1 | 14,00 | -7,50 | 21,00 | -7,50 | 7,00 |
| 2 | 21,00 | -7,50 | 21,00 | -15,00 | 7,50 |
| 3 | 21,00 | -15,00 | 14,00 | -15,00 | 7,00 |
| 4 | 14,00 | -15,00 | 14,00 | -7,50 | 7,50 |

**Support:**

| **n°** | **Name** | **dimensions** | **coordinates** | **edge** |
|---|---|---|---|---|

| | | (m) | x | y | |
|---|---|---|---|---|---|
| 42 | linear | 0,25 / 21,00 | 10,50 | -7,50 | — |
| 39 | linear | 15,00 / 0,30 | 14,00 | -7,50 | — |
| 43 | point | 0,35 / 0,35 | 14,00 | -7,50 | — |
| 37 | point | 0,30 / 0,30 | 14,00 | -15,00 | — |
| 37 | linear | 0,25 / 14,00 | 14,00 | -15,00 | — |
| 34 | linear | 15,00 / 0,30 | 21,00 | -7,50 | — |
| 35 | point | 0,30 / 0,30 | 21,00 | -7,50 | — |
| 36 | point | 0,30 / 0,30 | 21,00 | -15,00 | — |

* - head present

## 5.5. Calculation results:

### 5.5.1. Maximum moments + reinforcement for bending

| | Ax(+) | Ax(-) | Ay(+) | Ay(-) |
|---|---|---|---|---|
| Provided reinforcement (mm2/m): | 2234 | 654 | 2513 | 982 |
| Modified required reinforcement (mm2/m): | 1984 | 618 | 2079 | 701 |
| Original required reinforcement (mm2/m): | 1984 | 618 | 2079 | 701 |
| Coordinates (m): | 14,00;-7,50<br>21,00;-11,41 | 18,13;-15,00 | 14,00;-7,50 | |

### 5.5.2. Maximum moments + reinforcement for bending

| | Ax(+) | Ax(-) | Ay(+) | Ay(-) |
|---|---|---|---|---|
| Symbol: required area/provided area | | | | |
| Ax(+) (mm2/m) | **1984/2234** | 0/559 | 1984/2234 | 0/0 |
| Ax(-) (mm2/m) | 0/0 | **618/654** | 0/0 | 502/654 |
| Ay(+) (mm2/m) | 2079/2513 | 7/503 | **2079/2513** | 0/0 |
| Ay(-) (mm2/m) | 0/0 | 478/491 | 0/0 | **701/982** |
| | **SLS** | | | |
| Mxx (kN*m/m) | 149,37 | -48,64 | 149,37 | -7,39 |
| Myy (kN*m/m) | 151,99 | -5,46 | 151,99 | -54,78 |
| Mxy (kN*m/m) | -1,41 | 7,66 | -1,41 | -1,34 |
| Nxx (kN/m) | -5,09 | -2,94 | -5,09 | -0,05 |
| Nyy (kN/m) | -4,51 | 0,11 | -4,51 | -3,97 |
| Nxy (kN/m) | 0,01 | 0,44 | 0,01 | 0,41 |
| | **ULS** | | | |
| Mxx (kN*m/m) | 194,95 | -63,42 | 194,95 | -9,64 |
| Myy (kN*m/m) | 198,37 | -7,12 | 198,37 | -71,42 |
| Mxy (kN*m/m) | -1,86 | 10,00 | -1,86 | -1,75 |
| Nxx (kN/m) | -6,55 | -3,80 | -6,55 | -0,06 |
| Nyy (kN/m) | -5,81 | 0,15 | -5,81 | -5,14 |
| Nxy (kN/m) | 0,02 | 0,56 | 0,02 | 0,52 |
| Coordinates (m) | 14,00;-7,50<br>21,00;-11,41 | 18,13;-15,00 | 14,00;-7,50 | |
| Coordinates* (m) | 14,00;7,50;8,50<br>21,00;3,59;8,50 | 18,13;0,00;8,50 | 14,00;7,50;8,50 | |

* - Coordinates in the structure global coordinate system

### 5.5.4. Deflection

|f(+)| = 0 (mm) <= $f_{dop}$(+) = 30 (mm)
|f(-)| = 14 (mm) <= $f_{dop}$(-) = 30 (mm)

### 5.5.5. Cracking

upper layer
ax = 0,21 (mm) <= $a_{dop}$ = 0,40 (mm)
ay = 0,24 (mm) <= $a_{dop}$ = 0,40 (mm)
lower layer
ax = 0,00 (mm) <= $a_{dop}$ = 0,40 (mm)
ay = 0,00 (mm) <= $a_{dop}$ = 0,40 (mm)

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