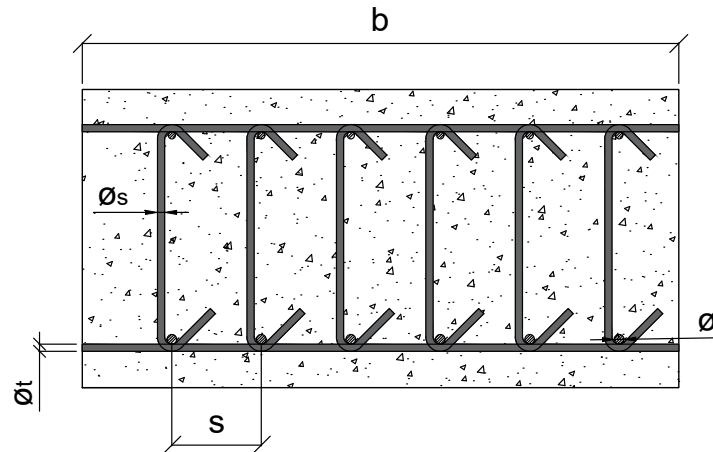




The Impact of the New Eurocode Implementation for Concrete Structures

Master's thesis in the Master's Programme Structural Engineering and Building Technology

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ABSTRACT

Eurocodes represent an important tool for structural designers in the European Union since they establish a common framework for the design of structures while simultaneously fulfilling safety and serviceability requirements. However, the current codes are based on methods, theories, and data that need to be updated to the current state-of-the-art, a task for EU member states. SS-EN 1992-1-1:2023 Eurocode 2 for concrete structures is part of a second generation of standards that will be used in Europe. Production of the new Eurocode is still in progress, the new Eurocode will become effective by July 2027, and the current version will have been replaced completely by March 30th, 2028.

The thesis aims to describe the differences between the new SS-EN 1992-1-1:2023 Eurocode 2 and the current version (SS-EN 1992-1-1:2005 + SS-EN 1992-2:2005), and explain how these differences will impact the design and implementation of concrete structures. A literature study was made, where the two Eurocodes were compared, and potential reasons for the changes were discussed. This was further investigated in a parametric study, in which the structural capacity of a simplified concrete slab was investigated using both versions of the Eurocode, and differences were identified. In addition, a case study was performed for an existing concrete bridge where sectional capacities were re-evaluated using the new standard.

It was found that several important design criteria had been subjected to major revisions in the new Eurocode. These changes were found to result in a higher shear capacity in many design situations, especially for simply supported edges. An increase in bending moment and shear capacity was also observed for sections with a depth of more than 200 mm. The capacity in Serviceability Limit States due to crack control was found to be higher for bars placed at the bottom of a slab. However, bars in the top face were found to have a similar capacity to the current Eurocode. Furthermore, a significant increase in anchorage length was observed for bars with a diameter larger than 16 mm.

Keywords: Eurocode, Generation 2, SS-EN 1992-1-1:2023, Standard, Concrete, Construction, Design

Påverkan av implementeringen av den nya Eurokoden för betongkonstruktioner
Examensarbete inom masterprogrammet Konstruktionsteknik och Byggnadsteknologi

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SAMMANFATTNING

Eurokoden är ett viktigt verktyg för ingenjörer inom EU. Den fungerar som ett gemensamt ramverk för dimensionering av byggnadsverk, vilket säkerställer att krav på funktion och säkerhet uppnås. De nuvarande regelverken är baserade på metoder, teorier och empirisk data som behöver uppdateras för att vara relevant med det nuvarande forskningsläget; en uppgift för medlemsländerna i EU. SS-EN 1992-1-1:2023 Eurocode 2 för betongbyggnad är en del av nästa generations standarder som kommer att användas i Europa. Sammanställningen av den nya Eurocoden har redan påbörjats, vilken kommer att införas juli 2027, och kommer att helt ersätta den nuvarande versionen senast 30e mars, 2028.

Syftet med tesen är att identifiera skillnaderna mellan den nya Eurocoden och den nuvarande: (SS-EN 1992-1-1:2005 + SS-EN 1992-2:2005), samt att förklara hur dessa ändringar kan komma att påverka dimensioneringen av betongbyggnader. En litteraturstudie gjordes, där de två Eurokoderna jämfördes. Därefter undersöktes de tydligaste skillnaderna ytterligare i en parametrisk studie, där den strukturella kapaciteten av en förenklad betongplatta undersöktes med båda versionerna av Eurocoden och jämfördes. Utöver detta gjordes en fallstudie på en rambro från 2013, där bärlastkapaciteten i flera kritiska snitt beräknades på nytt med den nya standarden.

Resultaten i tesen visar att flera viktiga dimensioneringskriterier har ustötts för betydande revideringar. En konsekvens av dessa förändringar var en ökad tvärkraftskapacitet för många tvärsnitt, speciellt vid fritt upplagda stöd. Dessutom observerades en ökad momentkapacitet för sektioner med en höjd på över 200 mm. Kapaciteten i bruksgränstillstånd med avseende på sprickbildning visades öka för sektioner med dragarmering i underkant. För dragarmering i överkant var kapaciteten samma som i den nuvarande Eurokod. Utöver detta observerades en tydlig ökning i förankringslängd för armering med en diameter större än 16 mm.

Nyckelord: Eurokod, SS-EN1992-1-1:2023, Standard, Betong, Konstruktion, Dimensionering

PREFACE

This thesis investigates the new generation of Eurocode with regard to concrete structures (SS-EN 1992:1-1:2023) in relation to the current generation (SS-EN 1992:1-1:2005 and SS-EN 1992-2:2005) at Chalmers University of Technology. The work was conducted in collaboration between Norconsult and Chalmers during the period January to June 2026.

We would like to thank our supervisor, Axel Kristensson at Norconsult, for his reliability, guidance, and thorough support, as well as Morgan Johansson and Emanuel Trolin for their valuable input on the thesis. We are also grateful to our examiner, Ignasi Fernandez, at the Division of Structural Engineering for his broad knowledge and expertise in concrete structures.

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Matilda Andersson and Lukas Olsson, Gothenburg, June 2026

ACRONYMS

Below is the list of acronyms that have been used throughout this thesis, listed in alphabetical order:

Ann.	Annex
AI	Artificial Intelligence
CEN	European Committee for Standardization
EC	Eurocode
EU	European Union
Gen 1	SS-EN 1992-1-1:2005 + SS-EN 1992-2:2005
Gen 2	SS-EN 1992-1-1:2023
NA	National Annex
NPD	Nationally Determined Parameter
SCM	Supplementary Cementitious Material
SLS	Servicability Limit States
TSFS	Swedish Transport Agency's Code of Statutes (Transportstyrelsens föfattningssamling)
ULS	Ultimate Limit States

NOTATIONS

Below is the nomenclature of indices, sets, parameters, and variables that have been used throughout this thesis.

Latin upper case letters

A	Cross sectional area
A_c	Cross-sectional area of concrete
$A_{c,conf}$	Cross-sectional area of confined concrete
$A_{c,eff}$	Effective concrete area
A_{cc}	Area under compression
A_{ct}	Area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack
A_{deck}	Area of bridge deck, case study
A_i	Area of a joint
A'_p	Area of pre or post-tensioned tendons within $A_{c,eff}$
A_s	Cross-sectional area of ordinary reinforcement
$A_{s,conf}$	Cross-sectional area of one leg of confinement reinforcement
$A_{s,min}$	Minimum cross-sectional area of reinforcement
$A_{s,min,w1},$ $A_{s,min,w2}$	Limits for minimum reinforcement area at most/least tensioned face to control cracking
A_{s,min,σ_s}	Minimum amount of reinforcement required for crack control
$A_{s,web}$	Additional longitudinal reinforcement for for deep beams to prevent cracking at the sides
A_{si}	Cross-sectional area of one reinforcement bar
A_{sl}	Area of the tensile reinforcement
A_{sw}	Cross-sectional area of shear reinforcement
$C_{Rd,c}$	NDP in Gen 1
C_{v1}	Interface roughness coefficient
C_{v2}	Interface roughness coefficient
D_{lower}	Minimum value of the maximum aggregate sieve size

D_{max}	Value of maximum sieve size of aggregates
$E_c, E_{c(28)}$	Tangent modulus of elasticity of normal weight concrete at stress of $\sigma_c = 0$ and at 28 days
$E_{c,eff}$	Effective modulus of elasticity
E_{cm}	Secant modulus of elasticity of concrete
E_{dyno}	Dynamic modulus of elasticity
E_s	Modulus of elasticity of reinforcing steel
F_{cd}	Design value of the compression force in a compression chord or in a strut (compression positive)
F_{td}	Design value of the tension force in a tension chord or tie or in the transverse reinforcement
M_{cr}	Bending moment which causes cracking
M_{Ed}	Design value of the applied bending moment
M_k	Bending moment under characteristic load combination, assuming long-term analysis
M_{Rd}	Design moment resistance of cross section
N_{Ed}	Design value of the applied axial force
RH	Relative humidity of the ambient environment in %
RH_0	Relative humidity at full saturation (100%)
RH_{eq}	Internal relative humidity of concrete at equilibrium, accounting for self-desiccation in high-performance concrete
$V_{c,red}$	Reduction of concrete volume, case study related
V_{deck}	Volume of bridge deck, case study
V_{Ed}	Design shear force in the considered section
$V_{Ed,red}$	Reduced design shear force in the considered section
$V_{Ed,i}$	Design shear force in the interface
V_{Rd}	Design shear resistance
$V_{Rd,c}$	Design shear resistance of the member without shear reinforcement
$V_{Rd,max}$	Design value of the maximum shear force which can be sustained by the member, limited by crushing of the compression struts.
$V_{Rd,s}$	Design value of the shear force which can be sustained by the yielding shear reinforcement
$V_{s, supp. red}$	Reduced volume of steel at support, case study

Latin lower case letters

a	Distance/Geometrical data
a_{cs}	Effective shear span with respect to the control section
a_{crit}	Distance to the control section / critical section

a_{sup}	Width of support
a_v	Mechanical shear span ; Distance from support to concentrated load
$a_{y,i}$	Distance from the centerline of reinforcement to the face of the concrete
b	Overall width of cross-section
b_1	Distance between longitudinal reinforcement bars fixed by confinement reinforcement
b_i	Width of interface
b_w	Width of web/beam
$b_{cs,x}, b_{cs,y}$	The width of confined concrete in x- and y directions
c	Concrete cover of reinforcement (distance from the face of concrete to the surface of the main reinforcement)
c_{act}	Actual concrete cover, may be larger than c_{nom} due to detailing.
c_{bot}	Concrete cover in the bottom of the bridge deck slab, case study
c_d	Nominal concrete cover used for anchorage calculations
$c_{min,dur}$	Minimum concrete cover due to durability requirement
c_{nom}	Nominal value of the concrete cover c which is specified in drawings.
c_s	Clear distance between parallel reinforcement bars
c_{top}	Concrete cover in the top of the bridge deck slab, case study
d	Effective depth of cross-section/slab
d_d	Design effective depth
d_g	Largest nominal aggregate size
d_{dg}	Aggregate size parameter used for detailing rules
d_{nom}	Nominal effective depth
d_v	Shear-resisting effective depth
d_x, d_y	Effective depth in x or y direction for planar members
e_p	Eccentricity of normal force from the center of gravity of the section for shear force capacity
f_{bd}	Design value of the ultimate bond stress
f_{cd}	Design compressive strength of concrete
$f_{cd,c}$	Design value of strength of confined concrete
f_{cm}	Mean value of concrete cylinder compressive strength
f_{cm0}	Reference mean compressive strength
$f_{cm,28}$	Mean compressive strength of concrete at 28 days of curing
f_{ctd}	Design tensile strength of concrete
$f_{ct,eff}$	Mean tensile strength at crack formation
f_{ck}	Characteristic compressive cylinder strength of concrete
$f_{ck,c}$	Characteristic compressive strength of confined concrete

$f_{ck,ref}$	Reference characteristic compressive strength
$f_{ctk,0.05}$	5% fractile characteristic tensile strength
f_{ctm}	Mean tensile strength
f_{yd}	Design yield strength of reinforcement
f_{ywd}	Design yield strength of shear reinforcement
h	Overall depth of a cross-section or of a part of a cross-section
h_0, h_n	Notional size of member
$h_{c,eff}$	Effective height of concrete for crack control
h^*	Height of section, with a maximum value of 1 m
k	Factor for concrete shear resistance, taking into account effective depth; Factor depending on height for crack control
k_1	NDP for concrete shear capacity; Factor considering effect of axial force and eccentricity on the concrete shear capacity; Bond condition coefficient
k_{1b}	Factor taking into account the effect of casting position on bond conditions for anchorage
$k_{1/r}$	Factor that considers the effect of curvature on crack formation
k_{1s}	NDP for computing lap length
k_2	Strain distribution factor
k_{3-4}	NDP for Maximum crack spacing, $s_{r,max}$
k_b	Factor taking into account the effect of casting position of bond conditions for crack control
k_c	Stress distribution coefficient for crack control
k_{cp}	Coefficient accounting for casting effect on bond conditions considering anchorage length
k_e	Aggregate-dependent NDP for modulus of elasticity
k_{fl}	Factor taking into account the stress state for crack spacing
k_h	Coefficient dependent on sectional geometry to account for the effect of non-uniform self-equilibrating stresses; Coefficient dependent on notional size, used for shrinkage calculations
k_{surf}	Factor considering the effect or ratio of actual cover/minimum cover
k_t	Crack control factor dependent on the duration of the load; Reference time factor
k_{tc}	Time factor
k_{tt}	Sustained load factor
k_{vp}	Factor considering the effect of axial forces on the concrete shear capacity
k_v	Factor depending on roughness of interface
k_w	NDP for crack control

k_{vp}	Coefficient considering effect of axial force
k_{dowel}	Factor depending on the roughness of the interface
$k_{conf,b}, k_{conf,s}$	Effectiveness factor for confinement
k_{pp}	Factor for axial compression effect on punching
l	Length; Span
l_0	Design lap length
$l_{0,min}$	Minimum lap length of reinforcing steel
l_{bd}	Design anchorage length of reinforcing steel
$l_{bd,min}$	Minimum anchorage length
$l_{b,rqd}$	Basic required anchorage length
l_c	Length of span
$m_{c,red}$	Reduced mass of concrete, case study related
m_{CO_2}	Mass of CO_2
m_s	Mass of steel
n	Non-dimensional axial force
n_b	Number of anchored bars or pairs of lapped bars in the potential splitting failure surface
n_σ	Exponent to consider effect of steel stress on anchorage length
q	Distributed load
s	Center to center spacing of reinforcement
s_c	Spacing of the confinement reinforcement along the bar to be anchored
s_l	Spacing of longitudinal reinforcement
$s_{r,max}$	Maximum crack spacing
$s_{r,m,cal}$	Calculated mean crack spacing when all cracks have formed or where not all cracks have formed, the maximum length along which there is slip between concrete and steel adjacent to a crack
s_t	Spacing of transverse reinforcement
s_y	Spacing between layered bars in y-direction
t	Thickness of a strut; Length over which the support reaction is distributed; Time being considered, age of the concrete
t_0	Age of concrete when the event under consideration occurs (pre-stressing, settlement, start of drying, loading age)
$t_{0,adj}$	Concrete age at loading adjusted for strength class of cement and temperature
t_s	Age of concrete at the beginning of drying
t_{ref}	Reference curing time
u	Perimeter exposed to drying
v_0	Resulting shear force in planar members

v_x, v_y	Out-of-plane shear force in planar members on the cross-sections perpendicular to the x and y directions
v_{Ed}	Design shear force per unit width for planar members
v_{min}	Factor used in shear resistance calculations without shear reinforcement
$v_{Ed,x}, v_{Ed,y}$	Design shear force per unit width in x or y directions for planar members
w	Width across which the shear force is distributed in planar members
w_k	Characteristic crack width
$w_{k,cal}$	Calculated crack width
$w_{lim,cal}$	Limiting crack width to be compared with the calculated crack width $w_{lim,cal}$
w_{max}	Limiting calculated crack width
w_{min}	Minimum width across which the shear force is distributed for planar members
x	Depth of the neutral axis at serviceability limit state
x_{cs}	Distance between the neutral axis and the axis of confinement reinforcement
x_I	Height of the neutral axis in the uncracked state
y_{cs}	Distance to concentrated load from support for design shear force in cantilever
z	Inner lever arm of internal forces for shear design

Greek letters

α	Angle; Ratio; Coefficient; Inclination of reinforcement across interface
α_{1-5}	Coefficient considering the influence of concrete strength, for creep calculations; Factors for computing anchorage lengths
α_6	Factor for calculating lap length
α_{bs}	NDP for the influence of cement type on basic shrinkage
α_c	Concrete strength factor
α_{ct}	Concrete tensile strength factor
α_{cc}	Long-term compressive strength reduction factor
α_{cw}	NDP shear capacity with stirrups
α_{ds}	Factor dependent on speed of concrete strength development and concrete class
$\alpha_{ds1}, \alpha_{ds2}$	Shrinkage-related coefficient dependent on the type of cement
α_e	Modular ratio E_s/E_{cm}
$\alpha_{e,eff}$	Effective modular ratio $E_s/E_{c,eff}$

α_h	Reduction coefficient for length or height
$\alpha_{NDP,b}$	NDP for basic shrinkage
$\alpha_{NDP,d}$	NDP for drying shrinkage
β	Angle; Ratio; Coefficient;
$\beta_{as}(t)$	Function describing development of autogenous shrinkage strain
$\beta_{bc,f_{cm}}$	Function describing the effect of concrete strength on basic creep
$\beta_{bc,t-t_0}$	Function describing the time development of basic creep
$\beta_{bs,t}$	Function describing time development of basic shrinkage
$\beta_c(t, t_0)$	Coefficient describing the development of creep with time after loading
$\beta_{dc,f_{cm}}$	Function describing the effect of concrete strength on drying creep
$\beta_{dc,RH}$	Function describing the effect of relative humidity on drying creep
β_{dc,t_0}	Function describing the effect of the adjusted concrete age at loading ($t_{0,adj}$) on drying creep
$\beta_{dc,t-t_0}$	Function describing the time development of drying creep
$\beta_{ds,t}$	Factor considering basic shrinkage over time
$\beta_{ds,t-t_s}$	Factor considering notional size and time development for calculating drying shrinkage
$\beta(f_{cm})$	Factor considering concrete strength on the notional creep coefficient; Factor considering notional size and concrete strength on the drying creep coefficient
β_h	Factor considering relative humidity and notional member size with regards to creep
β_{incl}	Angle between support and concentrated load for loads close to supports
β_{new}	Ratio of longitudinal force in new concrete due to composite action
β_{RH}	Shrinkage factor considering the effect of relative humidity
β_{t_0}	Factor considering the effect of concrete age at loading on the notional creep coefficient
γ_c	Partial factor for concrete
γ_s	Partial factor for steel
γ_v	Partial factor for shear
γ_{CE}	Partial factor including execution effects
Δb	Deviation in width
Δc	Deviation in cover
Δd	Reduction of effective depth
Δf_{cd}	Deviation in the design compressive strength of concrete
Δh	Deviation in height to quasi-permanent load combination
ε	Strain
ε_c	Concrete strain

ε_{c2}	Compressive strain in the concrete at the peak stress f_c
$\varepsilon_{c2,c}$	Value of ε_{c2} in case of confined concrete
ε_{c3}	Parabolic strain at peak stress
ε_{ca}	Autogenous shrinkage
$\varepsilon_{ca}(\infty)$	Final autogenous shrinkage strain
ε_{cbs}	Basic shrinkage
$\varepsilon_{cbs,f_{cm}}$	Basic shrinkage dependent on concrete strength
$\varepsilon_{cd}, \varepsilon_{cds}$	Drying shrinkage
$\varepsilon_{cd,0}$	Nominal unrestrained drying shrinkage
$\varepsilon_{cd,f_{cm}}$	Drying shrinkage dependent on concrete strength
ε_{cm}	Mean strain in the concrete between cracks
ε_{cs}	Total concrete shrinkage
$\varepsilon_{cs}(t, t_s)$	Mean shrinkage strain
ε_{cu}	Ultimate strain in the concrete
ε_{cu2}	Ultimate strain in the concrete at peak stress f_c
$\varepsilon_{cu2,c}$	Ultimate strain in the confined concrete at peak stress
$\varepsilon_{cu,c}$	Ultimate strain in the confined concrete
ε_s	Strain in reinforcing steel
ε_{sm}	Mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond the state of zero strain of the concrete at the same level is considered
η	Compression block factor; Ratio
η_1	Factor taking into account the casting position on bond conditions for anchorage
η_2	Anchorage length factor considering bar diameter
η_{cc}	Factor to account for the difference between the undisturbed compressive strength of a cylinder and the effective compressive strength that can be developed in the structural component
η_{sys}	Coefficient accounting for the performance of punching shear reinforcing systems
θ	Angle between the compression field and the member axis; Rotation under bending moment
θ_i	Inclination representing a geometrical imperfection
θ_{min}	Minimum allowed value of θ
λ	Compression block height factor
μ	Roughness interface factor
μ_v	Coefficient of friction at concrete interfaces

ν	Poisson's ratio; Strength reduction factor for concrete cracked due shear or other actions
ν_1	Efficiency factor for shear capacity with stirrups (NDP)
ν_{min}	NDP for concrete shear capacity
ξ_1	Adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel
ξ_v	Effective damping ratio
ρ	Reinforcement ratio; Density
ρ'	Reinforcement ratio of steel in compression
ρ_c	Density of concrete
ρ_{conf}	Ratio of the reinforcement providing confinement referred to the diameter of the bar to be anchored or spliced
ρ_i	Ratio of bonded reinforcement across interface
ρ_l	Reinforcement ratio for bonded longitudinal reinforcement in the tensile zone due to bending referred to the nominal concrete area $d \cdot b_w$
ρ_{min}	Minimum reinforcement ratio
$\rho_{p,eff}$	Effective reinforcement ratio of bonded tendon
ρ_s	Density of reinforcement
ρ_v	Shear reinforcement ratio
ρ_w	Shear reinforcement ratio
$\rho_{w,min}$	Minimum shear reinforcement ratio
ΣA_{st}	Cross-sectional area of the transverse reinforcement along the design anchorage length l_{bd}
$\Sigma A_{st,min}$	Cross-sectional area of the minimum transverse reinforcement
$\Sigma \xi_{v,st}$	Sum of the effective structural damping
$\Sigma \xi_{v,non,st}$	Sum of the effective damping from non-structural components
σ	Stress
σ_2	Principal lateral stress
σ_c	Compressive stress in the concrete
σ_{cd}	Design value of compressive stress in the concrete
σ_{c2d}	Confinement stress
σ_{cp}	Axial stress
σ_n	Compressive stress across interface; Stress per unit area
σ_s	Serviceability value of steel stress, determined on the basis of a cracked section
σ_{sd}	Design value of the reinforcing steel stress at the cross-section
σ_{sr}	Design value of stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking

τ	Shear stress
τ_{Ed}	Average acting shear stress over a cross-section
$\tau_{Ed,i}$	Design value of the shear stress at interfaces
τ_{Rd}	Shear resistance governed either by yielding of shear reinforcement or crushing of concrete
$\tau_{Rd,c}$	Shear stress resistance of members without shear reinforcement (average shear stress over a cross-section)
$\tau_{Rd,c,min}$	Minimum shear stress resistance allowing to avoid a detailed verification for shear (average shear stress over a cross-section)
$\tau_{Rd,c,max}$	Maximum shear capacity of concrete when considering axial forces
τ_{Rdi}	Shear stress resistance at interfaces
$\tau_{r,Rd,sw}$	Torsional shear stress resistance governed by yielding of shear reinforcement
$\tau_{t,Rd,sl}$	Torsional shear stress resistance governed by yielding of longitudinal reinforcement
$\tau_{t,Rd,max}$	Torsional shear stress resistance governed by crushing of the concrete in the compression field
ϕ	Diameter of a reinforcing bar
ϕ_w	Reinforcement diameter of stirrups/shear reinforcement
φ	Creep related coefficients
φ_0	Notional creep coefficient
$\varphi(t, t_0)$	Creep coefficient, defining creep between times t and t ₀ , related to elastic deformation at 28 days
$\varphi(\infty, t_0)$	Final value of creep coefficient
$\varphi_{bc}(t, t_0)$	Basic creep coefficient
$\varphi_{dc}(t, t_0)$	Drying creep coefficient
φ_{RH}	Factor considering the effect of relative humidity on the notional creep coefficient

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1

INTRODUCTION

The Eurocodes represent an important tool for structural designers in the European Union, establishing a common framework for the design of structures while simultaneously fulfilling safety and serviceability requirements. However, the current codes are based on methods, theories, and data that need to be updated to the current state-of-the-art; a task for EU member states. SS-EN 1992-1-1:2023 Eurocode 2 for concrete structures is part of a new generation of standards that will be used in Europe. Production of the new Eurocode is still in progress, and the Swedish Transport Agency is planning on having set the national parameters by the fall of 2026, to then report it to the EU by spring 2027. The new Eurocode will become effective by July 2027, and the current version will be phased out by March 30th, 2028.

There are several differences between the two generations of Eurocode, which may have an impact on the design of concrete structures. For example, the new Eurocode will allow the designer to account for differences in aggregate and binder types when compared to the standard concrete mix used in the current Eurocode. There are also revisions to how shear capacity is modeled, among many other changes, potentially influencing design. It is important to understand these differences to know how the design of concrete structures will be affected. Knowing the differences and their impact will also give insight as to whether previous methods have been conservative or not concerning structural capacity and material efficiency, and if the ease of use of the codes has been improved or not.

The thesis is carried out in collaboration with Norconsult Sverige AB, Bridge and Analysis Department, who wish to get an understanding of how the implementation of the new Eurocode could impact the construction industry and to be prepared for the upcoming shift of standards.

1.1 Aim

The thesis aims to describe the differences between the new SS-EN 1992-1-1:2023 Eurocode 2 and the current version (SS-EN 1992-1-1:2005 + SS-EN 1992-2:2005), and explain how these differences will impact the design and implementation of concrete structures. Reasons for the changes are also evaluated. The thesis also aims to give advice on the choice of national parameters for the final version of the new Eurocode.

1.2 Scope & Limitations

The thesis project is limited to “Concrete Structures”, SS-EN 1992-1-1:2005 and SS-EN 1992-2:2005 (Gen 1), and focuses on specific and relevant aspects that have been modified in the new Eurocode SS-EN 1992-1-1:2023 (Gen 2). The thesis aims to give a summary of these aspects through a literature study, but does not offer proof of concepts or models behind the revisions since these have already been covered in the background document for the next generation of Eurocode, see [9].

The scope of the thesis primarily includes areas of the Eurocode where significant changes are identified in the initial literature study. A detailed analysis is performed through a parametric study and a case study, where capacities from both versions of the Eurocode are calculated and compared. This includes changes to partial factors and material models, bending moment, shear force capacity, crack widths, and anchorage and lapping. Another area, which was concluded to have significant revisions but, due to time constraints and other limitations, was decided not to be included, is the chapter considering shear punching. Shear punching, together with other design criteria not studied in detail, was included in Chapter 8.

In the design methods where the application of Nationally Determined Parameters (NDPs) is required, the recommended NDPs from the Swedish Transport Agency (TSFS-2018:57) are used for the current Eurocode [15]. For Gen 2 of the Eurocode, the preliminary NDPs proposed in TSFS are implemented [18]. However, these values may change in the future. Therefore, the thesis may offer suggestions to the new NDPs for Gen 2 of the Eurocode.

The thesis does not include structural analyses of load cases and sectional forces, since the second generation of Eurocode for loads on structures is, at the time of the thesis’s work (spring 2026), not yet released. Instead, resistances are calculated from the geometries of the studied sections. In cases where the capacity of a given section depends on the stress state, assumptions are made based on expected utilization ratios.

1.3 Methodology

The project consists of a literature study, a parametric study, and a case study to ensure that the comparison and analysis of the new version of Eurocode are thorough.

1.3.1 Literature study

The literature study is used to obtain comprehensive knowledge about the implementation of the new Eurocode. Relevant sources include the current and new Eurocode, and a background document to the first and second generations of Eurocode, containing derivations and motivations behind the models and parameters. The background documents also contain explanations of the values of the preliminary recommended NDPs. Other documents, such as reports and literature relevant to the subject, are also used, for example, parametric studies made by the Swedish Transport Agency to investigate and determine possible values for the Swedish recommended NDPs.

The information from the literature study is first reviewed in chapter 2. General changes of SS-EN 1992-1-1:2023 are summarized. Then, each design criterion is graded based on the significance of the revision (Table 2.1), including references to the old and new versions. The grading is used to determine the design criterion to be analyzed further, and the motivational section (Section 2.4) reinforces the criterion's place in the thesis. An orientational section with references to relevant chapters in the thesis is presented in Section 2.3.

Secondly, a more in-depth literature review of the design criteria of interest is made to provide further details for the revisions (see Table 2.2). The criteria with the most significant changes are given their own chapters, while the less significant sections are included in chapter 8. Motivations for the revisions are included, and for chapters 3-8, descriptions of the design procedure in both versions of the Eurocode are also provided.

The literature study was conducted simultaneously with the parametric study and aims to explain the reason behind the calculative differences between the two Eurocodes. The information gathered is used throughout the project to understand the differences and their potential consequences on the design of concrete structures.

1.3.2 Parametric study

The parametric study consists of analytical calculations of selected design criteria, according to both versions of Eurocode. The calculations are performed using the mathematical software program MathCAD Prime 10.0.1.0 and summarized in Microsoft Excel. The design criteria are applied for a one-meter-wide concrete section with varying input parameters, including cross-sectional height, concrete quality, and reinforcement diameter. The concrete section is subjected to varying forces

and moments, and other design parameters such as reinforcement ratio, concrete cover, or concrete strength. The parametric study also investigates the impact of the preliminary NDPs proposed by the Swedish Transport Agency. In addition, the impact of the modified partial factors in the new Eurocode is investigated. The parametric study aims to achieve a comprehensible understanding of the impact of these revisions.

1.3.3 Case study

A case study is also performed, where the design methods of the current and new Eurocode 2 are applied for a typical Swedish reinforced concrete frame bridge. The aim is to compare resistances in critical sections and to conclude practical differences in terms of material use, such as the amount of reinforcement and the quantity of concrete. The case study covers the same design criteria that are examined in the parametric study, while using the Swedish NDPs proposed by TSFS.

1.4 Societal, Ethical, and Ecological Aspects

The thesis may act to guide the civil engineering industry in the European Union through the transition period when the new code is implemented. By summarizing changes and giving examples of differences (capacities, crack widths, and anchorage lengths/lap lengths) through parametric analyses and the case study, the contrast between the codes will become more apparent. The analysis of the national parameters will primarily aid the Swedish industry in a similar manner and might provide advice for adjustments to the National Annex. The potential differences in material can also have an ecological impact, since the results can be used to predict future material use in the construction sector in the EU. Additionally, old structures, designed with the current Eurocode or older standards, may be considered overly conservative or unsafe when re-evaluated with the next generation of Eurocode. This could potentially lead to changes in how the Swedish Transport Agency implements calculations for structural analyses in the future.

1.5 Application of AI

The artificial intelligence tool ChatGPT has been used to transfer mathematical expressions from MathCAD to $\text{\LaTeX}$. Mathematical expressions can be hard to generate in $\text{\LaTeX}$, but artificial intelligence can transform the calculations much faster than what can be done by hand. ChatGPT was also utilized to help with the formatting of tables, figures, equations, and text in $\text{\LaTeX}$, to help present the information of the thesis in a clear and coherent way. It is important to note that ChatGPT was not used to create original texts or to gather information.

2

CHANGES IN THE NEXT GENERATION OF EUROCODE - AN OVERVIEW

The second-generation Eurocode 2, SS-EN 1992-1-1:2023, contains several significant changes to the modeling of material and structural behavior compared to previous codes. The main objective of this chapter is to review the general changes of each section in Eurocode (SS-EN 1992-1-1:2005 + SS-EN 1992-2:2005 (Gen 1) and SS-EN 1992-1-1:2023 (Gen 2), and to conclude which design criteria are most affected by these changes. The background document from the European Committee for Standardization (CEN) has been used as a guide for the overview, see [9].

2.1 General changes

A focus point for the new version is enhanced ease of use. One of the main changes implemented in the new Eurocode to fulfill this goal is the merging of Eurocodes EN 1992-2 (bridges) and SS-EN 1992-1-1:2005 (General rules and rules for buildings) and SS-EN 1992-3 (Liquid retaining and containment structures). The number of Nationally Determined Parameters (NDPs) is also limited in the new version, which serves to enhance ease of use [8].

Simplified versions of specific verifications have been made, while more complex verifications have been moved to annexes (e.g., deflection). Calibration of the Eurocode has been made to make design on paper, transfer more accurately to reality, revising or introducing new coefficients and limits for certain cases, where they have previously been over- or underconservative (anchorage/lap length), or where special cases lead to unrealistic results (which is the case for the definition of the effective area for crack control) [29].

Furthermore, Gen 2 has broadened the assessment and retrofitting of existing structures by introducing new rules for reuse and sustainability [8], pushing the construction industry to become more conscious regarding environmental impact.

2.2 Evaluation of design criteria

Important design criteria in the two Eurocodes have been graded from one to three depending on the extent of differences between the codes, according to the following system:

Grade 1: No or minor differences

Grade 2: Moderate differences

Grade 3: Substantial differences

Grade 1 is given to clauses where the changes most likely result in small or negligible consequences. A clause can therefore have gone through many changes on paper, but mainly consists of a restructuring of equations and renaming coefficients, with an insignificant impact on design. **Grade 2** describes clauses that have gone through visible changes to coefficients and model choices, but where the impact on design is expected to be limited. **Grade 3** is designated to clauses that have been subjected to significant revisions, which can include added sections, complete rework of models, or where consequences are expected to be severe, resulting in major changes to dimensions and capacities.

Table 2.1: Grading of design criteria

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	GR
Materials	2.4, 3.1, Ann. A	4.3, 5.1, Ann. A and B	2
Durability and concrete cover	EN 1992-1, 4.2, 4.4	6.3, 6.5	2
Structural analysis	1992-2, 5.2, 5.5, 5.6, 5.8	7.2, 7.3, 7.4	1
Bending	3.1, 6.1, 1992-2, Ann. LL	8.1, Ann. G.4	1
Shear	6.2	8.2	3
Torsion and combined action	6.3	8.3, Ann. G	2
Punching*	6.4	6.4, 8.4, Ann. I	3
Design with strut and tie	6.5	8.5	2
Partially loaded areas	6.7	8.6, Ann. I	1
Stress limitation and crack control	7.3, 7.4	9.2	3
Deflection control	7.4	9.3	1
Vibrations*	-	9.4	3
Fatigue	6.8	10.4, 10.5, Ann. C and E	2
Detailing of reinforcement and tendons	8.3, 8.4, 8.6, 8.7, 8.9, 8.10	11.3-11.7, Ann. C.4	3
Detailing and particular rules	9.2-9.7	12.2-12.7	3
Precast concrete	8.10	13.5	2
Lightly reinforced concrete	12.6	14.4	1

* see motivation in Section 2.4.6.

2.3 Orientation of design criteria

All design criteria that were graded in Table 2.1 are summarized in the following Table 2.2, combined with references to the chapters in the thesis where more information can be found. Only certain design criteria were determined to be analyzed more in depth. These criteria have their own chapters (see "Lit. study" in Table 2.2), and are further examined in the parametric study (Chapter 9) and the case study (Chapter 10).

The detailed motivation, whether a design criterion was determined to need further examination and therefore be included in the parametric study and the case study, or if the literature study was deemed sufficient, is included in Section 2.4.

Table 2.2: Orientation of where to find clauses in the thesis

Design criteria	Lit. study	Par. study	Case study
Materials	3	9.1	10.1.2, 10.1.3
Bending moment	4	9.2	10.3
Shear capacity without shear reinforcement	5	9.3	10.4
Shear capacity with shear reinforcement	5	9.4	10.4
Crack control	6	9.5	10.5
Anchorage of straight bars	7.1	9.6.1	10.6
Lapping of straight bars	7.2	9.6.3	10.6
Durability and concrete cover	8.1	-	-
Structural analysis	8.2	-	-
Punching*	8.3	-	-
Torsion and combined actions	8.4	-	-
Design with strut and tie models and stress fields	8.5	-	-
Partially loaded areas	8.6	-	-
Deflection	8.7	-	-
Vibrations*	8.8	-	-
Fatigue	8.7	-	-
Detailing of reinforcement and tendons	8.10	-	-
Detailing of members and particular rules	8.11	-	-
Additional rules for precast concrete elements and structures	8.12	-	-
Plain and lightly reinforced concrete structures	8.13	-	-

* see motivation in Section 2.4.6.

2.4 Motivation of design criteria

This section works to motivate the decision of the arrangement and expansion - or lack thereof - of design criteria in the parametric study and the case study of the thesis. Only the design criteria that were concluded to be highly important and/or have seen major rework in the new generation of Eurocode are analyzed in depth. The design criteria that have not been investigated thoroughly in this thesis - either due to low grading, or time constraints - are summarized in Chapter 8.

2.4.1 Materials

The materials chapter is important to examine, even though it was graded a 2 in Table 2.1, since the partial factors have been modified in the new Eurocode to include a new partial factor for shear. The designer also has the option of reducing the partial factors for shear and reinforcing steel [11] (Annex A.3). This can be applied when using a design value of the effective depth for calculating sectional capacities, see Section 3.1. The changes made to the partial factors permeate the calculation process of any structure, and the impact of the revisions is therefore important to examine.

The new model for designing concrete strength can, in some cases, result in a different capacity than in Gen 1, especially for high-strength concretes, which are influenced by a new strength reduction factor [40]. Even though long-term effects, such as shrinkage and creep, have seen changes, they are not included in the parametric study. This is to reduce the number of variables and simplify the results. However, these effects are still taken into account in the case study.

The design concrete strength, along with modifications to partial factors, could be of importance, since these changes can influence material use and environmental impact, and are therefore included in the parametric study and the case study (see Table 2.2). However, the impact of the materials chapter is not analyzed in its own parametric study and case study. Section 9.1 (parametric study) and 10.1.2 (case study) are therefore only used to define the input parameters of the materials, then used in the forthcoming chapters.

2.4.2 Bending moment

The procedure to determine moment capacity has not changed much between the Eurocodes, therefore giving it grade 1 in Table 2.1. However, the above-mentioned effects, such as changes to partial factors and design strength, could influence the moment capacity. Bending is also a highly important aspect to consider during the design of concrete structures, which is why it is studied in detail in the parametric study and the case study (see Table 2.2).

2.4.3 Shear force

The clauses handling shear in the new Eurocode are completely revised [35], therefore earning it grade 3 in Table 2.1. Capacity calculations without and with shear reinforcement are especially interesting to examine since these could have a large impact on the design, and are therefore included in the parametric study, as well as the case study (see Table 2.2).

2.4.4 Crack control

Cracking control was graded a 3 in the evaluation section, Table 2.1, due to significant revisions to the models between Gen 1 and Gen 2. It is therefore included in the detailed analysis of this thesis (see Table 2.2). The chapter for crack control includes a new model for the calculation of crack width and changes to the maximum allowed crack width limits [4].

2.4.5 Anchorage and laps

Requirements for anchorage and lapping have also undergone major changes, earning it a 3 in grade in the evaluation chapter, Table 2.1. Detailing of reinforcement has been given a new model for calculating anchorage lengths and lap lengths [31]. Anchorage and laps are investigated in the parametric study and case study, where analyses of straight bars are made (see Table 2.2).

2.4.6 Other design criteria

Although several other design criteria have been revised considerably, they are not treated in their own chapter and are instead included in Chapter 8. This is the case for shear punching, which was given grade 3 in the evaluation of design criteria in Table 2.1 due to major changes in the new Eurocode [39]. However, it is not included in the parametric study due to the other above-mentioned design criteria being considered more important and due to time constraints. Similarly, clauses for vibrations were also given grade 3 in Table 2.1, and detailing of reinforcement for other than straight bars is not studied in detail because these clauses are not deemed to be commonly utilized for design. The remaining design criteria included in Chapter 8 have been placed there due to the motivations written at the beginning of this Section, 2.4.

3

MATERIALS

Material properties, such as concrete strength, elasticity, and partial factors, have been revised in the second generation of Eurocode 2. The apparent differences, with references to the Eurocodes, can be found in Table 3.1. Reasons for changes, together with potential consequences, are discussed in this chapter.

Table 3.1: Overview of changes for materials

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Partial factors	2.4.2.4, Ann. A	4.3.3, Ann. A	Reduced partial factors when uncertainty is limited. New partial factor for shear.
Concrete strength	3.1.2	5.1.3 Ann. B.4	New strength class. Longer curing times, inclusion of SCMs.
Modulus of elasticity	3.1.3	5.1.4, Ann. B.4	Dependant on aggregate type and size. Effects of SCMs included.
Creep and shrinkage	3.1.4, Ann. B	5.1.5, Ann. B.5, B.6	Modifications to models.
Concrete design factors	3.1.6	5.1.6	New strength reduction factor.

3.1 Partial factors

In SS-EN 1992-1-1:2005, partial factors are used to limit material and geometric uncertainties via clause 2.4.2.4 and Annex A. There is also a possibility to reduce these partial factors when uncertainties can be reduced. This is kept in SS-EN 1992-1-1:2023 in clause 4.3.3, but now the designer is given the option to adjust partial factors with formulas provided in Annex A, when statistical data for uncertainties can be proven to be more favorable than what is assumed in the Eurocode [30]. Additionally, the designer is given the option to reduce partial factors for steel and shear, γ_s and γ_v , when using design values for effective depth [11] [30].

A new partial factor for shear and punching resistance without shear reinforcement is also included in SS-EN 1992-1-1:2023 (Table 4.3). This new factor is introduced on the basis that the effect of variability on material strength, geometrical uncertainties, and model uncertainties is reduced, since the compressive strength of concrete has an exponent of 1/3 in the design formula for shear [30].

3.1.1 Design in Gen 1

The partial factors used for design in Eurocode 2 SS-EN 1992-1-1:2005, clause 2.4, are shown in Table 3.2.

Table 3.2: Partial safety factors for materials in EC 2 Gen 1

Design situations	γ_c for concrete	γ_s for reinforcing steel	γ_s for prestressing steel
Persistent & Transient	1.5	1.15	1.15
Accidental	1.2	1.0	1.0
Serviceability limit state	1.00	1.00	—

If measures are taken to limit deviations in cross-section dimensions and position of reinforcement following Table 3.3, the partial factor for reinforcing steel, γ_s , can be reduced according to NPDs; however, a recommended value is 1.1. Following these rules, the partial factor for concrete, γ_c , can also be reduced to 1.4, given that the coefficient of variation is below 10% according to Annex A.2 in SS-EN 1992-1-1:2005 [13].

If capacity calculations are based on field measurement of an existing structure, the partial factors for steel and concrete can be reduced to 1.05 and 1.45, respectively. For the same case, if the coefficient of variation can be proven to be less than 10%, the partial factor for concrete can be set to 1.35.

Table 3.3: Reduced deviations for dimensions and reinforcement position from Table A.1 in SS-EN 1992-1-1:2005

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

Worth noting is that the intermittent values not included in Table 3.3 can be calculated by interpolation.

3.1.2 Design in Gen 2

Partial factors implemented in SS-EN 1992-1-1:2023 clause 4.3.3 are shown in Table 3.4, where a new partial factor for shear and punching is introduced.

Table 3.4: Partial factors for materials by design situation in Gen 2

Design situations — Limit states	γ_s for reinforcing and prestressing steel	γ_c and γ_{CE} for concrete	γ_v for shear and punching resistance without shear reinforcement
Persistent and transient design situation	1.15	1.50	1.40
Accidental design situation	1.00	1.15	1.15
Serviceability limit state	1.00	1.00	—

Partial factors for reinforcing steel and shear in concrete can be reduced, given that the design effective depth, d_d , is used for calculating design resistances following Annex A.3 in SS-EN 1992-1-1:2023.

$$d_d = d_{nom} - \Delta d \quad (3.1)$$

where

$$\Delta d = \begin{cases} 15 \text{ mm} & \text{for reinforcing steel} \\ 5 \text{ mm} & \text{for prestressing steel} \end{cases}$$

and d_{nom} is the nominal distance from the top of the section to the center of reinforcement. As a result, the coefficient of variation for geometric uncertainty can be set to zero, and the bias factor set to 1. Recommended values for other bias factors and coefficients of variation are found in Annex A in SS-EN 1992-1-1:2023 [11]. Following these recommendations, the modified partial factors for steel and concrete are:

$$\gamma_s = 1.03$$

and

$$\gamma_v = 1.29$$

The Eurocode recommends that d_d can be used for capacity calculations, but the application of d_d can be limited by the National Annex. If measures are taken to reduce uncertainty, the partial factors can be reduced further by utilizing the equations found in Annex A in SS-EN 1992-1-1:2023 [11].

3.2 Concrete strength

The current Eurocode contains concrete classes, where each class has a different strength and stiffness. These properties are defined for $t_{ref} = 28$ days of curing in the current EC, while Gen 2 allows the designer to set a reference time between 28 and 91 days [11] (clause 5.1.3). This allows the designer to take into account the longer curing times of concretes with varying properties [40]. An updated model for strength development over time is introduced in Annex B.4 in the Eurocode, where the development of strength and stiffness depends on t_{ref} and the type of binders used for the cement. Additionally, Gen 2 introduces a new concrete class C100/115 [11] (Table 5.1 in SS-EN 1992-1-1:2023).

3.3 Design concrete strength

Currently, the concrete design strength is dependent on a national parameter, α_{ct} set to 1 or 0.85 depending on National Annexes (clause 3.1.6 in SS-EN 1992-1-1:2005). In Gen 2, clause 5.1.6 introduces three new factors that influence the design strength, η_{cc} , k_{tc} and k_{tt} . The value of η_{cc} (Eq. 3.5) takes into account the strength difference between undisturbed concrete cylinders during testing and structural concrete used in practice. This change leads to a reduction of f_{cd} by 7% for concrete class C50 and 21% for C80 for countries where $\alpha_{ct} = 1$, which is the case in Sweden. Countries with $\alpha_{ct} = 0.85$ would instead see an increase in strength for lower concrete classes and a small decrease for classes with higher strengths [40]. However, the negative effect of the strength reduction factor may be counteracted by the favorable effects from continuous cement hydration and the increased positive effect from confinement reinforcement [40].

3.3.1 Design in Gen 1

The following procedure is used in SS EN1992-1-1:2005 clause 3.1.6 to find the design compressive strength:

$$f_{cd} = \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} \quad (3.2)$$

where α_{cc} accounts for long-term effects on the compressive strength and unfavorable effects due to the application of load, the recommended value is 1.

To calculate the design tensile strength, the following expression is used:

$$f_{ctd} = \alpha_{ct} \frac{f_{ctk,0.05}}{\gamma_c} \quad (3.3)$$

where α_{ct} fulfills the same function as α_{cc} , the recommended value is 1, but can be chosen according to the National Annex. In Sweden, the recommended value is set to 1.0 according to TSFS 2018:57 [15].

3.3.2 Design in Gen 2

In SS-EN 1992-1-1:2005 clause 5.1.6, the design compressive strength is calculated as follows:

$$f_{cd} = \eta_{cc} \cdot k_{tc} \cdot \frac{f_{ck,0,05}}{\gamma_c} \quad (3.4)$$

Where the strength reduction factor η_{cc} is defined as

$$\eta_{cc} = \left(\frac{f_{ck,ref}}{f_{ck}} \right)^{1/3} \leq 1 \text{ with } f_{ck,ref} = 40 \text{ MPa} \quad (3.5)$$

and k_{tc} is set to 1 unless a specific reference time is chosen other than the standard 28 days for regular concrete.

The design tensile strength is taken as:

$$f_{ctd} = k_{tt} \frac{f_{ctk,0.05}}{\gamma_C} \quad (3.6)$$

where k_{tt} considers the effect of high sustained loads and time of loading on concrete tensile strength. The factor should be set to 0.80 if the reference time is less than or equal to 28 days for concrete classes CR and CN, or less than 56 days for concrete class CS. For other cases, the value of k_{tt} should be set to 0.7. However, it is proposed by the Swedish Transport Agency that the factor k_{tt} should instead be set to 1 [18].

3.4 Concrete elasticity

The modulus of elasticity for concrete is defined differently in the two versions of the Eurocode. The design procedure to obtain the modulus of elasticity of concrete for SS-EN 1992-1-1:2005 and SS-EN 1992-1-1:2023 is presented in this section.

3.4.1 Design in Gen 1

In Gen 1, the modulus of elasticity is specified in a table (Table 3.1, SS-EN 1992-1-1:2005). Alternatively, the modulus can be calculated as:

$$E_{cm} = 22 \cdot (f_{cm}/10)^{0.3} \quad (3.7)$$

These values apply to quartzite-based aggregates, and can be reduced or increased if other types of stones are used, such as sandstone or limestone, where the modulus of elasticity is reduced by 30% and 10%, respectively. For basalt aggregates, the E-modulus can instead be increased by 20% [13] (clause 3.1.3).

3.4.2 Design in Gen 2

For SS-EN 1992-1-1:2023, a new factor, k_e , is used to calculate the modulus of elasticity. This factor varies between 5000 and 13000 and depends on the type of aggregates used, with 9500 being a recommended value for quartzite aggregates [11] (clause 5.1.4). The reason for this change is that the stiffness of the rocks used for aggregates varies depending on the region from which the rocks are mined. Thus, the value of k_e should be specified by the National Annexes [40]. Countries where quartzite-based aggregates are commonly used, such as Sweden, would likely use $k_e = 9500$. The modulus of elasticity in Gen 2 is calculated as follows:

$$E_{cm} = k_e \cdot f_{cm}^{1/3} \quad (3.8)$$

Furthermore, the development of stiffness over time now also depends on the binder type, such as SCMs (Supplementary Cementitious Materials) and t_{ref} [11].

3.5 Stress-strain relationship

The stress-strain relationship of concrete can be used to model the structural response of concrete.

3.5.1 Design in Gen 1

The stress-strain relationship in SS-EN 1992-1-1:2005, clauses 3.1.5 and 3.1.7, is defined separately for the design of nonlinear structures and for the design of cross-sections. The model used for sections in bending is described in Section 4.1.1. For a nonlinear analysis, the following model can be used:

$$\frac{\sigma_c}{f_{cm}} = \frac{k \cdot \eta - \eta^2}{1 + (k - 2) \cdot \eta} \quad (3.9)$$

Where

$$\eta = \frac{\varepsilon_c}{\varepsilon_{c1}} \quad (3.10)$$

with

$$\varepsilon_{c1} = 0.7 \cdot (f_{cm})^{0.31} < 2.8 \text{ ‰} \quad (3.11)$$

and

$$k = 1.05 \cdot E_{cm} \cdot \frac{|\varepsilon_{c1}|}{f_{cm}} \quad (3.12)$$

It should be noted that this model takes into account the stiffness, which can be important since the stiffness of concrete depends on the aggregate type and may differ from the standard values. The model applies to strains below the ultimate strain:

$$\varepsilon_c < \varepsilon_{cu1} = 2.8 + 27 \cdot \left[\frac{(98 - f_{cm})}{100} \right]^4 \quad (3.13)$$

3.5.2 Design in Gen 2

The model for the design of cross-sections in bending in SS-EN 1992-1-1:2023 is summarized in Section 4.1.2. An alternative model is presented for concrete under uniaxial compression under short-term loading. This model is the same in Gen 1 for nonlinear analysis, described in Section 3.5.1 above. However, the ultimate strain limit of concrete is instead calculated as:

$$\varepsilon_c < \varepsilon_{cu1} = 2.8 + 14 \cdot \left(1 - \frac{f_{cm}}{108}\right)^4 \leq 3.5 \text{ ‰} \quad (3.14)$$

3.6 Creep

The layout of the chapter covering creep has been revised in the new Eurocode, where the creep coefficient can be found in a table for 50 years of loading. This table takes into account notional size, relative humidity, and different concrete types. In comparison, the current Eurocode uses graphs and formulas, but the resulting creep coefficient should be similar between the two codes [40]. A more detailed creep analysis for structures with lifetimes other than 50 years can be found in SS-EN 1992-1-1:2023, Annex B [11]. Annex B is required for nonlinear creep, as well as in the design of road- and railroad-bridges where the loading time can span up to 120 years.

3.6.1 Design in Gen 1

The current version of the Eurocode includes a simplified method for finding the final creep coefficient through graphs in Figure 3.1 in SS-EN 1992-1-1:2005. These graphs apply for either 50% or 80% relative humidity, for different strength classes, notional sizes, and types of concrete. For design situations that do not comply with the simplified case, Annex B can be used.

Following Annex B in SS-EN 1992-1-1:2005, the creep coefficient at a certain time, t [days], for a given time of loading t_0 , can be calculated as:

$$\varphi(t, t_0) = \varphi_0 \cdot \beta_c(t, t_0) \quad (3.15)$$

where

$$\varphi_0 = \varphi_{RH} \cdot \beta_{(f_{cm})} \cdot \beta_{(t_0)} \quad (3.16)$$

and

$$\varphi_{RH} = \begin{cases} 1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h_0}} & \text{for } f_{cm} \leq 35 \text{ MPa} \\ \left[1 + \frac{1 - RH/100}{0.1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1 \right] \cdot \alpha_2 & \text{for } f_{cm} > 35 \text{ MPa} \end{cases} \quad (3.17)$$

Here, φ_{RH} depends on the relative humidity of the surroundings, $RH[\%]$, and the notional size of the member, which is defined as:

$$h_0 = \frac{2 \cdot A_c}{u} \quad [\text{mm}] \quad (3.18)$$

where u is the part of the perimeter which is exposed to drying and A_c is the area of concrete in the section.

The coefficient $\beta_{f_{cm}}$ takes into account the mean strength of the concrete at 28 days of curing:

$$\beta_{(f_{cm})} = \frac{16.8}{\sqrt{f_{cm}}} \quad (3.19)$$

In addition, the time of loading is taken into account:

$$\beta_{(t_0)} = \frac{1}{(0.1 + t_0^{0.2})} \quad (3.20)$$

And the creep development over time is considered as follows:

$$\beta_c(t, t_0) = \left[\frac{(t - t_0)}{(\beta_{RH} + t - t_0)} \right]^{0.3} \quad (3.21)$$

where

$$\beta_h = \begin{cases} 1.5 \cdot (1 + (0.012 \cdot RH)^{18}) \cdot h_0 + 250 \leq 1500 & \text{for } f_{cm} \leq 35 \text{ MPa} \\ 1.5 \cdot (1 + (0.012 \cdot RH)^{18}) \cdot h_0 + 250 \cdot \alpha_3 \leq 1500 \cdot \alpha_3 & \text{for } f_{cm} \geq 35 \text{ MPa} \end{cases} \quad (3.22)$$

The α -coefficients consider the strength of the concrete and are defined as:

$$\alpha_1 = \left[\frac{35 \text{ MPa}}{f_{cm}} \right]^{0.7} \quad (3.23)$$

$$\alpha_2 = \left[\frac{35 \text{ MPa}}{f_{cm}} \right]^{0.2} \quad (3.24)$$

$$\alpha_3 = \left[\frac{35 \text{ MPa}}{f_{cm}} \right]^{0.5} \quad (3.25)$$

3.6.2 Design in Gen 2

In SS-EN 1992-1-1:2023 clause 5.1.5, the creep coefficient for 50 years of loading can be found in table 5.2 in [11], which depends on time of loading, strength development class, relative humidity, and notional size, h_n (same as h_0 in Gen 1). For structures with longer lifespans, Annex B.5 in [11] can be used, which is summarized below.

The creep coefficient in Gen 2 consists of both basic creep, $\varphi_{bc}(t, t_0)$, and drying creep $\varphi_{dc}(t, t_0)$:

$$\varphi(t, t_0) = \varphi_{bc}(t, t_0) + \varphi_{dc}(t, t_0) \quad (3.26)$$

Where the basic creep coefficient is calculated according to:

$$\varphi_{bc}(t, t_0) = \beta_{bc, f_{cm}} \cdot \beta_{bc, t-t_0} \quad (3.27)$$

The β -factors take into account the strength of concrete and the development of basic creep over time.

$$\beta_{bc, f_{cm}} = \frac{1.8}{(f_{cm,28})^{0.7}} \quad (3.28)$$

$$\beta_{bc, t-t_0} = \ln \left(\left(\frac{30}{t_{0,adj}} + 0.035 \right)^2 \cdot (t - t_0) + 1 \right) \quad (3.29)$$

Where $t_{0,adj}$ is the adjusted time of loading, which can be modified to take into account differences in the speed of strength development, or temperature during curing. The standard value is $t_{0,adj} = t_0$ [days] for cement class CN (normal hardening rate) and a temperature of 20°C. See [11], (B.17-18) for other conditions.

The formula for drying creep is defined as follows:

$$\varphi_{dc}(t, t_0) = \beta_{dc, f_{cm}} \cdot \beta_{dc, RH} \cdot \beta_{dc, t_0} \cdot \beta_{dc, t-t_0} \quad (3.30)$$

Where the β_{dc} -factors take into account the effect of concrete strength, relative humidity, time of loading, and development of drying creep:

$$\beta_{dc, f_{cm}} = \frac{412}{f_{cm,28}^{1.4}} \quad (3.31)$$

$$\beta_{dc, RH} = \frac{1 - \frac{RH}{100}}{\sqrt[3]{0.1 \cdot \frac{h_n}{100}}} \quad (3.32)$$

$$\beta_{dc, t_0} = \frac{1}{0.1 + t_{0,adj}^{0.2}} \quad (3.33)$$

$$\beta_{dc, t-t_0} = \left[\frac{t - t_0}{\beta_h + (t - t_0)} \right]^{\gamma(t_{0,adj})} \quad (3.34)$$

where

$$\gamma(t_{0,adj}) = \frac{1}{2.3 + \frac{3.5}{\sqrt{t_{0,adj}}}} \quad (3.35)$$

$$\beta_h = 1.5 h_n + 250 \alpha_{f_{cm}} \leq 1500 \alpha_{f_{cm}} \quad (3.36)$$

$$\alpha_{f_{cm}} = \left(\frac{35}{f_{cm,28}} \right)^{0.5} \quad (3.37)$$

Here, h_n is the notional size of the member, which is defined as:

$$h_n = \frac{2 \cdot A_c}{u} \quad [\text{mm}] \quad (3.38)$$

where u is the part of the perimeter which is exposed to drying.

Gen 2 also includes lower and upper values for the characteristic creep coefficient, which can be computed as follows:

$$\varphi_{k;0,10} = 0.6 \cdot \varphi(t, t_0) \quad \text{and} \quad \varphi_{k;0,05} = 0.5 \cdot \varphi(t, t_0) \quad (3.39)$$

and

$$\varphi_{k;0,90} = 1.4 \cdot \varphi(t, t_0) \quad \text{and} \quad \varphi_{k;0,95} = 1.5 \cdot \varphi(t, t_0) \quad (3.40)$$

These values should be used for design in cases where displacement has a significant influence.

3.7 Shrinkage

Similarly to creep, the clause for shrinkage has been shortened and revised [40]. In Gen 2, the total shrinkage strain after 50 years can be found in a table, depending on concrete type, notional size, and relative humidity. Formulas of drying shrinkage over time and autogenous shrinkage are now found in Annex B [11]. There have also been changes to the model of shrinkage, and as a result, the final shrinkage coefficient can, in some cases, become higher when using the model in Gen 2. Comparatively, SS-EN 1992-1-1:2005 may in some cases predict approximately 30 to 40% lower shrinkage [40].

3.7.1 Design in Gen 1

Shrinkage in SS-EN 1992-1-1:2005 clause 3.1.4 and Annex B.2 consists of two components: drying shrinkage and autogenous shrinkage.

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \quad (3.41)$$

Autogenous shrinkage occurs immediately after casting, even if drying is not possible, and is defined as:

$$\varepsilon_{ca}(t) = \beta_{as}(t) \cdot \varepsilon_{ca}(\infty) \quad (3.42)$$

where

$$\varepsilon_{ca}(\infty) = 2.5 \cdot (f_{ck} - 10) \cdot 10^6 \quad (3.43)$$

and

$$\beta_{as} = 1 - \exp(-0.2t^{0.5}) \quad (3.44)$$

The drying shrinkage at a certain time is defined as:

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s) \cdot k_h \cdot \varepsilon_{cd,0} \quad (3.45)$$

where t is the considered time, and t_s is the time at which the drying shrinkage starts to take place. k_h is calculated from the notional size of the member, h_0 :

$$h_0 = \frac{2 \cdot A_c}{u} \quad (3.46)$$

where u is the length of the surface which is exposed to drying and A_c is the concrete area.

Based on this, k_h can be interpolated from:

h_0	k_h
100	1.0
200	0.85
300	0.75
≥ 500	0.70

In addition, the variation of drying shrinkage over time, dependent on the notional size, is considered through the following factor:

$$\beta_{ds}(t, t_s) = \frac{t - t_s}{(t - t_s) + 0.04\sqrt{h_0}^3} \quad (3.47)$$

The value for $\varepsilon_{cd,0}$ can be found in [13] (Table 3.2) or can alternatively be calculated using the following equations from Annex B in [13]:

$$\varepsilon_{cd,0} = 0.85 \left[(220 + 110 \cdot \alpha_{ds1}) \cdot \exp \left(-\alpha_{ds2} \cdot \frac{f_{cm}}{f_{cmo}} \right) \right] \cdot 10^{-6} \cdot \beta_{RH} \quad (3.48)$$

Where the values of α_{ds1} and α_{ds2} are 4 and 0.12 respectively for class N, and

$$\beta_{RH} = 1.55 \left[1 - \left(\frac{RH}{RH_0} \right)^3 \right] \quad (3.49)$$

3.7.2 Design in Gen 2

The total shrinkage strain after 50 years can be found in Table 5.3 in SS-EN1992-1-1:2023, which presents shrinkage strains for different strength classes, strength development classes, and relative humidity [11]. However, Annex B can be used to calculate the shrinkage strain for structures with lifetimes other than 50 years, or for conditions different from those listed in Table 5.3 [11] of the Eurocode, which could be relevant for bridge design.

The mean shrinkage strain in Gen 2 consists of basic and drying shrinkage and is defined as:

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cbs}(t) + \varepsilon_{cds}(t - t_s) \quad (3.50)$$

The basic shrinkage occurs immediately after casting, even if drying is prevented:

$$\varepsilon_{cbs}(t) = \varepsilon_{cbs, f_{cm}} \cdot \beta_{bs, t} \cdot \alpha_{NDP, b} \quad (3.51)$$

Where $\alpha_{NDP, b} = 1$ is the recommended value, and

$$\beta_{bs, t} = 1 - \exp(-0.2\sqrt{t}) \quad (3.52)$$

$$\varepsilon_{cbs, f_{cm}} = \alpha_{bs} \cdot \left(\frac{f_{cm, 28}}{60 + f_{cm, 28}} \right)^{2.5} \cdot 10^6 \quad (3.53)$$

The value for α_{bs} depends on the hardening rate of the concrete during curing, and is set to $\alpha_{bs} = 700$ for cement class CN (Normal strength development). $f_{cm, 28}$ is the mean concrete strength after 28 days of curing.

The drying shrinkage is calculated as:

$$\varepsilon_{cds}(t - t_s) = \varepsilon_{cds, f_{cm}} \cdot \beta_{RH} \cdot \beta_{ds, (t-t_s)} \cdot \alpha_{NDP, d} \quad (3.54)$$

where

$$\varepsilon_{cds, f_{cm}} = (220 + 110 \alpha_{ds}) \cdot \exp(-0.012 f_{cm, 28}) \cdot 10^{-6} \quad (3.55)$$

The factor β_{RH} depends on the relative humidity during drying, in relation to the relative humidity at equilibrium, RH_{eq} .

$$\beta_{RH} = \begin{cases} 1.55 \left[1 - \left(\frac{RH}{RH_{eq}} \right)^3 \right] & \text{for } 20\% \leq RH \leq RH_{eq} \\ 1.55 \left[1 - \left(\frac{RH}{RH_{eq}} \right)^2 \right] & \text{for } RH_{eq} < RH < 100\% \\ 1.55 \left[1 - \left(\frac{RH}{RH_{eq}} \right)^2 \right] - 0.25 & \text{for } RH = 100\% \end{cases} \quad (3.56)$$

$$RH_{eq} = 99 \left(\frac{35}{f_{cm, 28}} \right)^{0.1} \leq 99 \quad (3.57)$$

$$\beta_{ds, t-t_s} = \left[\frac{t - t_s}{0.035 h_n^2 + (t - t_s)} \right]^{0.5} \quad (3.58)$$

The recommended value for $\alpha_{NDP, d}$ is 1, and α_{ds} depends on the speed of strength development, where $\alpha_{ds} = 4$ for class CN.

Similarly to creep, the shrinkage strains in Gen 2 have upper and lower characteristic values, which are defined as:

$$\varepsilon_{k;0,10} = 0.6 \cdot \varepsilon(t, t_0) \quad \text{and} \quad \varepsilon_{k;0,05} = 0.5 \cdot \varepsilon(t, t_0) \quad (3.59)$$

and

$$\varphi_{k;0,90} = 1.4 \cdot \varphi(t, t_0) \quad \text{and} \quad \varphi_{k;0,95} = 1.5 \cdot \varphi(t, t_0) \quad (3.60)$$

3.8 Aggregates

The influences of aggregate type and size have been reworked in the new Eurocode 2, now influencing the strength and elasticity of concrete.

3.8.1 Design in Gen 1

As mentioned in Section 3.4, the type of aggregates can influence the modulus of elasticity in Gen 1. However, the effect of size and aggregate type does not have an influence on the strength of concrete.

3.8.2 Design in Gen 2

The effect of aggregate size is included in SS-EN 1992-1-1:2023 through the addition of the new factor d_{dg} , which depends on the minimum value of the maximum sieve size for aggregates, D_{lower} , and the characteristic concrete strength. (clause 8.2.1) [11].

$$d_{dg} = \begin{cases} 16 \text{ mm} + D_{lower} \leq 40 \text{ mm} & \text{for } f_{ck} \leq 60 \text{ MPa} \\ 16 \text{ mm} + D_{lower} \left(\frac{60}{f_{ck}} \right)^2 \leq 40 \text{ mm} & \text{for } f_{ck} > 60 \text{ MPa} \end{cases} \quad (3.61)$$

Where D_{lower} should be larger than 8 mm. If D_{lower} is unknown, it can be replaced with D_{max} , which is the sieve size of the coarsest aggregate fraction[11].

This is not included in the current Eurocode, but plays a role in the shear strength of concrete [35]. As mentioned in Section 3.4, the aggregate type also influences the modulus of elasticity of concrete.

4

BENDING AND CONFINEMENT

The procedure for determining the bending moment capacity is similar for the current and new Eurocodes, but there are some differences. The model for confinement has also been revised. The main changes are summarized in Table 4.1 and are further discussed in this chapter.

Table 4.1: Overview of changes for bending and confinement

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Sectional response	6.1, 3.1.7	8.1.1, 8.1.2	Simplified compressive zone Independence of strength class.
Bending in slabs	1992-2, Ann. LL (informative)	8.1.3, Ann. G.4 (normative)	Models for slabs and membranes in normative Annex G.
Confined concrete	3.1.9	8.1.4	More favorable effect. Reworked formulas for confinement stresses. Aggregate size effect included.
Confinement reinforcement	9.3.1, 9.5.3	12.3.2(6), 12.6(3)	Reinforcement detailing. Confinement in compressive reinforcement of slabs and columns.

4.1 Bending - sectional response

The method of deriving the compression block factors to calculate the design moment resistance is similar between the generations of Eurocode. However, some changes have been made, which are reviewed in this section.

4.1.1 Gen 1

The sectional response is obtained in SS-EN 1992-1-1:2005 clauses 6.1 and 3.1.7, the compressive zone can be simplified according to Figure 4.1 below, where $\eta = 1$, $\lambda = 0.8$, and the ultimate strain in concrete is $\varepsilon_{cu} = 0.0035$. The value of ε_{c2} , which is the strain at reaching maximum strength, depends on the specific concrete class (Table 4.2).

Table 4.2: Parameters for determining the compression block of the concrete according to SS-EN 1992-1-1:2005 [13].

Class	12/15-50/60	55/67	60/75	70/85	80/95	90/105
ε_{c2} [‰]	2.0	2.2	2.3	2.4	2.5	2.6
n	2.0	1.75	1.6	1.45	1.4	1.4

The compression block factors can also be obtained analytically by using the parabolic compression block for a rectangular cross section, seen in Figure 4.1. The concrete stress can then be calculated depending on the current strain:

$$\sigma_c(\varepsilon_c)_{Gen1} = \begin{cases} \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] f_{cd} & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \\ f_{cd} & \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu2} \end{cases} \quad (4.1)$$

where n is defined in Table 4.2.

σ_c is then used to calculate the area of the compression block:

$$A = \int_0^{\varepsilon_{cu}} \sigma_c(\varepsilon_c) d\varepsilon_c \quad (4.2)$$

which gives the values of α and β accordingly:

$$\alpha = \frac{A}{f_{cd} \cdot \varepsilon_{cu}} \quad (4.3)$$

$$\beta = 1 - \frac{\int_0^{\varepsilon_{cu}} \sigma_c(\varepsilon_c) \varepsilon_c d\varepsilon_c}{A \cdot \varepsilon_{cu}} \quad (4.4)$$

The exact values of α and β for the parabolic compression block, for a rectangular cross section, are found in Table 4.3. As observed, the compressive block is reduced for high-strength concretes.

Table 4.3: Compression block factors according to SS-EN 1992-1-1:2005 [13] [1].

Class	12/15-50/60	55/67	60/75	70/85	80/95	90/105
α	0.810	0.742	0.695	0.637	0.599	0.583
β	0.416	0.392	0.377	0.362	0.355	0.353

In addition, the stress-strain relationship in Gen 1 includes both a bilinear relationship with $\varepsilon_{c2} = 0.002$ and a parabolic relationship with $\varepsilon_{c3} = 0.00175$, with different values for high-strength concretes (Table 3.1, SS-EN 1992-1-1:2005) [13].

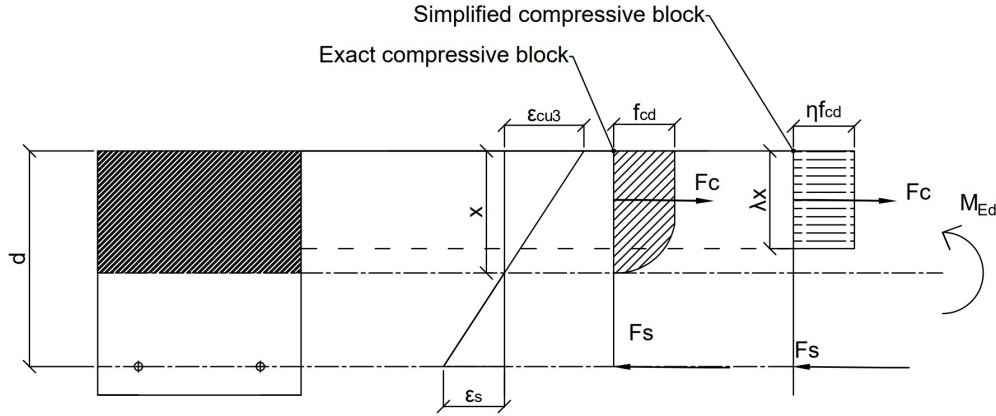


Figure 4.1: Compressive zone of concrete in EC 2 Gen 1

Through the equilibrium of the cross-section, the moment capacity can then be calculated as:

$$M_{Rd} = \alpha \cdot f_{cd} \cdot b \cdot x (d - \beta \cdot x) - N_{Ed} \left(d - \frac{h}{2} \right) \quad (4.5)$$

using the aforementioned values of η and λ .

4.1.2 Gen 2

The calculations of the stress block factors for Gen 2 are provided in SS-EN 1992-1-1:2023 clause 8.1.2, as well as Appendix A.2. For Gen 2, the same compressive block is used for both high and low-strength concretes, shown in Figure 4.1. The strain limits are always $\varepsilon_{c2} = 0.002$ and $\varepsilon_{cu} = 0.0035$, independent of strength class, the value of n is also constant and set to 2 [11].

The strain limits determine the concrete stress accordingly:

$$\sigma_c(\varepsilon_c)_{Gen2} = \begin{cases} \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}} \right)^2 \right] f_{cd} & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \\ f_{cd} & \text{for } \varepsilon_{c2} \leq \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (4.6)$$

Using the strain limits for a parabolic-rectangular stress distribution in Figure 4.1, and integrating the area of the compression block according to Eq. 4.2, results in the same expressions for α and β as in Gen 1 (Eq. 4.3 and 4.4). Since the value of ε_{c2} and n are static for Gen 2, the values of α and β become 0.81 and 0.416 respectively. However, for the simplified rectangular compressive block, they are $\lambda = 0.8$ and $\eta = 1$. The expression to calculate the moment capacity is the same as in Gen 1 (Eq. 4.5).

4.2 Bending of slabs

Models for planar structures, such as slabs, shells, and membranes, have been moved to the new normative Annex G. This is currently included in the informative Annex LL in SS-EN 1992-2, but only shell elements are included here, and rules for membrane elements are found in clause 6.109 [14]. Today, slabs in bridges and other structures are usually modeled using guidelines from Recommendations for FEA [10]. These models are based on old Swedish bridge standards.

The new Annex G in SS EN 1992-1-1:2023 provides models for determining in-plane stresses in slabs by adopting a sandwich model, and provides required moment capacities for reinforcement in Tables G.2 and G.3, and simplified design conditions are found in clause 8.1.3 [11]. It should be noted that the change from an informative annex to a normative one could influence the design of planar structures. This will not be studied in detail in this thesis, but it is a recommended subject of further study in a future thesis.

4.3 Confinement

During triaxial compression, the confinement of concrete has a positive effect on the ultimate strength and critical strain of concrete [13]. This effect is included in both versions of the Eurocode, but the models for determining the effect of confinement have been revised.

A reason for the revision is that the resistance of the current Eurocode is considered to be overly conservative for intermediate transverse stresses ($0.1f_{ck} \leq \sigma_2 \leq 0.6f_{ck}$). Therefore, SS-EN 1992-1-1:2005 uses different limits for the effective lateral stress, together with simplified formulas in which the positive contribution is added to the design strength of concrete [44].

Gen 2 also includes the effect of confinement on concretes and mortars with lower aggregate sizes, where the positive effect of confinement is reduced for smaller aggregates [11]. Gen 1 does not include this effect, and is thus considered inaccurate when using smaller aggregate sizes, such as mortars [44].

Additionally, the current Eurocode only briefly covers some detailing rules regarding confinement reinforcement, but no design rules and formulas are presented [44]. Gen 2 includes these effects by providing formulas for calculating the lateral effective stress, σ_{c2d} , for different types of sections and reinforcement arrangements, including examples of detailing, see Figure 4.2.

Gen 2 also requires that longitudinal reinforcement, which is included in capacity calculations, be confined by transverse reinforcement for beams, slabs, and columns

[11] (clause 12.3.3(6)). The effect of including confinement reinforcement can be significant, especially when using multiple reinforcement bars. As a consequence of the mentioned changes, the reduction in design strength for countries currently using $\alpha_{cc} = 1.0$ and $\alpha_{ct} = 1.0$ may be counteracted for sections with multiple bars of confinement reinforcement or circular sections in Gen 2 [44].

4.3.1 Design in Gen 1

Confinement is included in the current Eurocode (SS-EN 1992-1-1:2005, clause 3.1.9) by enhancing the characteristic strength of concrete. This is modeled with a bi-linear relationship, where the slope of the increase in compressive strength depends on the state of lateral stresses, σ_2 .

$$f_{ck,c} = \begin{cases} f_{ck} \left(1 + 5 \cdot \frac{\sigma_2}{f_{ck}} \right) & \text{for } \sigma_2 \leq 0.05 \cdot f_{ck} \\ f_{ck} \left(1.125 + 2.5 \cdot \frac{\sigma_2}{f_{ck}} \right) & \text{for } \sigma_2 > 0.05 \cdot f_{ck} \end{cases} \quad (4.7)$$

The limits of concrete strain are also influenced by the confinement effect:

$$\varepsilon_{c2,c} = \varepsilon_{c2} \cdot \left(\frac{f_{ck,c}}{f_{ck}} \right)^2 \quad (4.8)$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \cdot \sigma_2 / f_{ck} \quad (4.9)$$

4.3.2 Design in Gen 2

Gen 2 (SS-EN 1992-1-1:2023, clause 8.1.4) models the effect of confinement as an increase in the design compressive strength. This effect applies to various types of sections, including sections subjected to both flexure and axial forces.

$$\Delta f_{cd} = \begin{cases} 4 \cdot \sigma_{c2d} & \text{for } \sigma_2 \leq 0.6 \cdot f_{cd} \\ 3.5 \cdot \sigma_{c2d}^{3/4} \cdot f_{cd}^{1/4} & \text{for } \sigma_2 > 0.6 \cdot f_{cd} \end{cases} \quad (4.10)$$

The strength modification is reduced when the aggregate size parameter d_g is less than 32 mm, where the reduction is proportional to $d_{dg}/32$.

The design stress from confinement reinforcement for a section under combined axial force and flexure is calculated as follows:

$$\sigma_{c2d} = \min \left[\frac{\Sigma A_{s,confx}}{b_{cs,x}}, \frac{\Sigma A_{s,confy}}{b_{cs,y}} \right] \cdot \frac{f_{yd}}{s} \quad (4.11)$$

Gen 2 also provides formulas for a variety of sections under different loading conditions. The average increase in compressive strength due to confinement, $f_{cd,c}$, is then calculated within the area enclosed by confinement reinforcement, see Figure 4.2.

$$f_{cd,c} = f_{cd} + k_{conf,b} \cdot k_{conf,s} \cdot \Delta f_{cd} \quad (4.12)$$

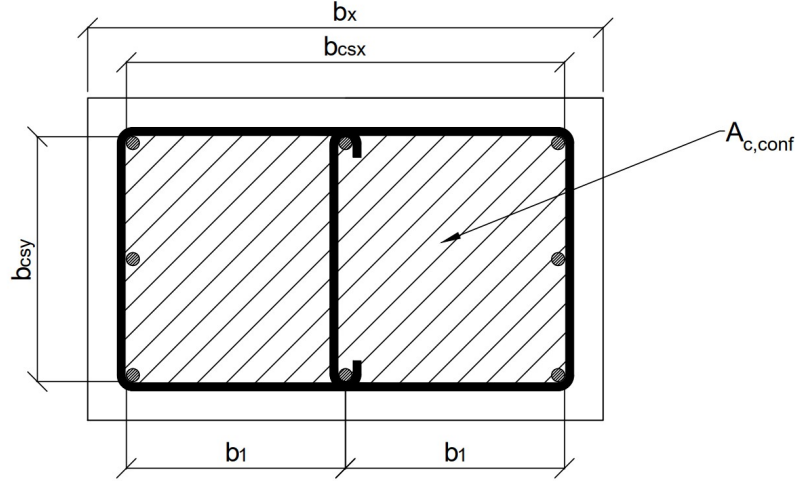


Figure 4.2: Rectangular confined section in Gen 2

The factors $k_{conf,b}$ and $k_{conf,s}$ below apply for a section under combined flexure and axial force (Table 8.1 case d) in SS-EN 1992-1-1:2023).

$$k_{conf,b} = \frac{A_{c,conf} - \frac{1}{6} \cdot \Sigma b_1^2}{A_{cc}} \quad (4.13)$$

where A_{cc} is the area of the part of the section under compression.

$$k_{conf,s} = \left[1 - \frac{s}{4 \cdot x_{cs}} \right] \cdot \left[1 - \frac{s}{2 \cdot b_{csy}} \right] \quad (4.14)$$

where x_{cs} is the distance to the neutral axis, which should be less than $b/2$.

The increase in the strain limits of concrete can be taken into account.

$$\varepsilon_{c2,c} = \varepsilon_{c2} \cdot \left(1 + 5 \cdot \frac{\Delta f_{cd}}{f_{cd}} \right) \quad (4.15)$$

$$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.2 \cdot \frac{\sigma_{c2d}}{f_{cd}} \quad (4.16)$$

5

SHEAR

Chapter 8.2 in the Gen 2 of Eurocode handling shear has undergone an almost complete revision compared to the previous version, see Table 5.1. The alterations will be explained and further documented in their separate sections below.

Table 5.1: Overview of changes for shear force

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Design shear force	(Rec. for finite element analysis, 2012)	8.2.1(6), 8.2.2(3)	Critical section for planar members.
Capacity calculations without shear reinforcement	6.2.2	8.2.1, 8.2.2	d_{dg} -aggregate factor. Definition of effective depth. New model for capacity. Removal of long term prestress from f_{yd} .
Capacity calculations with shear reinforcement	6.2.3	8.2.3	Limit of $\cot \theta$. Re-defined ν -factor. Additional scenarios. New model for shear near supports.
Flange shear	6.2.4, 6.2.106	8.2.4, 8.2.5	Flange shear calculations. Flange compression field.
Shear at interface	6.2.5	8.2.6	τ_{Rdi} , sufficient/insufficient anchorage. Surface roughness coefficients.

5.1 Design shear force

The design shear force, V_{Ed} , is the shear force acting on the control section at a certain distance from the support. The procedure for finding the design shear force in Gen 1 and Gen 2 of Eurocode 2 is explained in this section.

5.1.1 Design in Gen 1

The following section explains how to find the design shear force by following SS-EN 1992-1-1:2005.

Beams

In the current version of Eurocode, the distance to the control section is determined as

$$a_{crit} = z \cdot \cot(\theta) \quad (5.1)$$

which is the distance from the support edge, see Figure 5.1. The lever arm, z , is approximately defined as $0.9 \cdot d$, where d is the distance from the top of the section to the centerline of reinforcement [1].

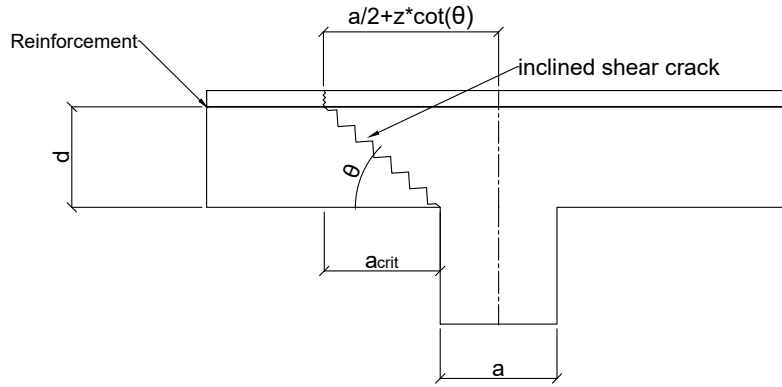


Figure 5.1: Location of control section for slabs and beams

The shear crack angle θ is 45 degrees for sections without shear reinforcement. For reinforced members, it is within the following limits

$$1 \leq \cot \theta \leq 2.5 \quad (5.2)$$

For a simply supported beam with an evenly distributed load, the design shear force can be determined from Equation 5.3 [26] (eq 5.26).

$$V_{Ed} = q \cdot \left(\frac{l}{2} - \left(a_{crit} + \frac{a}{2} \right) \right) \quad (5.3)$$

Here, a is the length of the support and a_{crit} is the distance to the critical section.

Planar members

The current Eurocode does not offer recommendations for control sections and distribution widths in slabs, shells, and membranes. Instead, "Recommendations for finite element analysis" [10] is often used for design, and relevant models are summarized below. Using these guidelines, the control section is defined using Equation 5.1, which is visualized in Figure 5.1.

The resulting shear force acting on any section is defined as:

$$v_0 = \sqrt{v_x^2 + v_y^2} \quad (5.4)$$

where v_x and v_y are the shear forces perpendicular to the x and y directions. The shear force is distributed across the width w , orthogonal to the direction of the resulting shear force acting on the control section. The design shear force is then taken as the average shear force acting on the width w . This distribution width is the same as that used for the redistribution of moments, but with some additional limitations.

The following limits apply for planar members between supports:

$$w = \begin{cases} \min\left(3h, \frac{l_c}{10}\right) & \text{for } \frac{x}{d} = 0.45 \\ \min\left(5h, \frac{l_c}{5}\right) & \text{for } \frac{x}{d} = 0.3 \\ \frac{l_c}{4} & \text{for } \frac{x}{d} = 0.25 \\ \frac{l_c}{2} & \text{for } \frac{x}{d} = 0.15 \\ \min\left(5h, \frac{l_c}{5}\right) & \text{for } \frac{x}{d} = 0 \end{cases} \quad (5.5)$$

where l_c is the span length, h is the section's height, and x is the height of the compressive zone. A conservative minimum value for w independent of ductility is

$$w_{min} = 2h + a \quad (5.6)$$

where a is the width of the support.

For cantilevers, the value of w can instead be taken between the following maximum

and minimum values:

$$w_{max} = \max \begin{cases} 7d + b + t \\ 10d + 1.3y_{cs} \end{cases} \quad \text{for } y = 0 \quad (5.7)$$

and

$$w_{min} = \min \begin{cases} 7d + b + t \\ 10d + 1.3y_{cs} \end{cases} \quad \text{for } y = y_{max} \quad (5.8)$$

where y_{cs} is the distance to an applied concentrated force from the support.

Loads near supports

If a load is applied within the following distance to a support, the design shear force can be reduced

$$0.5 \cdot d \leq a_v \leq 2 \cdot d \quad (5.9)$$

where a_v is defined in Figure 5.2

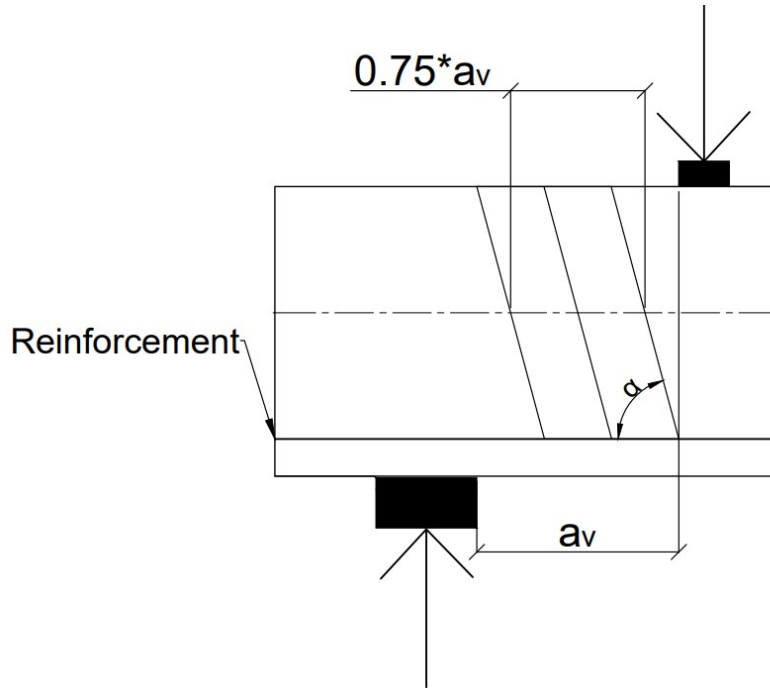


Figure 5.2: Illustration of load near supports

$$V_{Ed,red} = \frac{a_v}{2d} \cdot V_{Ed} \quad (5.10)$$

This only applies if the reduced shear force satisfies the condition

$$V_{Ed,red} < A_{sw} \cdot f_{ywd} \cdot \sin \alpha \quad (5.11)$$

where the angle α is defined in Figure 5.2.

5.1.2 Design in Gen 2

The procedure for finding the design shear force by following SS-EN 1992-1-1:2023, clause 8.2.1 and 8.2.2 is explained below.

Beams

The same control section can be used for determining the design shear force as in Gen 1, Equation 5.1 for linear members. To determine the design shear force for a simply supported beam, Equation 5.3 can be used, which is the same as in Gen 1. However, the effective depth can be modified following the equation 3.1, following Annex 3 in [11].

Planar members

The design shear force per unit width for slabs, shells, and membranes can be calculated according to

$$v_{Ed} = \sqrt{v_{Ed,x}^2 + v_{Ed,y}^2} \quad (5.12)$$

where $v_{Ed,x}$ and $v_{Ed,y}$ are the out-of-plane shear forces per unit width that act perpendicularly to the x and y directions.

The control section for planar members is defined similarly to that of beams according to Equation 5.1. However, if the shear force is not constant along the control section, the design shear force may be averaged along a width of $w = 4 \cdot d$ around the peak in shear force, with d according to Equation 5.13 [11]. It should be noted that this is a significant difference compared to the case for cantilevers in Gen 1, where the shear force is distributed over a width greater than $7h$ [10].

The effective depth depends on the ratio of shear forces in the x and y directions

$$d = \begin{cases} d_x & \text{for } \frac{v_{Ed,y}}{v_{Ed,x}} \leq 0.5 \\ 0.5(d_x + d_y) & \text{for } 0.5 < \frac{v_{Ed,y}}{v_{Ed,x}} < 2 \\ d_y & \text{for } \frac{v_{Ed,y}}{v_{Ed,x}} \geq 2 \end{cases} \quad (5.13)$$

Here, d_x and d_y are effective depths in the x and y directions. Design values for effective depth can also be used according to Equation 3.1, which allows for more favorable partial factors to be used.

Loads near supports

In Gen 2, the design load is not modified close to supports as it is in Gen 1. Instead, the resistances are modified within a certain distance from the support which is further explained in the following sections.

5.2 Capacity without shear reinforcement

A new model for shear capacity without reinforcement is used, which is based on a mechanical model, as opposed to the empirical model used in the current Eurocode [35]. The changes mentioned in Table 5.1 have been made due to underestimation of the size effect, conservative approaches regarding tensile stresses, neglected aggregate interlock capacity, and the effect of shear slenderness, and shear resistance equations being derived in a manner that applying them to scenarios outside of their original range could turn dangerous [35]. Alterations of this subchapter work to remove these wrong calibrations by introducing a new factor accounting for the impact of concrete and aggregate properties, d_{dg} , changing the definitions of effective depth, and making the shear capacity not only influenced by the shear acting on the section, but also by the bending moment [35].

5.2.1 Design in Gen 1

Regions of a member where $V_{Ed} \leq V_{Rd,c}$, only minimum required reinforcement is needed, i.e., no shear reinforcement is necessary according to SS-EN 1992-1-1:2005 clause 6.2.2.

The design resistance for shear sliding failure for members reinforced in flexure, but not requiring design shear reinforcement, is calculated as

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k (100 \cdot \rho_l \cdot f_{ck})^{1/3} + k_1 \cdot \sigma_{cp} \right] b_w \cdot d \quad (5.14)$$

with a minimum of

$$V_{Rd,c} = (v_{\min} + k_1 \cdot \sigma_{cp}) b_w \cdot d \quad (5.15)$$

where

$$\rho_l = \frac{A_{sl}}{b_w \cdot d} \leq 0.02;$$

A_{sl} is the area of the tensile reinforcement;

b_w is the smallest width of the cross-section in the tensile area [mm];

σ_{cp} $N_{Ed}/A_c < 0.2f_{cd}$ [MPa];

N_{Ed} is the axial force in the cross section due to loading or pre-stressing.

Influence of imposed deformations may be ignored;

A_c Area of the cross section.

The values of $C_{Rd,c}$, v_{min} , and k_1 are NPDs which can be modified via the National Annex. Their recommended values for Sweden are [15]:

$$\begin{aligned} C_{Rd,c} &= \frac{0.18}{\gamma_c} \\ v_{min} &= 0.035 \cdot k^{3/2} \cdot f_{ck}^{(1/2)} \\ k_1 &= 0.15 \end{aligned}$$

where

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2 \quad \text{using } d \text{ in mm} \quad (5.16)$$

Resistance to shear compression failure should also be verified according to

$$V_{Ed} \leq 0.5 \cdot b_w \cdot d \cdot \nu \cdot f_{cd} \quad (5.17)$$

With the strength reduction factor defined as follows

$$\nu = 0.6 \left(1 - \frac{f_{ck}}{250} \right) \quad \text{for } f_{ck} \text{ in MPa} \quad (5.18)$$

5.2.2 Design in Gen 2

Detailed verification of shear resistance concerning shear sliding failure in reinforced concrete members can be neglected provided that $\tau_{Ed} \leq \tau_{Rd,c,min}$ according to clause 8.2.1 in SS-EN 1992-1-1:2023. The minimum shear resistance is calculated as

$$\tau_{Rd,c,min} = \frac{11}{\gamma_v} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d}} \quad (5.19)$$

An effective value for d can be used, which allows the partial factors for shear and steel γ_v and γ_s to be reduced following the procedure explained in Chapter 3.

If detailed verification is needed, then the design for shear reinforcement can be neglected if $\tau_{Ed} \leq \tau_{Rd,c}$. If a region is without geometric discontinuities, the transformation of the design shear force to the average design shear stress over the cross-section can be calculated as

$$\tau_{Ed} = \frac{V_{Ed}}{b_w \cdot z} \quad (5.20)$$

For a region without shear reinforcement, the design shear stress resistance should be calculated according to

$$\tau_{Rd,c} = \frac{0.66}{\gamma_v} \cdot \left(100\rho_1 \cdot f_{ck} \cdot \frac{d_{dg}}{d} \right)^{1/3} \geq \tau_{Rd,c,min} \quad (5.21)$$

for

$$\rho_l = \frac{A_{sl}}{b_w \cdot d} \quad (5.22)$$

where

- A_{sl} is the effective area of tensile reinforcement at the distance d ;
- b_w is the width of the cross-section of linear members. For cross sections with variable width or for circular cross sections, clause 8.2.3(9) should be followed;
- d is the nominal value of the effective depth, d_{nom} .

If the effective shear span with respect to the moment and shear in the control section, a_{cs} calculated from Expression 5.24, is shorter than $4d$, the value of d in Eq. 5.21 can be replaced by the mechanical shear span, a_v , given by

$$a_v = \sqrt{\frac{a_{cs}}{4} \cdot d} \quad (5.23)$$

$$a_{cs} = \left| \frac{M_{Ed}}{V_{Ed}} \right| \geq d \quad (5.24)$$

If the control section is affected by axial forces (N_{Ed}), the value d or a_v (whichever chosen) should be multiplied by k_{vp} accordingly

$$k_{vp} = 1 + \frac{N_{Ed} \cdot d}{|V_{Ed}| \cdot 3 \cdot a_{cs}} \geq 0.1 \quad (5.25)$$

$$d_{N_{Ed}} = \begin{cases} d \cdot k_{vp} & \text{or} \\ a_v \cdot k_{vp} \end{cases} \quad (5.26)$$

Thus, the expression for the concrete shear resistance affected by axial forces can be written as:

$$\tau_{Rd,c} = \frac{0.66}{\gamma_v} \cdot \left(100\rho_1 \cdot f_{ck} \cdot \frac{d_{dg}}{d_{N_{Ed}}} \right)^{1/3} \geq \tau_{Rd,c,min} \quad (5.27)$$

Gen 2 also presents an alternative expression for the concrete shear resistance when the section is subjected to axial forces, where the following condition should be satisfied:

$$\tau_{Rdc,min} \leq \tau_{Rdc,c} = \tau_{Rdc,0} - k_1 \cdot \sigma_{cp} \leq \tau_{Rdc,max} \quad (5.28)$$

where

$$\tau_{Rd,c,0} = \frac{0.66}{\gamma_v} \cdot \left(100\rho_1 \cdot f_{ck} \cdot \frac{d_{dg}}{d} \right)^{1/3} \quad (5.29)$$

where the value of d can be replaced by a_v from Equation 5.23 without considering the effect of prestressing or axial forces (should not be multiplied by k_{vp}).

The stress in the concrete resulting from the axial force is defined as:

$$\sigma_{cp} = \frac{N_{Ed}}{A_c} \quad (5.30)$$

N_{Ed} is a compressive axial force acting on the section with an eccentricity e_p to the center of gravity (positive towards the side in tension). This applies to both prestressing forces and other compressive forces.

$$k_1 = \frac{0,5}{a_{cs,0}} \left(e_p + \frac{d}{3} \right) \cdot \frac{A_c}{b_w \cdot z} \leq 0,18 \cdot \frac{A_c}{b_w \cdot z} \quad (5.31)$$

Here, $a_{cs,0}$ is taken from Equation 5.24 excluding the additional moment and shear caused by the axial force N_{Ed} .

The shear capacity under axial forces should be limited by

$$\tau_{Rdc,max} = 2.15 \cdot \tau_{Rdc,0} \cdot \left(\frac{a_{cs,0}}{d} \right)^{0.6} \leq 2.7 \cdot \tau_{Rdc,0} \quad (5.32)$$

When concentrated loads are applied within a distance of a_q from the support, the strut and tie model should be used to verify the shear capacity of the concrete. This verification can be neglected for planar members on line supports without shear reinforcement, given that the distance from load to support, $a_q < d$ and $\tau_{Ed} < 2\tau_{Rdc,min}$.

5.3 Capacity with shear reinforcement

The subchapter managing capacity calculations with shear reinforcement (8.2.3) differs from Gen 1 by the update of values for certain parameters. The variable-angle truss model from SS-EN 1992-1-1:2005 has predominantly been kept. The minimum allowable angle of the compression field has been reduced for members affected by significant axial forces, but increased for cases using ductility class A reinforcement. The efficiency factor for the effective compressive strength of concrete has been modified to consider the strain state more accurately. A design model considering load application near supports has also been made, and replaces the previous empirical formulation. Moreover, Gen 2 contains additional cases for cross sections and combined actions for axial forces and shear. [38]

5.3.1 Design in Gen 1

The following section summarizes the design procedure in SS-EN 1992-1-1:2005 clause 6.2.3. In regions where $V_{Ed} \geq V_{Rd,c}$, shear reinforcement should be added to fulfill the following criteria: $V_{Ed} \leq V_{Rd,s}$. The shear resistance model is based on a truss system shown in Figure 5.3 below.

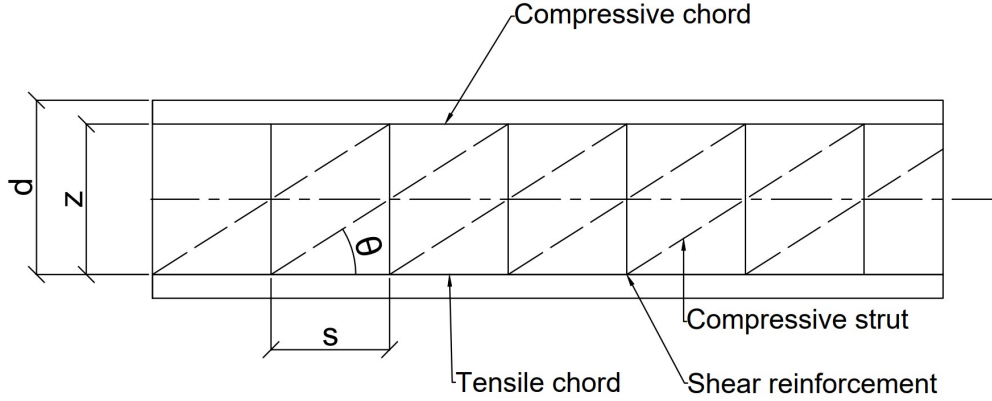


Figure 5.3: Truss model for shear capacity with stirrups

where $z = 0.9d$, and θ is the angle of the compressive chord defined in Expression 5.2. Based on this model, the shear resistance with stirrups is defined as

$$V_{Rd,s} = \frac{A_{sw}}{s} z \cdot f_{ywd} \cdot \cot \theta \quad (5.33)$$

limited by the maximum resistance corresponding to the crushing of concrete:

$$V_{Rd,max} = \alpha_{cw} \cdot b_w \cdot z \cdot \nu_1 \frac{f_{cd}}{(\cot \theta + \tan \theta)} \quad (5.34)$$

where α_{cw} is an NDP, where the recommended value is 1 (without prestressing), and ν_1 is the efficiency factor dependent on the concrete strength:

$$\nu_1 = \begin{cases} 0.6 & \text{for } f_{ck} \leq 60 \text{ MPa} \\ 0.6 - \frac{f_{ck}}{100} > 0.5 & \text{for } f_{ck} > 60 \text{ MPa} \end{cases} \quad (5.35)$$

5.3.2 Design in Gen 2

The method for determining shear force resistance with stirrups in SS-EN 1992-1-1:2023 clause 8.2.3 is summarized below.

If the criteria $\tau_{Ed} \leq \tau_{Rd,c}$ can not be fulfilled, then shear reinforcement should be designed to satisfy $\tau_{Ed} \leq \tau_{Rd,sy}$. Similar to Gen 1, the model for shear resistance

with reinforcement is based on the same truss model, see Figure 5.3.

The shear stress resistance of reinforced members is defined as

$$\tau_{Rd,sy} = \rho_v \cdot f_{ywd} \cdot \cot \theta \quad (5.36)$$

With the reinforcement ratio

$$\rho_v = \frac{A_{sw}}{b_w \cdot s} \quad (5.37)$$

and where the angle of the compression field is limited according to

$$1 \leq \theta \leq \theta_{min} \quad (5.38)$$

The upper limit $\cot \theta_{min}$ is 2.5 for reinforced members with no axial force. The value of $\cot \theta_{min}$ can be increased for sections with high axial compressive stress, and reduced for axial tension. The limit is further reduced by 20% when using ductility class A reinforcement. An additional check for the concrete stress in the compressive field is required.

$$\sigma_{cd} = \tau_{ed} \cdot (\cot \theta + \tan \theta) < \nu \cdot f_{cd} \quad (5.39)$$

where the efficiency factor ν is 0.5, unless the inclination of the compression field is higher than the limits in Eq. 5.38. In this case, a refined calculation of the efficiency factor is presented, see (Eq. 8.45, Gen 2) in [11].

Gen 2 also provides a formula for optimization of the compression field angle to achieve simultaneous yielding of the stirrups and crushing of the concrete:

$$\cot \theta_{min} \geq \cot \theta = \sqrt{\frac{\nu \cdot f_{cd}}{\rho_w \cdot f_{ywd}}} - 1 \geq 1 \quad (5.40)$$

While satisfying the following condition:

$$\tau_{Rd,sy} = \rho_v \cdot f_{ywd} \cdot \cot \theta \leq \frac{\nu \cdot f_{cd}}{2} \quad (5.41)$$

Loads near supports

An increase in shear resistance can be considered for loads applied near supports within a distance of

$$a_v = z \cdot \cot \beta_{incl} < z \cdot \cot \theta \quad (5.42)$$

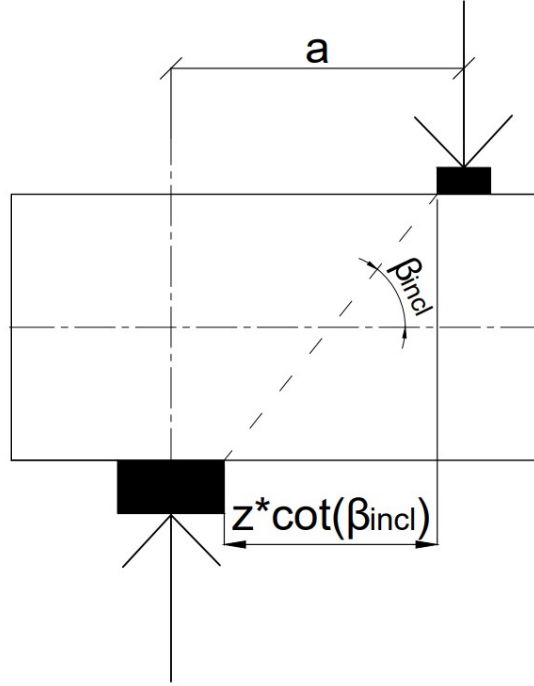


Figure 5.4: Illustration of loads near supports

The shear stress resistance within this distance can be increased according to

$$\tau_{Rd} = \nu \cdot f_{cd} \frac{\cot \theta - \cot \beta_{incl}}{1 + \cot^2 \theta} + \rho_w \cdot f_{ywd} \cdot \cot \beta_{incl} \leq \nu \cdot f_{cd} \frac{\cot \theta}{1 + \cot^2 \theta} \quad (5.43)$$

5.4 Shear between web and flanges

For shear in between web and flanges, the upper limits of $\cot \theta_f$ for compression flanges have been increased (clause 8.2.4 in SS-EN 1992-1-1:2023). The formulation of the shear calculation is also reworked for easier application (clause 8.2.5 in SS-EN 1992-1-1:2023) [11].

5.5 Shear at interface

Shear at the interface has a broadened scope and additional guidelines that clarify when verifications are needed. Calculation of shear stress resistance has been complemented with a supporting equation in the case of insufficient anchorage, leading to yielding of the interface reinforcement since the previous resistance calculation was not applicable for this scenario (Eqs. 8.76 and 8.77 in SS-EN 1992-1-1:2023).

The new Equation (Eq. 8.77) for insufficient anchorage can result in higher degrees of interface reinforcement, but can - in many cases - be compensated, resulting in a reduced anchorage length and shorter reinforcing bars [22].

The shear stress resistance equations include surface roughness coefficients (c_{v1} , c_{v2} , μ_v , k_v , k_{dowel}) [11] (Table 8.2), these values have been changed compared to Gen 1, and summarized into a table and an explanatory figure. Detailing guidelines for composite slabs requiring interface reinforcement have been updated with additional spacing rules, as well as a directory for minimum required interface reinforcement when delamination along edges by permanent loads can not be prevented [22].

6

CRACK CONTROL

Several modifications to coefficients, formulas, and models have been made in Gen 2, summarized in Table 6.1. Since the changes are significant, the design procedure will be compared in detail in this chapter.

Table 6.1: Overview of changes for stress limitations and crack control

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Crack width limits	7.3.1(5)	9.2.1(6)	Surface cover. New limit values. Added exposure class, XF.
Minimum reinforcement	7.3.2	9.2.2(2)	New coefficients. Different equations for axial/bending and combined action.
Additional longitudinal reinf. for crack control	-	9.2.2(4)	Requirement for additional longitudinal reinforcement for beams.
Crack width	7.3.4	9.2.3	New coefficients. New limit value.
Effective area in tension	7.3.2(3)	9.2.3(3)	New definition of effective area around reinforcement. Different formula for multiple layers of bars.
Mean crack distance	7.4.3	9.2.3(6)	Mean crack distance instead of max distance, uses new coefficients and new limit. Effect of casting position.
Restraint cracking	SS-EN 1992-3:2006	9.2.3(4), Ann. D, Ann. H	Wider and more general application.

The limits for allowed crack widths are presented in a table in the current Eurocode, where the maximum allowed crack width, w_{max} , depends on exposure classes, Table 6.2 (Table 7.1N in SS-EN 1992-1-1:2005) [13]. This layout is kept in Gen 2 (SS-EN 1992-1-1:2023), but new exposure classes are included (XF1-4) corresponding to freeze-thaw attack. The given crack width limits are also multiplied by a new factor, k_{surf} , that depends on the concrete cover (Table 9.2, Gen 2) [11].

The current Eurocode provides one formula for calculating the minimum reinforcement area for crack control (Eq 7.1) [13]. Gen 2 instead provides three different formulas for different loading cases: bending, tension, and combined bending and axial stress. Additional longitudinal reinforcement is also required for beams with a down-stand depth of over 600 mm (clause 9.2.2) [11].

Gen 2 introduces the clause 9.2.3, "Refined control of cracking", where an updated model for crack width calculation is presented. The basis of these changes is inconsistencies in the modeled cracking behavior in the current EC, where the cracks do not match reality well. This new model includes important factors that are neglected in Gen 1, such as the effect of casting position and the effect of curvature [4]. The effect of the casting position of the bars under tension is currently only considered for anchorage lengths, but the effect is neglected for crack widths. This is now included in Gen 2 through the coefficient k_b , which allows for a reduction of the crack spacing and crack widths for bars located at the bottom of a section (with good conditions) [4]. The effect of curvature is included through the introduction of a new factor, $k_{1/r}$, that takes into account the curvature of the most tensioned fiber. There is also a new limit for the difference between steel and concrete strain in Gen 2 (Eq 9.11), where the limit now depends on loading duration (clause 9.2.3) [11].

Additionally introduced in clause 9.2.3 are expressions to calculate the strain of members subjected to imposed strains through restraints. Calculation of strains, stresses, and crack widths due to restraints was previously included in Eurocode 2 in document SS-EN 1992-3:2006, considered to mainly be applicable for liquid retaining and containment structures of concrete. Examples are primary loaded walls of a containment structure, reservoirs or silos, or in general structures that contain or carry liquids. Since the current version was published around 2005, the concept of restraint has become more widely known, and applications now include other structures than containment structures [14] [13]. Therefore, the expressions that handle restraints have been moved to the main document of Eurocode 2 (SS-EN 1992-1-1-2023).

Connected to restraint cracking, Gen 2 introduces Annex D (Evaluation of early-age and long-term cracking due to restraint) and H (Guidance on design of concrete structures for water-tightness), serving to replace SS-EN 1992-3:2006 along with clause 9.2.3(4). Annex D's objective is to guide the restriction of early age cracking by considering the effects of temperature changes from the concrete's hardening process and basic shrinkage combined with imposed deformations at later ages [28]. Annex H mostly acts to replace the parts of SS-EN 1992-3:2006 relating to water-tightness of concrete structures and enhance ease of use and consistency to the new Eurocode's formulation of cracking [2].

The definition of effective area has also been reworked to avoid unrealistic behavior in the current Eurocode. For example, the effective areas around bars with long spacing are now limited [4].

The outcomes of all the above-mentioned changes are improved ease of use and improved reliability, since the new model better matches test results [4].

6.1 Crack width limits

Both versions of the Eurocode include limiting values for the maximum crack width. These values are part of the National Annexes and are presented below.

6.1.1 Design in Gen 1

The design procedure for finding the maximum allowed crackwidth in Gen 1, SS-EN 1992-1-1:2005, is summarized below. Clause 7.3.1 limits the crack widths depending on the exposure class, bond conditions, and loading situations. These limits are defined in a National Annex for each country, and the EC-recommended limits are shown in Table 6.2 below.

Table 6.2: Recommended crack width limits in [mm], (SS-EN 1992-1-1:2005, Table 7.1N) [13]

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0.4	0.2
XC2, XC3, XC4	0.3	0.2
XD1, XD2, XS1, XS2, XS3	0.3	Decompression

However, the Swedish National Annex used different values than those recommended by the Eurocode. Following the Swedish National Annex, the maximum crack widths for 100 years are summarized in Table 6.3 below:

Table 6.3: Allowed crack width limits for durability for 100 years based on TSFS 2018:57 [15]

Exposure class	w_{max} [mm]
XC2	0.4
XC3, XC4	0.3
XS1, XS2	0.2
XS3	0.15
XD1, XD2	0.2
XD3	0.15

6.1.2 Design in Gen 2

Clause 9.2.1 in SS-EN 1992-1-1:2023 provides different limitations of crack widths related to appearance and durability. These limits range from 0.4 mm to 0.2 mm, and can be found in Tables 9.1 and 9.2 in SS-EN 1992-1-1:2023. A factor k_{surf} is used to modify the allowable crack width depending on concrete cover:

$$k_{surf} = \frac{c_{act}}{10 + c_{min,dur}} \leq 1.5 \quad (6.1)$$

where c_{act} is defined as the real cover thickness, which can be greater than c_{nom} due to detailing or execution.

The Swedish Transport Agency has proposed to use the same crack width limits as in the National Annex for Gen 1, table 6.3 above. However, these maximum crack widths can be augmented by the factor k_{surf} to increase the limits for large concrete covers [18].

6.2 Minimum reinforcement area

A minimum amount of reinforcement is required for crack control in parts of sections under tension.

6.2.1 Design in Gen 1

The model for minimum reinforcement area for crack control according to SS-EN 1992-1-1:2005 clause 7.3.2 is summarized below, and can be designed and calculated as:

$$A_{s,min,\sigma_s} = k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} \quad (6.2)$$

where

$$k = \begin{cases} 1 & \text{for } h \leq 300 \text{ mm} \\ 0.65 & \text{for } h \geq 800 \text{ mm} \end{cases} \quad (6.3)$$

and

$$k_c = \begin{cases} 1 & \text{for pure tension} \\ 0.4 \cdot \left[1 - \frac{\sigma_c}{k_1 \left(\frac{h}{h^*} \right) f_{ct,eff}} \right] \leq 1 & \text{for combined action} \end{cases} \quad (6.4)$$

and σ_c is the mean value of concrete stress, and it should be noted that N_{Ed} is defined as positive for compression.

$$\sigma_c = \frac{N_{Ed}}{b \cdot h} \quad (6.5)$$

h^* is the same as the height of the section, but is no larger than 1 m. The coefficient k_1 includes the effect of axial forces

$$k_1 = \begin{cases} 1.5 & \text{if } N_{Ed} \text{ is a compressive force} \\ \frac{2h^*}{3h} & \text{if } N_{Ed} \text{ is a tensile force} \end{cases} \quad (6.6)$$

A_{ct} is the area of concrete within the tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before the formation of the first crack.

6.2.2 Design in Gen 2

Clause 9.2.2 of SS-EN 1992-1-1:2023 states that minimum reinforcement should be provided with an adequate area in which yielding is avoided while balancing the moment and axial force that causes cracking. Gen 2 presents a simplified model for calculating the minimum reinforcement where three different cases are identified:

1) for pure bending

$$A_{s,min.w1} \geq \frac{0.2 k_h f_{ct,eff} A_c}{f_{yk}} \quad (6.7)$$

2) for pure tension

$$A_{s,min.w1} = A_{s,min.w2} \geq \frac{0.5 k_h f_{ct,eff} A_c}{f_{yk}} \quad (6.8)$$

3) for combined flexure and axial stress

$$A_{s,min.w1} = \begin{cases} \geq \frac{0.3 N_{Ed} + 0.2 k_h f_{ct,eff} A_c}{f_{yk}} \\ \geq 0 \\ \leq \frac{0.5 k_h f_{ct,eff} A_c}{f_{yk}} \end{cases} \quad (6.9)$$

and

$$A_{s,min,w2} = \frac{N_{Ed}}{f_{yk}} - A_{s,min,w1} \begin{cases} \leq A_{s,min,w1} \\ \geq 0 \end{cases} \quad (6.10)$$

where $A_{s,min,w1}$ and $A_{s,min,w2}$ are the minimum reinforcement areas in the most tensioned face, and the least tensioned of the section, respectively. (If the full section is under varying tension).

k_h is a coefficient that depends on the geometry of the section, defined as follows

$$0.5 \leq k_h = 0.8 - 0.6 (\min(b, h) - 0.3) \leq 0.8 \quad (6.11)$$

and $f_{ct,eff}$ is the mean value of the tensile strength of concrete at time t_{ref} .

Additional longitudinal reinforcement is required for beams with down-stand depth > 600 mm when the reinforcement is placed at the most tensioned fiber. This is to prevent cracking at the sides of the beam.

$$A_{s,web} = 0.2 \frac{f_{ct,eff}}{f_{yk}} b_w (d - x - a_1) \quad (6.12)$$

This reinforcement is to be placed along the web surfaces limited by the neutral axis and $(d - a_1)$ with a maximum spacing of 300 mm, where a_1 is defined to be 150 mm.

6.3 Calculation of crack widths

The procedure for calculating the crack widths in both versions of the Eurocode is summarized below.

6.3.1 Design in Gen 1

The characteristic crack width listed in clause 7.3.4 is calculated based on maximum crack spacing and the strain difference between concrete and reinforcement in the following formula.

$$w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm}) \quad (6.13)$$

The difference in strain depends on the steel stress for a cracked section and can be expressed as

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \frac{f_{ct,eff}}{\rho_{p,eff}} (1 + \alpha_e \rho_{p,eff})}{E_s} \geq 0.6 \frac{\sigma_s}{E_s} \quad (6.14)$$

where α_e is the elasticity ratio between reinforcement and concrete

$$\alpha_e = \frac{E_s}{E_c} \quad (6.15)$$

and $\rho_{p,eff}$ the reinforcement ratio defined as

$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A'_p}{A_{c,eff}} \quad (6.16)$$

The effective area $A_{c,eff}$ is visualized in figure 6.1 below.

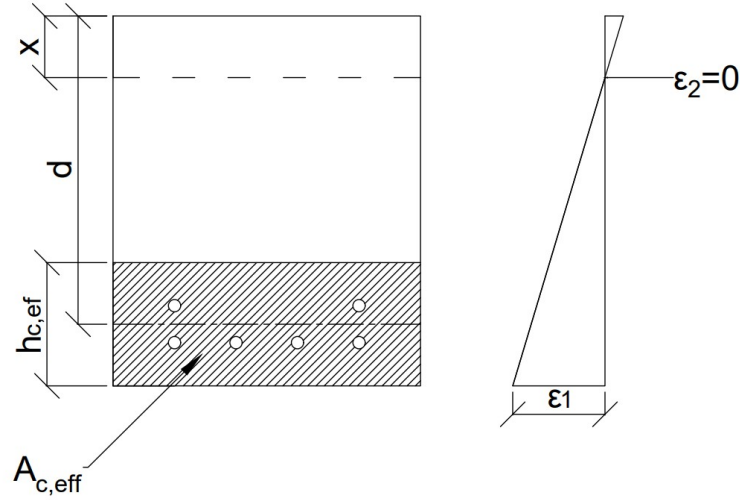


Figure 6.1: Effective area for cracked cross sections according to SS-EN 1992-1-1:2005

The coefficient k_t depends on the duration of loading

$$k_t = \begin{cases} 0.6 & \text{for short term loading} \\ 0.4 & \text{for long term loading} \end{cases} \quad (6.17)$$

ξ_1 and A'_p are included for prestressed members, see Eq 7.5 in SS-EN 1992-1-1:2005 for definition.

Given that the bonded reinforcement is fixed with a spacing $< 5(c + \phi/2)$, it is possible to calculate the maximum crack spacing as follows

$$s_{r,max} = k_3 \cdot c + \frac{k_1 \cdot k_2 \cdot k_4 \cdot \phi}{\rho_{p,eff}} \quad (6.18)$$

where k_1 depends on the bond condition

$$k_1 = \begin{cases} 0.8 & \text{for good bond} \\ 1.6 & \text{for plain bars} \end{cases} \quad (6.19)$$

and k_2 considers the strain distribution

$$k_2 = \begin{cases} 0.8 & \text{for bending} \\ 1.6 & \text{for tension} \end{cases} \quad (6.20)$$

The values of k_3 and k_4 are 3.4 and 0.424, respectively. They can, however, be modified via the National Annex. The Swedish Transport Agency (TSFS 2018:57 [15]) recommends the following expression for k_3 :

$$k_3 = \frac{7\phi}{c} \quad (6.21)$$

and keeping the Eurocode recommended value of k_4 . It should be noted that the coefficient k_3 works to remove the influence of concrete cover on crack width, which can benefit sections with thick concrete covers.

6.3.2 Design in Gen 2

The procedure for crack control in Gen 2 SS-EN 1992-1-1:2023 clause 9.2 is summarized in this section. The characteristic crack $w_{k,cal}$ width is calculated in clause 9.2.3 based on a mean crack spacing, $s_{r,m,cal}$, and also depends on the effect of curvature and the difference in strain between concrete and steel. This value should be compared to the crack width limit values given in tables, but does not match in-situ measurements.

$$w_{k,cal} = k_w \cdot k_{1/r} \cdot s_{r,m,cal} (\varepsilon_{sm} - \varepsilon_{cm}) \quad (6.22)$$

The coefficient k_w is an NDP, and the recommended value is 1.7.

$k_{1/r}$ takes into account the effect of curvature.

$$k_{1/r} = \frac{h - x}{h - a_{y,i} - x} \quad \text{for the most tensioned fiber} \quad (6.23)$$

and

$$k_{1/r} = \frac{|x|}{a_{y,i} + |x|} \quad \text{for the least tensioned fiber (both faces in tension)} \quad (6.24)$$

Here, a_y is the distance from the face of the concrete in tension to the centerline of the tensile reinforcement.

The difference in strain is calculated as follows

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \frac{f_{ct,eff}}{\rho_{p,eff}} (1 + \alpha_e \rho_{p,eff})}{E_s} \geq (1 - k_t) \frac{\sigma_s}{E_s} \quad (6.25)$$

which is the same as in SS-EN 1992-1-1:2005, but the limit value now depends on the type of loading, where $k_t = 0.6$ for short-term/instant loading, or long-term /repeated loading, but in the crack formation stage. Alternatively, $k_t = 0.4$ when cracking has stabilized after long-term continuous/repeated loading. Other parameters and coefficients are defined in Section 6.3.

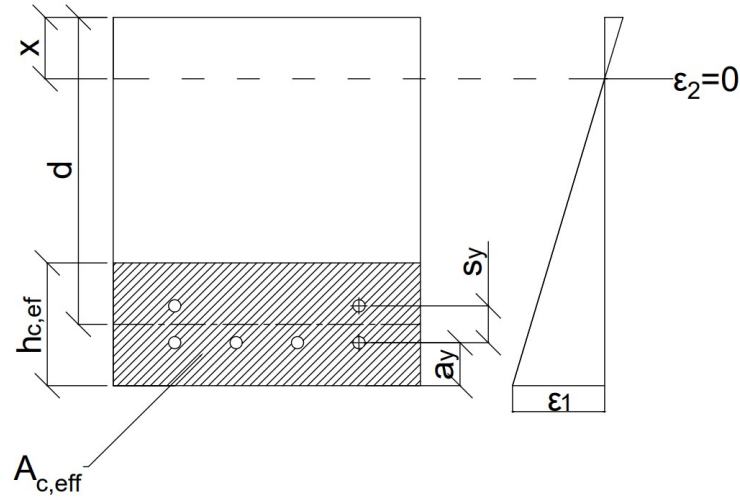


Figure 6.2: Effective area for different sections

The effective area for a section subjected to bending is shown in Figure 6.2, where the effective height can be calculated from:

$$h_{c,eff} = \begin{cases} \min(a_y + 5\phi, 10\phi, 3.5a_y, h - x, h/2) & \text{for one layer of bars} \\ \min\left(\min(a_y + 5\phi, 10\phi, 3.5a_y), (n-1)s_y, h - x, h/2\right) & \text{for } n \text{ layers of bars} \end{cases} \quad (6.26)$$

It should be noted that the effective area can only be taken into account within a width of 10ϕ around a group of bars as shown in Figure 9.3 in SS-EN 1992-1-1:2023 [11].

The mean value of the distance between cracks is calculated as follows

$$s_{r,m,cal} = 1.5c + \frac{k_{fl} k_b}{7.2} \frac{\phi}{\rho_{p,eff}} \leq \frac{1.3}{k_w} (h - x) \quad (6.27)$$

with

$$k_{fl} = \begin{cases} \frac{h - h_{c,eff}}{h} & \text{with pure bending} \\ 1 & \text{with pure tension} \\ 0.5 \left(1 + \frac{h - x_I - h_{c,eff}}{h - x_I} \right) & \text{otherwise} \end{cases} \quad (6.28)$$

where x_I is the height of the neutral axis in stage 1 (uncracked). k_b is defined as 1.2 for bad bond conditions and 0.9 for good bonding between reinforcement and concrete.

From equation 6.27 above, it can be observed that the crack spacing increases with concrete cover with a factor of 1.5. In addition, it is no longer possible to reduce the crack spacing for large concrete covers with the National Annex, as is the case in Gen 1. Therefore, this may lead to larger crack widths for thick concrete covers in Gen 2.

6.4 Restraint cracking

Directions to handle restraint cracking are included as an additional document named SS-EN 1992-3:2006 in the current Eurocode, which also gives design instructions for liquid retaining and containment structures in concrete. As mentioned in the introduction of Chapter 6, the new generation of Eurocode includes clauses handling restraint cracking in the main document (SS-EN 1992-1-1:2023), since the knowledge of restraint has become more generally known and its application wider than it previously was.

Design in Gen 1

Restraint cracking in the current Eurocode is included in SS-EN 1992-3:2006, where it divides the difference in strain ($\varepsilon_{sm} - \varepsilon_{cm}$) dependent on the restraint scenario according to Figures 6.3a and 6.3b [12].

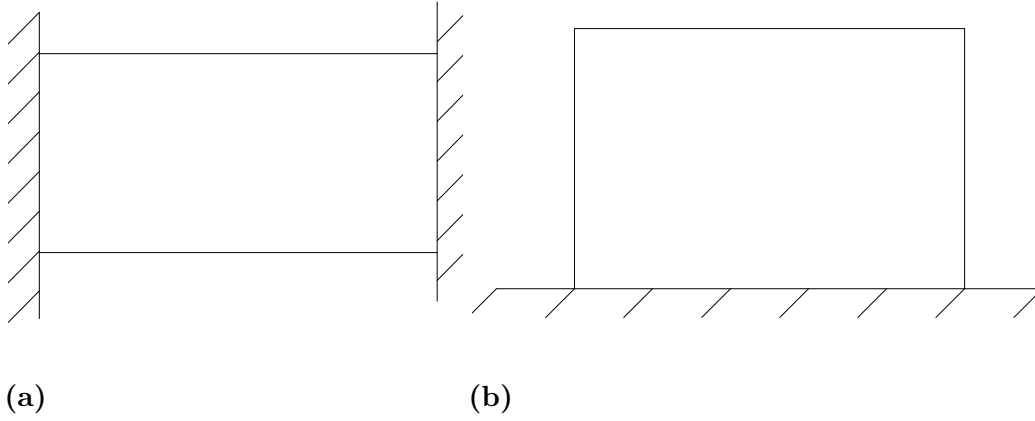


Figure 6.3: Restraint conditions a) restraint at ends, and b) restraint at base according to SS-EN 1992-3:2006 [12]

For restraint at the ends of the element (Figure 6.3a), the definition of $\varepsilon_{sm} - \varepsilon_{cm}$ is the same as in Expression 6.14. For the scenario of restraint along the edge of a long element (Figure 6.3b), the strain becomes:

$$\varepsilon_{sm} - \varepsilon_{cm} = R_{ax} \varepsilon_{free} \quad (6.29)$$

where

R_{ax} is the restraint factor from external axial restraint;
 ε_{free} strain imposed on the element in the case of no restraint

Clause L.2 in SS-EN 1992-3:2006 states that restraint factors for usual cases can be found in Figure L.1 and Table L.1. The value of R depends on the relationship between length and the height of the element, L/H . After the difference of strain has been calculated, the procedure in Section 6.3 can be followed to calculate the final crack spacing and crack width.

Design in Gen 2

Clause 9.2.3 for refined control of cracking in SS-EN 1992-1-1:2023 also includes directions for $\varepsilon_{sm} - \varepsilon_{cm}$ for three types of restrained members subjected to imposed strains (Figure 9.2 in SS-EN 1992-1-1:2023), two types which are the same as in Gen 1 (Figure 6.3), and one additional restraint scenario (Figure 6.4). For members subjected to imposed strains and restrained at the edges according to Figure 6.3a, the expression of strain becomes:

$$\varepsilon_{sm} - \varepsilon_{cm} = R_{ax} \varepsilon_{free} - k_t \frac{f_{ct,eff}}{E_{cm}} \geq 0 \quad (6.30)$$

where R_{ax} is the restraint factor calculated as:

$$R_{ax} = 1 - \frac{\varepsilon_{restr}}{\varepsilon_{imp}} \quad (6.31)$$

At the base of a wall, a value of $R_{ax} = 0.5$ can be assumed, then, the strains are defined as:

- ε_{restr} the strain that develops in the restrained element,
- ε_{imp} the imposed strain (from free shrinkage, free temperature strain). The ratio $\varepsilon_{restr}/\varepsilon_{imp}$ may be estimated according to linear elastic analysis, and if relevant, may account for staged construction,
- ε_{free} imposed strain developing after the construction stage when the restraint is applied (e.g., precast elements).

The difference in strain, $\varepsilon_{sm} - \varepsilon_{cm}$ for members restrained only at the ends as shown in Figure 6.3b is defined the same as in Eq. 6.25.

The last scenario is applicable if the member is restrained at both ends and at the edges according to Figure 6.4.

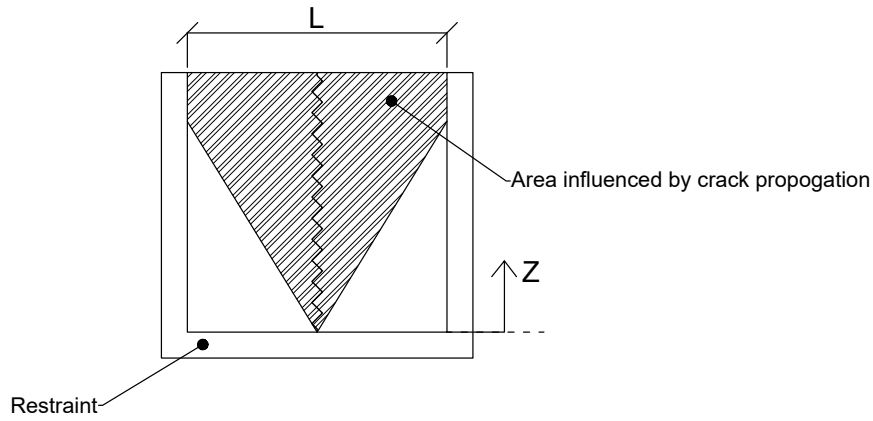


Figure 6.4: Restraint at base according to SS-EN 1992-1-1:2023

Then, the expression to be used is determined by the relationship between height, z , and the length of the member, L , accordingly:

$$\text{if } z = \begin{cases} > L/2 & \text{use Expression 6.25} \\ \leq L/2 & \text{use Expression 6.30} \end{cases}$$

After $\varepsilon_{sm} - \varepsilon_{cm}$ has been obtained, the final mean crack spacing can be calculated according to Expression 6.27.

7

ANCHORAGE AND LAPS OF STRAIGHT BARS

This chapter gives an overview of the differences and consequences related to anchorage and lapping of straight bars in the new Eurocode. A more detailed comparison of design methods for this type of anchorage and lapping is also provided.

Table 7.1: Overview of changes for detailing of anchorage and laps for straight bars

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Anchorage of straight bars	8.4.3(2), 8.4.4, 8.6(2)	11.4.2	Table summary, includes favorable effects.
Laps	8.7.2, 8.7.3	11.5.2	New method, different anchorage length.

7.1 Anchorage of straight bars

The calculation of anchorage length for straight bars in the current Eurocode was divided into three steps (8.4.2-8.4.4) [13] containing many parameters and an extensive calculation process [31]. The new version (clause 11.4.2) gives a choice between table values for common cases (Table 11.1 in SS-EN 1992-2:2023) or for the anchorage length to be determined directly through a formula [11] (Eq. 11.3 in SS-EN 1992-2:2023). Additional clauses mention favorable scenarios when the anchorage length can be reduced, e.g., in the presence of confinement reinforcement [31].

7.1.1 Design in Gen 1

The calculation of the basic anchorage length of straight bars according to SS-EN 1992-1-1:2005 can be calculated through clause 8.4.3, which is then followed by the calculation of the design anchorage length in 8.4.4. For welded bars, the anchorage length can be reduced via 8.6(2). The design lap length is calculated in clause 8.7.3.

The anchorage length depends on the bar diameter ϕ , design steel stress σ_{sd} , and design bond strength f_{bd} .

$$l_{b,rqd} = \left(\frac{\phi}{4}\right) \left(\frac{\sigma_{sd}}{f_{bd}}\right) \quad (7.1)$$

where

$$f_{bd} = 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} \quad (7.2)$$

f_{ctd} is the design tensile strength of concrete, and should be limited to strength classes below C60/75, and

$$\eta_1 = \begin{cases} 1 & \text{with good bond conditions} \\ 0.7 & \text{otherwise} \end{cases} \quad (7.3)$$

Here, "good bond" applies to bars located 250 mm or less from the bottom face of a slab/beam with a height of less than 250 mm, or for bars with an inclination of $45 < \alpha < 90$ degrees from the longitudinal axis. For sections with a height of more than 600 mm, "other" conditions apply within 300 mm from the top surface, but also in all cases unless "good bond" can be proven to exist.

The bond strength is also reduced for bar diameters above $\phi 32$ mm:

$$\eta_2 = \begin{cases} 1 & \text{for } \phi \leq 32 \text{ mm} \\ \frac{132 - \phi}{100} & \text{for } \phi \geq 32 \text{ mm} \end{cases} \quad (7.4)$$

Equation 7.1 also applies to bent bars, where the anchorage length follows the bar centerline.

The design value for anchorage length is calculated as follows

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \geq l_{b,min} \quad \text{where} \quad \alpha_2 \alpha_3 \alpha_5 \geq 0.7 \quad (7.5)$$

Coefficients α_i depend on cover thickness, confinement, and reinforcement detailing. Recommended values of α_i can be found in Table 8.2, SS-EN 1992-1-1:2005.

The minimum anchorage length can be computed as:

$$l_{bd,min} > \begin{cases} \max(0.3 l_{b,rqd}, 10 \phi, 100 \text{ mm}) & \text{for bars in tension} \\ \max(0.6 l_{b,rqd}, 10 \phi, 100 \text{ mm}) & \text{for bars in compression} \end{cases} \quad (7.6)$$

7.1.2 Design in Gen 2

To calculate the anchorage length of straight bars in SS-EN 1992-1-1:2023, one has to follow clause 11.4.2. The anchorage length is dependent on the type of bar used, its diameter, and nominal concrete cover, as well as the bond conditions. It is designed to be able to safely transfer the forces from the reinforcement to the surrounding concrete without spalling or bond failures.

Depending on the aforementioned conditions, Table 11.1 in SS-EN 1992-1-1:2023 can be used to get the anchorage length of a straight bar, which was also mentioned above in Section 7.1. If a common case is not present, or if a more detailed calculation is needed, the following equation can be used:

$$l_{bd} = k_{lb} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435} \right)^{n_\sigma} \cdot \left(\frac{25}{f_{ck}} \right)^{\frac{1}{2}} \cdot \left(\frac{\phi}{20} \right)^{\frac{1}{3}} \cdot \left(\frac{1.5\phi}{c_d} \right)^{\frac{1}{2}} \geq 10\phi \quad (7.7)$$

where c_d is the nominal cover, k_{cp} is a coefficient for casting effects, k_{lb} and n_σ depend on the anchorage and load combinations, and are applied according to the recommended values or the National Annex.

The ratios given in Expression 7.7 should be limited by:

$$\frac{\phi}{20} \geq 0.6 \quad \text{and} \quad \frac{25}{f_{ck}} \geq 0.3$$

The parameters are taken as:

$$c_d = \min(0.5 c_s; c_x; c_y; 3.75 \phi) \quad (7.8)$$

$$k_{cp} = \begin{cases} 1 & \text{for bars with good bond conditions,} \\ 1.2 & \text{for poor bond conditions and for all bars used in slip form} \\ & \text{construction unless it is shown that the vertical bars cannot} \\ & \text{move during casting,} \\ 1.4 & \text{for all bars executed under bentonite or similar slurries,} \\ & \text{unless data is available for the specific slurry to be used.} \end{cases} \quad (7.9)$$

Here, "good bond" generally applies to reinforcement bars located within 300 mm from the bottom of a beam or slab, and "poor bond" accounts for bars at the top face, more than 300 mm from the bottom. $k_{cp} = 1.4$ can be applied under certain geotechnical conditions. Additionally, k_{lb} is a NDP with the recommended values:

$$k_{lb} = \begin{cases} 50 & \text{for persistent and transient design situations} \\ 35 & \text{for accidental design situations} \end{cases} \quad \text{with } n_\sigma = \frac{3}{2} \quad (7.10)$$

However, the Swedish Transport Agency has proposed preliminary values for these NDPs [18]. These values are defined as:

$$k_{lb} = \begin{cases} 40 & \text{for persistent and transient design situations} \\ 30 & \text{for accidental design situations} \end{cases} \quad \text{with } n_\sigma = 1 \quad (7.11)$$

The reasoning behind the choices of these NDPs is presented in Section 9.6.

As presented in Chapter 7.1, the anchorage length can be reduced in the presence of favorable cases, such as confinement. These are specified in clauses 11.4.2(5) and 11.4.2(6).

7.2 Lapping of straight bars

The instructions for lapping of straight bars are also applicable to all other types of laps, as included in Section 8. The calculation of the required lap length differs between Gen 2 and Gen 1. Gen 2, SS-EN 1992-2:2023 clause 11.5.2, clarifies the design of laps through added clauses (4) and (5), and through figure simplification. For example, visualizing the difference between transverse reinforcement and confinement reinforcement (Figure 11.12, SS-EN 1992-2:2023) [31].

7.2.1 Design in Gen 1

The calculation of the design lap length of straight bars according to SS-EN 1992-1-1:2005 can be calculated via clause 8.7.3. Clause 8.7 states that the forces between bars can be transmitted through the lapping of bars, welding, or mechanical coupling. These laps should carry forces from one bar to another, while avoiding spalling or cracking of concrete. Additionally, laps should not be located in sections of maximum moment/shear force, and their layout should be symmetrical. Limitations on the layout of laps are visualized in Figure 7.1, where the distance between adjacent laps should be more than 20 mm, or $2 \cdot \phi$.

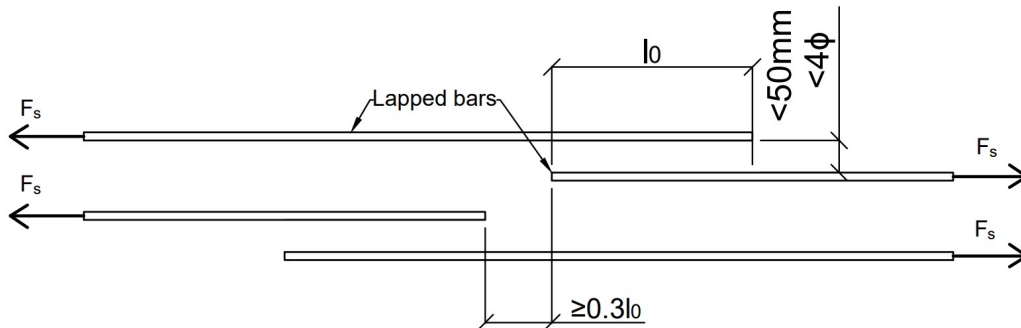


Figure 7.1: Arrangement of lapped bars

Following the provisions in SS-EN 1992-1-1:2005 clause 8.7.3, the design lap length is defined as:

$$l_0 = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,rqd} \geq l_{0,min} \quad (7.12)$$

where the minimum lapping length is defined as:

$$l_{0,min} > \max(0.3 \cdot \alpha_6 \cdot l_{b,rqd}, 15 \phi, 200 \text{ mm}) \quad (7.13)$$

The factors $\alpha_{1,2,3,5}$ are the same as in Eq.7.5 except for α_3 , where the area, $\Sigma A_{st,min}$ should instead be calculated as:

$$A_{st,min} = A_s \frac{\sigma_{sd}}{f_{yd}} \quad (7.14)$$

and α_6 can be taken from Table 7.2.

Table 7.2: Values for α_6

ρ_1	<25%	33%	50%	>50%
α_6	1	1.15	1.4	1.5

where ρ_1 is the reinforcement ratio of lapped bars within a distance of $0.65 l_0$ from the center of the considered lap.

The most common value of ρ_1 for main reinforcement is 50%, giving α_6 a value of 1.4. For secondary or compression reinforcement, ρ_1 usually equals 100%, which makes the value of α_6 equal to 1.5.

7.2.2 Design in Gen 2

To calculate the lap length of straight bars in SS-EN 1992-1-1:2023, one has to follow clause 11.5.2. Much like the current version, SS-EN 1992-1-1:2023 also (clause 11.5.2) states that forces should be transmitted through lapping, mechanical coupling, or welding. They should be designed to transfer forces and prevent spalling or cracking of the concrete.

For straight bars with tension or compression laps, the lap length can be defined as

$$l_{sd} = k_{ls} \cdot l_{bd} \geq 15 \phi \quad (7.15)$$

where k_{ls} equals 1.2, unless the National Annex is applied, and l_{bd} is the anchorage length. A preliminary value for the Swedish National Annex is:

$$k_{ls} = 1.5 \quad (7.16)$$

which should be used in combination with the NDP k_{lb} for anchorage length [18].

7.3 Preliminary NDPs proposed by the Swedish Transport Agency

The anchorage lengths are expected to be greater in Gen 2 for bars with large diameters and high steel stress ($\phi 16$ mm or above), especially for good bond conditions. For smaller bar diameters and poor bond conditions, the anchorage lengths are instead expected to be significantly lower [31].

To avoid overly conservative anchorage lengths for bars with large diameters, the Swedish Transport Agency has proposed preliminary changes to the NDPs for anchorage lengths and lap lengths k_{lb} , see 7.11 and 7.16 [18]. These changes are motivated by the lack of data points corresponding to large bar diameters and high steel stresses used for calibrating the model for anchorage lengths in Gen 2. However, for lap lengths, the data bank used to calibrate the lapping coefficient included these higher stresses and larger bar diameters. Therefore, the purpose of the new lap length coefficient is to counteract the change of the anchorage coefficient. The goal was to keep the lapping lengths as the recommended value provided by the Eurocode, since the Swedish Transport Agency argued that the calibration of the lapping coefficient was justified, since a larger data bank was used. [20].

It should be noted that these modified parameters are not yet implemented and could be changed in the future. These changes are motivated by the lack of data points corresponding to the larger bar diameters (above $\phi 16$) used to calibrate the model for anchorage length in Gen 2 [20].

8

REVIEW OF OTHER CHANGES

The following sections contain summaries of changes between Eurocode SS-EN 1992-1-1:2005, SS-EN 1992-2:2005, and Eurocode SS-EN 1992-1-1:2023. This chapter excludes comparison for chapters regarding Materials, Bending, Shear, Crack control, and Anchorage/Lap lengths, since these are covered in previous chapters. The differences concluded in this chapter are not analyzed in detail, and the parts of Eurocode 2 covered here will not be included further in the Master's Thesis.

8.1 Durability and concrete cover

The majority of changes in this chapter have been made to simplify the application of the code [45]. The changes are summarized in Table 8.1.

Table 8.1: Overview of changes for durability and concrete cover

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Exposure classes	4.2, 4.4.1.2	6.3.	Ease of use. General table.
Minimum concrete cover	4.4.1.2	6.5.2.1	Modified equation for minimum concrete cover.
Minimum cover for durability	4.4.1.2	6.5.2.2	$c_{min,dur}$ for carbon reinforcing steel, divided by carbonation and chlorides.
Allowable deviation in concrete cover	4.4.1.3	6.5.3	Reported differently.

Chemical classes

Exposure classes for mechanical attack by abrasion have been added to the general table of exposure classes in SS-EN 1992-1-1:2023 clause 6.3 [11] (Table 6.1). These exposure classes existed in the old version (SS-EN 1992-1-1:2005, clause 4.2 and 4.4.1.2), but were not included in the summarized table [45].

Concrete cover

$c_{min,dur}$ for carbon reinforcing steel is also divided into a table specified for carbonation (Table 6.3 (NDP), and another for chlorides (Table 6.4 (NDP)) in SS-EN 1992-1-1:2023 clause 6.5.2 [11]. For certain conditions, $c_{min,dur}$ can be reduced according to clauses 6.5.2.2(2), 6.5.2.2(3), while for pre-stressing tendons, they should be increased according to 6.5.2.2(4) [11]. Gen 2 also includes the allowable deviation in concrete cover in a table, which is explained through text in the current Eurocode. The main result of these changes is improved ease of use [45].

8.2 Structural analysis

Changes to the Structural Analysis section have been made to simplify verifications and correct previous uncertainties and are summarized in the following Table 8.2.

Table 8.2: Overview of changes for structural analysis

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Geometric imperfections	1992-2, 5.2	7.2.1.2	New limits for α_h .
Plastic analysis for beams, frames, and slabs	5.6.2.2	7.3.3.2	Same ductility limit for all strength classes.
LEA with redistribution	5.5(5)	7.3.2	(3) Can be done without an explicit check on rotation capacity. (5) New formulation for rotation capacity.
2nd order analysis	5.8.4	7.4, Ann. O	Separating local and global effects.

Geometric imperfections

The effect of geometric imperfection is included in the current Eurocode (SS-EN 1992-1-1:2005 clause 5.2), which introduces an inclination, θ_i . The inclination depends on the height of the structure and the number of structural members. This model is kept in SS-EN 1992-2:2023, but a lower limit for the imperfection factor, α_h , is included via clause 7.2.1.2 [11]. This could potentially reduce the effect of geometric imperfections since the designer can use a lower inclination in design.

Geometric imperfections are also included in SS-EN 1992-2:2005 (bridges), which uses the same model as explained above, but the effect of multiple members is not included (clause 5.2) [14]. Since Gen 2 combines SS-EN 1992-2:2005 and SS-EN 1992-1-1:2005, it is now possible to take into account the effect of multiple members on the imperfection factor for bridges since the same formula is used for buildings and bridges.

Plastic and linear elastic analysis

For plastic analysis, the limit for ductility is independent of strength class in Gen 2 (clause 7.3.3.2) [11], as opposed to Gen 1, where it varies for high-strength concretes (clause 5.6.2) [13]. Gen 2 also includes a simplification of the analyses for linear elastic analysis with redistribution that enables the analysis to be made with or without explicit verification of rotation capacity [47]. The verification of rotation capacity has also been revised due to previous uncertainties in derivation and incorrect curves, among other points mentioned in the background document [7].

Second order effects

The influence of second-order effects has seen some changes in Gen 2 (clause 7.4), where the simplified analysis has been moved to Annex O. Gen 1 (clause 5.8.4) provides simplified checks for slenderness for isolated members and global second-order effects. These models also exist in Gen 2, but some modifications have been made to formulas and coefficients. However, the general method for second-order effects is not changed significantly in Gen 2 [6].

8.3 Punching

The clause for shear punching has been considerably revised in the new Eurocode, with differences listed in Table 8.3 below. These changes require further investigation, but as mentioned in the motivational section (Section 2.4.6) was decided to be deprioritised to have time to examine the other highly important design criteria. It was also considered less important since punching is a more specific design case, which is not always relevant to design. However, the important differences and potential consequences of the revisions of the punching clauses are discussed in this section.

Table 8.3: Overview of changes for punching shear

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Control perimeter	6.4.2(1)(2)	8.4.2(2)	Reduced control section.
Reduction of force for foundation slabs	6.4.4 (2)	8.4.2(6)	Reduction for ground slabs without shear reinforcement.
Concentration of shear forces coefficient, β	6.4.3(3)(4)(5)	8.4.2(6)	Approximate values for different supports. Additional refined equations.
Capacity without shear reinforcement	6.4.4(1)(2)	8.4.3(1)	New mechanical model. New coefficients. d_g -factor. Beneficial effects of compressive membrane in action in continuous slab-column connections
Capacity with shear reinforcement	6.4.5(1)	8.4.4 (1), Ann.I.8.5	New mechanical model
Capacity limit	6.4.5(1)	8.4.4(5)(6), Ann.I.8.5	Updated shear resistance limits
Outer control perimeter	6.4.5(4)	6.4.4(7)(8)	Reduced outer control section

The design for shear punching in the current Eurocode, SS-EN 1992-1-1:2005, has been criticized for shortcomings and inconsistencies in the design method and ease of use [39]. Examples of shortcomings in the current Eurocode are the neglected effect of slenderness, overestimated punching resistance due to incorrect modeling of the size effect, and overestimation of the punching resistance of reinforced slabs. The design procedure has therefore been revised in Gen 2, where the same method is used for multiple design situations, such as foundations, columns, and slabs [39].

Control perimeter

The control perimeter for punching is changed, where clause 6.4.2 in SS-EN 1992-1-1:2005, calculates punching resistance within a distance of $2 \cdot d_v$ [13]. In SS-EN 1992-1-1:2023 clause 8.4.2(2), the control perimeter is reduced to $0.5 \cdot d_v$, where d_v is the shear-resisting effective depth [11]. This is a significant difference. However, this is not further investigated in the thesis due to time constraints. It is recommended to be investigated in a future study.

Design shear stress

Additionally, the factor β used to compute design shear stress is now given in a table for different design situations (clause 8.4.2(6), Table 8.3 in SS-EN 1992-1-1:2023), whereas the current Eurocode uses an extensive formula for this (clause 6.4.3 in SS-EN 1992-1-1:2005) [11]. This change aims to improve ease of use since the factor can easily be found without a thorough calculation.

In Gen 1, the shear stress is verified within the control perimeter of $2d$ on both sides of the support. For ground slabs, the upward pressure from soil is deducted from the shear stress within the control whole perimeter (clause 6.4.5) [13]. In Gen 2, this deduction is only allowed within a distance of $0.67d_v$ from the face of the support (clause 8.4.4) [11]. Additionally, the outmost control perimeter has been reduced from $1.5d$ in Gen 1, to $0.5d_v$ in Gen 2.

Resistance without reinforcement

There are also changes to the punching resistance for slabs without reinforcement, with a complete revision of the formula and a new mechanical model. The new model in clause 8.4.3 depends on aggregate size and punching shear gradient, and more refined models are presented for specific cases. The effect of slabs with axial compressive forces, or prestressing, is also taken into account through the addition of a new factor, k_{pp} [11].

Resistance with reinforcement

Design for punching resistance with reinforcement is completely revised in clause 8.4.4(1), with a new mechanical model. The new formula depends on the aggregate, the ratio between the moment resistance without reinforcement and the design punching shear stress, and the vertical shear reinforcement ratio. A maximum value for punching shear resistance is also included and is modified by a factor η_{sys} [11].

8.4 Torsion and combined actions

Improvements have been made for torsion and combined actions and can be observed in Table 8.4. Changes have been proposed to widen the scope and clarify design rules. They have also been made to complete the preceding section with clauses that did not previously exist, to enhance ease of use, and simplify the verification of members subjected to combined action [23].

Table 8.4: Overview of changes for torsion and combined action

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Torsional capacity equations	-	8.3.4	New equations.
Maximum nominal cover	-	8.3.4(5)	Determination of how a thin-walled cross section carries torsion.
Combined action	6.3.2(4)	8.3.5, Ann.G, G.3(3)	More combinations. σ_{cd} in Ann. G. Reduction of ν for verification of bottom wall.
Interaction for combination of internal forces	-	8.3.6	New equation.

Many of the torsion capacity equations in SS-EN 1992-1-1:2023 clause 8.3.4 are new. The capacity calculations have been added to cover capacity governed by yielding of the shear reinforcement $\tau_{r,Rd,sw}$, yielding of the longitudinal reinforcement $\tau_{t,Rd,sl}$, or crushing of the concrete compression field $\tau_{t,Rd,max}$ [11]. The equations simplify verification of members subjected to combined actions [23].

The requirement of maximum nominal cover in clause 8.3.4(5) applies to a thin-walled cross-section. Previously, there was the risk of overestimation of the torsional capacity for square cross-section due to spalling of the concrete cover. To correct this estimation, the torsional capacity is now based on a reduced cross-section with a reduced cover [23].

Design rules for combined action in SS-EN 1992-1-1:2023 clause 8.3.5 have been completed to widen the scope and enable application for a larger set of combinations compared to clause 6.3.2(4) in SS-EN 1992-1-1:2005. This general method of interaction between a combination of actions will be especially useful for the design of structures such as box bridge girders [23].

A linear interaction formula has been introduced via clause 8.3.6 for cases when the dominant force on the cross-section is a combination of internal forces and not torsion. This interaction formula acts to provide a simple and conservative approach to verification [23].

8.5 Design with strut-and-tie models and stress fields

Some changes have been made to the design of strut and tie models, which are summarized in Table 8.5. This section, as with other sections, has updated contents based on advancements in knowledge in the field [41].

Table 8.5: Overview of changes for design with strut-and-tie models and stress fields

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Strength reduction factor, ν	6.5.2(2)	8.5.2(3)(4)(5)	Factor θ_{cs} . Refined approach. (5) Value for compression fields in cracked zones.
Model of struts and ties	6.5.3	8.5.3	Modeling of tie forces.
Design of nodes	6.5.4	8.5.4	Extended.
Spreading of struts	-	8.5.5(3)	Prevention of local cracking.

The general clause has been improved and extended to clarify the analogies of the strut-and-tie models with the stress field model.

The strength reduction factor of concrete, ν , has been revised in SS-EN 1992-1-1:2023 clause 8.5.2 to depend on the cracking state of the compression field (compared to 6.5.2(2) in SS-EN 1992-1-1:2005). Three scenarios are given:

1. Compression fields and struts crossed or deviated by a tie at an angle.
2. Compression fields and struts in a region without transverse cracking.
3. Compression fields in cracked zones.

The first scenario is beneficial for preliminary design, when the exact angle of the strut and tie is unknown. The second case allows for a refined calculation for the precise calculation of the resistance. The third scenario is mostly applicable for automated calculations and gives a local value of the strength reduction factor based on the principal tensile strain [11] [41].

Design of nodes is extended in Gen 2 clause 8.5.4, and shall be included in the general verification for the transfer of forces between different strut-and-ties. Different types of nodes have also been clarified to simplify the verification.

Spreading of struts has been added as clause 8.5.5(3) to specify design criteria for transverse reinforcement needed to carry spreading forces when a load is applied near an edge, and to prevent uncontrolled cracking near the loaded region [41].

8.6 Partially loaded areas

The most significant difference between the old and new versions regarding the section for Partially loaded areas is the calculation of design resistance (Eq. 8.126 in SS-EN 1992-1-1:2023) as listed in Table 8.6 [11].

Table 8.6: Overview of changes for partially loaded areas

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Design resistance	6.7	8.6, Ann. I8.6	Wider range of application. Less conservative approach.

The background document mentions that this section previously was very conservative with respect to compression failure, Gen 2 has therefore a wider area of application and a less conservative approach [5].

8.7 Deflection

The new Eurocode provides additional methods for deflection control meant to simplify verification according to Table 8.7

Table 8.7: Overview of changes for deflection control

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Deflection control by span/depth ratio	7.4.2(2)	9.3.2(1)	New method for slenderness limits.
Long-term deflections for reinforced concrete structures	7.4.3	9.3.3	New simplified method.

An additional method for long-term deflection of reinforced concrete structures has been presented due to the complex application of the ξ -method previously used in SS-EN 1992-1-1:2005 clause 7.4.3. This new simplified method, given in SS-EN 1992-1-1:2023 clause 9.3.3, obtains the same results as the aforementioned method with no risk of inconsistencies. From this general simplified formulation of deflection, a general equation determining the slenderness limit can be derived, simplifying also the deflection control by span/depth ratio, resulting in an updated version of the limiting span/effective depth ratios l/d for buildings [3].

8.8 Vibrations

This section was not included in Gen 1 of Eurocode 2, but is covered in the newer version as seen in Table 8.8.

Table 8.8: Overview of changes for vibrations

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Vibrations	-	9.4	New.

SS-EN 1992-1-1:2023 provides guidelines for verifications, dynamic modulus of elasticity, and damping ratios to be used (clause 9.4) [11]. According to Gen 2, verification of vibrations should follow instructions given in EN 1990:2023 (A.1.8.3, A.2.9.2, A.2.9.3, A.2.9.4) [11].

8.9 Fatigue

The changes to the Eurocode regarding Fatigue mainly contain simplifications of verifications to enhance ease of use. Some new verifications have also been added. The revisions are included in the Table 8.9.

Table 8.9: Overview of changes for fatigue

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Complex verification	6.8.5, 6.8.4(2)	Ann. E (2.1-2.3, 3.1-3.3), E.5	Complex verification moved to annex, clause 10 includes simplified verification.
Verification for reinforcing and prestressing steel	6.8.4	10.4 + Ann. E.4 + Ann. C.4	New simplified verification.
Design fatigue strength	6.8.7(1)	10.5(1)	New equation.
Verification of concrete	6.8.7	Ann. E.4.3	New formula for verifying concrete fatigue resistance.

Fatigue of reinforcing steel needs to be verified for structural elements that are exposed to stress cycles, caused by repeated changes in load or deformation (clause 10.1, SS-EN 1992-1-1:2023) [11]. Today, prestressing steel is used as reinforcement to a further extent, and bar diameters are smaller compared to when the current Eurocode (SS-EN 1992-1-1:2005) was released. Tack welding is also often used to fix bars together, but this damages fatigue performance. To include the negative effects of tack welding, some changes have been made to the S-N curves used for fatigue verification, resulting in changes in Gen 2 (clause 10.4 and 10.5) [21]. The new Eurocode provides simplified methods for fatigue verification, where bars with a diameter less than 12 mm have a higher fatigue strength, which is decreased for welded bars (clauses 10.4-10.7, SS-EN 1992-1-1:2023). Refined fatigue verifications using damage equivalent stress range, and the Palmgren-Miner method previously included in clauses 6.8.4 and 6.8.4 in SS-EN 1992-1-1:2005 are moved to Annex E.

8.10 Detailing of reinforcement and tendons

The revision of the following section includes new guidelines for various types of anchorage and laps, e.g. anchorage and laps of U-bar loops previously not covered by the old Eurocode [31] [25], as well as new rules for post tensioning tendons [17] and handling of deviation forces due to curved chords [33] which are summarized in Table 8.10. This chapter also gives an overview of the differences and consequences related to detailing in the new Eurocode. Details regarding anchorage and lapping of straight bars are provided in Chapter 7, which also gives a detailed comparison between SS-EN 1992-1-1:2005 and SS-EN 1992-1-1:2023 for the design methods for anchorage and lapping of said bars.

Table 8.10: Overview of changes for detailing of reinforcement and tendons

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Permissible mandrel diameter, general	8.3	11.3	New simplified rules.
Anchorage of bundles	8.9.2	11.4.3	Equivalent diameter introduced
Anchorage, bars w/ bends	8.4.3(3), 8.4.4	11.4.4	Reduction of anchorage length for specific cases.
Anchorage of bars, welded transverse reinforcement	8.4.4, 8.6	11.4.5	Allowed reduction of anchorage length.
Anchorage of U-bar loops	-	11.4.6	New.
Anchorage of headed bars in tension	-	11.4.7	New.
Anchorage of bonded post installed reinforcing steel	-	11.4.8, Ann. C.4, Ann. C.8	New condition.
Laps	8.7.2, 8.7.3	11.5.2	New method, different anchorage length.
Laps of bundles	8.9.3	11.5.3	Bundles with mixed diameter included.
Laps using U-bar loops	-	11.5.4	New equation for capacity.
Laps using headed bars	-	11.5.5.	New equation for capacity.
Curvature and tendon lengths adjacent to anchorages	-	11.6.3	New minimum radius for curvature of tendons.
Deviation forces	-	11.7	New design provisions and rules for verification of curved reinforcement.
Prestressing post tensioning	8.10.1.3	11.6.2	Bundled tendons allowed New minimum radius of curvature adjacent to anchorage.

8.10.1 Permissible mandrel diameters for bent bars

The current generation (SS-EN 1992-2:2005, clause 8.3) was proven to perform poorly when estimating the response of bent reinforcement [36]. The new section (SS-EN 1992-2:2023, clause 11.3) simplifies the detailing rules to cover the majority of practical cases. It prevents change in mandrel size due to bending, by allowing the large-scale bending to be performed through progressive smaller-scale bending [11]. Keeping the mandrel size constant prevents the need for different machinery, leading to more efficient production. Simplified rules are also added for standard hook and bend anchorages, by defining a minimum clear distance, c_x , between the bar and an edge parallel to the bend. Furthermore, an equation enhancing the capacity of the steel stress, σ_{sd} , in the scenario of transverse bars being placed within a bend is added [36].

8.10.2 Anchorage of reinforcing steel

Clause 11.4 (SS-EN 1992-2:2023) for anchorage of reinforcing steel has seen several revisions in Gen 2, including new models and new cases of application. It also contains simplified design methods with enhanced ease of use. The effect of casting position is also included, where the poor bond conditions for bars cast at the top face of slabs and beams are taken into account [31].

Anchorage of bundles

The improvement of the anchorage of bundles is the explicit clarification that bundles can exist of bars with mixed sizes [31], clause 11.4.3.

Anchorage of bars with bends and hooks

According to SS-EN 1992-2:2023, clause 11.4.4, if the bend and/or hook is in compression, the anchorage capacity of the bars can be neglected unless spalling can be prevented. If spalling is prevented, the anchorage length can be reduced approximately the same as for straight bars in compression, 15ϕ , however, with a minimum length of 10ϕ [31]

Anchorage of bars with welded transverse reinforcement

The guidelines of anchorage of bars with welded transverse reinforcement in clause 11.4.5 have improved ease of use [31]. Reduction can be made to the anchorage length depending on the size and number of bars, given that the anchorage length does not go beneath 5ϕ [11].

Anchorage of U-bar loops

As mentioned at the beginning of this chapter, verification of this connection did not exist previously (clause 11.4.6) [11]. The loop details must follow the same guidelines as for permissible mandrel diameters [11]. In case where the legs of the U-bar loop have different forces, the anchorage length follows the guidelines for straight bars, but with a reduction of 20ϕ with a minimum length of 10ϕ [31].

Anchorage of headed bars in tension

The model for anchorage of headed bars in tension (clause 11.4.7) is new [32]. Requirements for whether the tensile forces in the reinforcing steel can be set to one given value ($\sigma_{sd} = 435$ MPa) are given. If the requirements are not fulfilled, or if a more detailed value is desirable, an equation calculating the maximum stresses in the bars is also defined. In the situation where the tensile forces are larger than the anchorage capacity of the head, an equation for the anchorage length required to carry the remaining stresses is given [11].

Anchorage of bonded post-installed reinforcing steel

This section also did not exist in Gen 1 [11]. Clause 11.4.8 in SS-EN 1992-2:2023 introduces a bond efficiency factor, $k_{b,pi}$, which compares the anchorage performance to that of cast-in bars, and acts to increase the anchorage length of the bonded post-installed reinforcement [11] [31].

8.10.3 Laps of reinforcing steel and mechanical couplers

This section has also become broader than it was in Gen 1, SS-EN 1992-2:2005, by proposing new methods, conditions, and cases of application.

All types of laps

The calculation of the required lap length differs between Gen 2 and Gen 1. Gen 2, SS-EN 1992-2:2023 clause 11.5.2, clarifies the design of laps through added clauses (4) and (5), and through figure simplification. For example, visualizing the difference between transverse reinforcement and confinement reinforcement (Figure 11.12, SS-EN 1992-2:2023) [31].

Laps of bundles

As with anchorage of bundles, the revision allows bundles of mixed diameter to be lapped (clause 11.5.3, SS-EN 1992-2:2023) [31].

Laps using U-bar loops

A new clause (11.5.4 in SS-EN 1992-2:2023) provides instruction on detailing of laps using U-bar loops, which is included in Gen 2 [11], which is not part of the current Eurocode. These connections are generally used to secure prefabricated concrete elements, such as bridge decks. The clause also contains a simple capacity calculation based on the theory of plasticity using an upper bound approach [25].

Laps using headed bars

The section regarding detailing of laps using headed bars is new (11.5.5(2) and (3) in SS-EN 1992-2:2023), and shall be combined with the section for anchorage of headed bars [11]. Three cases (Figure 11.15 in SS-EN 1992-1-1:2023) can be chosen for design, which affect the calculation of the capacity of the short-headed bar laps in tension (Eq. 11.17 in SS-EN 1992-1-1:2023). Laps using headed bars also call for transverse reinforcement to avoid brittle behavior (Eq. 11.21 in SS-EN 1992-1-1:2023). The capacity and ductility of the headed bars can be increased using tie-down reinforcement, the relevant area requirements are included in clause 7 in SS-EN 1992-1-1:2023 [46].

8.10.4 Post-tensioning tendons

The definition of the minimum radius of curvature and straight length of tendons adjacent to anchorages was previously prescribed by the manufacturer of the detailing through European Technical Assessment documents (ETA). However, in SS-EN 1992-1-1:2023, it is more clearly defined in the standard via clause 11.6.3, which may be used to prevent damage to the unconfined concrete and avoid a reduction of the axial tensile strength [17].

8.10.5 Deviation forces due to curved tensile and compressive chords

Gen 1's proposition was proved to be uneconomical and unnecessary for many cases of application. Since deviation due to curved tensile or compressive chords often needs to be considered, a new verification method was created through clause 11.7 in SS-EN 1992-1-1:2023. The new method was also proposed to reduce the risk of accidents [33].

8.11 Detailing of members and particular rules

The major difference between the old and new versions of this section of the Eurocode is the ease of use, which has been made possible by summarizing NDPs in tables along with relevant clauses and requirements addressing the point given [27] [11]. Changes are listed in the Table 8.11

Table 8.11: Overview of changes for detailing of members and particular rules

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Minimum reinforcement use	9.2.1.1	12.2	(2) Resistance of cracked section (6) Clarifications of anchorage and lapping
Minimum shear reinforcement for ductility classes B and C	-	12.4(4)	New
Shear reinforcement detailing	9.2.2, 9.3.2	12.3.3, 12.4.2	Slabs
Beams	9.2	12.3	Clarifications of NDPs and anchorage in tension/compression zone
Beam, shear, and torsion reinforcement	9.2.3	12.3.3	Anchorage in tension zones. Additional cases and restrictions for laps in stirrups in tension zones.
Punching shear reinforcement	9.4.3	12.5.1	Updated to be consistent with revised punching shear models.
Integrity reinforcement in flat slabs	9.4.1(3)	12.5.2	Amount of integrity reinforcement dependent on punching shear reinforcement. Allows a combination of contributions of shear, hogging, and integrity reinforcement.
Detailing requirements for reinforcement in columns	9.5.2	12.6	Clarifications for interpretation of prestressed columns. Limitations to spacing of longitudinal bars. Minimal number of longitudinal bars in circular columns.
Walls and deep beams	9.6, 9.7	12.7	Combined requirements for walls and deep beams.
Foundations	9.8	12.8	Clarifications of strut and tie. Footings and pile caps without shear reinforcement.
Tying systems for robustness of buildings	9.10	12.9	Clarifications of tie forces. Adaptability due to NDPs.

8.11.1 Minimum reinforcement rules

Rules on minimum reinforcement are included in both versions of Eurocode, but there are some differences in the requirements. The current Eurocode provides a formula for calculating the minimum reinforcement area directly (clause 9.2.11, SS-EN 1992-1-1:2005) [13], whereas SS-EN 1992-1-1:2023 clause 12.2 simply states that the minimum reinforcement should be able to carry at least the cracking load. For beams and slabs loaded with torsion or shear, minimum reinforcement is required. In Gen 2, the reinforcement ratio for minimum reinforcement in shear or torsion can be reduced by 10- and 20% for ductility classes B and C, respectively [11] (clause 12.2). This reduction is motivated by the fact that the deformation capacity of the reinforcement becomes governing when reinforcement ratios are low, which is the case when using minimum reinforcement. This effect also influences clause 8.2.3 in SS-EN 1992-1-1:2023, where members using shear reinforcement with ductility class A are penalized with a 20% reduction of the allowable angle of the compression field [37].

8.11.2 Beams

Since anchorage in the tension zone can be less efficient, and SS-EN 1992-1-1:2005 clause 9.2 did not distinguish between whether the anchorage was in tension or compression, this clarification was made in SS-EN 1992-1-1:2023 via clause 12.3 [27]. Continued issues with anchorage in tension zones in SS-EN 1992-1-1:2005, clause 9.2.3, have also led to a reassessment of Figure 12.2, now placed in SS-EN 1992-1-1:2023 clause 12.3.3, providing additional cases to showcase what detailing is and is not accepted in tension zones [11].

8.11.3 Slabs

Both generations of Eurocode give recommendations for detailing of slabs, but there have been some changes to the layout of the clauses. Gen 2 includes a table summarizing NDPs (Table 12.2 (NDP) in clause 12.4 SS-EN 1992-1-1:2023). Additional clarifications in the form of nodes have also been included. Another change is that Gen 2 now states that shear reinforcement can be detailed such that all except the outermost layers of longitudinal reinforcement bars are enclosed if the slab in question is not a one-way slab (clause 12.4.2, Figure 12.6 in SS-EN 1992-1-1:2023). This could lead to significant consequences, since it allows for more favorable detailing arrangements where multiple layers of bars are used. Gen 2 also provides cases where minimum shear reinforcement of slabs may be omitted (clause 12.4.2(3)).

8.11.4 Slab-column connections and column bases

Revisions of clause 9.4.3 in SS-EN 1992-1-1:2005 had to be made to stay consistent with the new models for punching shear, mainly leading to changed limitations, for

example, shear reinforcement spacing [27], resulting in clause 12.5.1, SS-EN 1992-1-1:2023. Gen 2 also introduces a maximum diameter for punching shear reinforcement, with different maximum values depending on the type of reinforcement (clause 12.5.1) [11]. This limit was introduced to avoid unfavorable effects from using a few bars with a very high diameter, which can harm punching resistance [43].

Flat slabs are also required to have integrity reinforcement that continues to carry load after punching failure has occurred. This is particularly relevant for flat slabs, which are prone to punching failure in a brittle manner [34]. In the current Eurocode, this is accounted for by prescribing at least two (2) reinforcement bars in each orthogonal direction at the bottom of slab-column connections (clause 9.4, SS-EN 1992-1-1:2005) [13]. This is kept in Gen 2 via clause 12.5.2, but the designer is also allowed to account for reinforcement bars used for other reasons, given that they fulfill requirements on anchorage, ductility, and placement. Gen 2 also includes a capacity calculation for integrity reinforcement, which is absent in the current EC. Favorable effect from hogging reinforcement on the tension side is also accounted for in clause 12.5.2 [11].

8.11.5 Columns

Clause 9.5.2 in SS-EN 1992-1-1:2005 has undergone revisions to clarify design, particularly for prestressed columns, resulting in clause 12.6 in SS-EN 1992-1-1:2023. A limit for spacing of longitudinal bars has been introduced [11]. For prestressed columns where all longitudinal bars in the column are prestressed, the minimum spacing based on bar size can be ignored [27]. Reinforcement bars within the compressive zone with a strain above 2‰ also need additional transverse reinforcement [11].

8.11.6 Walls and deep beams

The requirements for walls and deep beams have been combined in clause 12.7, SS-EN 1992-1-1:2023, due to difficulties in practice to determine if the behavior of the wall is that of a wall or if it behaves like a deep beam [27]. Elements are now differentiated by how they carry load, either axially loaded or by carrying normal and shear forces (Table 12.4, Gen 2) [11].

8.11.7 Foundations

Clarifications of the strut and tie model when used for detailing of foundations are included, with specific rules for the design of footings and pile caps without shear reinforcement [27] [11].

8.11.8 Tying systems for robustness of buildings

The old version of Eurocode for design of concrete structures, EN 1992-1-1:2005, was published before the standard covering accidental loads, EN 1991-1-7:2006, creating

some inconsistencies between standards. Clarifications of tie force requirements and horizontal ties have therefore been made. The second generation of EN 1991-1-7 was not available during the creation of EN 1992-1-1:2023; however, the risk of discrepancies has been reduced via the NDPs, allowing national bodies to modify them if required [27].

8.12 Additional rules for precast concrete elements and structures

Gen 2 declares new design rules for pre-tensioned tendons for precast concrete elements, since design criteria for pre-tensioning in the previous version only covered in-situ cast concrete, see Table 8.12.

Table 8.12: Overview of changes for precast concrete elements

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Minimum cover for pre-tensioning tendons	8.10, 4.4.1.2	13.5.1	Updated values.
Transmission length and anchorage length for pre-tensioning tendons	8.10.2.2	13.5.3, 13.5.4	New formulas. Updated coefficients.
Shear resistance without shear reinforcement	6.2.2	13.5.5	Moved, limiting effective depth.

The requirements for clear spacing between tendons and the required minimum concrete cover were previously separated for pre-tensioning tendons, they have now been merged. The minimum cover declared in clause 13.5.1 SS-EN 1992-1-1:2023 should depend on the type of pre-tensioning tendon, since these have different bond mechanisms, but also on the clear spacing. The value of the minimum concrete cover has been specified accordingly. The tendon-specific spacing for precast structures has also been updated [11] [42].

The calculation of the transmission length and anchorage length of pre-tensioning tendons has been simplified through clause 13.5.3 and 13.5.4, SS-EN 1992-1-1:2023 [16]. Comparing the two versions of Eurocode, they give very similar results for transmission lengths at low concrete strengths for strands. Regarding anchorage length for strands, the results are very similar for medium to high concrete strengths. Differences for wires are slightly larger for both the calculation of transmission length and the calculation of anchorage length. Clauses for calculating the transmission length (13.5.3) and the anchorage length (13.5.4) should be limited to a strand diameter no larger than 15.7 mm [16].

The calculation of shear resistance of pre-stressed members without shear reinforcement in clause 6.2.2, (Eq. 6.4 in SS-EN 1992 2:2005) has been moved to clause 13.5.5 (SS-EN 1992-1-1:2023) since pre-stressed members usually imply that they are

precast as well. The verification is no longer limited to simply supported members; it can now be applied to continuous members as well. The clause is, however, limited to an effective depth smaller than 500 mm, unless the size effect is considered [19].

8.13 Plain and lightly reinforced concrete structures

The new Eurocode introduces a simplified method for designing walls and columns where the axial capacity is calculated, which is listed in changes of the section in Table 8.13. This method takes the effect of eccentricity and second-order effects, such as creep, into account (clause 14.4) [11]. A similar method exists in the current Eurocode (clause 12.6.5.2, SS-EN 1992-1-1:2005), but second-order effects are not included.

Table 8.13: Overview of changes for plain and lightly reinforced concrete

Design Criteria	Ref. 1st Gen	Ref. 2nd Gen	Main Changes
Capacity for walls	12.6.5.2	14.4.5.2	Simplified design method.

9

PARAMETRIC STUDY

The behavior of a one-way concrete strip is analyzed for varying parameters corresponding to different design cases. The parameters were then changed one at a time, and the design capacities and behavior predicted by Gen 1 and Gen 2 were compared and visualized in graphs. The parameters chosen are specific to each design situation, but the general layout and geometry of the strip are summarized in Figure 9.1.

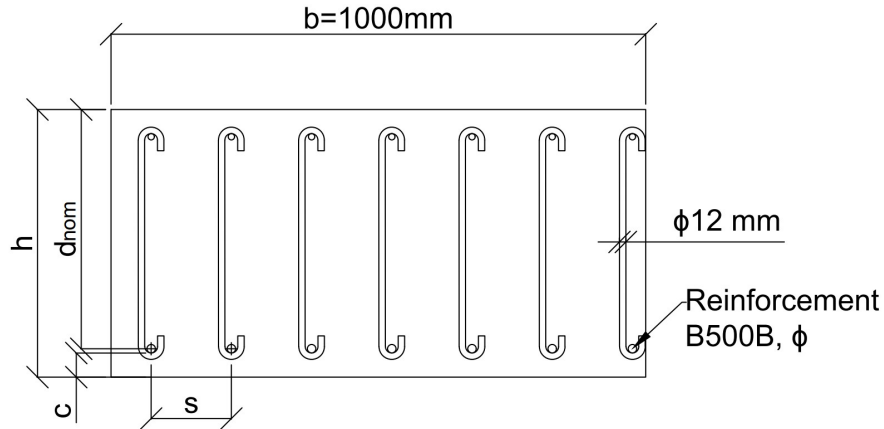


Figure 9.1: Specific section for the parametric study

- Width, $b = 1000$ mm
- Stirrup diameter, $\phi_s = 12$ mm (only included in relevant design scenarios)
- Characteristic steel strength, $f_{yk} = 500$ MPa
- Young's modulus for steel, $E_s = 200$ MPa

The term "ratio" refers to when the two standards have been compared according to:

$$\eta = \frac{\text{Result of Gen 2}}{\text{Result of Gen 1}} \quad (9.1)$$

and is used like "capacity ratio". Positive ratio (>1) indicates higher values when using the Gen 2 method, while a negative ratio (<1) means that Gen 1 predicts a higher value.

9.1 Materials and partial factors

The material data presented is included in the parametric studies.

Gen 1

- Concrete strength factor (NPD), $\alpha_c = 1.0$
- Concrete tensile strength factor (NPD), $\alpha_{ct} = 1.0$
- Partial factor concrete, $\gamma_c = 1.5$
- Partial factor steel, $\gamma_s = 1.15$

Gen 2

- Reference time factor, $k_t = 1.0$
- Minimum aggregate diameter for upper sieve size, $D_{lower} = 20$ mm
- Partial factor concrete, $\gamma_c = 1.5$
- Partial factor shear, $\gamma_v = 1.4$ for $d = d_{nom}$, and $\gamma_v = 1.29$ for $d = d_d$
- Partial factor steel, $\gamma_s = 1.15$ for $d = d_{nom}$, and $\gamma_s = 1.03$ for $d = d_d$

The capacity calculations in Gen 2 utilizes two different sets of partial factors, depending on whether the effective depth is based on a nominal value d_{nom} or a design value d_d according to Eq. 3.1. In the comparison, the combined partial factor and depth that result in the highest capacity is used. This effect could be significant in deep sections, where a reduction in effective depth of 15 mm relative to the nominal value is negligible compared to the total depth of the section. It should be noted that the application of the design effective depth for capacity calculations is limited by the National Annex. However, for this thesis, it is assumed that the application of d_d is allowed as recommended in the Eurocode.

9.2 Bending & compression

The parametric study for bending is made following the procedure explained in Chapter 4 to calculate the ultimate bending moment resistance. The height of the section varies, and is then subjected to an increasing centric axial force proportional to the capacity of the section in compression. The calculations were performed for varying reinforcement ratios. Two separate concrete classes, and thus concrete strengths, were also investigated. For Gen 2, the ultimate moment resistance varies depending on the nominal effective depth or the design effective depth; the largest value of M_{Rd} was chosen as the governing. The calculations can be found in Appendix A.1 with different input parameters depending on the scenario.

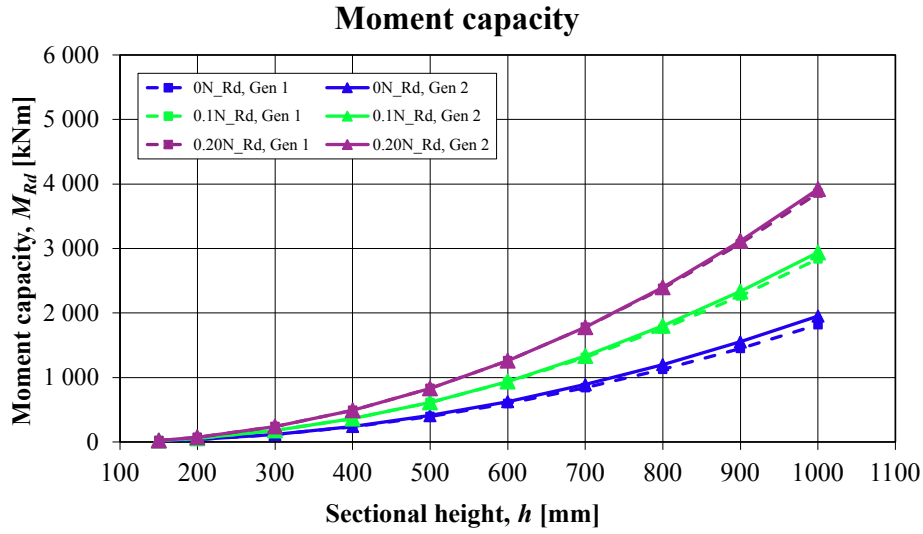
The ultimate moment resistance was calculated using the properties listed:

- Design compressive normal force, $N_{Ed} = [0, 5, 10, 15, 20]\% \cdot f_{cd} \cdot A_c$
- Height of section, $h = [150, 200, 300, 400, 500, 600, 700, 800, 900, 1000]$ mm
- Reinforcement diameter, $\phi = 16$ mm
- Reinforcement ratio, $\rho = [0.2, 0.5, 1]\%$
- Characteristic concrete strength, $f_{ck} = [35, 55]$ MPa
- Concrete cover, $c = 55$ mm

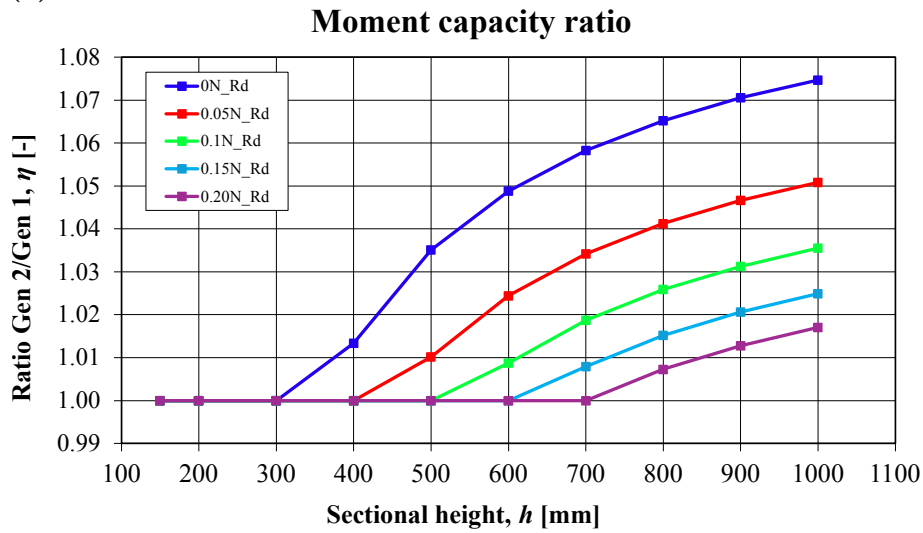
9.2.1 Influence of sectional height and compressive axial force for concrete class C35/45

The result from using the characteristic strength of 35 MPa to calculate the bending moment capacity is presented in Figure 9.2. Reinforcement ratios of $\rho_s = 0.2\%$ and $\rho_s = 1\%$ were also investigated, the graphs can be found in Appendix A.4.

For a reinforcement ratio of $\rho_s = 0.5\%$ shown in Figure 9.2, the moment capacity ratio increases with increased sectional height. The capacity varies between 2-8% larger for Gen 2 for the deepest examined sections, where the least increase is observed for the case with the highest added normal force. However, shorter sections result in less or no increase in bending moment resistance. Similarly, if the nominal value for the effective depth were used, there would be no difference in the moment capacity.



(a)

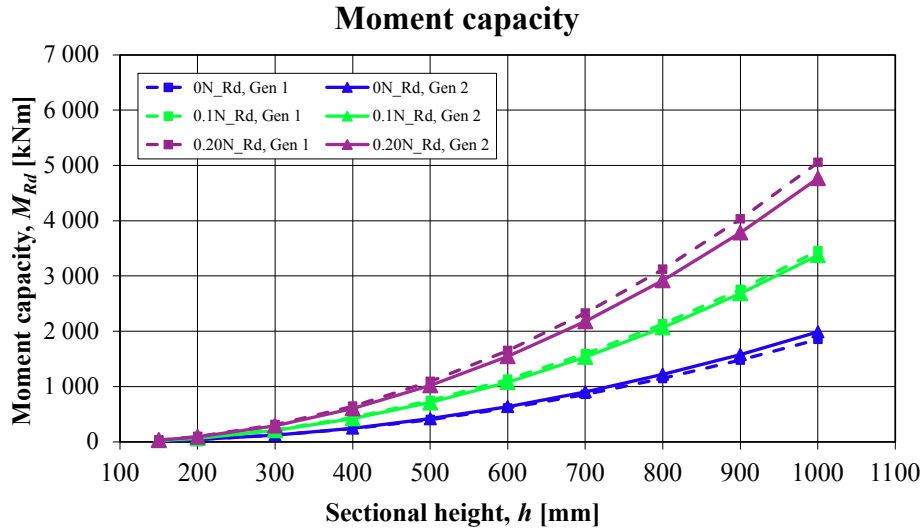


(b)

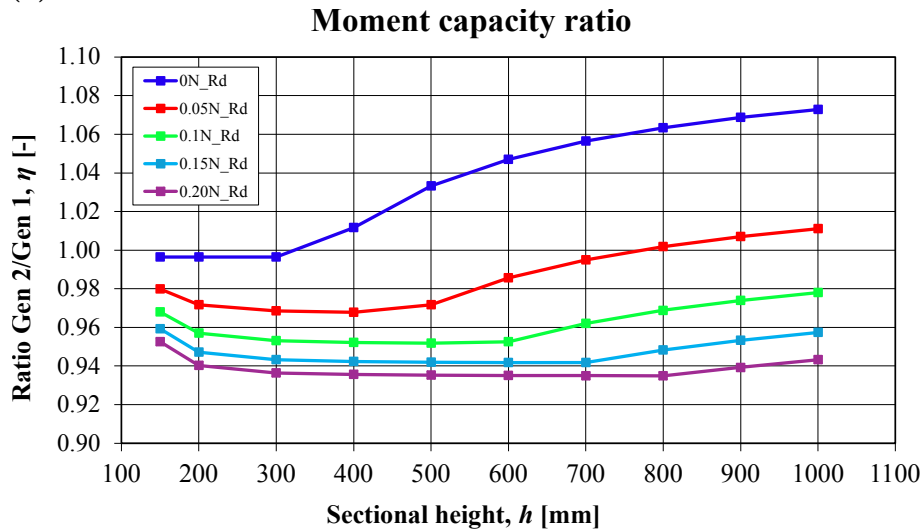
Figure 9.2: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 0.5\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

9.2.2 Influence of sectional height and compressive axial force for concrete class C55/67

The results from increasing the concrete strength to 55 MPa can be observed in Figure 9.3. Reinforcement ratios of $\rho_s = 0.2\%$ and $\rho_s = 1\%$ were also investigated, the graphs can be found in Appendix A.4.



(a)



(b)

Figure 9.3: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 0.5\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

When the reinforcement ratio is set to $\rho_s = 0.5\%$ as shown in Figure 9.3, the moment capacity increases for the taller sections. However, in the presence of axial load, the capacity is still lower than the bending moment capacity of Gen 1. At most, a 6% decrease in capacity can be observed when the large axial forces are applied to the sections.

9.2.3 Discussion

These results are only applicable if the design effective depth can be used for moment capacity calculations. As of now, it is uncertain if d_d can be used for these types of calculations in Sweden, since the application of d_d is controlled by the National

Annex. It could therefore be decided by the Swedish Transport Agency that the application of d_d is not allowed. In this case, there will be no increase in the bending moment capacity of Gen 2.

The result from using the concrete class C35/45 shows that taller sections have larger bending moment resistances for Gen 2 compared to Gen 1. The added axial load increases the overall moment capacity of the section but reduces the positive increase observed when using the Gen 2 design method.

For the higher concrete class (C55/67), the results show that Gen 2 has a lesser capacity than Gen 1 in the presence of larger axial forces, due to the strength reduction factor η_{cc} , introduced in Gen 2 (Eq. 3.5). For concrete strengths higher than $f_{ck,ref}$ (>40 MPa), η_{cc} becomes smaller than 1, leading to a reduction in the design compressive strength of concrete, f_{cd} (see Eq. 3.4).

The formula to calculate the axial force capacity N_{Rd} is:

$$N_{Rd} = A_c \cdot f_{cd} \quad (9.2)$$

The design axial force is then calculated accordingly:

$$N_{Ed} = [\%] \cdot N_{Rd} \quad (9.3)$$

meaning that the normal force is proportional to the axial force capacity of the cross section, determined by f_{cd} . The normal force of Gen 2, therefore, becomes smaller than that of Gen 1. Then, when the bending moment capacity is calculated using Equation 4.5, the positive impact from the normal force becomes larger where N_{Ed} is larger.

It is **highly important** to note that in the scenario where the amplitude of the axial force is identical in Gen 1 and Gen 2 (not proportional to the axial force resistance N_{Rd}), the difference in design moment resistance is similar to that presented in the Section 9.2.1 above, where there was no reduction of the design compressive strength for Gen 2.

Therefore, the conclusion to be made is that the bending moment resistance for the examined concrete strengths, sectional heights, and range of reinforcement ratios is nearly always favorable for Gen 2. The largest increase in capacity appeared for the tallest section with no added axial force. This increase happens when the design effective depth, d_d , is favored over the nominal effective depth, d_{nom} . Using d_d allows for the application of partial factor γ_{sd} for the reinforcement, which, in turn, allows the yield limit of the reinforcement, σ_s , to go from 435 MPa to 485 MPa. The compression zone is therefore increased, resulting in a greater moment capacity.

For smaller sections, the application of d_d decreases the moment capacity slightly, since d_d is the reduced nominal effective depth by 15 mm. Removing 15 mm from a shorter section has a more negative impact on the lever arm than for a taller section. The positive impact on the moment capacity from the increased yielding limit is also too small for shorter sections to be able to override the negative effect of the smaller lever arm.

9.3 Shear capacity without stirrups

The shear capacity with and without shear reinforcement is calculated following the design procedure explained in Chapter 5, where the capacities in Gen 1 and Gen 2 are compared for varying properties. In Gen 2, the design effective depth, along with reduced partial factors for shear and reinforcing steel, is used when favorable.

Additionally, forces acting on the section shown in Figure 9.1 are assumed to be the same in both Gen 1 and Gen 2. The design effective depth, d_d , was used to calculate the minimum shear capacity without stirrups and the shear capacity with stirrups.

However, d_d was not used for the detailed verification for shear without stirrups, which is solely determined based on the nominal effective depth, d_{nom} , and the effective shear span, a_{cs} , according to Section 5.2.2. The complete calculations can be found in Appendix B.1, where the input parameters vary depending on the scenario.

The shear capacity without stirrups is calculated for three different heights, each with varying reinforcement ratios and strength classes. The design shear stress, τ_{Ed} , was set to the minimum shear resistance of the section, $\tau_{Rd,min}$. Additionally, the effect of sectional forces is also included in the analysis, with a compressive normal stress applied at the center of gravity of the section and a moment. However, no effect of prestressing is included.

- Height of section, $h = [300, 500, 900]$ mm
- Characteristic concrete strength, $f_{ck} = 35$ MPa
- Concrete cover, $c = 55$ mm
- Reinforcement diameter, $\phi = 16$ mm
- Reinforcement ratio, $\rho_1 = [0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1]$ %
- Design shear force, $V_{Ed} = V_{Rd,c}$ kN (100% Utilization)
- Design compressive axial force, $N_{Ed} = [0, 0.05] \cdot N_{Rd}$ kN
- Design bending moment, $M_{Ed} = [0, 0.25, 0.5, 0.75] \cdot M_{Rd}$ kNm

Based on the input parameters, three different load scenarios are analyzed:

- 1) A slab only loaded with shear (a simply supported edge)
- 2) Shear force and moment (over a continuous support or fixed support)
- 3) Shear force and normal force

NDPs

Nationally Determined Parameters for Gen 1 were selected in accordance with recommendations from the Swedish Transport Agency, TSFS 2018:57 [15]. For Gen 2, the values recommended in the Eurocode were used, described in Chapter 5 together with preliminary recommendations from TSFS [18].

9.3.1 Influence of shear force

The shear capacity of concrete with and without any axial forces or moments is visualized in Figure 9.4. The figure shows the concrete shear capacity of a slab only loaded with shear. This is equivalent to the capacity of a control section located next to a simply supported edge.

The results show that the concrete shear capacity is significantly higher in Gen 2 for low reinforcement ratios, especially for the 300 and 500 mm deep sections, where the capacity is $\sim 42\%$ and $\sim 30\%$ larger, respectively. This vast increase in shear capacity for low reinforcement ratios is a result of the new model for minimum shear capacity in Gen 2 ($\tau_{Rdc,min}$), which becomes governing for sections with low reinforcement amounts.

For sections with a higher reinforcement ratio, the detailed shear capacity τ_{Rdc} becomes governing instead, with 5-24% higher concrete shear capacity in Gen 2. In addition, the increase in shear capacity is less significant for deep sections, indicating that Gen 2 of the Eurocode treats these heights more conservatively.

The influence of other aggregate sizes was also investigated in Annex B.3.5. The results show that a larger aggregate, with $D_{lower} = 24$ mm, results in an even higher shear capacity by 10-50%. For a smaller aggregate size, $D_{lower} = 16$ mm, the increase in capacity instead ranges from 0% for deep sections to 35% for squat sections.

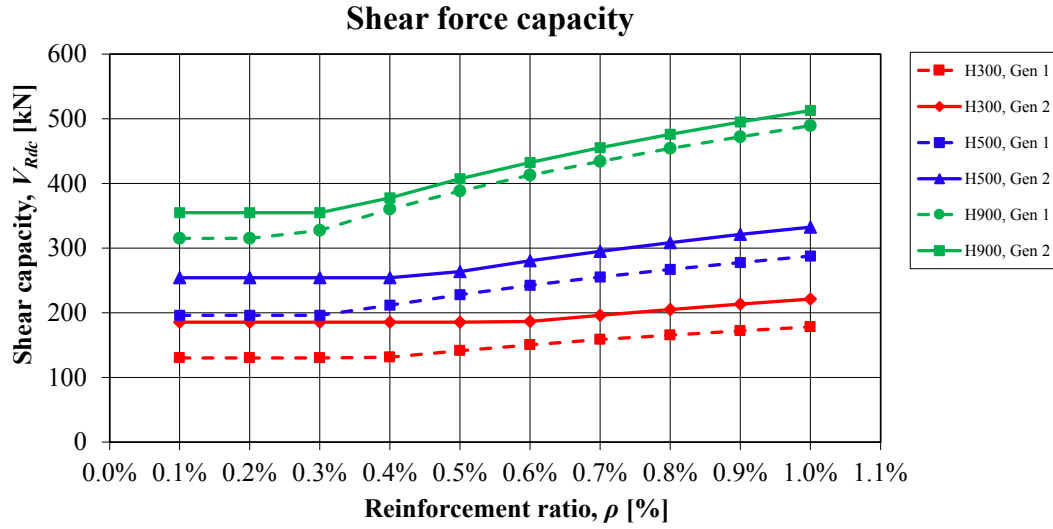
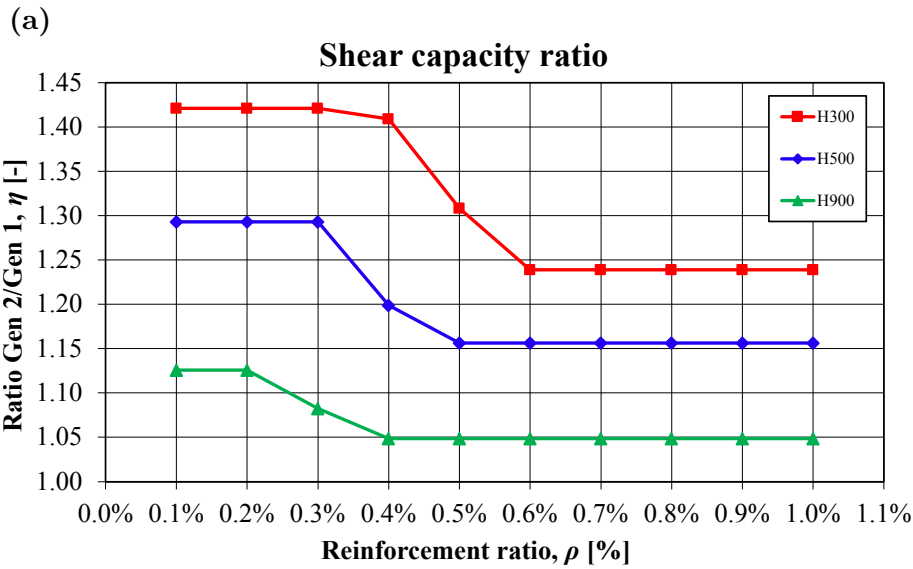


Figure 9.4: $V_{Ed} = V_{Rd}$, no axial load or bending moment for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

9.3.2 Influence of shear force combined with bending moment

As shown in Section 5.2.2, the effective shear span of SS-EN 1992-1-1:2023 depends on the relationship between the bending moment and the design shear force. This effect is not included in Gen 1. The interaction between shear force and bending moment can be relevant when investigating the mid-support of a continuous beam or a fixed support. Figure 9.5 shows the concrete shear capacity for a section subjected to a shear force equal to 100% of its minimum shear capacity, combined with a moment at 50% of its bending capacity ULS. Additionally, shear force combined with 25% and 75% moment utilization was also investigated, but is included in Appendix

B.3.

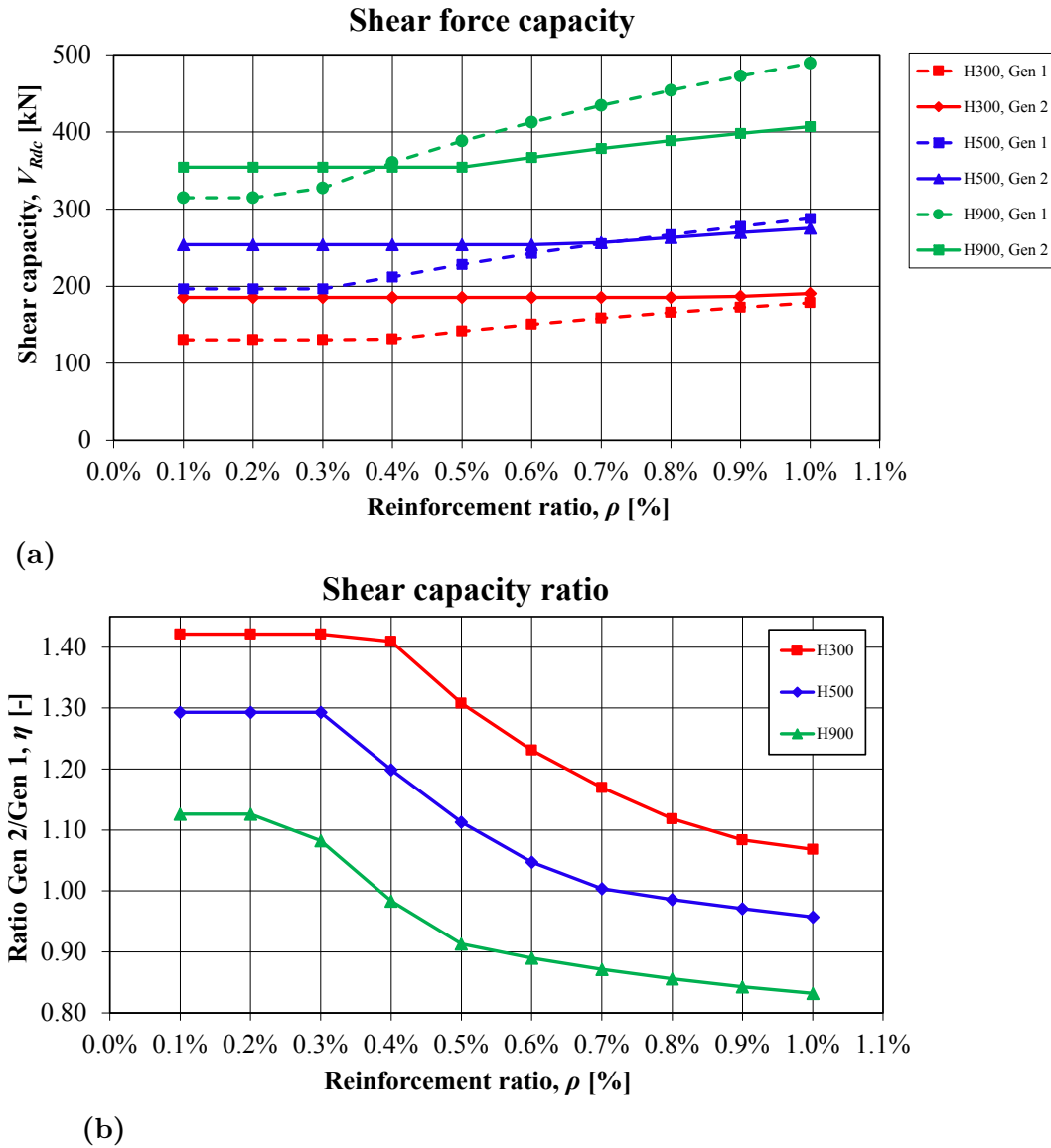


Figure 9.5: Loaded with shear $V_{Ed} = V_{Rd,c}$ and bending moment $M_{Ed} = 0.5M_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

Figure 9.5 shows that sectional moments have a negative influence on the shear capacity in Gen 2. This is especially evident for deep sections with high reinforcement ratios subjected to bending moment, where the concrete shear capacity was reduced by up to 17% compared to Gen 1. For shorter sections, Gen 2 gives greater shear capacity than the current Eurocode for the investigated range of reinforcement ratios. The height of 300 mm gives the largest difference, where Gen 2 gives 43% larger capacity for reinforcement ratios smaller than 0.4% (minimum shear capacity governs). Similarly to Figure 9.5, a 75% moment utilization leads to a 17% reduction in shear capacity. However, this reduction occurs for even lower reinforcement

ratios, since the design moment is greater. In comparison, a 25% moment utilization ratio results in a higher shear capacity in Gen 2 for nearly all reinforcement ratios.

9.3.3 Influence of shear force and compressive axial force

This scenario consists of a section loaded with shear force and compressive normal force applied at the center of gravity. The design shear force is the same as the shear capacity, while the compressive axial force equals 5% of the compressive capacity in ULS.

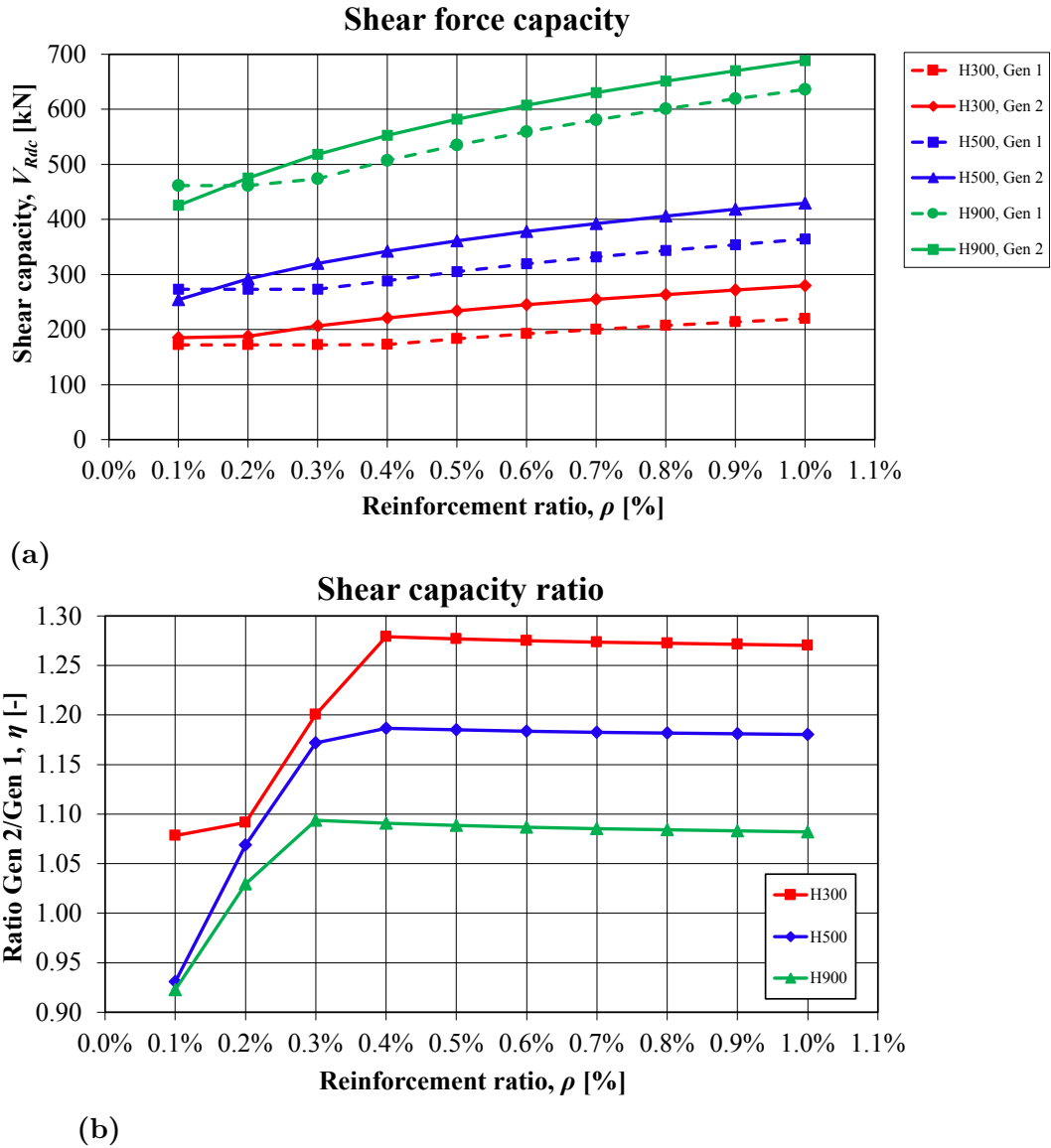


Figure 9.6: Loaded with shear $V_{Ed} = V_{Rd}$ and normal force $N_{Ed} = 0.05N_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

Figure 9.6 shows that the shear capacity increases by 8-28% for a section loaded with simultaneous shear and normal forces. For high reinforcement ratios, the increase in shear capacity in Gen 2 is higher than for a similar section without axial forces. An additional scenario with $N_{Ed} = 0.1N_{Rd}$ can be found in Appendix B.3.

9.3.4 Discussion

As shown in the graphs above, the concrete shear capacity is, in many cases, higher in Gen 2 compared to the current Eurocode. In cases with no moment or normal force acting on the section, the increase in capacity is about 5-43% compared to Gen 1. The increase is less prominent for deeper sections or sections with high reinforcement amounts. It was also found that a lower aggregate size negatively affects the concrete shear strength. However, it can be argued that small aggregates in combination with a deep section are unlikely to occur in reality.

A reason for these increases in shear capacity may be that the new Eurocode incorporates the effect of sectional forces. This is included through the introduction of a new mechanical shear span, a_{cs} , that is used to determine the concrete shear capacity based on the moment/shear ratio, following Equation 5.24. As a result, a greater shear capacity can be taken into account for sections where the moment is small relative to the shear force.

Another reason why the new Eurocode produces less conservative results may be that normal forces are considered in relation to the shear forces acting on the section through the factor k_{vp} , see Equation 5.25. This means that when the normal force is large compared to the shear force, the increase in shear strength becomes greater. Gen 2 also provides an alternative approach for considering the effect of normal forces, which becomes governing in some cases, resulting in even higher shear capacities.

It can therefore be concluded that the model in the old Eurocode likely considers a highly conservative case, where a high moment and low normal force in relation to the shear force is assumed, regardless of "real" forces acting on the section. With Gen 2, if it can be shown that the moment is low and the normal force is large in relation to shear, a higher shear capacity can be taken into account. This can be important for simply supported edges, with no moment acting on the section.

However, a decrease in the shear strength of concrete was observed for deep sections with high reinforcement ratios, subjected to both bending moment and shear force. This can be explained by the application of the mechanical shear span, a_v , and the effective shear span, a_{cs} . The interaction between shear force and bending moment results in a lower shear capacity for sections where the design moment is large compared to the design shear force. It should be noted that this effect is not included in the current Eurocode.

9.4 Shear capacity with stirrups

The shear capacity with and without shear reinforcement is calculated following the design procedure explained in Chapter 5, where the capacities in Gen 1 and Gen 2 are compared for varying properties. In Gen 2, the design effective depth, along with reduced partial factors for reinforcing steel, is used when favorable. Additionally, forces acting on the section shown in Figure 9.1 are assumed to be the same in both Gen 1 and Gen 2. The complete calculations can be found in Appendix B.1 for both generations, where the input parameters vary depending on the scenario.

The models for shear capacity with stirrups are similar between Gen 1 and Gen 2, which are based on the same truss model, see Chapter 5. They are thus expected to yield similar results. However, Gen 2 introduces a new effective depth, together with potential modifications to partial factors, and a new strength reduction factor that could be important for crushing capacity for high-strength concretes.

These effects are expected to influence the shear capacity with stirrups, especially for deep sections. To investigate this, calculations were performed for vertical stirrups, where the design shear stress was set equal to the capacity of the stirrups before yielding. These calculations were performed for different sectional heights. Additionally, the angle of the compression field, $\cot \theta$, was optimized with Equation 5.40 to achieve the highest shear capacity possible (usually 2.5). Other specific properties and dimensions are listed below:

- Height of section, $h = [150, 200, 300, 400, 500, 600, 700, 800, 900, 1000]$ mm
- Shear reinforcement diameter, $\phi_w = 12$ mm
- Longitudinal shear reinforcement spacing, $s_l = [230, 160, 140, 125]$ mm
- Transversal shear reinforcement spacing, $s_t = 200$ mm
- Characteristic concrete strength, $f_{ck} = 35$ MPa
- Design shear stress, $\tau_{Ed} = \tau_{Rdc}$

NDPs

Nationally Determined Parameters for Gen 1 were selected in accordance with recommendations from the Swedish Transport Agency, TSFS 2018:57 [15]. For Gen 2, the values recommended in the Eurocode were used, described in Chapter 5.

9.4.1 Influence of sectional height and longitudinal shear reinforcement ratio

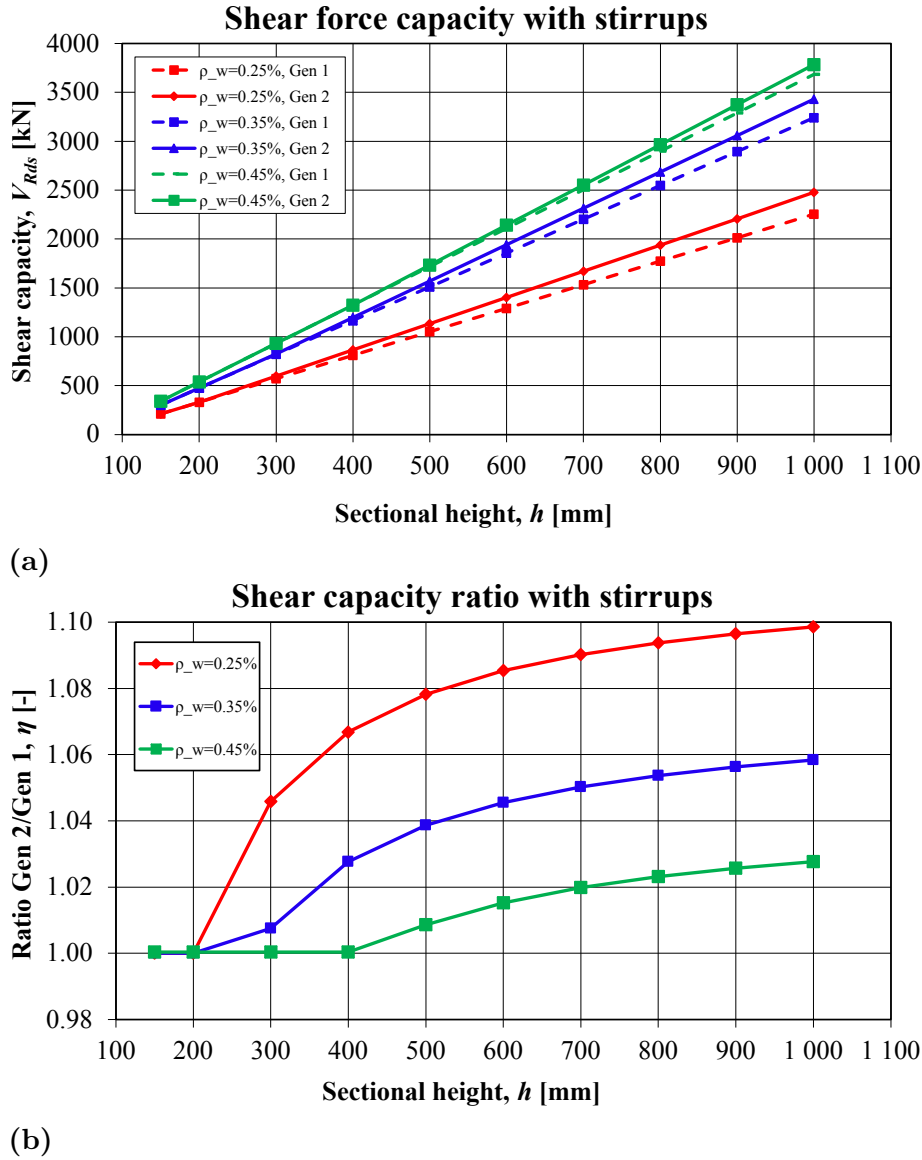


Figure 9.7: Shear capacity with stirrups for different heights and varying reinforcement ratios for the longitudinal shear reinforcement

Figure 9.7 shows the ratio, η , of shear capacity with stirrups for cross sections with varying height. The line in red represents shear sliding failure with yielding of shear reinforcement. Here, the shear capacity for heights of 200 mm or below is identical for the two Eurocodes. For deeper sections, an increase in shear capacity is observed, up to 10% higher capacity for deep sections. This is a result of the design effective depth, which allows for a more favorable partial factor for steel to be used. The differences are smaller for higher shear reinforcement ratios (crushing of concrete becomes limiting), represented by lines in blue and green in Figure 9.7.

9.4.2 Discussion

The results show that lower sections (≤ 200 mm) have similar shear capacity when including stirrups, a result that is expected since the same truss model is used in both generations. Taller sections show an increase in shear capacity, which can be explained by the modified partial factors for shear and reinforcing steel, which become governing for deeper sections. For very high shear reinforcement ratios, shear compression failure becomes governing, which is not influenced by the modified partial factors in Gen 2. Thus, the results of Gen 2 are similar to those of Gen 1 for larger ρ_w ratios.

The, in general, larger capacities of SS-EN 1992-1-1:2023 (observed in Figure 9.7) are likely caused by the new Expression 5.40, which allows the shear compression angle to be optimized to achieve simultaneous yielding of the shear reinforcement and crushing of the concrete.

9.5 Crack control

Surface crack widths are modeled for the slab in Figure 9.1. For simplicity, no long-term effects, such as creep or shrinkage, are included. The studied concrete slab is considered to be loaded under quasi-permanent conditions. The allowable crack widths of both versions of the Eurocode were compared for different exposure classes. Additionally, calculations for crack widths were performed for several different concrete covers, sectional heights, and reinforcement ratios, where the design moment is proportional to the reinforcement ratio. The concrete class of the examined cross-section was constant, giving a constant concrete strength of $f_{ck} = 35$ MPa. The calculations can be found in Appendix C.1 where the input parameters vary for the different scenarios.

9.5.1 Crack width limits

The maximum allowable crack widths in Gen 1 are based on values from national annexes. Recommendations from the Swedish Transport Agency, TSFS 2018:57 [15] (Table 12.2) were used. For Gen 2, no national annexes have yet been released as of the writing of this thesis. However, TSFS has proposed to use the same crack width limits in Gen 2 as in the current Eurocode, but including the factor k_{surf} [18]. Therefore, both the EC-recommended values and the TSFS-proposed values are investigated.

The limits apply for quasi-permanent load-combinations for a lifetime of 100 years, summarized in Table 9.1. The limits in Gen 2 can be increased using the factor k_{surf} , Eq. 9.4, for cases where the concrete cover exceeds the minimum cover for durability, $c_{min,dur}$, which depends on the exposure class.

$$k_{surf} = \frac{c_{act}}{10 \text{ mm} + c_{min,dur}} \leq 1.5 \quad (9.4)$$

Where c_{act} is the actual concrete cover, which may be larger than the nominal cover thickness because of detailing.

For example, when using exposure class XD2 with structural class 2, the concrete cover for durability becomes: $c_{min,dur} = 30$ mm. Using this, together with cover thickness of $c_{act} = [45, 55, 60]$ mm, the factor k_{surf} becomes:

$$k_{surf, XD2} = [1.125, 1.375, 1.5] \quad (9.5)$$

The resulting crack width limits in Gen 2 and Gen 1 are presented in Table 9.1 below, where all values from Gen 2 may be augmented with k_{surf} :

Table 9.1: Allowed crack width limits for durability, Gen 1 values are based on TSFS 2018:57 [15], with EC recommended values for Gen 2 according to SS-EN1992-1-1:2023 and the preliminary NDPs proposed by the Swedish Transport Agency [11].

Exposure class	100 years		
	Gen 1	Gen 2 Rec.	Gen 2 Prel. TSFS
XC2	0.4	0.3	0.4
XC3, XC4	0.3	0.3	0.3
XS1, XS2	0.2	0.3	0.2
XS3	0.15	0.3	0.15
XD1, XD2	0.2	0.3	0.2
XD3	0.15	0.3	0.15

For exposure class XD2, with concrete cover $c = 45$ mm, the crack width limits become:

Table 9.2: Allowed crack width limits for durability, XD2, with $c = 45$ mm

Exposure class	100 years		
	Gen 1	Gen 2 Rec.	Gen 2 Prel. TSFS
XC2	0.4	0.34	0.45
XC3, XC4	0.3	0.34	0.34
XS1, XS2	0.2	0.34	0.23
XS3	0.15	0.34	0.17
XD1, XD2	0.2	0.34	0.23
XD3	0.15	0.34	0.17

And for class XD2 with cover thickness $c = 60$ mm:

Table 9.3: Allowed crack width limits for durability, XD2, with $c = 60\text{mm}$

Exposure class	100 years		
	Gen 1	Gen 2 Rec.	Gen 2 Prel. TSFS
XC2	0.4	0.45	0.6
XC3, XC4	0.3	0.45	0.45
XS1, XS2	0.2	0.45	0.3
XS3	0.15	0.45	0.225
XD1, XD2	0.2	0.45	0.3
XD3	0.15	0.45	0.225

The relationship between allowed crack width and concrete cover was also investigated for exposure classes XD1 and XD2, visualized in Figure 9.8 below. The graph shows that the recommended method by Eurocode allows larger crack widths than the preliminary TSFS NDPs.

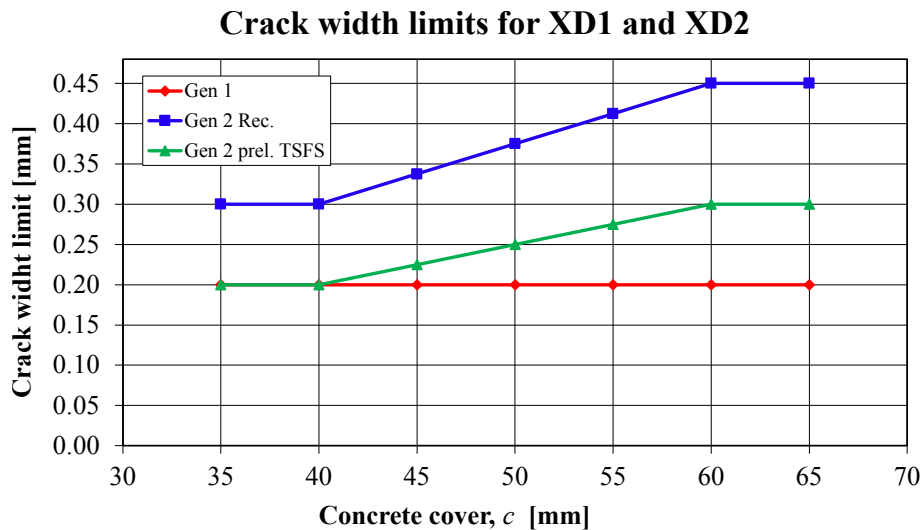


Figure 9.8: Crack width limits for both Gen 1 and Gen 2 recommended NDPs, and preliminary TSFS NDPs with exposure classes XD1 and XD2 for varying concrete cover

Tables 9.2 and 9.3 show that Gen 2 results in less conservative crack width limits when compared to the TSFS recommendations for the current Eurocode. This is further emphasized for sections where the thickness of the cover is greater than $c_{min,dur} + 10\text{ mm}$, where the crack width limits can be increased up to 50%. However, as shown in Figure 9.8 above, no additional favorable effect is achieved by increasing the concrete cover above 60 mm.

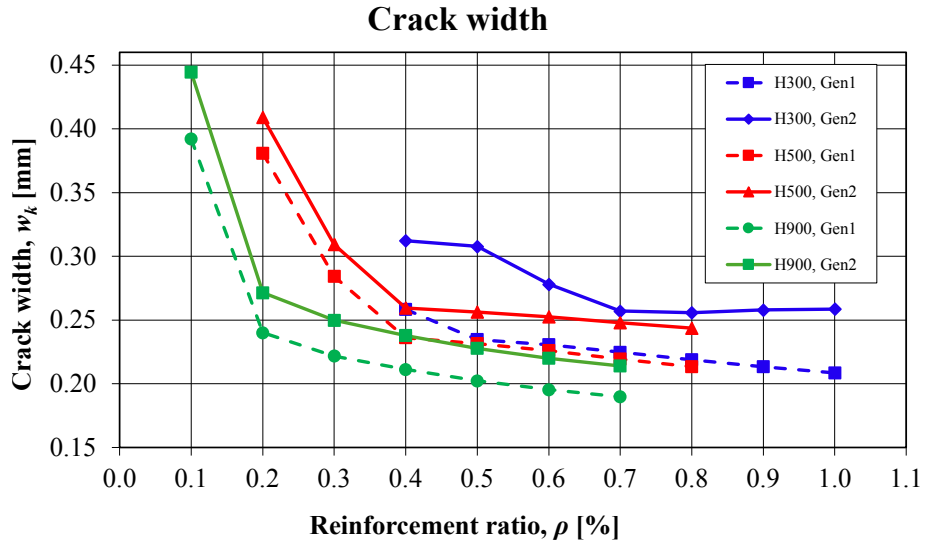
9.5.2 Influence of reinforcement ratio

The influence of the reinforcement ratio on the crack width was investigated for different sectional heights. Good bond conditions were assumed for a constant bar diameter of $\phi 16$ and a concrete cover of $c = 45$ mm corresponding to a slab in a road bridge in Sweden. Other necessary parameters are listed below:

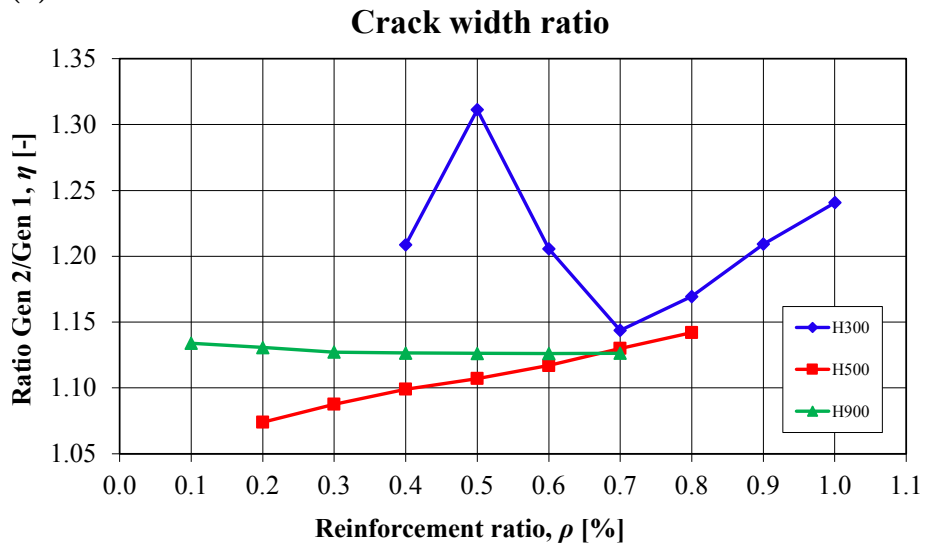
- Concrete cover, $c = 45$ mm
- Height of section, $h = [300, 400, 500, 900]$ mm
- Reinforcement diameter, $\phi = 16$ mm
- Reinforcement ratio ranging between $\rho \sim [0.1 - 1]\%$
- Bond conditions: Good

Calculations for crack widths were performed for a range of reinforcement ratios for the section shown in Figure 9.1. Since the bar diameter is constant, the reinforcement spacing was changed instead. For each reinforcement ratio, the design moment was set to 60% of the moment capacity in ULS. This was to avoid yielding of the reinforcement and to maintain a similar level of steel stress. Thus, the design bending moment is proportional to the reinforcement ratio. The resulting tensile stress in the reinforcing steel for each section was $\sigma_s = 240 - 260$ MPa (depending on reinforcement ratio), corresponding to a standard case in SLS.

The resulting crack widths are visualized in Figure 9.9, showing that it is evident that the estimated crack widths are greater in Gen 2 for most reinforcement ratios and sectional heights. The increase in crack width ranges from 7-20% for sections with a height of 400 mm or more. However, the crack widths are up to 30% higher for a shallow section with a high reinforcement ratio. The reason why H300 was only investigated for reinforcement ratios $\geq 0.4\%$ was that lower reinforcement ratios lead to unrealistically large bar spacings. The same can be said for H500 with reinforcement ratios $\geq 0.2\%$.



(a)



(b)

Figure 9.9: a) Crack widths, and b) crack width ratio, for varying heights and reinforcement ratios and good bond conditions with $\sigma_s \simeq 250$ MPa.

9.5.3 Influence of steel stress

The influence of steel stress was studied for the same section as in Figure 9.9. The variation of steel stress depends on the variation of the utilization ratio of the cross-section's moment capacity.

The input parameters are the following:

- Concrete cover, $c = 45$ mm
- Height of section, $h = [300, 500, 900]$ mm
- Reinforcement diameter, $\phi = 16$ mm
- Reinforcement spacing, $s = 100$ mm
- Moment resistance utilization, $\eta = [0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0]\%$
- Steel stress, $\sigma_s \approx 160 - 430$ MPa
- Bond conditions: Good

The difference in crack width was plotted for varying steel stresses, for three different sectional heights, visualized in Figure 9.10, showcasing that the crack width increases nearly linearly with increasing steel stress, for both versions of the Eurocode.

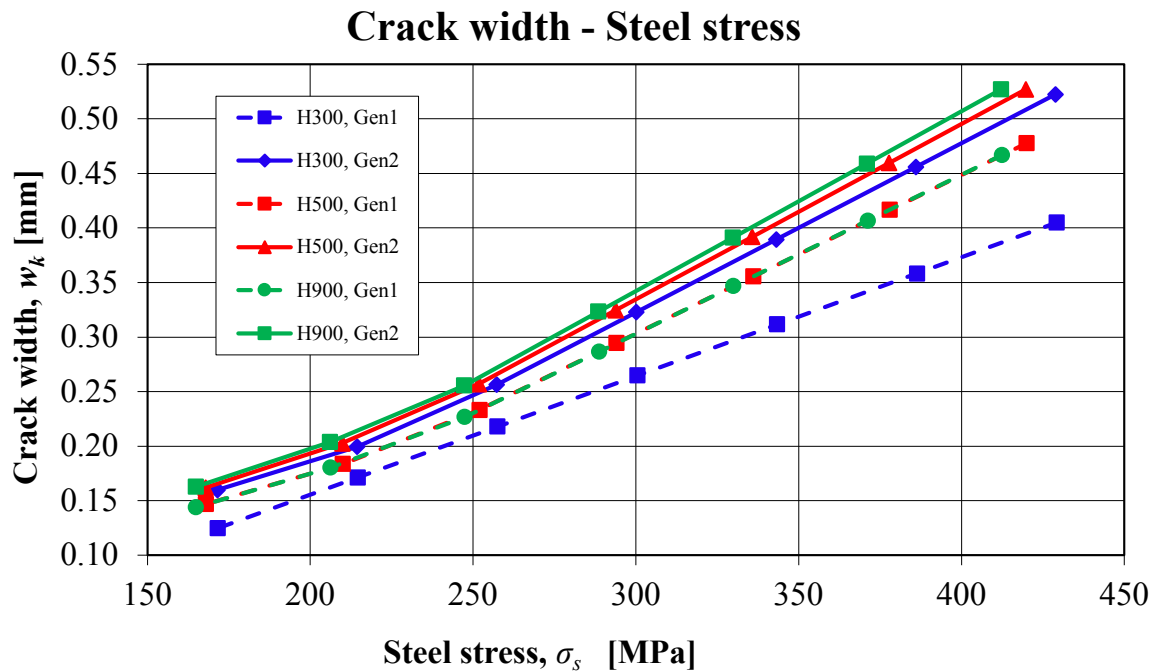


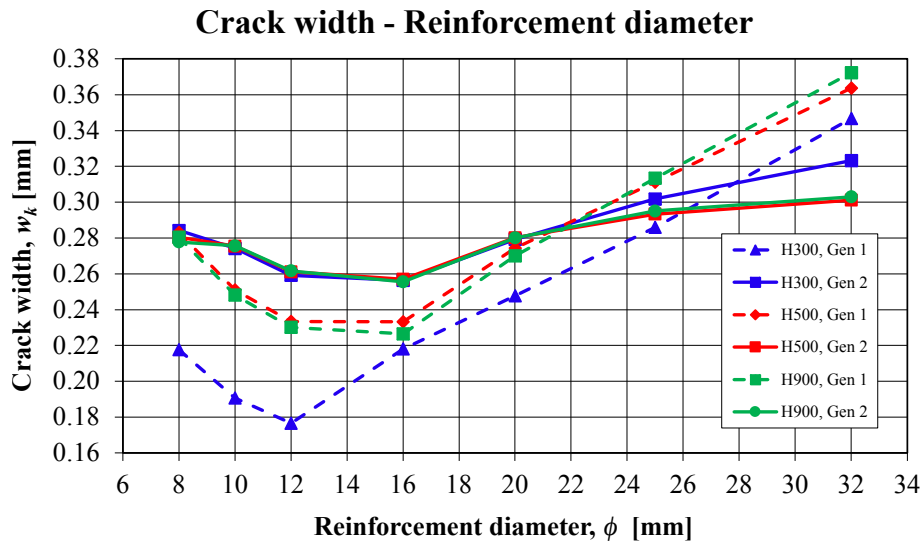
Figure 9.10: Crack width for different steel stresses and sectional heights in Gen 1 and Gen 2

9.5.4 Influence of reinforcement diameter

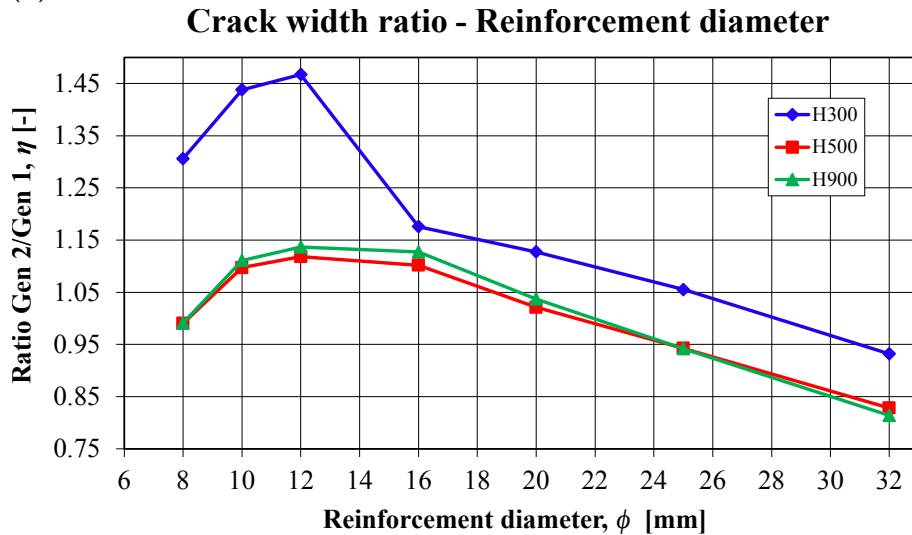
The differences in crack widths for different sectional heights and reinforcement diameters were also investigated. The same beam as in Figure 9.1 was studied, with constant reinforcement spacing, but with varying diameter. Thus, the reinforcement ratio increases with diameter. However, the steel stress was controlled by setting the design moment to 60% of the yield capacity, as explained in Section 9.5.2.

The following parameters were used:

- Concrete cover, $c = 45$ mm
- Sectional heights, $h = [300, 500, 900]$ mm
- Reinforcement diameter, $\phi = [8, 10, 12, 16, 20, 25, 32]$ mm
- Reinforcement spacing, $s = 100$ mm
- Design bending moment, $M_{Ed} = 0.6 \cdot d \cdot f_{yd} \cdot A_s$
- Steel stress, $\sigma_s = 236 - 249$ MPa (depending on reinforcement diameter)
- Bond conditions: Good



(a)



(b)

Figure 9.11: Crack widths for different reinforcement diameters

The crack widths are plotted for different concrete reinforcement diameters in Figure 9.11. The figure shows that the crack widths are higher in Gen 2 for diameters below $\phi 20$ mm, although the two codes follow a similar trend. The opposite is true for

diameters above 20 mm, where Gen 1 results in up to 15% wider cracks for sectional heights of 500 mm or more, and up to 45% wider cracks for a section with a height of 300 mm.

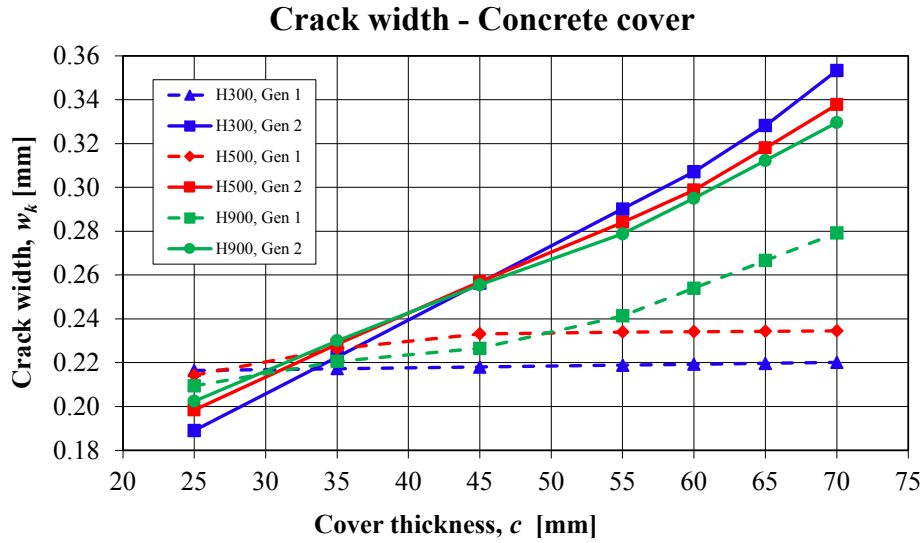
9.5.5 Influence of concrete cover thickness

The influence of the thickness of the concrete cover, c , was studied for the same section as in Figure 9.11 above, but with a diameter $\phi 16$ mm and a constant steel stress of $\sigma \simeq 250$ MPa and varying cover thickness. The total list of the input parameters is included below:

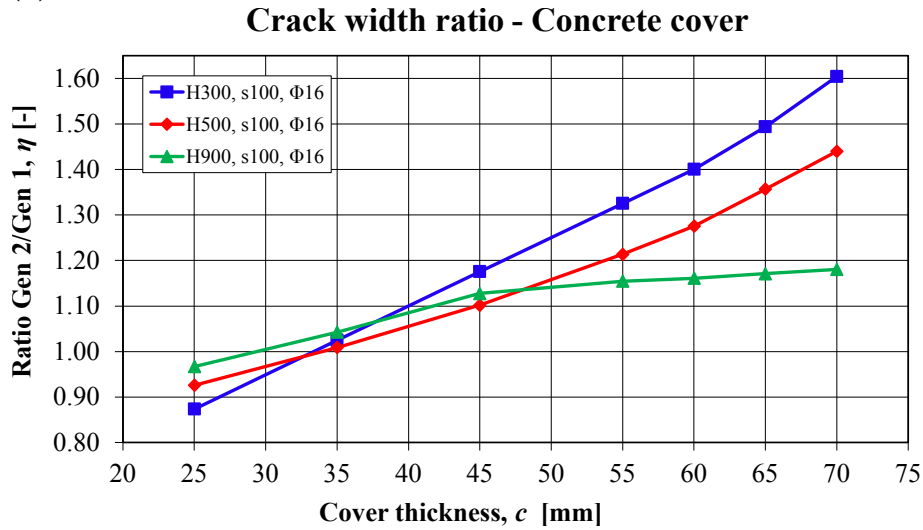
- Concrete cover, $c = [20, 25, 30, 40, 50, 55, 60]$ mm
- Sectional heights, $h = [300, 500, 900]$ mm
- Reinforcement diameter, $\phi = 16$ mm
- Steel stress, $\sigma_s = 247 - 259$ MPa
- Bond conditions: Good

The resulting crack widths were plotted for three different sectional heights in Figure 9.12 below. The crack widths in Gen 2 are significantly higher for larger concrete covers, with an increase of 20-60% for sections with large concrete covers. For such concrete covers, the allowable crack width can also increase by up to 50% with the factor k_{surf} . However, it should be noted that the crack widths continue to increase for concrete covers larger than 60 mm, which is the maximum thickness where k_{surf} provides a favorable effect. Thus, it may be unfavorable to increase the concrete cover beyond this point.

For thin concrete covers, such as $c = 30$ mm or less, the crack widths are instead observed to be similar, or lower compared to Gen 1.



(a)



(b)

Figure 9.12: Crack widths for different reinforcement diameters

9.5.6 Discussion

Based on the results shown in Figures 9.9, 9.11, and 9.12, it can be concluded that Gen 2 results in greater crack widths for many design situations, especially for high concrete cover thicknesses.

It should be noted that the crack widths are not entirely comparable between the two Eurocodes, since changes have been made to the allowable crack width limits. This means that the larger cracks in Gen 2 may be negated by the increase in crack width limits through the factor k_{surf} . This is further investigated in the case study, see Section 10.5.

9.6 Anchorage and laps of straight bars

The parametric study comparing the anchorage and lap length of Gen 1 and Gen 2 follows the method explained in Chapter 7. The scenario is based on the section in Figure 9.1 with straight bars. The study investigates the impact of the Gen 2 EC-recommended NDPs and the preliminary NDPs from the Swedish Transport Agency in relation to the already existing NDPs of SS-EN 1992-1-1:2005 for different reinforcement diameters. The input parameters used for the calculations are presented below:

- Concrete strength, $f_{ck} = 35$ MPa
- Concrete cover, $c = 45$ mm
- Reinforcement diameters, $\phi = [8, 10, 12, 16, 20, 25, 32]$ mm
- Reinforcement spacing, $s = 150$ mm
- Bond conditions: Good/Poor

The calculations can be found in Appendix D.1.

Gen 2 Recommended NDPs

The recommended values for NDPs in Gen 2 for anchorage and lap lengths are:

$$k_{1b.rec} = \begin{cases} 50 & \text{for transient design situations} \\ 35 & \text{for accidental design situations} \end{cases} \quad (9.6)$$

and

$$n_{\sigma.rec} = \frac{3}{2} \quad (9.7)$$

$$k_{1s.rec} = 1.2 \quad (9.8)$$

Gen 2 Preliminary NDPs proposed by the Swedish Transport Agency

The Swedish Transport Agency authorizes the code of statutes for Swedish infrastructure (TSFS). They have, through these codes [20], proposed modifications to the NDPs of Eurocode 2, SS-EN 1992-1-1:2023, with the following recommendations:

$$k_{1b.prel.TSFS} = \begin{cases} 40 & \text{for transient design situations} \\ 30 & \text{for accidental design situations} \end{cases} \quad (9.9)$$

and

$$n_{\sigma.prel.TSFS} = 1 \quad (9.10)$$

$$k_{1s.prel.TSFS} = 1.5 \quad (9.11)$$

However, these values are preliminary and may be changed in the future.

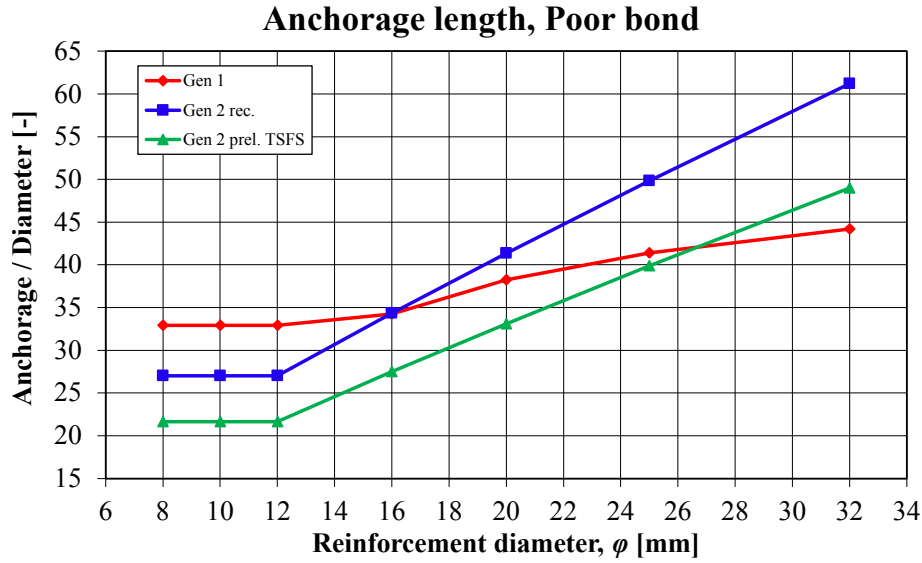
9.6.1 Anchorage length - Influence of reinforcement diameter

The design anchorage was calculated for a bar spacing of 150 mm, varying reinforcement diameter, and for concrete class C35/45. The design lengths in relation to the bar diameter are found in Figure 9.13.

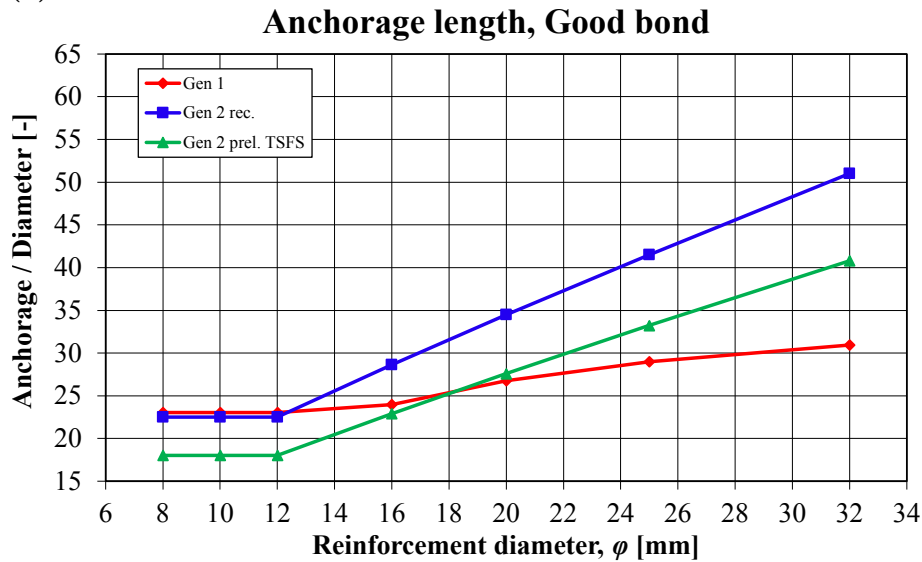
Figure 9.13a shows that the application of the recommended NDPs (blue line) in combination with poor bond conditions gives similar or lower anchorage lengths as Gen 1 (red line) for small reinforcement diameters, but the lengths differ significantly for larger bar diameters. When the preliminary TSFS NDPs are applied (green line), the anchorage lengths are reduced compared with the recommended NDPs. The design lengths using the preliminary NDPs are also shorter than in Gen 1 for the majority of investigated bar diameters, only resulting in larger anchorage lengths than Gen 1 once the bar diameter is larger than $\phi 25$.

A positive effect from the good bond conditions shown in Figure 9.13b results in all anchorage lengths being reduced. However, the positive effect of good bond conditions is less significant for Gen 2 while using the recommended NDPs. The recommended NDPs also result in longer anchorage lengths for larger bar diameters, which was also observed in the application of poor bond conditions. Using the preliminary NDPs proposed by TSFS results in shorter anchorage lengths than the recommended NDPs, and gives a better match to Gen 1. It should be noted that reinforcement spacings above 150 mm, such as 200 mm, result in the same anchorage lengths as that of 150 mm spacing (green line).

Results for other strength classes can be found in Annex D.3.1 and D.3.3. Recommendations for changes to the NDPs are presented in Section 9.6.5.



(a)



(b)

Figure 9.13: Design anchorage length/Diameter relationship for concrete class C35/45 with different NDPs using a) poor bond conditions and b) good bond conditions.

9.6.2 Discussion

The result from Figure 9.13 shows an overall increased anchorage length when comparing the two generations of Eurocode while using the recommended Gen 2 NDPs. The Swedish Transport Agency has proposed NDPs that counteract this increase in anchorage length in Gen 2, with the goal of making the curve more similar to Gen 1 for larger bar sizes. They argued that the recommended values of the anchorage length coefficients could result in a higher level of uncertainty for larger bar sizes based on information given in the background documents (mentioned in Section 7.3).

The Swedish Transport Agency, therefore, tailored its NDPs to fit the Gen 1 curve when a good bond is applied (Figure 9.13b), meaning that there is no distinction between bond conditions when applying the NDPs proposed by TSFS.

The consequence when a poor bond is applied is anchorage lengths that are significantly shorter in Gen 2 compared with the current values, especially for smaller bar diameters (Figure 9.13a).

The structural consequences of having such a large decrease in the anchorage for structures with poor bond conditions and small reinforcement diameters are not investigated in this thesis, but one can speculate that the application of using these preliminary NDPs could result in unsafe anchorage lengths.

In comparison, the recommended NDPs produce results that are much more similar to those of Gen 1 for diameters below $\phi 16$, which also appear justified in the background documents. A combination of the recommended NDPs and the preliminary TSFS factors to obtain the final Swedish NDPs could therefore be worthwhile investigating.

Strength class C25/30 and C55/67 were also investigated, considering anchorage length of straight bars. The graphs can be found in Appendix D.3.1 and D.3.3. A lower concrete strength class is shown to result in greater anchorage lengths for all investigated reinforcement diameters.

However, the ratio between Gen 1 and Gen 2 is shown to decrease for such concretes. The opposite is true for higher concrete strength classes, where the anchorage lengths in Gen 2 are even greater compared to Gen 1. Thus, higher concrete strengths are unfavorable compared to Gen 1 with regard to anchorage.

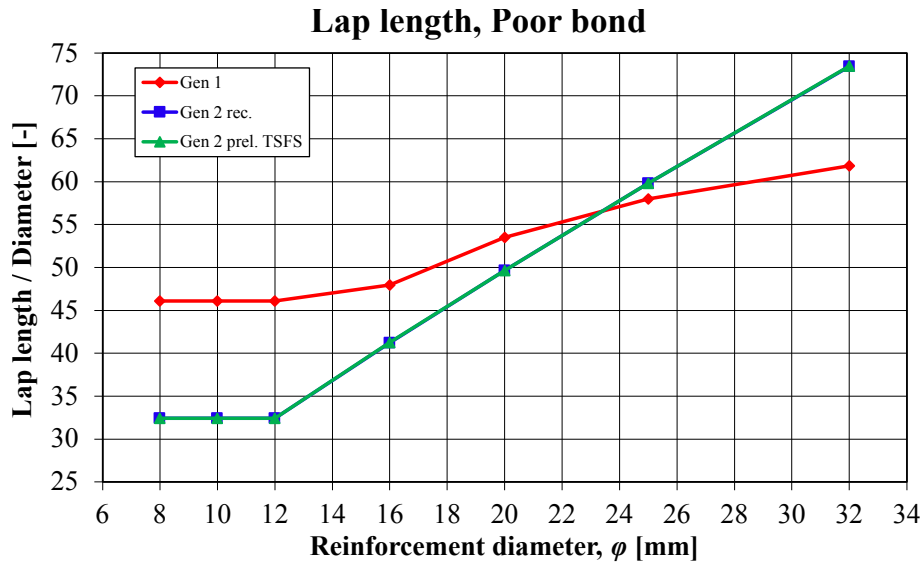
9.6.3 Lap length - Influence of reinforcement diameter

The lap length was calculated using the same premises as the anchorage lengths and the different NDPs provided above. The lap length in relation to the reinforcement diameter is provided in Figure 9.14.

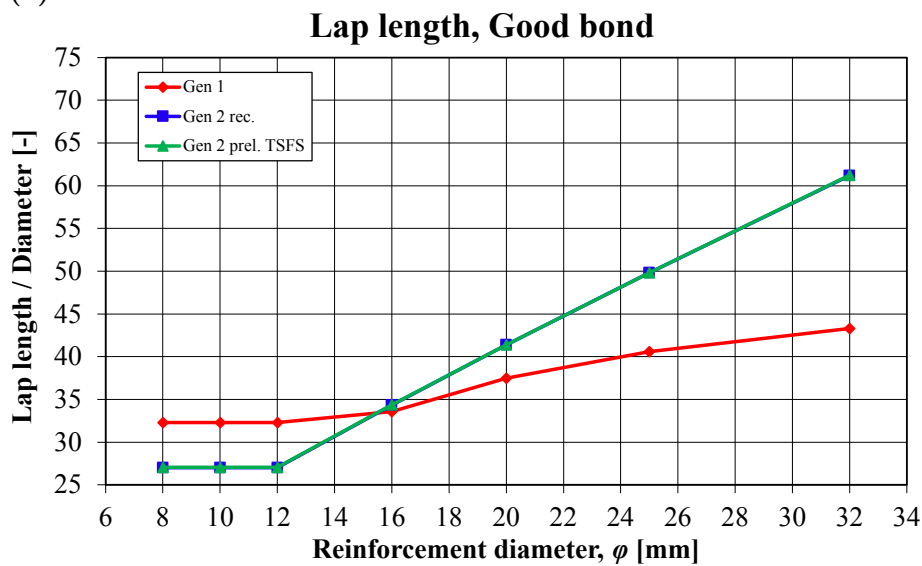
Figure 9.14a for the design lap length with poor bond conditions shows identical lengths when applying the recommended NDPs in comparison with the preliminary TSFS NDPs. For Gen 2, the lap lengths are up to 30% shorter for reinforcement diameters below 25 mm. For bar sizes equal or larger than 25 mm, the lap length is longer for Gen 2 for both types of NDPs, showing a 9% increase for $\phi 32$.

Likewise, as with poor conditions, the good-bond-condition-lap-lengths using the recommended NDPs are the same as when using the preliminary TSFS NDPs shown in Figure 9.14b. Furthermore, Gen 2 results in decreased lengths for diameters

smaller than 16 mm. The largest difference is for $\phi 12$ and smaller, resulting in 16% shorter lap length using the Gen 2 NDPs. For bar diameters larger than 16 mm, the lap length with the Gen 2 method increases compared to Gen 1, showing the largest difference (42%) for $\phi 32$.



(a)



(b)

Figure 9.14: Design lap length/Diameter relationship for concrete class C35/45 with different NDPs using a) poor bond conditions and b) good bond conditions.

Lap lengths for other strength classes can be found in Annex D.3.2 and D.3.4. Similar to anchorage lengths, a higher concrete class leads to a greater ratio between the lap length in Gen 2/Gen 1.

9.6.4 Discussion

The result from Figure 9.14, calculating the lap lengths for varying reinforcement diameters, shows that poor bond requires shorter lengths than previously assumed in Gen 1, except for bar diameters 25 mm and larger. For the smallest investigated bars (8-12 mm), the difference is greatest. For good bond, the differences are smaller, but diameters larger than 16 mm result in longer lap lengths than previously. The consequences of implementing the new generation of Eurocode could therefore be decreased material use for members requiring poor bond. The change of material use when a good bond is present is more uncertain. For bridge construction, which uses the larger reinforcement bars in the investigated range, one can assume an increased demand for steel. Meanwhile, house construction usually uses a smaller bar diameter, indicating a decreased material use.

As mentioned, the lap length for Gen 2 is identical even when applying the preliminary TSFS NDPs. That is due to the reason that the Swedish Transport Agency did not desire to change the lap length with the preliminary NDPs. They reasoned that calibration of the lap length coefficient is done using a sufficient foundation, including higher stresses and larger bar diameters - information was provided in the background documents (summarized in Section 7.3). The Swedish Transport Agency therefore argued that the resulting lap lengths are realistic for the desired range of reinforcement sizes.

As mentioned in the section for calculation of anchorage length above, concrete classes C25/30 and C55/67 were also investigated, the graphs can be found in Appendix D.3.2 and D.3.4. The results show that lower concrete classes show significantly shorter anchorage lengths for Gen 2 (for both recommended NDP and preliminary TSFS NDP) compared to Gen 1 when combined with poor bond conditions. For good bond conditions, the design lap length is generally shorter, unless for the larger examined reinforcement diameters ($\geq \phi 25$). Higher concrete classes show the opposite behaviour, with Gen 2 (both NDPs) having equal or much larger lap lengths for all investigated reinforcement bars.

9.6.5 Recommendation for changes to preliminary-TSFS-proposed-NDPs

Due to the potential unsafe anchorage lengths for small reinforcement diameters discussed above, this thesis recommends that the modified NDPs proposed by TSFS: k_{1b} and k_{1s} should only be applied for diameters greater than $\phi 16$ to avoid unsafe results. Therefore, these NDPs may be further modified according to:

$$k_{1b.mod} = 50 - 10 \cdot \frac{(\phi - \phi_b)}{\phi_b} \quad \text{for } \phi \geq \phi_b \text{ mm} \quad (9.12)$$

where

$$\phi_b = \begin{cases} 12 \text{ mm} & \text{for good bond conditions} \\ 16 \text{ mm} & \text{for poor bond conditions} \end{cases} \quad (9.13)$$

Equation 9.12 above assumes values of $k_{1b} = 50$ for diameters below $\phi 16$ mm, and is reduced for larger diameters. This is intended to result in safe anchorage lengths for smaller diameters while reducing the anchorage for larger bars, which is more in line with the model from Gen 1.

Similarly, k_{1s} (lap length NDP) may be modified following:

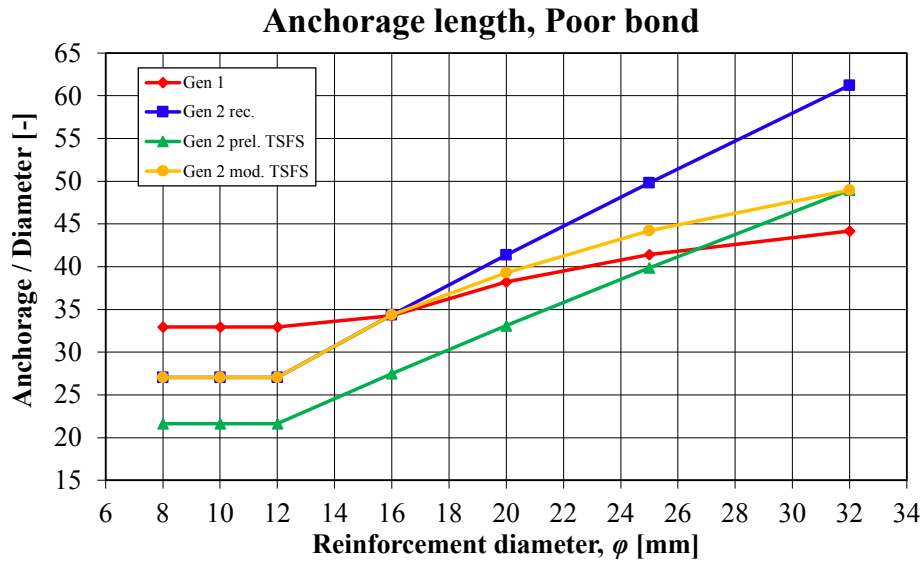
$$k_{1s.mod} = \frac{50 \cdot 1.2}{k_{1b.mod}} \quad (9.14)$$

which starts at 1.2 and assumes a value of 1.5 for $k_{1b} = 40$ (as recommended by TSFS) for $\phi 32$. This change is solely to adapt the lap lengths due to the change of the anchorage coefficient, the same method the Swedish Transport Agency utilized as they created the preliminary TSFS NDPs, to ensure that the lap length follows the recommended Gen 2 curve.

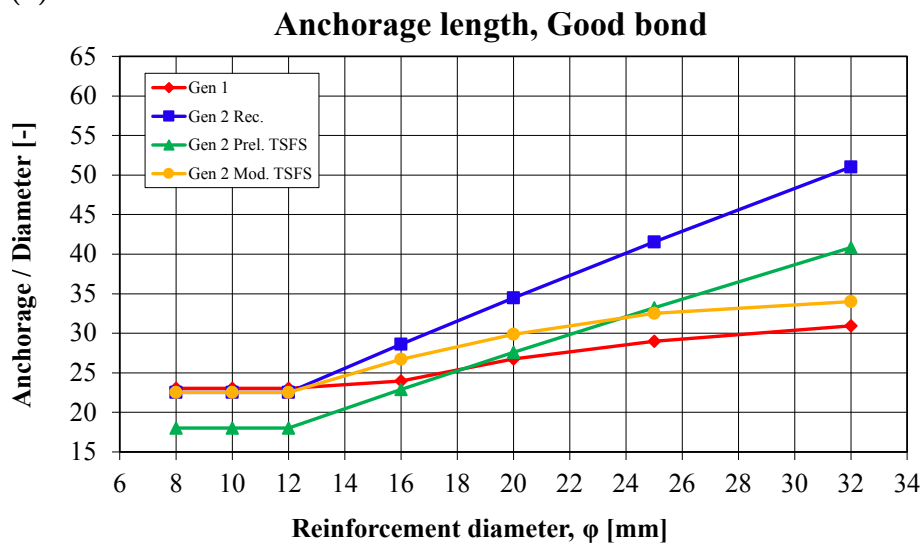
The value for n_σ recommended by TSFS may also be adopted:

$$n_{\sigma.mod} = 1 \quad (9.15)$$

The anchorage lengths obtained from the proposed NDPs mentioned above are visualized in Figure 9.15 below.



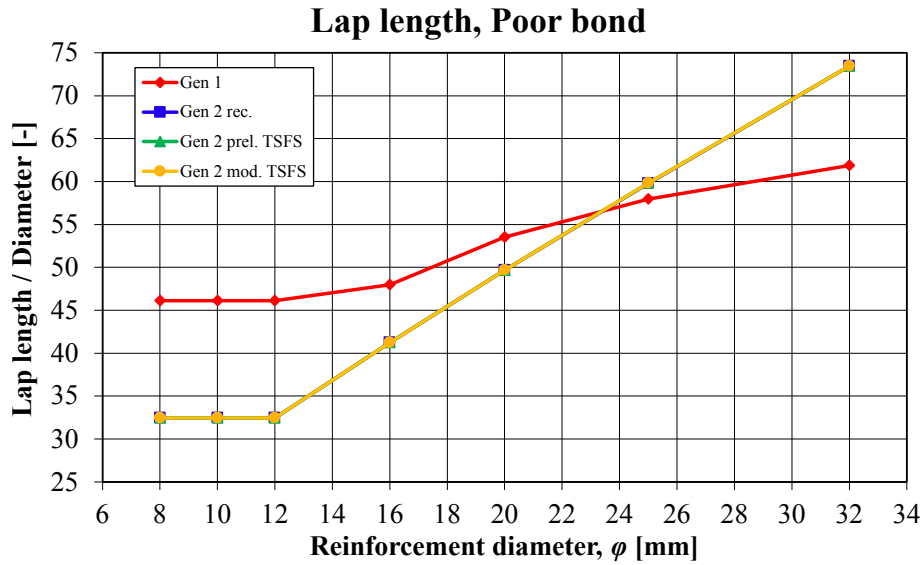
(a)



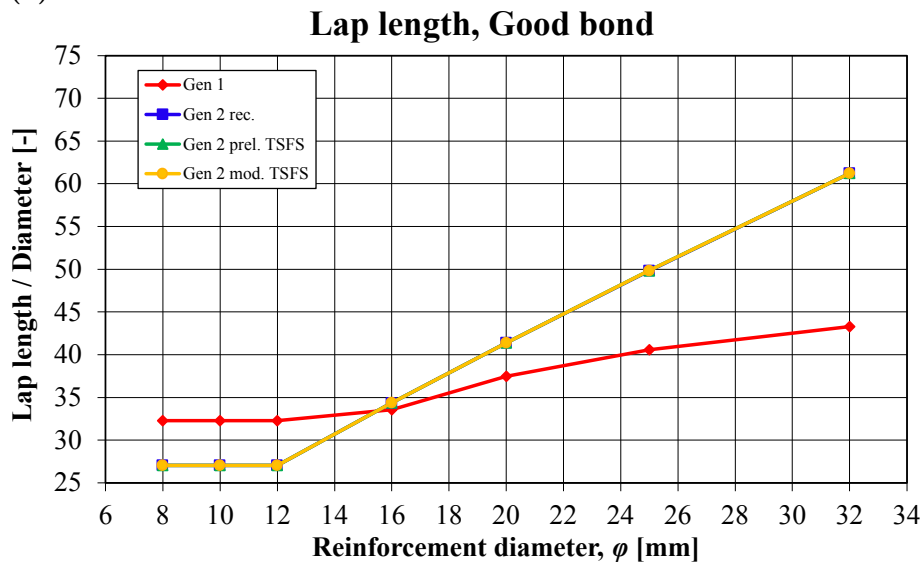
(b)

Figure 9.15: Design anchorage length/Diameter relationship for concrete class C35/45 with different NDPs (including the modified TSFS NDPs) using a) poor bond conditions and b) good bond conditions.

As observed in Figure 9.15, the modified TSFS NDPs (yellow line) combine the values of the recommended NDPs (blue line) and the preliminary TSFS NDPs (green line), resulting in anchorage lengths that start similar to the recommended values, and as the reinforcement diameter increases, changes to follow the curve of the preliminary TSFS NDPs. Through this modification, the potential underestimation of anchorage length for smaller bar diameters present with the previous TSFS method is eliminated, especially obvious when poor bond was applied (Figure 9.15a). Additionally, the change removes the uncertainties with the longer anchorage lengths that were present when the bar size increased, most prominently shown in the case of good bond conditions (Figure 9.15b).



(a)



(b)

Figure 9.16: Design anchorage length/Diameter relationship for concrete class C35/45 with different NDPs (including the modified TSFS NDPs) using a) poor bond conditions and b) good bond conditions.

Figure 9.16 shows that the modified TSFS NDPs result in the same lap lengths as the preliminary TSFS NDPs, which are also the same as the Gen 2 recommended NDPs.

9.6.6 Including the effect of confinement

As observed in the studies performed above, shown in Figures 9.15 and 9.13, the anchorage lengths are higher in Gen 2 than in Gen 1 for large bar diameters, even when using modified NDPs. In order to further reduce the anchorage lengths, the effect of confinement can be taken into account. This effect has been reworked in Gen 2, taking into account the beneficial effect of transverse reinforcement [31]. It is therefore interesting to examine the effect of this change on the anchorage lengths.

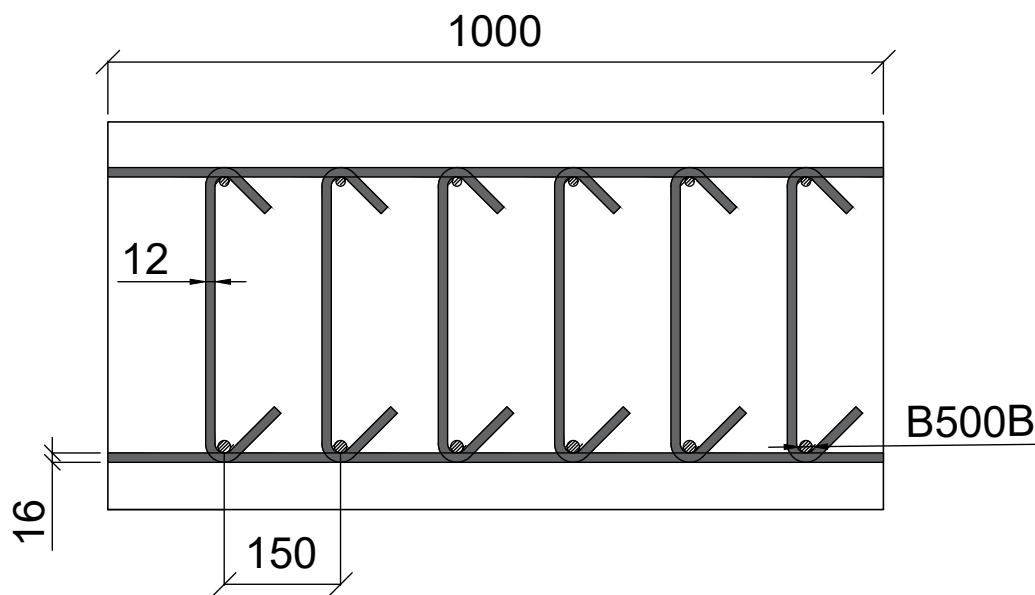
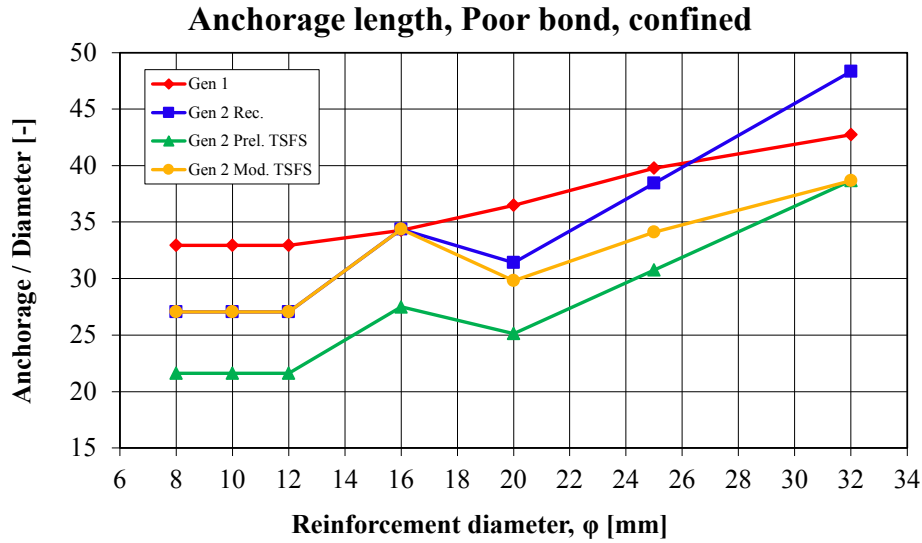


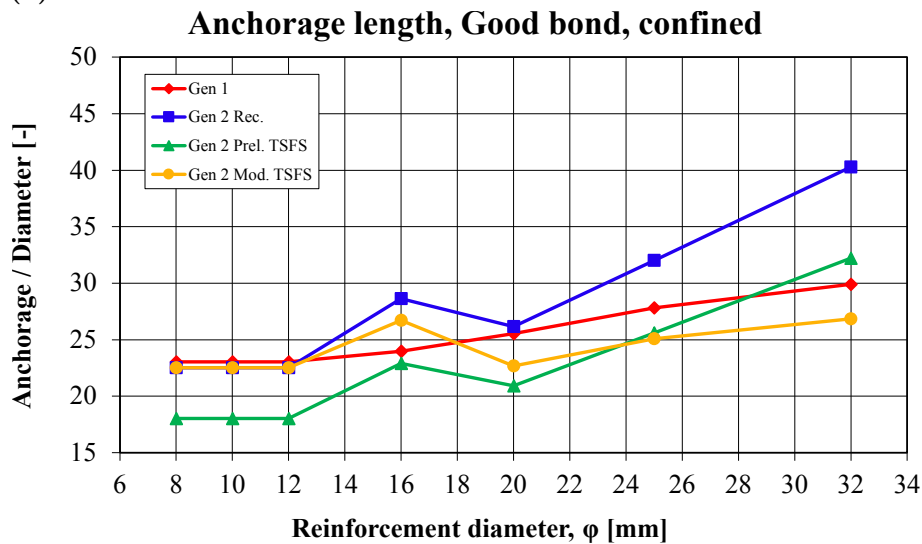
Figure 9.17: Detailing of confinement reinforcement

For the confinement study, confinement reinforcement with a diameter $\phi 12$ mm was used, with a transverse spacing of $s = 150$ mm (the spacing of flexural reinforcement to be anchored), see Figure 9.17. The effect of confinement was only included for reinforcement diameters of 20 mm or larger, whose size normally requires to be enclosed, therefore only providing a reduction of anchorage and lap length for these larger bars.

The calculation of the anchorage length including the confinement effect is shown in Figure 9.18. Comparing the Gen 2 recommended NDPs with Gen 1 shows that both bond conditions show similar behavior as was observed when ignoring the confinement effect, giving longer anchorage for larger reinforcement diameter.



(a)



(b)

Figure 9.18: Design anchorage length/Diameter relationship including confinement effect for concrete class C35/45 with different NDPs using a) poor bond conditions and b) good bond conditions.

For the preliminary TSFS NDPs and poor bond (Figure 9.18a), the confinement effect on the larger bars reduces the lengths noticeably, resulting in shorter anchorage lengths than Gen 1 for all investigated bar sizes, potentially providing the risk of unsafe anchorage. The modified TSFS NDPs proposed by the authors of this thesis provide anchorage that is slightly shorter than the current Eurocode for $\phi 20 \leq$.

Applying good bond conditions, as shown in Figure 9.18b, also shows similar behavior as when the confinement effect was excluded, with the recommended NDPs showing the longest anchorage for the largest investigated bar diameters. The preliminary NDPs proposed by the Swedish Transport Agency result in similar anchorage to

Gen 1, providing slightly shorter anchorage lengths for the largest examined bar size compared to what they did when ignoring the confinement effect. The modified TSFS NDPs show a more similar match to the Gen 2 recommended NDPs for reinforcement between 16-20 mm than the preliminary TSFS ones.

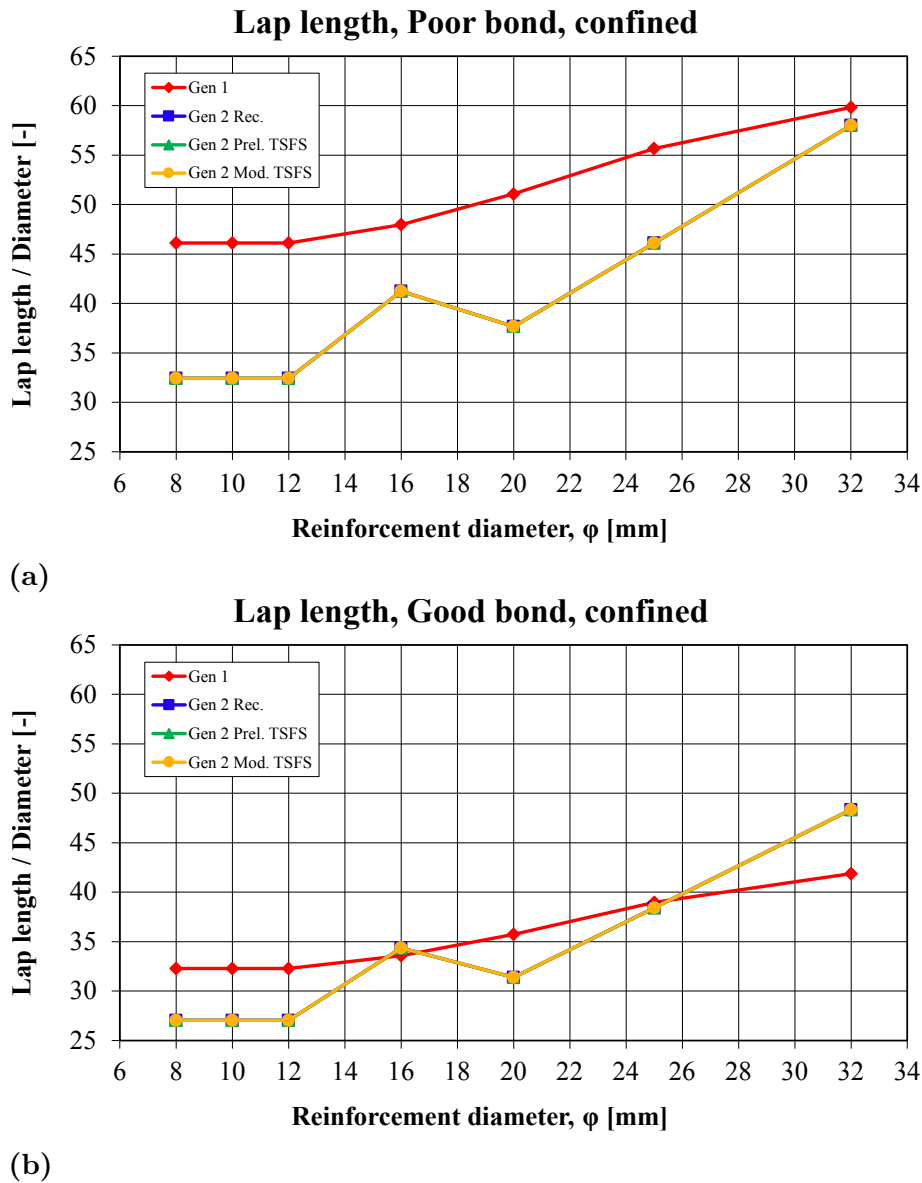


Figure 9.19: Design lap length/Diameter relationship including confinement effect for concrete class C35/45 with different NDPs using a) poor bond conditions and b) good bond conditions.

Figure 9.19 shows the result from the calculation of lap length, using the different NDPs. The confinement effect can be observed to decrease the design lap length of bars equal to or larger than $\phi 20$. Since the Gen 2 recommended NDP for lapping was considered satisfactory, the preliminary TSFS NDP and, therefore, also the modified NDP are all identical to one another. Poor bond (Figure 9.19a) shows that

reinforcement sizes below 28 mm provide shorter lap lengths, up to 30% decrease for the smallest diameters of 8-12 mm compared to Gen 1. The conclusion that the lap lengths required for Gen 1 are, in general, shorter than they were for Gen 1 is the same as when calculating lapping without confinement effect. Good bond conditions (Figure 9.19b) show similar lap lengths when using bar diameters 16-20 mm, where smaller bars require shorter lengths and larger bars have increased lap lengths compared to Gen 1.

10

CASE STUDY

A case study was conducted to investigate the impact of changes to the Eurocode, applied to a real bridge scenario. This was done for an existing frame bridge in Sweden, designed with Gen 1 of the Eurocode. A frame bridge is the most common bridge type used in Sweden [24], making the comparison important for future reference and design. Critical sections of the bridge were identified, where bending moment and shear capacities, crack widths, and anchorage lengths were calculated using the new Eurocode. These values were then compared with the corresponding values from Gen 1. It should be noted that no structural analysis of forces and stress distributions was performed. Changes in other parts of Gen 2 of the Eurocode that influence loads, load combinations, and analysis are therefore not compared in this study. The sectional capacities were instead calculated based on the existing geometry of the bridge, with a conservative estimate of 100% utilization ratio.

10.1 Bridge description

The bridge "Bro 14-2008-1" was used for the case study, which is a reinforced concrete frame bridge constructed in 2013 that spans the road E6 between the Pålen and Tanumshede area in the Bohuslän region of Sweden. Details and drawings of the bridge were provided with the courtesy of Mikael Hermansson at Norconsult Sverige. The bridge was designed for a lifetime of 120 (L100) years and was constructed to allow wildlife to pass beneath the bridge. The location of the bridge is provided in Figure 10.2.



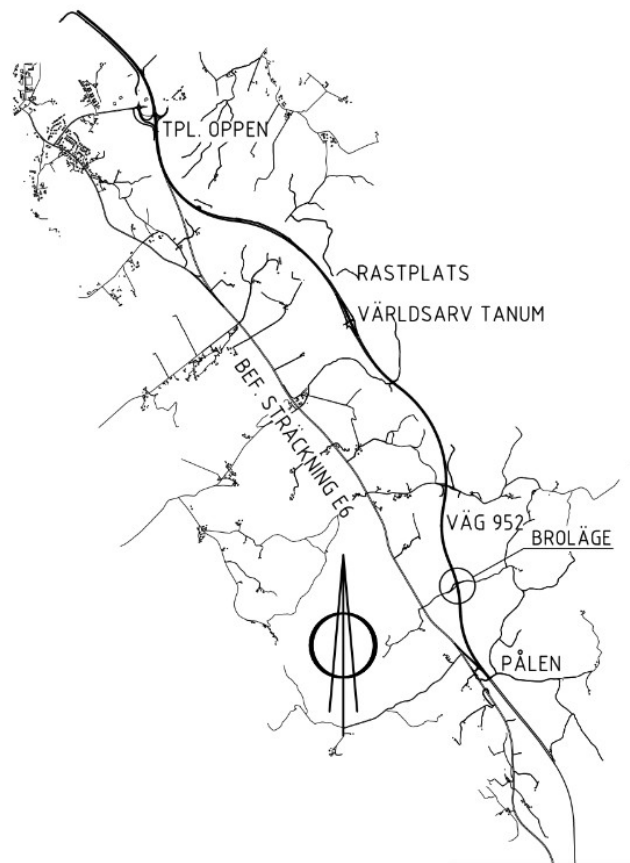


Figure 10.2: Location of the studied bridge

10.1.1 Geometry

The bridge consists of a reinforced concrete slab with a free span of 12 m and a width of 20.5 m. The height varies over the length of the span, with the height $h = 830$ mm at the support and $h = 452$ mm in the mid section, visualized in Figures 10.3 and 10.4.

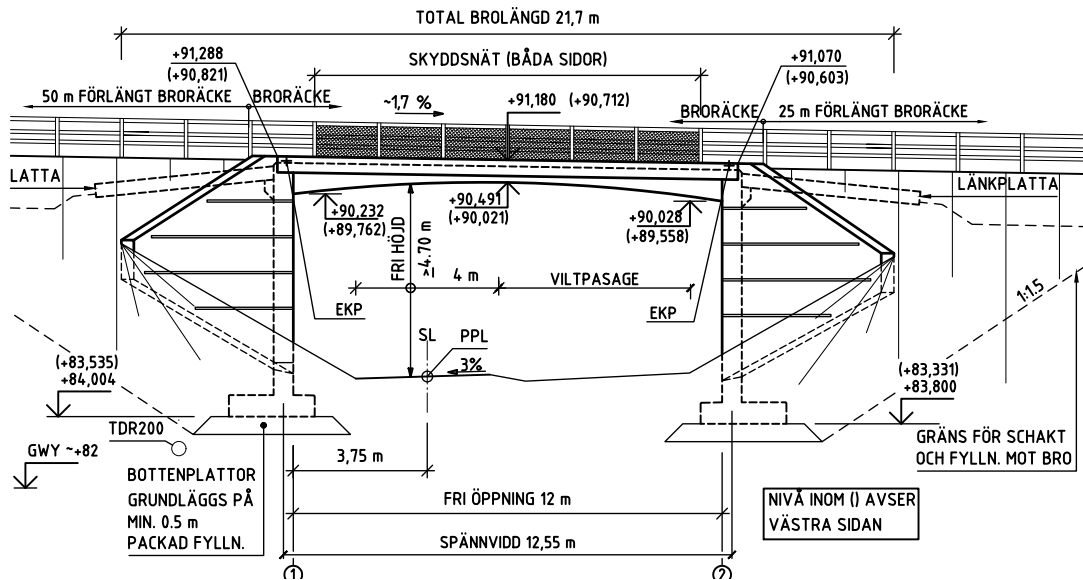


Figure 10.3: Bridge span viewed from side

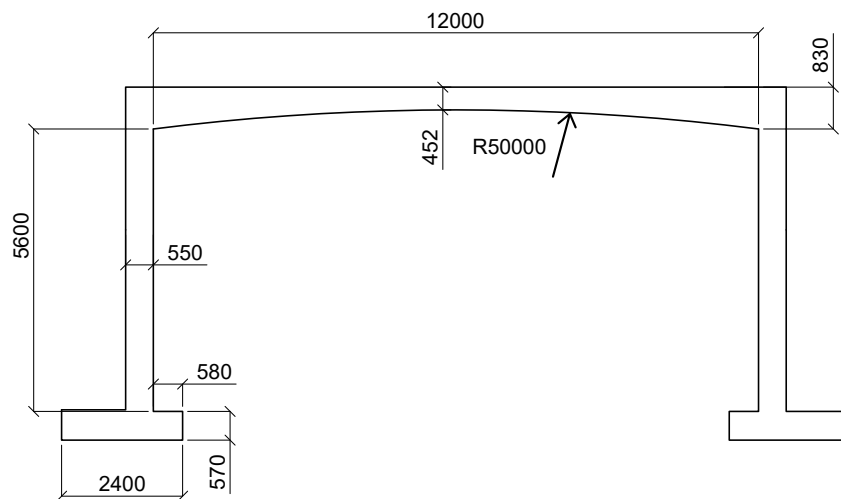


Figure 10.4: Model of bridge span and height dimensions in [mm]

10.1.2 Material

Concrete class C35/45 is used, with an exposure class of XD1 and XF4. Class B500B was used for the reinforcing steel, with bar diameters $\phi 20$, $\phi 16$, and $\phi 10$, depending on the section studied. The slab also has a clear concrete cover of 40 mm. The material properties are summarized in Table 10.1 below:

Table 10.1: Material properties of the bridge

Material	Strength [MPa]	Elasticity [GPa]	Partial factors
C35/45	$f_{ck} = 35$ $f_{ck,ref} = 40$ $f_{cm} = 43$ $f_{ctm} = 3.2$	$E_{cm} = 34$ $k_e = 9500$	$\gamma_c = 1.5$ $\gamma_v = [1.4; 1.29]$ $\alpha_{cc} = 1$ $k_{tc} = 1$
B500B	$f_{yk} = 500$	$E_s = 200$	$\gamma_s = [1.15; 1.03]$

10.1.3 Long term effects

The long-term effects were calculated for $t = 100$ years (lifespan of the bridge). The time of loading was assumed to be $t_0 = 5$ days, and the drying was assumed to begin at $t_s = 0$ days. Regular CEM-1 cement was used, class CN (normal hardening rate). The drying shrinkage and creep were modeled to only take place at the bottom face of the slab, with all other faces considered covered. The full calculations for long-term effects can be found in Appendix C.2.

The resulting creep coefficients after 100 years are:

$$\varphi = \begin{cases} 1.88 & \text{Gen 1} \\ 2.06 & \text{Gen 2} \end{cases} \quad (10.1)$$

and the shrinkage are strains:

$$\varepsilon = \begin{cases} 2.27 \cdot 10^{-4} & \text{Gen 1} \\ 2.17 \cdot 10^{-4} & \text{Gen 2} \end{cases} \quad (10.2)$$

10.2 Critical sections

The bridge can be divided into a mid-strip and an edge strip, with different reinforcement content and detailing. Based on this, three different critical sections were identified concerning moment, shear, crack formation, and anchorage. The locations of the critical sections are visualized in Figure 10.5 below.

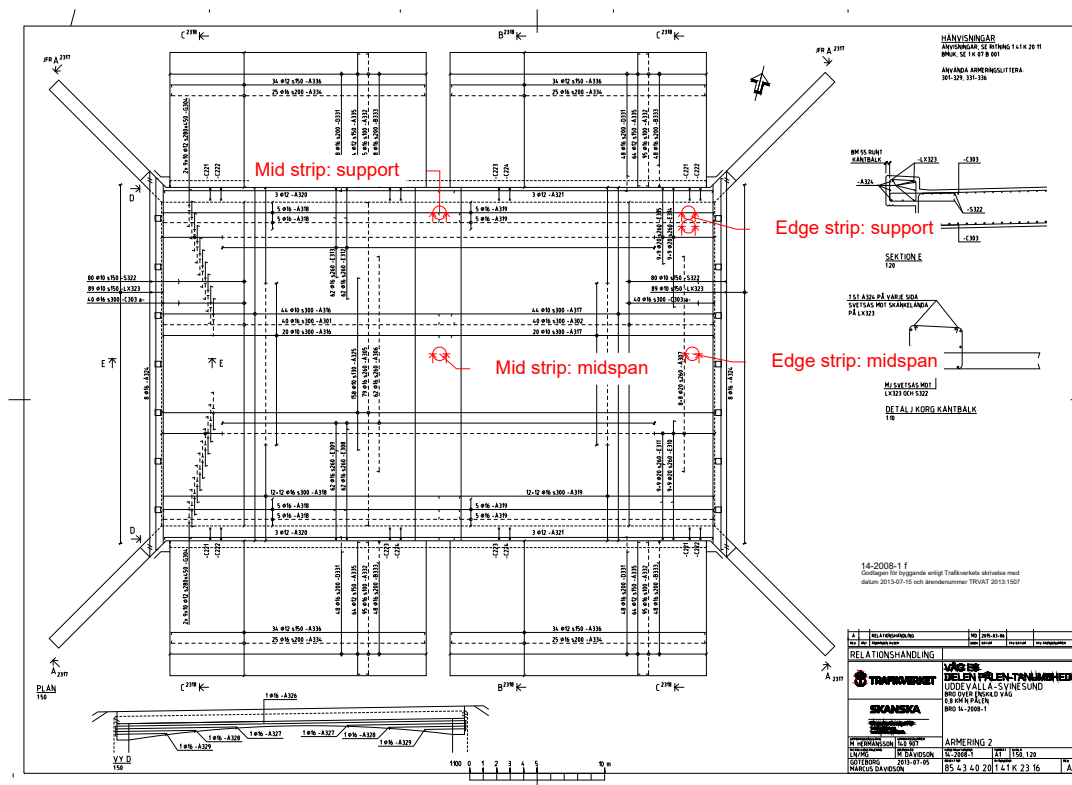


Figure 10.5: Bridge deck viewed from above, with the critical sections marked in red

10.2.1 Support sections

The sections supporting the slab are tensioned at the top and under compression at the bottom, and can be divided into a mid strip and an edge strip with different reinforcement arrangements. The mid strip does not have shear reinforcement, while the edge strip contains stirrups, visualized in Figures 10.6 and 10.7 below.

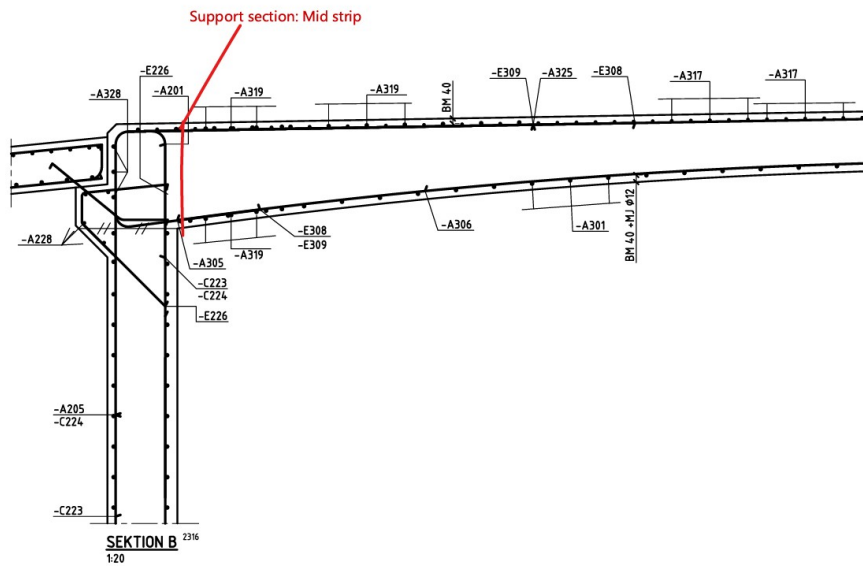


Figure 10.6: Location of support section of the mid strip

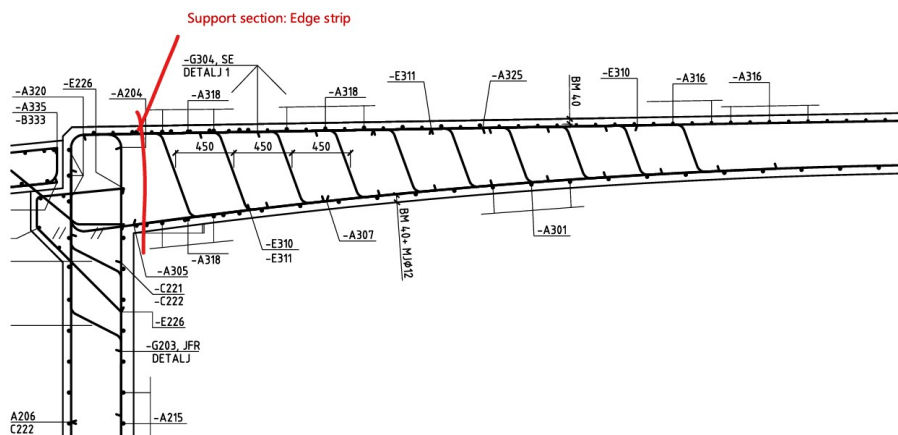


Figure 10.7: Location of support section of the edge strip

Mid strip

The critical section in the mid strip is visualized in Figure 10.8 below, where the tensioned reinforcing steel has a diameter of 16 mm with a spacing of $s = 130$ mm, with no shear reinforcement. The bottom reinforcement has a spacing of 260 mm. Additional transverse reinforcement encloses the main reinforcement on both the top and bottom, with a diameter of $\phi = 16$ mm and spacing $s = 300$ mm. The bottom of the section also includes mounting irons with a diameter of 12 mm. Thus, the cover thickness at the top of the slab is:

$$c_{top} = 40 + 16 = 56 \text{ mm} \quad (10.3)$$

and in the bottom:

$$c_{bot} = 40 + 16 + 12 = 68 \text{ mm} \quad (10.4)$$

The geometry of the slab in the mid strip at the support section of the bridge is visualized in Figure 10.8 below.

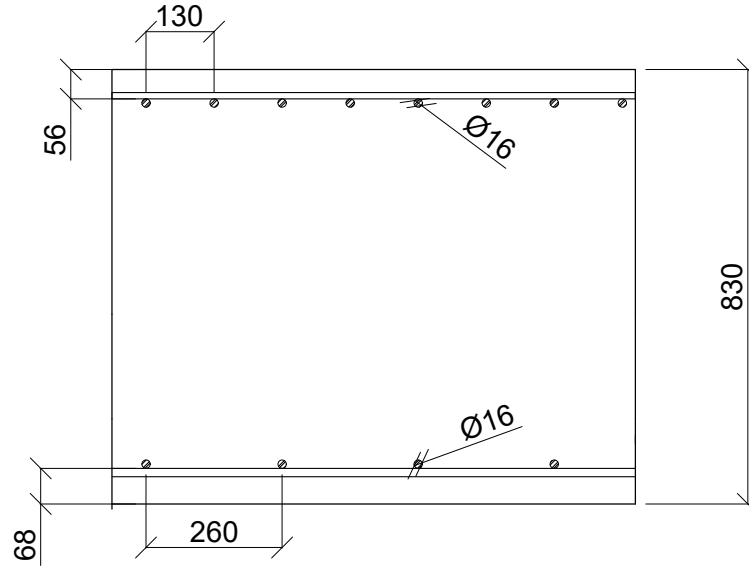


Figure 10.8: Geometry of slab, mid strip

Edge strip

The edge strip of the slab has a different reinforcement layout than the mid strip. The top of the section is under tension, with reinforcement of diameter $\phi = 20$ mm and spacing $s = 130$ mm. The bottom of the section is under compression with bars of diameter 16 mm with a spacing of 260 mm. Stirrups are used, with a diameter $\phi_w = 12$ mm, 70 degree inclination, and with longitudinal and transverse spacing of $s_l = 450$ and $s_t = 280$ mm, respectively. The shear reinforcement is distributed over a width of 2.25 m and spans 5 m from the support on both sides.

The geometry of the edge strip in the support section of the bridge is visualized in Figure 10.9 below. The cover thickness is the same as for the mid strip.

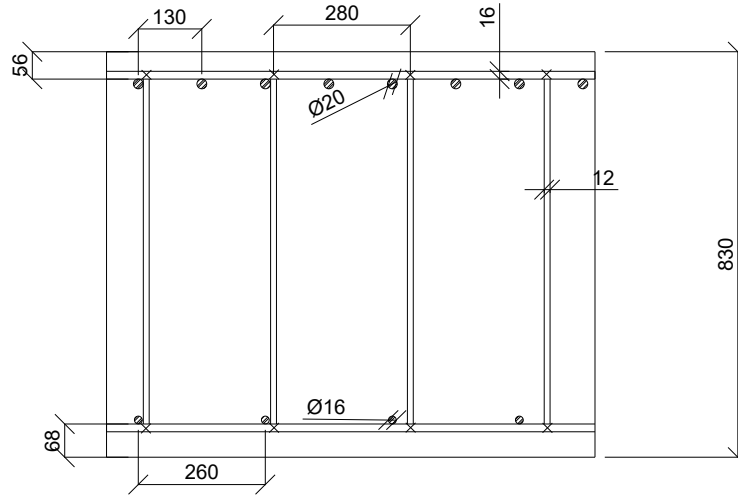


Figure 10.9: Support section of edge strip

10.2.2 Midspan sections

The reinforcement detailing of the midspan sections of the bridge is different from that at the supports and contains no shear reinforcement.

Edge strip

The mid section of the edge strip has a similar height to the mid strip, but with a larger reinforcement diameter. The slab is under tension at the bottom of the slab, and compression at the top. The height in the midsection is 452 mm. The compressed reinforcement consists of $\phi 10$ bars with a spacing of 130 mm, while the tensile reinforcement is $\phi 20$ mm in diameter with spacing 260 mm and $\phi 16$ with spacing 260 mm (both in the same layer). Thus, actual spacing is $s = 130$ mm. The longitudinal and transverse reinforcement is the same as for the support section of the edge strip, declared in Chapter 10.2.1. However, since the verification of the mid-section is made on the bottom side of the section, the nominal cover thickness is:

$$c_{nom} = 40 + 16 + 12 = 68 \text{ mm} \quad (10.5)$$

since it includes the reinforcement used for mounting (monteringsj rn), which has a diameter of 12 mm. The cross-section is shown in Figure 10.10 below.

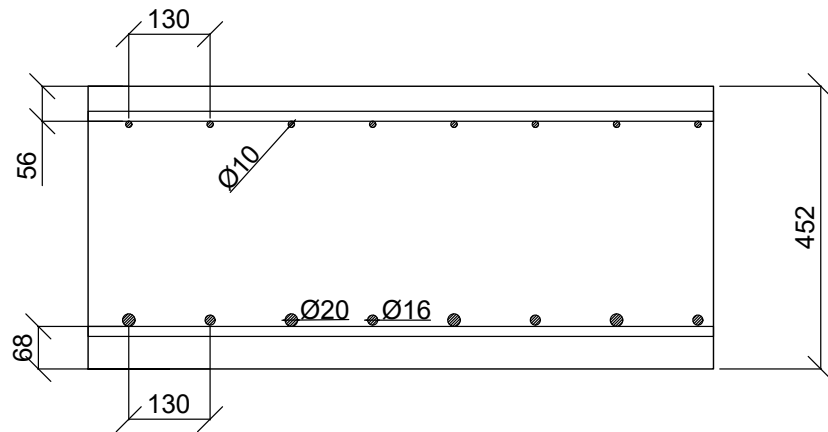


Figure 10.10: Midspan section of edge strip

Mid strip

The mid section of the mid strip has identical input parameters as the edge strip defined in Figure 10.10, **except** that the tensile reinforcement consists solely of $\phi 16$ with a spacing of $s = 130$ mm.

10.3 Bending

All four critical sections were investigated, considering the calculation of the bending moment resistance, to thoroughly examine the capacity. The support section at the edge strip is especially important since it is more heavily reinforced, thus having a higher stiffness, with greater forces and moments. The design moment resistance was calculated following Chapter 4, and is clarified in Appendix A.

10.3.1 Support sections

The calculation for the moment capacity in the support section of the edge strip and mid strip can be observed in Appendix A.3. It should be noted that the geometry is the same in both sections, except for the tension reinforcement diameter, which is 16 mm in the mid strip and 20 mm in the edge strip.

The calculations for the edge strip resulted in the values presented in Table 10.2:

Table 10.2: Negative bending moment resistance at support section

	M_{Rd} [kNm]
Edge strip	
Gen 1	781
Gen 2	848
Difference [%]	8.5
Mid strip	
Gen 1	516
Gen 2	555
Difference [%]	7.5

Thus, the bending capacity in Gen 2 is 8.5% higher in the edge strip, and 7.5% higher in the mid strip. This means that the geometry of the sections could be modified to save material.

As a result of the higher capacity in Gen 2, the height of the section can be reduced by 50 mm to $h = 780$ mm, resulting in a similar moment capacity as in Gen 1.

10.3.2 Midspan sections

The calculation procedure can be found in Appendix A.3, and the resulting bending moment capacities of the mid sections are presented in Table 10.3.

Table 10.3: Positive bending moment resistance at mid section

	M_{Rd} [kNm]
Edge strip	
Gen 1	308
Gen 2	325
Difference [%]	5.6
Mid strip	
Gen 1	247
Gen 2	259
Difference [%]	4.7

Similarly to the capacity near the supports, it is possible to reduce the height of the section in Gen 2 by 12 mm, reaching a new height of $h = 440$ mm while reaching a similar bending capacity as in Gen 1.

10.3.3 Discussion

The calculation using the methodology of Gen 2 results in larger moment resistances than using Gen 1. This follows the behavior which was observed in the parametric study (Chapter 9), where deeper sections were observed to result in a greater increase in moment capacity. The edge strip has a higher capacity ratio since the reinforcement diameter is higher there, benefiting from the improved partial factor for steel.

10.4 Shear

To calculate the design shear force capacity of the bridge slab, two critical sections were determined: the support of the mid strip (without shear reinforcement), as well as the support of the edge strip (with stirrups). The calculation of the design shear resistance follows the principles of Chapter 5, details can be found in Appendix B.

10.4.1 Support sections

The shear capacities of both critical sections were calculated, see Section 10.2.1. For the mid-strip, since it has no stirrups, the shear force capacity of the section was determined without considering shear reinforcement.

The calculations are presented in Appendix B.2. The values of the design shear force capacity for the edge and mid strip at the support section are shown in Table 10.4.

Table 10.4: Shear force resistance at support section

	V_{Rd} [kN]
Edge strip	
Gen 1	722
Gen 2	787
Difference [%]	9
Mid strip	
Gen 1	295
Gen 2	340
Difference [%]	15

Based on the result from the edge strip, where Gen 2 results in a higher capacity, there is a possibility to reduce the height of the section with regard to shear capacity to $h = 770$ mm as opposed to the current height of $h = 830$, resulting in the same shear capacity as in Gen 1. This is a reduction of 60 mm.

Simultaneously, the result from the mid strip indicates that the height of the critical section with regard to shear could be reduced to $h = 650$ mm (180 mm reduction), while still maintaining the same shear capacity as in Gen 1. However, doing so would also decrease the moment capacity too much, making it unrealistic in this case.

10.4.2 Discussion

The resulting shear capacities are higher in Gen 2 for both critical sections that were studied. The difference is greatest in the middle strip, with an 18% higher shear capacity in Gen 2. This increase in capacity without stirrups is what was expected based on the parametric study.

For the section located in the edge strip, the increase in shear capacity is smaller, with an increase of 9% compared to the capacity from Gen 1. This is also in line with what was observed in the parametric study, and is a result of the modified partial factor for steel, which governs deep sections.

10.5 Crack control

Crack widths were checked at all four critical sections. The study consisted of increasing the bending moment applied to the section until the characteristic crack width reached its maximum allowed value. The maximum bending moment that could be applied with regard to crack control was then compared for the two Eurocodes. The calculation is presented in Appendix C.

The bond conditions were assumed poor for the bars in the top face of the slab for Gen 2 (top in tension), but with good conditions for bars in the bottom. For Gen 1, the bond properties were assumed to be good, since the effect of casting position is not taken into account in the model. The calculations were performed for long-term conditions, including creep and shrinkage from Equations 10.1 and 10.2.

10.5.1 Crack width limits

The minimum concrete cover for durability, $c_{min,dur}$, was determined from Table 4.4N from SS-EN 1992-1-1:2005, to be 30 mm, considering the exposure class XD1. For SS-EN 1992-1-1:2023, the same value for $c_{min,dur}$ was assumed.

Thus, the limiting crack width was increased by 40% for top reinforcement and 50% for bottom reinforcement in Gen 2. The limiting crack width depends on the coefficient k_{surf} clarified in Eq. 6.1. With the given values of the bridge, the value of k_{surf} for top reinforcement becomes:

$$k_{surf} = \frac{68 \text{ mm}}{30 \text{ mm} + 10 \text{ mm}} \leq 1.5 = 1.5 [-] \quad (10.6)$$

and for reinforcement located at the bottom:

$$k_{surf} = \frac{56 \text{ mm}}{30 \text{ mm} + 10 \text{ mm}} \leq 1.5 = 1.4 [-] \quad (10.7)$$

Based on the recommended crack width limits in SS-EN 1992-1-1:2005 for Gen 1 and SS-EN 1992-1-1:2023 for Gen 2 for exposure classes XD1 and XD2, the final crack width limits dependent on the chosen NDPs are shown in Table 10.5.

Table 10.5: Limiting crack widths for different NDPs using exposure classes XD1 and XD2 for the top and bottom of sections defined in Section 10.2.

Limiting crack widths	Gen 1	Rec. Gen 2	Gen 2 Prel. TSFS
Top of section [mm]	0.2	0.42	0.28
Bottom of section [mm]	0.2	0.45	0.3

10.5.2 Support sections

The maximum allowed bending moment with regard to crack control was verified in the mid strip and edge strip with poor bonding conditions, with the tensile reinforcement at the top of the slab. The iterative process to find the design bending moment that provides a crack width equal to the crack width limit was made in Appendix C.3.

Edge strip

Table 10.6 shows the maximum allowed bending moments concerning crack control in the Edge strip, with $\phi 20$ reinforcement, which are presented below. Both Gen 2 EC-recommended crack width limits and the preliminary limits proposed by TSFS were used.

Table 10.6: Maximum allowed bending moments concerning crack control for poor bond conditions for the support section of the edge strip, Appendix C.3

Bending moment	M_{Ed} [kNm]	Difference [%]
Gen 1	305	-
Gen 2 Recommended	465	52.0
Gen 2 Prel. TSFS	312	2.3

Mid strip

Maximum allowed bending moments in SLS for the $\phi 16$ bars in the mid strip are shown in Table 10.7:

Table 10.7: Maximum allowed bending moments concerning crack control for poor bond conditions for the support section of the mid strip, Appendix C.3

Bending moment	M_{Ed} [kNm]	Difference [%]
Gen 1	187	-
Gen 2 Recommended	283	51.0
Gen 2 Prel. TSFS	190	1.6

Thus, the mid strip, with $\phi 16$ reinforcement, is governing the moment capacity. It is possible to reduce the height of the support section in Gen 2 by 10 mm to $h = 820$ mm to obtain the same allowed bending moments concerning crack control as in Gen 1, using the TSFS-proposed NDPs.

10.5.3 Midspan sections

Calculation of the crack widths at the mid-sections of the edge strip and mid strip was made considering good bond conditions, since the verification is made at the bottom face of the slab. The calculations for crack widths due to bending moments are shown in Appendix C.3.

Edge strip

The results from the edge strip, with reinforcement diameters $\phi 16$ and $\phi 20$, are shown in Table 10.8.

Table 10.8: Maximum allowed bending moment with regard to crack control for good bond conditions in the span section of the edge strip, Appendix C.3

Bending moment	M_{Ed} [kNm]	Difference [%]
Gen 1	145	-
Gen 2 Recommended	190	31.0
Gen 2 Prel. TSFS	124	-17.0

Mid strip

The results for bars with a diameter of 16 mm are presented in Table 10.9 below:

Table 10.9: Maximum allowed bending moment with regard to crack control for good bond conditions in the span section of the mid strip, Appendix C.3

Bending moment	M_{Ed} [kNm]	Difference [%]
Gen 1	110	-
Gen 2 Recommended	146	33.0
Gen 2 Prel. TSFS	95	-16.0

To achieve the same allowed bending moment in Gen 2 as in Gen 1 for crack control, the height of the section in the middle of the span would need to be increased by 58 mm.

10.5.4 Discussion

The span-sections have a lower allowed bending moment with regard to crack control compared to Gen 1 of the Eurocode. This decrease in bending moment is caused by the concrete cover, which is 68 mm at the bottom of the midspan sections due to the detailing of reinforcement (mounting irons and transverse reinforcement). The maximum concrete cover that k_{surf} gives a favorable effect is $c = 60$ mm, while the crack widths continue to increase for larger concrete covers. At the same time, the Swedish National Annex of Gen 1 includes an NDP that reduces the crack widths for large concrete covers (k_3), which has a significant effect for $c = 68$ mm. Because of this, there is a major difference between the results in Gen 1 and Gen 2. If the concrete cover had been set to $c = 60$ mm in the same sections, the difference would become much smaller, producing similar results between the two Eurocodes. This is what is observed in the support sections, where the concrete cover is $c = 56$ mm. Here, the capacity is, instead, slightly higher in Gen 2.

It should also be noted that the choice of NDPs for the crack width limits has a major impact on the result. Here, the values from TSFS are the main focus, since these are expected to be implemented in Sweden.

10.6 Anchorage & lap length

The critical sections for the calculation of the anchorage length are both sections of the edge strip, as well as the support section for the mid strip. All critical sections are important to compare since they differ in reinforcement diameter, spacing, cover thickness, and bond conditions, which all affect the resulting anchorage length.

The design anchorage length and design lap length were calculated according to Chapter 7. Different values are presented depending on which NDPs were used, explained in Section 9.6.1 of the parametric study. The exact calculations are included in Appendix D.

The calculations are performed both with and without considering the effect of confinement from transverse reinforcement. The stirrups present at the edge strip are considered non-confining of the section, and will therefore not be included in the confinement effect. The anchorage lengths will be controlled on the tensile side to obtain the bridge's governing lengths.

10.6.1 Support sections

The edge strip and the mid strip at the support section of the bridge deck will have different anchorage and lap lengths, since the mid strip has a reinforcement diameter of 16 mm, while the edge strip has $\phi 20$.

Mid strip

The anchored reinforcement is located at the top of the section (poor bond conditions), with a diameter of $\phi = 16$ mm and cover thickness of $c = 56$ mm. The full calculations are included in Appendix D.2, and showcased in Table 10.10:

Table 10.10: Anchorage and lap length at critical section: mid strip - support section

$\phi 16$	[mm]	Difference [%]
Anchorage length	l_{bd}	
Gen 1	527	-
Gen 2 Recommended	488	-7.4
Gen 2 Prel. TSFS ¹	391	-25.8
Gen 2 Mod. TSFS ¹	488	-7.4
Lap length	l_0 or l_{sd}	
Gen 1	664	-
Gen 2 Recommended	586	-11.7
Gen 2 Prel. TSFS ¹	586	-11.7
Gen 2 Mod. TSFS ¹	586	-11.7

¹Same calculation as in Appendix D.2, but with NDPs from Section 9.6 and 9.6.5

Edge strip

The anchored reinforcement is located at the top of the section (poor bond conditions), with a diameter of $\phi = 20$ mm and cover thickness of $c = 56$ mm. The calculations can be found in Appendix D.2, resulting the values presented in Table 10.11:

Table 10.11: Anchorage and lap length at critical section: edge strip - support section

$\phi 20$	[mm]	Difference [%]
Anchorage length	l_{bd}	
Gen 1	659	-
Gen 2 Recommended	749	13.6
Gen 2 Prel. TSFS ¹	599	-9.1
Gen 2 Mod. TSFS ¹	711	8
Lap length	l_0 or l_{sd}	
Gen 1	910	-
Gen 2 Recommended	898	-1.3
Gen 2 Prel. TSFS ¹	898	-1.3
Gen 2 Mod. TSFS ¹	898	-1.3

10.6.2 Midspan sections

The anchored reinforcement has a diameter of $\phi = 20$ mm in the edge strip, and a diameter of $\phi = 16$ mm in the mid strip, both with a cover thickness of $c = 68$ mm. The tensile reinforcement is located at the bottom of the section, with good bond conditions. The final anchorage lengths and lap lengths are shown in Table 10.12 and 10.13 and follow the procedure presented in Appendix D.2.

Table 10.12: Anchorage and lap length at critical section: edge strip - mid section

$\phi 20$	[mm]	Difference [%]
Anchorage length	l_{bd}	
Gen 1	461	-
Gen 2 Recommended	624	35.4
Gen 2 Prel. TSFS ¹	499	8.2
Gen 2 Mod. TSFS ¹	540	17.0
Lap length	l_0 or l_{sd}	
Gen 1	637	-
Gen 2 Recommended	749	17.6
Gen 2 Prel. TSFS ¹	749	17.6
Gen 2 Mod. TSFS ¹	749	17.6

¹Same calculation as in Appendix D.2, but with NDPs from Section 9.6 and 9.6.5

Table 10.13: Anchorage and lap length at critical section: mid strip - mid section

$\phi 16$	[mm]	Difference [%]
Anchorage length	l_{bd}	
Gen 1	369	-
Gen 2 Recommended	407	10.3
Gen 2 Prel. TSFS ¹	326	-13.2
Gen 2 Mod. TSFS ¹	380	3.0
Lap length	l_0 or l_{sd}	
Gen 1	491	-
Gen 2 Recommended	488	-0.6
Gen 2 Prel. TSFS ¹	488	-0.6
Gen 2 Mod. TSFS ¹	488	-0.6

10.6.3 Discussion

The anchorage lengths are greater in Gen 2 for bars with good bond conditions. The difference is most significant when using the recommended NDPs of the EC, with a 10-35% longer anchorage in the middle of the span.

Sections with poor bonding conditions have lower anchorage lengths than Gen 1 for bars with a diameter $\phi = 16$ mm. Here, a reduction of 26% is observed when using the preliminary TSFS-proposed NDPs. This is also in line with what was observed in the parametric study, which showed that the anchorage lengths were lower in Gen 2 for smaller reinforcement diameters, especially when using the preliminary NDPs from TSFS. In comparison, the anchorage lengths are closer to the values from Gen 1 when using the modified NDPs proposed in this thesis. This leads to safer results for anchorage with poor bond.

The lap lengths were 1-18% greater for bars with good bond, while a reduction of 1-12% was observed for bars with poor bond. However, there is no difference between the results when using EC-recommended NDPs and the preliminary ones proposed by TSFS.

¹Same calculation as in Appendix D.2, but with NDPs from Section 9.6 and 9.6.5

10.7 Consequences of SS-EN 1992-1-1:2023

The application of the new Eurocode on a standard bridge was found to result in higher capacities for many critical sections. As a consequence, it is possible to reduce the geometry at the support of the slab, while still obtaining the same capacities as in Gen 1.

For the support sections, the maximum allowed bending moment in SLS with regard to crack control was found to be governing. In this section, the height of the slab could be reduced by 10 mm to a height of 830 mm while maintaining the same capacities as in Gen 1, or higher.

The sections in the middle of the span were also governed by the bending moment in SLS due to crack control. However, here, the height of the section would need to be increased by 58 mm.

Therefore, the total amount of concrete in the bridge would likely be higher if the bridge were designed today. This is mainly due to the high concrete cover in the mid sections of the bridge, which becomes the governing factor for crack control in SLS.

Due to the significant increase in anchorage and lap lengths for bars with good bond conditions, the volume of steel in the bottom face of the bridge is expected to increase. However, the required volume of steel from Gen 2 is lower in the top face of the slab. Whether there would be an increase or decrease in the total steel volume depends on the choice of national parameters. The Gen 2 recommended NDPs would result in a net increase in steel volume, while the preliminary NDPs proposed by TSFS would instead result in a decrease in steel volume.

11

DISCUSSION

One of the main objectives of this thesis was to identify and analyze the design criteria that have been subjected to the most significant revisions in the new Eurocode and to explain the reasons for the changes. This was done through an extensive literature study, starting with a review and evaluation to conclude the most crucial differences.

The decision of key changes to be further analyzed was based on:

- i) the result from the overview (Table 2.1),
- ii) the time limitations of the Master's thesis,
- iii) the importance of the design criteria in the design of concrete bridges.

Based on Table 2.1 and (iii), it was concluded that the design regarding shear force resistance, crack control, and detailing of reinforcement had undergone major changes. Additionally, the chapter considering material design and bending moment resistance was included based on (iii).

The chapter regarding Vibrations included in Gen 2 was decided not to be further investigated, even though major changes have been made, simply because it was not covered in Gen 1, and a comparison could therefore not be made.

The chapter covering shear punching was not investigated further either due to (ii). The proclaimed key changes were examined in their own chapters, including design differences and motivations for the changes. Here, the background documents to the Eurocode were utilized to obtain motivations and derivations of the new models.

A parametric study was then carried out, examining the key changes identified in the literature study, aiming to visualize and give examples of the consequences of the application of Gen 2. The results of the parametric study show that the implementation of the new Eurocode often leads to higher structural capacities than for Gen 1. This is also shown in the case study, which agrees with the results from the parametric study. It can therefore be argued that the new Eurocode is less conservative (higher capacity, more favorable) in many design situations.

One example of this is the shear capacity without stirrups, where sections without external moment were shown to have a significantly higher capacity in Gen 2 of the Eurocode. This is highly relevant for simply supported slabs, where the supported edges are usually without a bending moment. Based on this, it is expected that Gen 2 will result in less conservative behavior for simply supported slabs, which both benefit from the favorable effects of casting position on crack control and the effective shear span for shear verification without stirrups.

However, the methodology in Gen 2 often requires additional knowledge to be able to take into account these favorable effects. For example, the stress state of the studied section needs to be known to utilize the effective shear span to find the shear capacity. This could make the design process more complicated.

Another reason for the higher capacities in Gen 2 is the addition of the design effective depth, d_d . This removes geometric uncertainty by reducing the effective depth by 15 mm, which allows for more favorable partial factors to be used (see Eq. 3.1). In fact, the application of d_d is the only reason that an increase in bending moment capacity (Chapters 10.3, 9.2) and shear capacity with stirrups (Chapters 9.4, 10.4) is observed.

Using the design effective depth is beneficial for structures with a great height, where the reduction in effective depth by 15 mm is nearly negligible compared to the total depth of the section. This is the case for beams or slabs in bridges. However, thinner sections with a height of 200 mm or less, such as slabs in buildings, will likely be unaffected by this change. It should be noted that d_d can be used unless limited by the National Annex.

If it were decided that d_d is not allowed to be used, there would be no increase in moment capacity and shear capacity with stirrups in the new Eurocode. However, it is preliminarily proposed by the Swedish Transport Agency that Annex A.3 of the Eurocode should be included [18]. This means that d_d can be used for capacity calculations.

In some cases, the new Eurocode was shown to produce more conservative results than the current Eurocode. This could imply that Gen 1 is unsafe in certain design situations. One such example is the anchorage lengths for reinforcement bars with large diameters and with good bond conditions. In practice, this could result in much longer anchorage for bars in the bottom face of a slab. This is highly relevant for bridges, since large bar diameters are generally used, potentially leading to increased material usage and costs.

The required anchorage can be reduced to better match the lengths in Gen 1 by adopting the new NDPs for anchorage proposed by the Swedish Transport Agency, see Eq. 7.11. However, these NDPs resulted in significantly shorter anchorage for small reinforcement diameters (below 16 mm) and with poor bond conditions. It can be argued that this is on the unsafe side, since the EC-recommended anchorage lengths in Gen 2 appear to be justified for these diameters, matching test data according to the background document [31]. Therefore, it is suggested that the reduction in anchorage should be limited to only include bars with diameters 16 mm or above, see Eq. 9.12.

Gen 2 of the Eurocode was also found to be more conservative towards the concrete shear capacity for slabs with a large height subjected to substantial bending moments. This is caused by the effective shear span, where the ratio $\frac{V_{Ed}}{M_{Ed}}$ is used to determine the shear capacity without stirrups. This becomes a problem when the design shear force is small relative to the design moment, which can be the case for deep slabs without stirrups, but with very high flexural reinforcement ratios (above $\rho = 0.4\%$). Shear reinforcement may therefore be needed to a greater extent for such sections.

In addition, Gen 2 resulted in a decreased maximum bending moment in SLS due to crack control for sections with great concrete covers. This could have a negative impact on bridge construction, since bridges generally have large concrete covers (due to strict exposure classes and detailing). This means that crack control may become even more governing in future design scenarios.

It should be noted that the results and conclusions from this thesis mainly apply to slabs in bridges. As a consequence, the exact results may not be directly applicable to other types of construction. However, overall conclusions of the general behavior in Gen 2 may be applied to structures other than bridge slabs.

12

CONCLUSIONS

12.1 Literature study and parametric study

The literature study showed that several important design criteria have been subjected to significant revisions in the new Eurocode. This included changes to partial factors, material models, shear capacity, crack control, and anchorage. The effects and consequences of these modifications were thoroughly investigated. Based on the parametric study from Chapter 9, it could be concluded that:

- 1) The moment capacity and shear capacity with stirrups are the same in both versions of the Eurocode. However, when using the design effective depth d_d together with an improved partial factor for reinforcing steel, higher capacities can be reached for sections with a height above 200 mm. This can result in up to 10% higher capacity for shear and moment for sections with a height of 1000 mm with a high reinforcement ratio.
- 2) A reduction in moment capacity was observed for sections with a high concrete strength class. This is caused by the new strength reduction factor, η_{cc} , which reduces the design strength for concrete with a compressive strength $f_{ck} = 45$ MPa or above. However, this negative effect can, in some cases, be counteracted by the favorable effect of the modified partial factor for steel when using d_d for capacity calculations.
- 3) A significant increase in the shear capacity of concrete was observed for slabs devoid of moment, such as for simply supported edges. This is believed to be a result of the new model for shear in Gen 2, which includes the effect of moment, aggregates, more favorable partial factors for shear, and steel. This resulted in a 5-40% higher concrete shear capacity, depending on the height of the section and reinforcement ratio, and aggregate size.
- 4) The interaction of moment and shear force was found to negatively affect the shear capacity of concrete in Gen 2. This results in a decrease in shear capacity by up to 17% for deep sections subjected to significant moments with high reinforcement ratios.

- 5) The crack widths in Gen 2 were found to be significantly larger, with an increase of up to 60% for sections with thick concrete covers. However, the maximum crack width can also be increased by up to 50% for a concrete cover of 60 mm.
- 6) Reinforcement bars with large diameters require greater anchorage lengths in Gen 2 of the Eurocode. However, bars with diameters smaller than 16 mm instead require lower anchorage lengths, especially with poor bond conditions. The anchorage lengths can be reduced through NDPs. However, the national parameters proposed in TSFS lead to much shorter anchorage lengths for bars with poor bond conditions and small diameters, which could potentially be unsafe. A modification to these NDPs is therefore suggested.

12.2 Case study

A case study was also performed to provide deeper insight into how the changes to the new Eurocode will impact realistic design situations. Therefore, the structural capacity of a concrete frame bridge, designed with the current Eurocode, was investigated in several critical sections. Based on the results of the case study in Chapter 10 for a Swedish concrete bridge, the following conclusions can be drawn:

- 1) The bending moment capacity for a Swedish concrete bridge is expected to increase by 5-8% when using the new Eurocode. This is a result of the design effective depth, together with a modified partial factor for steel.
- 2) The bridge had a 15% higher concrete shear capacity. The shear capacity with stirrups was 9% higher, as a result of adopting the design effective depth.
- 3) The maximum design bending moment M_{Ed} , due to crack control, is expected to decrease for sections with a concrete cover thicker than 60 mm. It was shown that a cover thickness of $c = 56$ mm resulted in a cracking behavior similar to that in Gen 1, while $c = 68$ mm led to a reduction in M_{Ed} by 16%. This is because the beneficial effect from the surface coefficient k_{surf} is maximized at $c = 60$ mm for exposure classes XD1 and XD2 (conditions for a typical Swedish bridge).
- 4) Using the Gen 2 recommended NDPs, the required volume of reinforcing steel is expected to increase because of the longer anchorage and lap lengths. However, Sweden will likely use the recommendation from the Swedish Transport Agency, which preliminarily proposed NDPs to reduce the anchorage and lap lengths. Using these NDPs leads to a reduction in the total volume of steel. They were also found to result in 25% shorter anchorage lengths compared to Gen 1 for sections with poor bond conditions, which could be unsafe. Therefore, this thesis proposes changes to the NDPs, see Section 9.6.5. This eliminates unsafe anchorage lengths for bars with poor conditions, while reducing the anchorage for bars with larger diameters and good bond conditions.

12.3 Further studies

Due to the limitations of the thesis, some important aspects of the Eurocode could not be studied in detail. Therefore, the authors of this thesis suggest future studies to analyze the following clauses with respect to changes in the new Eurocode.

- **Planar members in shear and bending:** The current Eurocode treats planar members in an informative annex, which has been changed into a normative Annex with new models in the new Eurocode. It is therefore important to investigate these changes.
- **Shear punching:** This chapter has been extensively revised in the new Eurocode, but it was not included in this thesis due to time constraints. It is therefore interesting to compare and analyze the consequences of these changes.
- **Restraint cracking:** This is a new chapter that is not included in the current Eurocode. Its application has been known previously, but its inclusion in the Eurocode is new. The consequences of this addition should therefore be analyzed.
- **Additional case studies and parameters:** Investigations on structures other than bridge decks / or with other parameters, such as for more concrete classes.
- **Actions on structures:** Gen 2 of Eurocode SS-EN 1991-1-1.
- **Design of steel structures:** Gen 2 of Eurocode SS-EN 1993-1-1.

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A

BENDING

A.1 Calculation for parametric study

The same calculative sheet was used for varying input parameters listed in Section 9.2.

Bending moment - SS-EN1992-2005

clause 6.1

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1 Input parameters

1.1 Geometry

Height	$h := 1000 \text{ mm}$
Width	$b := 1000 \text{ mm}$
Reinforcement diameter	$\phi := 16 \text{ mm}$
Concrete cover	$c := 55 \text{ mm}$
Bar spacing	$s := 145 \text{ mm}$
Effective depth	$d := h - c - \frac{\phi}{2} = 937 \text{ mm}$

1.2 Materials

Characteristic compressive strength	$f_{ck} := 35 \text{ MPa}$
E-module	$E_{cm} := 33 \text{ GPa}$
Partial safety factor	$\gamma_C := 1.5$
Long term effects coefficient	$\alpha_{cc} := 1$
Compressive block factors	$\alpha := 0.742$ $\beta := 0.392$
Ultimate strain	$\epsilon_{cu} := 3.5 \cdot 10^{-3}$

Characteristic yielding strength

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

E-modulus, reinforcement

$$E_s := 200 \text{ GPa}$$

Partial factor reinforcement

$$\gamma_s := 1.15$$

2 Calculation

2.1 Design material

Design compressive strength

$$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield limit, reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

Area of concrete

$$A_c := b \cdot h = 1 \text{ m}^2$$

Reinforcement ratio

$$\rho := 0.2 \cdot 10^{-2}$$

Area of tensile reinforcement

$$A_s := \rho \cdot d \cdot b = (1.874 \cdot 10^3) \text{ mm}^2$$

2.2 Normal force

Normal force capacity

$$N_{Rd} := -A_c \cdot f_{cd} = -2.33 \cdot 10^4 \text{ kN}$$

Normal force utilization

$$\eta := 5\%$$

Design normal force

$$N_{Ed} := \eta \cdot N_{Rd} = -1166.67 \text{ kN}$$

2.2 Ultimate moment resistance

Calculation according to section 6.1 in EN1992-1-1-2005

Yielding strain

$$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.17 \cdot 10^{-3}$$

Assumption, reinforcement yields

$$\sigma_s := f_{yd} = 435 \text{ MPa}$$

Finding the compressive zone, x

$$x_1 := \frac{\sigma_s \cdot A_s}{\alpha \cdot f_{cd} \cdot b} = 47.06 \text{ mm}$$

Deformation condition

$$\varepsilon_s := \frac{d - x_1}{x_1} \cdot \varepsilon_{cu} = 0.07$$

Assumption: Reinforcement doesn't yield

Finding the compressive zone, x

Solve for x_2 values

$$x_2 := 5 \text{ mm} \quad (\text{guess})$$

$$\sigma_s = E_s \cdot \frac{(d - x_2)}{x_2} \cdot \varepsilon_{cu}$$

$$x_2 := \text{find}(x_2) = 578 \text{ mm}$$

Check of steel strain

$$x := \begin{cases} \text{if } \varepsilon_s \geq \varepsilon_{sy} \\ \parallel x_1 \\ \text{else} \\ \parallel x_2 \end{cases} = 47.06 \text{ mm}$$

Ultimate moment resistance

$$M_{Rd_N} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot (d - \beta \cdot x) - N_{Ed} \cdot \left(d - \frac{h}{2} \right) = 1258.254 \text{ kN} \cdot \text{m}$$

Bending moment - SS-EN1992-2023

clause 8.1

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1 Input parameters

1.1 Geometry

Height	$h := 1000 \text{ mm}$
Width	$b := 1000 \text{ mm}$
Reinforcement diameter	$\phi := 16 \text{ mm}$
Concrete cover	$c := 55 \text{ mm}$
Bar spacing	$s := 145 \text{ mm}$
Nominal effective depth	$d_{nom} := h - c - \frac{\phi}{2} = 937 \text{ mm}$
Deviation of effective depth	$\Delta d := 15 \text{ mm}$
Design effective depth	$d_d := d_{nom} - \Delta d = 922 \text{ mm}$

1.2 Materials

Characteristic compressive strength	$f_{ck} := 35 \text{ MPa}$
Partial safety factor	$\gamma_C := 1.5$
Ultimate strain	$\epsilon_{cu} := 3.5 \cdot 10^{-3}$

Compressive strain at peak stress

$$\varepsilon_{c2} := 0.002$$

Reference characteristic compressive strength

$$f_{ck_ref} := 40 \text{ MPa}$$

Design factors, concrete

$$k_{tc} := 1$$

$$t_{ref} \leq 28 \text{ days}$$

Characteristic yielding strength

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

E-modulus, reinforcement

$$E_s := 200 \text{ GPa}$$

Partial factor reinforcement

$$\gamma_s := 1.15$$

Partial factor reinforcement, for d_d

$$\gamma_{sd} := 1.03$$

2 Calculation using d_{nom}

2.1 Design material

Concrete strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\frac{1}{3}}, 1 \right) = 1$$

Design compressive strength

$$f_{cd} := \eta_{cc} \cdot k_{tc} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield limit, reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

Area of concrete

$$A_c := b \cdot h = 1 \text{ m}^2$$

Reinforcement ratio

$$\rho := 0.2\%$$

Area of tensile reinforcement

$$A_s := \rho \cdot d_{nom} \cdot b = (1.874 \cdot 10^3) \text{ mm}^2$$

Stress block factors, derived from
SS-EN1992-2023 clause 8.1.2,
see Appendix A.2

$$\alpha := 0.810$$

$$\beta := 0.416$$

2.2 Normal force

Normal force capacity

$$N_{Rd} := -A_c \cdot f_{cd} = -2.33 \cdot 10^4 \text{ kN}$$

Normal force utilization

$$\eta := 5\%$$

Design normal force

$$N_{Ed} := \eta \cdot N_{Rd} = -1166.67 \text{ kN}$$

2.3 Ultimate moment resistance

Yielding strain

$$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.17 \cdot 10^{-3}$$

Assumption, reinforcement yields

$$\sigma_s := f_{yd} = 435 \text{ MPa}$$

Finding the compressive zone, x

$$x_1 := \frac{\sigma_s \cdot A_s}{\alpha \cdot f_{cd} \cdot b} = 43.11 \text{ mm}$$

Deformation condition

$$\varepsilon_s := \frac{d_{nom} - x_1}{x_1} \cdot \varepsilon_{cu} = 0.07$$

Assumption: Reinforcement doesn't yield

Finding the compressive zone, x

Solve for x2	$x_2 := 10 \text{ mm} \quad (\text{guess})$
	$\sigma_s = E_s \cdot \frac{(d_{nom} - x_2)}{x_2} \cdot \varepsilon_{cu}$
	$x_2 := \text{find}(x_2) = 578 \text{ mm}$

Check of steel strain

$$x := \begin{cases} \text{if } \varepsilon_s \geq \varepsilon_{sy} & = 43.11 \text{ mm} \\ \parallel & x_1 \\ \text{else} & \\ \parallel & x_2 \end{cases}$$

Ultimate moment resistance

$$M_{Rd_nom} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot \left(d_{nom} - \beta \cdot x \right) - N_{Ed} \cdot \left(d_{nom} - \frac{h}{2} \right) = 1258.67 \text{ kN} \cdot \text{m}$$

3 Calculation using d_d **3.1 Design material**

Design compressive strength

$$f_{cd} = 23.33 \text{ MPa}$$

Design yield limit, reinforcement

$$f_{yd_d} := \frac{f_{yk}}{\gamma_{sd}} = 485 \text{ MPa}$$

Area of concrete

$$A_c = 1 \text{ m}^2$$

Reinforcement ratio

$$\rho = 2 \cdot 10^{-3}$$

Area of tensile reinforcement

$$A_s := \rho \cdot d_d \cdot b = (1.844 \cdot 10^3) \text{ mm}^2$$

Stress block factors

$$\alpha = 0.81$$

$$\beta = 0.416$$

3.2 Normal force

Normal force capacity

$$N_{Rd} = -2.33 \cdot 10^4 \text{ kN}$$

Normal force utilization

$$\eta = 0.05$$

Design normal force

$$N_{Ed} = -1166.67 \text{ kN}$$

3.2 Ultimate moment resistance

Yielding strain

$$\varepsilon_{sy} := \frac{f_{yd,d}}{E_s} = 2.43 \cdot 10^{-3}$$

Assumption, reinforcement yields

$$\sigma_s := f_{yd,d} = 485.44 \text{ MPa}$$

Finding the compressive zone, x

$$x_1 := \frac{\sigma_s \cdot A_s}{\alpha \cdot f_{cd} \cdot b} = 47.36 \text{ mm}$$

Deformation condition

$$\varepsilon_s := \frac{d_d - x_1}{x_1} \cdot \varepsilon_{cu} = 0.06$$

Assumption: Reinforcement doesn't yield

Finding the compressive zone, x

Solve for Guess Values

$$\begin{aligned} x_2 &:= 10 \text{ mm} \quad (\text{guess}) \\ \sigma_s &= E_s \cdot \frac{(d_d - x_2)}{x_2} \cdot \varepsilon_{cu} \\ x_2 &:= \text{find}(x_2) = 544.44 \text{ mm} \end{aligned}$$

Check of steel strain

$$x := \begin{cases} \text{if } \varepsilon_s \geq \varepsilon_{sy} & = 47.36 \text{ mm} \\ \parallel & x_1 \\ \text{else} & \\ \parallel & x_2 \end{cases}$$

Ultimate moment resistance

$$M_{Rd,d} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot (d_d - \beta \cdot x) - N_{Ed} \cdot \left(d_d - \frac{h}{2} \right) = 1300.02 \text{ kN} \cdot \text{m}$$

4 Comparison of M_{Rd}

The largest ultimate moment resistance will be the governing value

$$M_{Rd} := \max(M_{Rd,nom}, M_{Rd,d}) = 1300.021 \text{ kN} \cdot \text{m}$$

A.2 Deriving compression block factors, Gen 2

The compression block factors are derived in the literature chapter to be applied in the parametric study and case study.

Deriving stress block factors for bending - SS-EN1992-2023 clause 8.1.2

Partial safety factor

$$\gamma_C := 1.5$$

Ultimate strain

$$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$$

Compressive strain at peak stress

$$\varepsilon_{c2} := 0.002$$

Characteristic compressive strength

$$f_{ck} := 35 \text{ MPa}$$

Reference compressive strength

$$f_{ck_ref} := 40 \text{ MPa}$$

Design factors, concrete

$$k_{tc} := 1 \quad t_{ref} \leq 28 \text{ days}$$

Strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\frac{1}{3}}, 1 \right) = 1$$

Design compressive strength

$$f_{cd} := \eta_{cc} \cdot k_{tc} \cdot \frac{f_{ck}}{\gamma_C} = 23.33 \text{ MPa}$$

Concrete stress

$$\sigma_c(\varepsilon_c) := \begin{cases} \text{if } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \\ \left\| f_{cd} \cdot \left(1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}} \right)^2 \right) \right\| \\ \text{else} \\ \left\| f_{cd} \right\| \end{cases}$$

Area of compressive zone

$$A := \int_0^{\varepsilon_{cu}} \sigma_c(\varepsilon_c) d\varepsilon_c = (6.61 \cdot 10^4) \text{ Pa}$$

Stress block factor alpha

$$\alpha := \frac{A}{f_{cd} \cdot \varepsilon_{cu}} = 0.81$$

Stress block factor beta

$$\beta := 1 - \frac{\int_0^{\varepsilon_{cu}} \sigma_c(\varepsilon_c) \cdot \varepsilon_c d\varepsilon_c}{A \cdot \varepsilon_{cu}} = 0.416$$

A.3 Case study calculations

Calculation is made at the critical edge strip section defined in Section 10.2.1. The same calculation sheet is also used to calculate the bending moment resistance with the input parameters defined in Section 10.2.2 for the middle section, located at the edge strip of the bridge deck.

Bending moment resistance, SS-EN1992-2005

clause 6.1 - support section, edge strip

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1 Input parameters

1.1 Geometry

Height	$h := 830 \text{ mm}$
Width	$b := 1000 \text{ mm}$
Reinforcement diameter, compression	$\phi' := 16 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Diameter monterinsj�rn	$\phi_{mj} := 12 \text{ mm}$
Concrete cover, top	$c := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Concrete cover, bottom	$c_{bot} := 40 \text{ mm} + \phi_t + \phi_{mj} = 68 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Bar spacing, compression	$s' := 260 \text{ mm}$
Effective depth, first layer	$d := h - c - \frac{\phi}{2} = 764 \text{ mm}$
Effective depth, to compressive layer	$d' := c_{bot} + \frac{\phi'}{2} = 76 \text{ mm}$

1.2 Materials

Characteristic compressive strength	$f_{ck} := 35 \text{ MPa}$
Partial safety factor	$\gamma_C := 1.5$

Long term effects coefficient

$$\alpha_{cc} := 1$$

Compressive block factors

$$\alpha := 0.81$$

$$\beta := 0.416$$

Ultimate strain

$$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$$

Reinforcement

Characteristic yielding strength

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

E-modulus, reinforcement

$$E_s := 200 \text{ GPa}$$

Partial factor reinforcement

$$\gamma_s := 1.15$$

1.3 Loads

Design normal force (negative = compression)

$$N_{Ed} := 0 \text{ kN}$$

2 Calculation

2.1 Design material

Design compressive strength

$$f_{cd} := \alpha_{cc} \cdot \frac{f_{ck}}{\gamma_C} = 23.33 \text{ MPa}$$

Design yield limit, reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

Area of concrete

$$A_c := b \cdot h = 0.83 \text{ m}^2$$

Area of compressive reinforcement at d'

$$A_{s'} := \frac{\pi \cdot \phi'^2}{4} \cdot \frac{b}{s'} = 773.32 \text{ mm}^2$$

Area of tensile reinforcement at d_nom

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Reinforcement ratio, top layer

$$\rho' := \frac{A_{s'}}{d \cdot b} = 1.01 \cdot 10^{-3}$$

Reinforcement ratio, bottom layer

$$\rho := \frac{A_s}{d \cdot b} = 3.16 \cdot 10^{-3}$$

2.2 Ultimate moment resistance

Yielding strain

$$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.17 \cdot 10^{-3}$$

Assumption, reinforcement yields

$$\sigma_s := f_{yd} = 435 \text{ MPa}$$

Guess the value of the compressive zone

$x := 80 \text{ mm}$

guess

Equilibrium condition

$$f(x) := \alpha \cdot f_{cd} \cdot b \cdot x + E_s \cdot \frac{(x - d')}{x} \cdot \epsilon_{cu} \cdot A_{s'} - \sigma_s \cdot A_s$$

Finding the compressive zone, x

$$x := \text{root}(f(x), x) = 62.04 \text{ mm}$$

Deformation condition

$$\epsilon_s := \frac{d - x}{x} \cdot \epsilon_{cu} = 0.04$$

Steel stress, compression

$$\sigma_s := E_s \cdot \frac{(x - d')}{x} \cdot \epsilon_{cu} = -157.53 \text{ MPa}$$

Check of steel strain, does it yield?

$$\epsilon_s \geq \epsilon_{sy} = 1$$

$$x = 62.04 \text{ mm}$$

Ultimate moment resistance

$$M_{Rd} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot (d - \beta \cdot x) + \sigma_s \cdot A_{s'} \cdot (d - d') - N_{Ed} \cdot \left(d - \frac{h}{2}\right) = 781.733 \text{ kN} \cdot \text{m}$$

Bending moment resistance, SS-EN1992-2023

clause 8.1 - support section, edge strip

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3 Calculation using d_d	3
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4 Comparison of M_{Rd}	4

1 Input parameters

1.1 Geometry

Height	$h := 830 \text{ mm}$
Width	$b := 1000 \text{ mm}$
Reinforcement diameter, compression	$\phi' := 16 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Diameter monterinsj�rn	$\phi_{mj} := 12 \text{ mm}$
Concrete cover, top	$c := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Concrete cover, bottom	$c_{bot} := 40 \text{ mm} + \phi_t + \phi_{mj} = 68 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Bar spacing, compression	$s' := 260 \text{ mm}$
Effective depth, first layer	$d_{nom} := h - c - \frac{\phi}{2} = 764 \text{ mm}$
Deviation of effective depth	$\Delta d := 15 \text{ mm}$
Design effective depth	$d_d := d_{nom} - \Delta d = 749 \text{ mm}$
Effective depth, to compressive layer	$d'_{nom} := c_{bot} + \frac{\phi'}{2} = 76 \text{ mm}$
Design effective depth, to compressive layer	$d'_d := d'_{nom} - \Delta d = 61 \text{ mm}$

1.2 Materials

Characteristic compressive strength

$$f_{ck} := 35 \text{ MPa}$$

Partial safety factor

$$\gamma_C := 1.5$$

Ultimate strain

$$\varepsilon_{cu} := 3.5 \cdot 10^{-3}$$

Compressive strain at peak stress

$$\varepsilon_{c2} := 0.002$$

Reference characteristic compressive strength

$$f_{ck_ref} := 40 \text{ MPa}$$

Design factors, concrete

$$k_{tc} := 1 \quad t_{ref} \leq 28 \text{ days}$$

Characteristic yielding strength

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

E-modulus, reinforcement

$$E_s := 200 \text{ GPa}$$

Partial factor reinforcement

$$\gamma_s := 1.15$$

Partial factor reinforcement, for d_d

$$\gamma_{sd} := 1.03$$

1.3 Loads

Design normal force (negative = compression)

$$N_{Ed} := 0 \text{ kN}$$

2 Calculation using d_{nom}

2.1 Design material

Concrete strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\frac{1}{3}}, 1 \right) = 1$$

Design compressive strength

$$f_{cd} := \eta_{cc} \cdot k_{tc} \cdot \frac{f_{ck}}{\gamma_C} = 23.33 \text{ MPa}$$

Design yield limit, reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

Area of concrete

$$A_c := b \cdot h = 0.83 \text{ m}^2 \quad 1.55 \cdot 10^3 - 1.44 \cdot 10^3 = 110$$

Area of compressive reinforcement at d'

$$A_{s'} := \frac{\pi \cdot \phi'^2}{4} \cdot \frac{b}{s'} = 773.32 \text{ mm}^2$$

Area of tensile reinforcement at d_{nom}

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Reinforcement ratio, top layer

$$\rho' := \frac{A_{s'}}{d \cdot b} = 1.01 \cdot 10^{-3}$$

Reinforcement ratio, bottom layer	$\rho := \frac{A_s}{d_{nom} \cdot b} = 3.16 \cdot 10^{-3}$
Stress block factors, derived from SS-EN1992-2023 clause 8.1.2	$\alpha := 0.810$ $\beta := 0.416$

2.2 Ultimate moment resistance

Calculation according to section 6.1 in EN1992-1-1-2005

Yielding strain	$\varepsilon_{sy} := \frac{f_{yd}}{E_s} = 2.17 \cdot 10^{-3}$
-----------------	---

Assumption, reinforcement yields	$\sigma_s := f_{yd} = 435 \text{ MPa}$
---	--

Guess the value of the compressive zone	$x := 80 \text{ mm}$	guess
---	----------------------	-------

Equilibrium condition	$f(x) := \alpha \cdot f_{cd} \cdot b \cdot x + E_s \cdot \frac{(x - d'_{nom})}{x} \cdot \varepsilon_{cu} \cdot A_{s'} - \sigma_s \cdot A_s$
-----------------------	---

Finding the compressive zone, x	$x := \text{root}(f(x), x) = 62.04 \text{ mm}$
---------------------------------	--

Deformation condition	$\varepsilon_s := \frac{d_{nom} - x}{x} \cdot \varepsilon_{cu} = 0.04$
-----------------------	--

Steel stress, compression	$\sigma_{s'} := E_s \cdot \frac{(x - d'_{nom})}{x} \cdot \varepsilon_{cu} = -157.53 \text{ MPa}$
---------------------------	--

Check of steel strain, does it yield?	$\varepsilon_s \geq \varepsilon_{sy} = 1$
--	---

$$x = 62.04 \text{ mm}$$

Ultimate moment resistance

$$M_{Rd_nom} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot (d_{nom} - \beta \cdot x) + \sigma_{s'} \cdot A_{s'} \cdot (d_{nom} - d'_{nom}) - N_{Ed} \cdot \left(d_{nom} - \frac{h}{2} \right) = 781.73 \text{ kN} \cdot \text{m}$$

3 Calculation using d_d

3.1 Design material

Design compressive strength	$f_{cd} = 23.33 \text{ MPa}$
Design yield limit, reinforcement	$f_{yd_d} := \frac{f_{yk}}{\gamma_{sd}} = 485 \text{ MPa}$
Area of concrete	$A_c = 0.83 \text{ m}^2$
Reinforcement ratio, top layer	$\rho' := \frac{A_{s'}}{d \cdot b} = 1.03 \cdot 10^{-3}$

Reinforcement ratio, bottom layer

$$\rho := \frac{A_s}{d_d \cdot b} = 3.23 \cdot 10^{-3}$$

Stress block factors

$$\alpha = 0.81$$

$$\beta = 0.416$$

3.2 Ultimate moment resistance

Calculation according to section 6.1 in EN1992-1-1-2005

Yielding strain

$$\varepsilon_{sy} := \frac{f_{yd,d}}{E_s} = 2.43 \cdot 10^{-3}$$

Assumption, reinforcement yields

$$\sigma_s := f_{yd,d} = 485.44 \text{ MPa}$$

Guess the value of the compressive zone

$$x := 80 \text{ mm} \quad \text{guess}$$

Equilibrium condition

$$f(x) := \alpha \cdot f_{cd} \cdot b \cdot x + E_s \cdot \frac{(x - d'_d)}{x} \cdot \varepsilon_{cu} \cdot A_{s'} - \sigma_s \cdot A_s$$

Finding the compressive zone, x

$$x := \text{root}(f(x), x) = 61.73 \text{ mm}$$

Deformation condition

$$\varepsilon_s := \frac{d_d - x}{x} \cdot \varepsilon_{cu} = 0.04$$

Steel stress, compression

$$\sigma_s := E_s \cdot \frac{(x - d'_d)}{x} \cdot \varepsilon_{cu} = 8.28 \text{ MPa}$$

Check of steel strain, does it yield?

$$\varepsilon_s \geq \varepsilon_{sy} = 1$$

Yes it yields

$$x = 61.73 \text{ mm}$$

Ultimate moment resistance

$$M_{Rd,d} := \alpha \cdot f_{cd} \cdot b \cdot x \cdot (d_d - \beta \cdot x) + \sigma_s \cdot A_{s'} \cdot (d_d - d'_d) - N_{Ed} \cdot \left(d_d - \frac{h}{2} \right) = 848.31 \text{ kN} \cdot \text{m}$$

4 Comparison of M_{Rd}

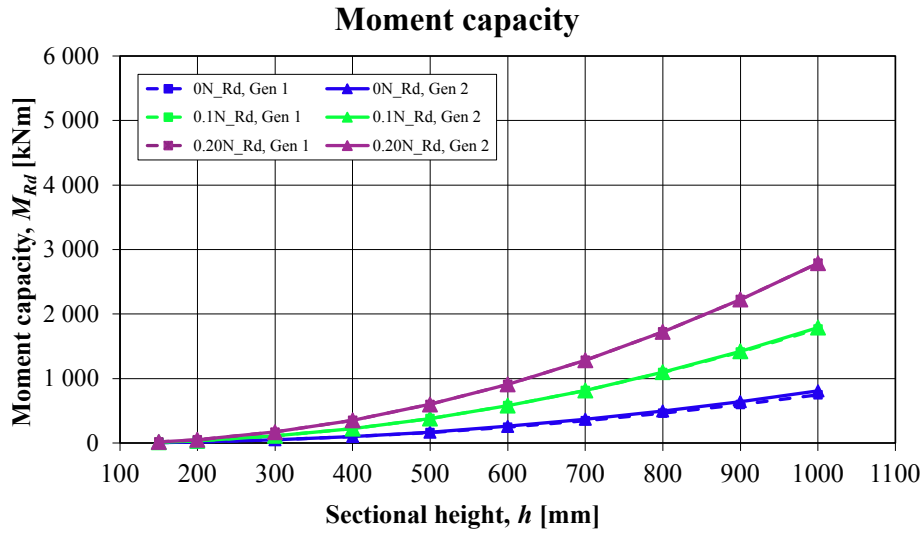
The largest ultimate moment resistance will be the governing value

$$M_{Rd} := \max(M_{Rd,nom}, M_{Rd,d}) = 848.309 \text{ kN} \cdot \text{m}$$

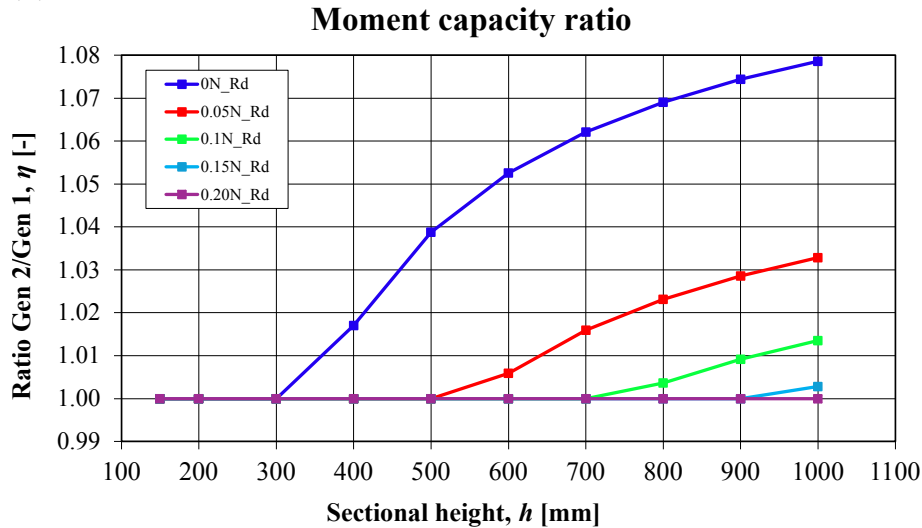
A.4 Additional graphs

These additional graphs cover additional reinforcement ratios than those included in the result of the parametric study for the bending moment capacity. Other input parameters are the same as presented in Section 9.2.

A.4.1 Concrete class C35/45, $\rho_s = 0.2\%$



(a)



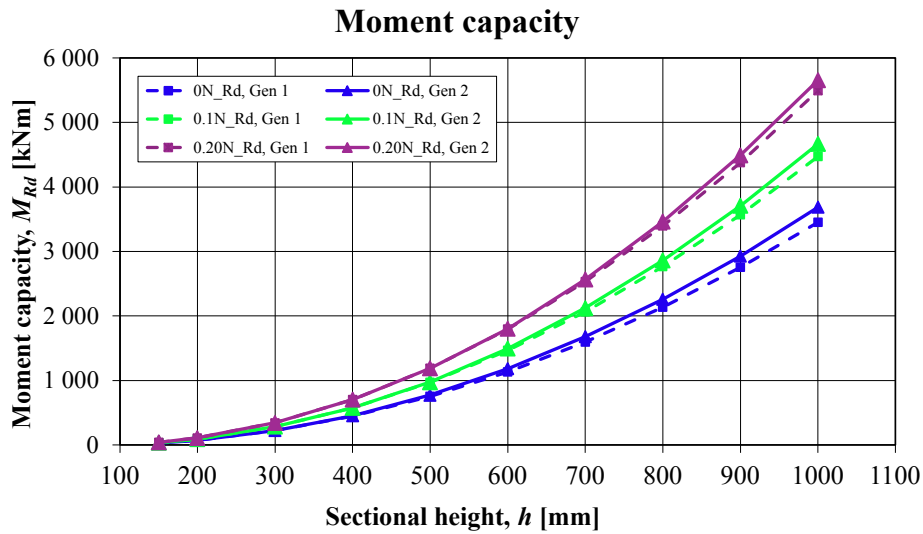
(b)

Figure A.1: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 0.2\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

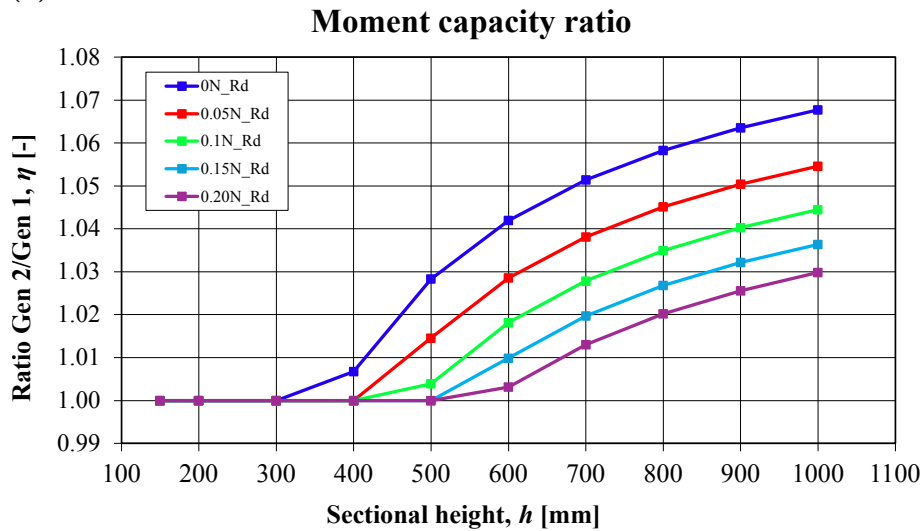
The moment capacity for a concrete slab with a reinforcement ratio of 0.2% increases between 0 to 8 % for a sectional height of $h = 150 - 1000$ mm when comparing the

generations of Eurocode (Figure A.1). The largest difference is observed when no axial force is added. For increased axial force, the difference in moment capacity ratio decreases, showing a negligible difference when the axial force becomes 20% of N_{Rd} . The additional 8% of moment resistance observed when $N_{Ed} = 0$ kN is due to the application of the design effective depth d_d , which becomes favoured for deeper sections.

A.4.2 Concrete class C35/45, $\rho_s = 1\%$



(a)



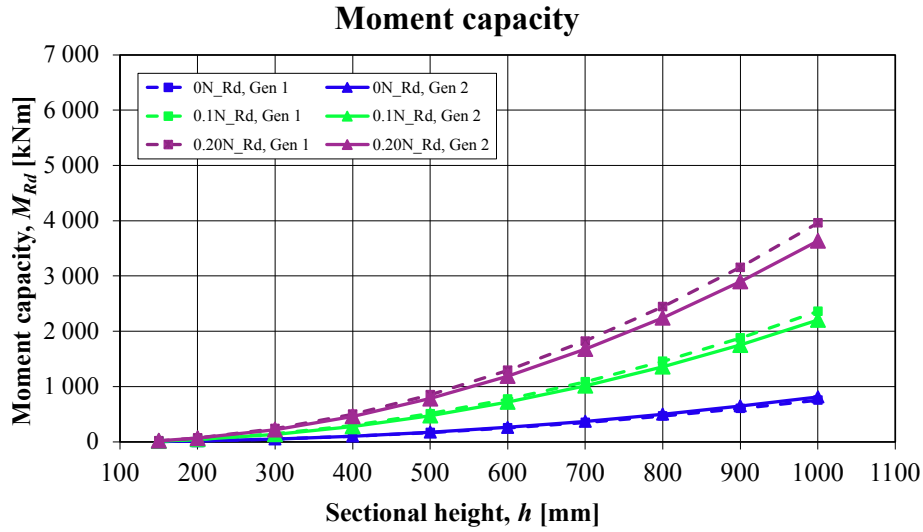
(b)

Figure A.2: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 1\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

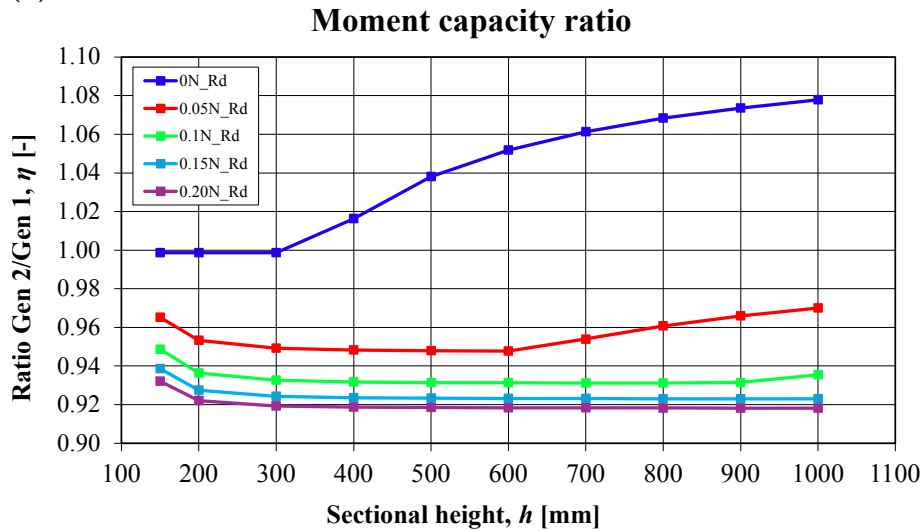
The same behaviour as has been observed with the lower reinforcement ratios can also be observed for $\rho_s = 1\%$ shown in Figure A.2, where the difference in capacity increases at increased sectional height but decreases at added axial force. The difference varies from 3-7% for the tallest sections, but shows a negligible difference for shorter sections.

A.4.3 Concrete class C55/67, $\rho_s = 0.2\%$

For a reinforcement ratio of 0.2% shown in Figure A.3, the increased concrete strength combined with axial force results in lower moment capacity for Gen 2. When no axial force is present, the behaviour of the moment capacity ratio is similar to that of Figure A.1b. When an axial load is present, the bending moment resistance decreases 3-8% for the tallest investigated sections.



(a)

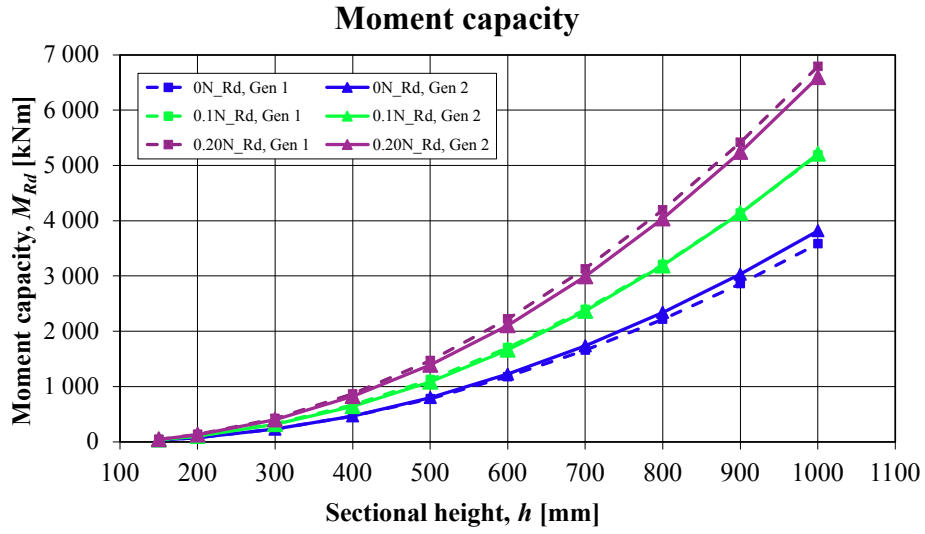


(b)

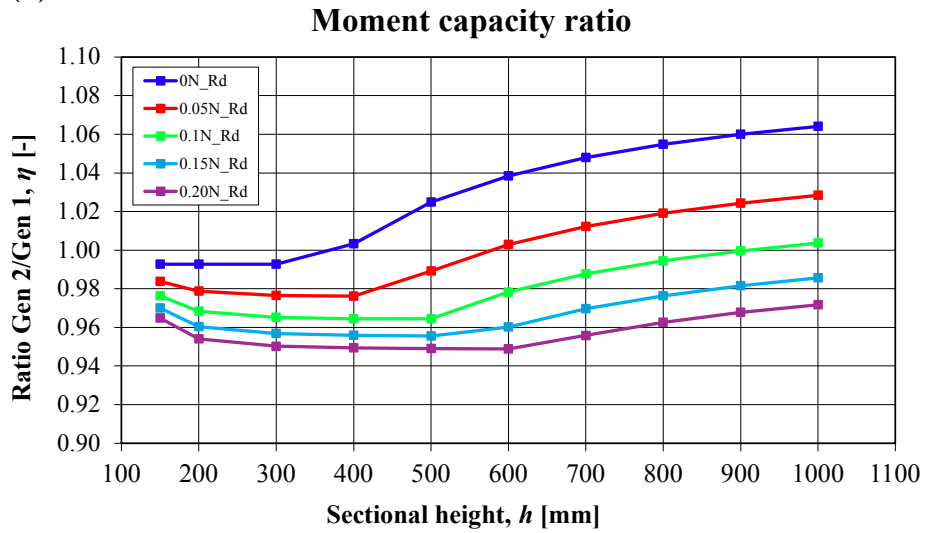
Figure A.3: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 0.2\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

A.4.4 Concrete class C55/67, $\rho_s = 1\%$

The overall behavior for concrete class C55/67 seen for $\rho_s = 0.2$ is also observed in Figure A.4 when $\rho_s = 1\%$, i.e., the normal force decreases the capacity ratio of Gen 2. However, the increased reinforcement ratio makes the negative impact from the axial load less significant than for lower ρ_s . For high axial loads, the bending moment resistance is still lower for Gen 2, even for the taller sections, than the result obtained from using the design method of Gen 1.



(a)



(b)

Figure A.4: Comparison of bending moment capacity in Gen 2 compared to Gen 1, with reinforcement ratio $\rho_s = 1\%$, for varying sectional height and increasing centric axial force. a) Bending moment capacity, M_{Rd} [kNm] and b) Moment capacity ratio, η [-].

B

SHEAR

B.1 Calculation for parametric study

The calculative sheet was used to calculate the shear force capacity with and without including stirrups. The resistance calculations excluding stirrups used input parameters listed in Section 9.3, while the variable parameters for the calculations including stirrups are listed in Section 9.4.

Shear verification - SS-EN1992-1-1:2005

Clause 6.2

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$t_1 := 500 \text{ mm}$
Spacing reinforcement	$s := 100 \text{ mm}$
Concrete cover reinforcement	$c := 55 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Diameter stirrups	$\phi_s := 12 \text{ mm}$
Effective depth	$d := t_1 - c - \frac{\phi}{2} = 437 \text{ mm}$
Spacing stirrups transversal	$s_t := 230 \text{ mm}$
Spacing stirrups longitudinal	$s_l := 200 \text{ mm}$
Reinforcement ratio	$\rho := 0.5\%$

1.2 Material

Compressive strength of concrete	$f_{ck} := 35 \text{ MPa}$
Factor concrete	$\alpha_c := 1$
Partial factor concrete	$\gamma_c := 1.5$

Characteristic yield strength reinforcement

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

Partial factor reinforcement

$$\gamma_s := 1.15$$

2 Calculation

2.1 Design material

Design concrete compressive strength

$$f_{cd} := \alpha_c \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

2.2 Sectional parameters

Area of concrete

$$A_c := b \cdot t_1 = 0.5 \text{ m}^2$$

Area of tensile reinforcement

$$A_s := \rho \cdot b \cdot d = (2.19 \cdot 10^{-3}) \text{ m}^2$$

2.3 Load

Normal force utilization

$$\eta_N := 5\%$$

Design normal force

$$N_{Ed} := -\eta_N \cdot A_c \cdot f_{cd} = -583.33 \text{ kN}$$

2.4 Members without shear reinforcement

EN 1992-1-1 6.2.2

National shear parameter

$$k_1 := 0.15$$

Crdc factor (NDP)

$$C_{Rdc} := \frac{0.18}{\gamma_c} = 0.12$$

k-factor

$$k := \min \left(1 + \sqrt{\frac{200 \text{ mm}}{d}}, 2 \right) = 1.68$$

Reinforcement ratio

$$\rho_1 := \min \left(\frac{A_s}{b \cdot d}, 0.02 \right) = 5 \cdot 10^{-3}$$

Axial stress (positive in compression)

$$\sigma_{cp} := \min \left(\frac{-N_{Ed}}{A_c}, 0.2 \cdot f_{cd} \right) = 1.167 \text{ MPa}$$

Design shear resistance without reinforcement

$$v_{min} := 0.035 \cdot k^{\frac{3}{2}} \cdot \left(\frac{f_{ck}}{\text{MPa}} \right)^{\frac{1}{2}} \cdot \text{MPa} = 0.45 \text{ MPa}$$

Shear sliding capacity

$$V_{Rdc} := \left(C_{Rdc} \cdot k \cdot \left(100 \cdot \rho_1 \cdot \frac{f_{ck}}{\text{MPa}} \right)^{\left(\frac{1}{3} \right)} \cdot \text{MPa} + k_1 \cdot \sigma_{cp} \right) \cdot b \cdot d = 304.73 \text{ kN}$$

Minimum capacity

$$V_{Rdc_min} := (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b \cdot d = 272.9 \text{ kN}$$

Strength reduction factor

$$v := 0.6 \cdot \left(1 - \frac{f_{ck}}{250 \cdot \text{MPa}} \right) = 0.52$$

Shear compression failure

$$V_{Rd_max} := 0.5 \cdot b \cdot d \cdot v \cdot f_{cd} = (2.63 \cdot 10^3) \text{ kN}$$

Design capacity

$$V_{Rdc} := \min(\max(V_{Rdc}, V_{Rdc_min}), V_{Rd_max}) = 304.73 \text{ kN}$$

2.5 Members with shear reinforcement

Angle of compression chord

$$\cot\theta := 2.5$$

Angle of stirrups (vertical)

$$\alpha := 75 \text{ deg}$$

Stress state coefficient

$$\alpha_{cw} := 1$$

Area of shear reinforcement

$$A_{sw} := \phi_s^2 \cdot \frac{\pi}{4} \cdot \frac{b}{s_t} = 491.73 \text{ mm}^2$$

Shear reinforcement ratio

$$\rho_w := \frac{A_{sw}}{s_l \cdot b \cdot \sin(\alpha)} = 2.55 \cdot 10^{-3}$$

Check minimum ratio
SS EN 1992-1-1-2005 (9.2.2)

$$\rho_{min} := \frac{\left(0.08 \cdot \sqrt{\frac{f_{ck}}{\text{MPa}}} \right)}{\frac{f_{yk}}{\text{MPa}}} = 9.47 \cdot 10^{-4}$$

$$\rho_w > \rho_{min} = 1$$

Lever arm for shear

$$z := 0.9 \cdot d = 0.39 \text{ m}$$

Shear resistance with stirrups

$$V_{Rds} := \frac{A_{sw}}{s_l} \cdot z \cdot f_{yd} \cdot \cot\theta = (1.05 \cdot 10^3) \text{ kN}$$

Maximum capacity of stirrups

$$V_{Rds_max} := \alpha_{cw} \cdot b \cdot z \cdot v \cdot \frac{f_{cd}}{\left(\cot\theta + \frac{1}{\cot\theta} \right)} = (1.63 \cdot 10^3) \text{ kN}$$

Design shear capacity with stirrups

$$V_{Rds} := \min(V_{Rds}, V_{Rds_max}) = (1.05 \cdot 10^3) \text{ kN}$$

Shear capacity with stirrups with inclination

$$V_{Rds_a} := \frac{A_{sw}}{s} \cdot z \cdot f_{yd} \cdot (\cot\theta + \cot(\alpha)) \cdot \sin(\alpha) = (2.25 \cdot 10^3) \text{ kN}$$

Maximum capacity of inclination stirrups

$$V_{Rds_max_a} := \alpha_{cw} \cdot b \cdot z \cdot v \cdot f_{cd} \cdot \frac{(\cot\theta + \cot(\alpha))}{1 + \cot\theta^2} = (1.81 \cdot 10^3) \text{ kN}$$

Design shear capacity with inclined stirrups

$$V_{Rds_a} := \min(V_{Rds_a}, V_{Rds_max_a}) = (1.81 \cdot 10^3) \text{ kN}$$

Shear verification - SS-EN1992-1-1:2023

clause 8.2

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$t_1 := 500 \text{ mm}$
Spacing reinforcement	$s := 100 \text{ mm}$
Concrete cover reinforcement	$c := 55 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Diameter stirrups	$\phi_s := 12 \text{ mm}$
Spacing stirrups transversal	$s_t := 230 \text{ mm}$

Spacing stirrups longitudinal

$$s_l := 200 \text{ mm}$$

Reinforcement ratio

$$\rho := 0.5\%$$

1.2 Material

Characteristic concrete compressive strength

$$f_{ck} := 35 \text{ MPa}$$

Partial factor shear for:
 d_{nom}

$$\gamma_v := 1.4$$

 d_d

$$\gamma_{vd} := 1.29$$

Partial factor concrete

$$\gamma_c := 1.5$$

Characteristic yield strength reinforcement

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

Partial factor steel for:
 d_{nom}

$$\gamma_s := 1.15$$

 d_d

$$\gamma_{sd} := 1.034$$

Reference compressive strength

$$f_{ck_ref} := 40 \text{ MPa}$$

Strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\left(\frac{1}{3} \right)}, 1 \right) = 1$$

Factor for reference time other than 28 days

$$k_{te} := 1$$

Minimum aggregate diameter for upper sieve size

$$D_{lower} := 20 \text{ mm}$$

Aggregate size parameter

$$d_{dg} := \begin{cases} f_{ck} < 60 \text{ MPa} \\ \left| \begin{array}{l} \min(16 \text{ mm} + D_{lower}, 40 \text{ mm}) \\ \text{if } f_{ck} > 60 \text{ MPa} \\ \left| \min \left(16 \text{ mm} + D_{lower} \cdot \left(\frac{60 \text{ MPa}}{f_{ck}} \right)^2, 40 \text{ mm} \right) \right| \end{array} \right| = 36 \text{ mm} \end{cases}$$

2 Calculation using d_{nom}

2.1 Design material

Design compressive strength concrete

$$f_{cd} := \eta_{cc} \cdot k_{te} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength rebar

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

2.2 Sectional parameters

Effective depth, nominal

$$d_{\text{nom}} := t_1 - c - \frac{\phi}{2} = 437 \text{ mm}$$

Concrete area

$$A_c := b \cdot t_1 = 0.5 \text{ m}^2$$

Area of reinforcement

$$A_s := \rho \cdot b \cdot d_{\text{nom}} = (2.19 \cdot 10^{-3}) \text{ m}^2$$

2.3 Minimum shear resistance

Minimum shear resistance with d_{nom}

$$\tau_{\text{Rdc_min}} := \frac{11}{\gamma_v} \cdot \sqrt{\frac{f_{\text{ck}}}{f_{\text{yd}}} \cdot \frac{d_{\text{dg}}}{d_{\text{nom}}}} \cdot \text{MPa} = 0.64 \text{ MPa}$$

2.4 Load

Change these values if needed

Shear force utilization

$$\eta_v := 100\%$$

Design shear force

$$V_{\text{Ed}} := \eta_v \cdot \tau_{\text{Rdc_min}} \cdot d_{\text{nom}} \cdot b \cdot 0.9 = 251.65 \text{ kN}$$

Moment utilization

$$\eta_M := 50\%$$

Design bending moment (for calculating required reinforcement area)

$$M_{\text{Ed}} := \eta_M \cdot A_s \cdot 0.9 \cdot d_{\text{nom}} \cdot f_{\text{yd}} = 186.82 \text{ kN} \cdot \text{m}$$

Normal force utilization

$$\eta_N := 5\%$$

Design normal force

$$N_{\text{Ed}} := -\eta_N \cdot A_c \cdot f_{\text{cd}} = -583.33 \text{ kN}$$

2.5 Detailed verification without shear reinforcement (8.2.2)

Effective shear span

$$a_{\text{cs}} := \max \left(\left| \frac{M_{\text{Ed}}}{V_{\text{Ed}}} \right|, d_{\text{nom}} \right) = 742.37 \text{ mm}$$

Coefficient for axial forces

$$k_{\text{vp}} := \max \left(1 + \frac{N_{\text{Ed}}}{|V_{\text{Ed}}|} \cdot \frac{d_{\text{nom}}}{3 \cdot a_{\text{cs}}}, 0.1 \right) = 0.55$$

Mechanical shear span

$$a_v := \begin{cases} \text{if } a_{\text{cs}} < 4 \cdot d_{\text{nom}} \\ \left\| \sqrt{\frac{a_{\text{cs}}}{4} \cdot d_{\text{nom}}} \right\| \\ \text{else} \\ \left\| d_{\text{nom}} \right\| \end{cases} = 284.79 \text{ mm}$$

Shear stress resistance

$$\tau_{\text{Rdc}} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho \cdot \frac{f_{\text{ck}}}{\text{MPa}} \cdot \frac{d_{\text{dg}}}{k_{\text{vp}} \cdot a_v} \right)^{\left(\frac{1}{3} \right)} \cdot \text{MPa} = 0.752 \text{ MPa}$$

Design value of shear stress resistance

$$\tau_{Rdc} := \max(\tau_{Rdc}, \tau_{Rdc_min}) = 0.75 \text{ MPa}$$

Shear stress converted to shear force

$$V_{Rdc_nom} := \tau_{Rdc} \cdot 0.9 \cdot d_{nom} \cdot b = 295.74 \text{ kN}$$

2.6 Alternative approach considering axial forces, clause 8.2.2(5)

Compressive stress

$$\sigma_{cp} := \frac{N_{Ed}}{A_c} = -1.17 \cdot 10^3 \text{ kPa}$$

Eccentricity of compressive force

$$e_p := 0$$

Inner lever arm

$$z := 0.9 \cdot d_{nom} = 0.39 \text{ m}$$

Factor from NDP (note! No moment from eccentricity of compressive force included in M_{Ed})

$$k_1 := \min\left(\frac{0.5}{a_{cs}} \cdot \left(e_p + \frac{d_{nom}}{3}\right) \cdot \frac{A_c}{b \cdot z}, 0.18 \cdot \frac{A_c}{b \cdot z}\right) = 0.12$$

Ultimate shear resistance

$$\tau_{Rdc_max} := 2.7 \cdot \tau_{Rdc} = 2.03 \text{ MPa}$$

Design value of shear stress

$$\tau_{Rdc_0} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho \cdot \frac{f_{ck}}{\text{MPa}} \cdot \frac{d_{dg}}{a_v}\right)^{\left(\frac{1}{3}\right)} \cdot \text{MPa} = 0.61 \text{ MPa}$$

Shear capacity with regards to compressive axial forces

$$\tau_{Rdc_alt} := \tau_{Rdc_0} - k_1 \cdot \sigma_{cp} = 0.76 \text{ MPa}$$

Final design capacity without stirrups

$$\tau_{Rdc_alt} := \max\left(\min(\tau_{Rdc_max}, \tau_{Rdc_alt}), \tau_{Rdc_min}\right) = 0.76 \text{ MPa}$$

Shear force resistance

$$V_{Rdc_alt_nom} := \tau_{Rdc_alt} \cdot 0.9 \cdot d_{nom} \cdot b = 298.82 \text{ kN}$$

2.7 Detailed verification with reinforcement (8.2.3)

Design shear force (assumption)

$$V_{Ed} := 3 \cdot V_{Rdc_alt_nom} = 896.46 \text{ kN}$$

Design shear stress

$$\tau_{Ed} := \frac{V_{Ed}}{b \cdot 0.9 \cdot d_{nom}} = 2.28 \text{ MPa}$$

Angle of compressive field

$$\cot\Theta := 2.5$$

Angle of stirrups (between 45 and 90 degrees)

$$\alpha := 75 \text{ deg}$$

Factor (For standard compression field angle)

$$v := 0.5$$

Area of stirrups

$$A_{sw} := \phi_s^2 \cdot \frac{\pi}{4} \cdot \frac{b}{s_t} = (4.92 \cdot 10^{-4}) \text{ m}^2$$

Reinforcement ratio stirrups

$$\rho_w := \frac{A_{sw}}{b \cdot s_l} = 2.46 \cdot 10^{-3}$$

Minimum shear reinforcement ratio

$$\rho_{w_min} := 0.08 \cdot \frac{\sqrt{\frac{f_{ck}}{\text{MPa}}}}{\frac{f_{yk}}{\text{MPa}}} = 9.47 \cdot 10^{-4}$$

Check shear reinforcement ratio

$$\rho_w > \rho_{w_min} = 1$$

2.7.1 Shear resistance with right angle stirrups

Shear resistance reinforcement yielding

$$\tau_{Rdc_sy} := \rho_w \cdot \frac{f_{yk}}{\gamma_s} \cdot \cot \Theta = 2.67 \text{ MPa}$$

Stress in compressive field verification

$$\sigma_{cd} := \tau_{Ed} \cdot \left(\cot \Theta + \frac{1}{\cot \Theta} \right) = 6.61 \text{ MPa}$$

$$\sigma_{cd_lim} := v \cdot f_{cd} = 11.67 \text{ MPa}$$

$$\sigma_{cd} < \sigma_{cd_lim} = 1$$

2.7.3 Shear stress to shear force

Shear force resistance for comparison with Gen 1

Right angle stirrups

$$V_{Rd_sy_nom} := \tau_{Rdc_sy} \cdot 0.9 \cdot d_{nom} \cdot b = (1.05 \cdot 10^3) \text{ kN}$$

3 Calculation using d_d

3.1 Design material

Design compressive strength concrete

$$f_{cd} = 23.33 \text{ MPa}$$

Design yield strength

$$f_{yd_d} := \frac{f_{yk}}{\gamma_{sd}} = 483.56 \text{ MPa}$$

3.2 Sectional parameters

Effective depth, design (A.8)

$$d_d := d_{nom} - 15 \text{ mm} = 422 \text{ mm}$$

Area of reinforcement

$$A_{s_d} := \rho \cdot b \cdot d_d = (2.11 \cdot 10^{-3}) \text{ m}^2$$

3.3 Minimum shear resistance

Minimum shear resistance with d_d

$$\tau_{Rdc_min2} := \frac{11}{\gamma_{vd}} \cdot \sqrt{\frac{f_{ck}}{f_{yd_d}} \cdot \frac{d_{dg}}{d_d}} \cdot \text{MPa} = 0.67 \text{ MPa}$$

3.4 Load

Shear force utilization

$$\eta_V = 1$$

Design shear force

$$V_{Ed} := \eta_V \cdot \tau_{Rdc_min2} \cdot d_d \cdot b \cdot 0.9 = 254.49 \text{ kN}$$

Moment utilization

$$\eta_M = 0.5$$

Design bending moment (for calculating required reinforcement area)

$$M_{Ed} := \eta_M \cdot A_{s_d} \cdot 0.9 \cdot d_d \cdot f_{yd_d} = 193.76 \text{ kN} \cdot \text{m}$$

Normal force utilization

$$\eta_N = 0.05$$

Design normal force

$$N_{Ed} := -\eta_N \cdot A_c \cdot f_{cd} = -583.33 \text{ kN}$$

3.5 Detailed verification without shear reinforcement (8.2.2)

Effective shear span

$$a_{cs} := \max \left(\left| \frac{M_{Ed}}{V_{Ed}} \right|, d_{nom} \right) = 761.37 \text{ mm}$$

Coefficient for axial forces

$$k_{vp} := \max \left(1 + \frac{N_{Ed}}{|V_{Ed}|} \cdot \frac{d_{nom}}{3 \cdot a_{cs}}, 0.1 \right) = 0.56$$

Mechanical shear span

$$a_v := \begin{cases} \text{if } a_{cs} < 4 \cdot d_{nom} \\ \sqrt{\frac{a_{cs}}{4} \cdot d_{nom}} \\ \text{else} \\ d_d \end{cases} = 288.41 \text{ mm}$$

Shear stress resistance

$$\tau_{Rdc} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho \cdot \frac{f_{ck}}{\text{MPa}} \cdot \frac{d_{dg}}{k_{vp} \cdot a_v} \right)^{\left(\frac{1}{3} \right)} \cdot \text{MPa} = 0.741 \text{ MPa}$$

Design value of shear stress resistance

$$\tau_{Rdc} := \max (\tau_{Rdc}, \tau_{Rdc_min2}) = 0.74 \text{ MPa}$$

Shear stress converted to shear force

$$V_{Rdc_d} := \tau_{Rdc} \cdot 0.9 \cdot d_{nom} \cdot b = 291.62 \text{ kN}$$

3.6 Alternative approach considering axial forces, clause 8.2.2(5)

Compressive stress

$$\sigma_{cp} := \frac{N_{Ed}}{A_c} = -1.17 \cdot 10^3 \text{ kPa}$$

Eccentricity of compressive force

$$e_p := 0$$

Inner lever arm

$$z := 0.9 \cdot d_{nom} = 0.39 \text{ m}$$

Factor from NDP (note! No moment from eccentricity of compressive force included in M_{Ed})

$$k_j := \min \left(\frac{0.5}{a_{cs}} \cdot \left(e_p + \frac{d_{nom}}{3} \right) \cdot \frac{A_c}{b \cdot z}, 0.18 \cdot \frac{A_c}{b \cdot z} \right) = 0.12$$

Ultimate shear resistance

$$\tau_{Rdc_max} := 2.7 \cdot \tau_{Rdc} = 2 \text{ MPa}$$

Design value of shear stress

$$\tau_{Rdc_0} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho \cdot \frac{f_{ck}}{\text{MPa}} \cdot \frac{d_{dg}}{a_v} \right)^{\left(\frac{1}{3}\right)} \cdot \text{MPa} = 0.61 \text{ MPa}$$

Shear capacity with regards to compressive axial forces

$$\tau_{Rdc_alt_d} := \tau_{Rdc_0} - k_1 \cdot \sigma_{cp} = 0.75 \text{ MPa}$$

Final design capacity without stirrups

$$\tau_{Rdc_alt_d} := \max \left(\min \left(\tau_{Rdc_max}, \tau_{Rdc_alt_d} \right), \tau_{Rdc_min2} \right) = 0.75 \text{ MPa}$$

Shear force resistance

$$V_{Rdc_alt_d} := \tau_{Rdc_alt_d} \cdot 0.9 \cdot d_d \cdot b = 286.2 \text{ kN}$$

3.7 Detailed verification with reinforcement (8.2.3)

Design shear force (assumption)

$$V_{Ed} := 3 \cdot V_{Rdc_alt_d} = 858.61 \text{ kN}$$

Design shear stress

$$\tau_{Ed} := \frac{V_{Ed}}{b \cdot 0.9 \cdot d_d} = 2.26 \text{ MPa}$$

Angle of compressive field

$$\cot \Theta := 2.5$$

Factor (For standard compression field angle)

$$\eta := 0.5$$

Area of stirrups

$$A_{sw} := \phi_s^2 \cdot \frac{\pi}{4} \cdot \frac{b}{s_t} = (4.92 \cdot 10^{-4}) \text{ m}^2$$

Reinforcement ratio stirrups

$$\rho_w := \frac{A_{sw}}{b \cdot s_l} = 2.46 \cdot 10^{-3}$$

Minimum shear reinforcement ratio

$$\rho_{w_min} := 0.08 \cdot \frac{\sqrt{\frac{f_{ck}}{\text{MPa}}}}{\frac{f_{yk}}{\text{MPa}}} = 9.47 \cdot 10^{-4}$$

Check shear reinforcement ratio

$$\rho_w > \rho_{w_min} = 1$$

3.7.1 Shear resistance with right angle stirrups

Shear resistance reinforcement yielding

$$\tau_{Rdc_sy} := \rho_w \cdot \frac{f_{yk}}{\gamma_{sd}} \cdot \cot \Theta = 2.97 \text{ MPa}$$

Stress in compressive field verification

$$\sigma_{cd} := \tau_{Ed} \cdot \left(\cot \Theta + \frac{1}{\cot \Theta} \right) = 6.56 \text{ MPa}$$

$$\sigma_{cd_lim} := v \cdot f_{cd} = 11.67 \text{ MPa}$$

$$\sigma_{cd} < \sigma_{cd_lim} = 1$$

Simultaneous yielding and crushing

$$\cot\Theta_{\text{lim}} := \sqrt{v \cdot \frac{f_{cd}}{\rho_w \cdot f_{yd_d}}} - 1 = 2.97$$

$$\tau_{\text{Rdc_sim}} := \rho_w \cdot \frac{f_{yk}}{\gamma_{sd}} \cdot \cot\Theta_{\text{lim}} = 3.53 \text{ MPa}$$

$$\tau_{\text{Rdc_sim}} < v \cdot \frac{f_{cd}}{2} = 1$$

$$\tau_{\text{Rd}} := \min(\tau_{\text{Rdc_sim}}, \tau_{\text{Rdc_sy}}) = (2.97 \cdot 10^6) \text{ Pa}$$

$$V_{\text{Rd_sy_d}} := d_d \cdot 0.9 \cdot b \cdot \tau_{\text{Rd}} = (1.13 \cdot 10^3) \text{ kN}$$

4 Comparison of d_{nom} and d_d

4.1 Detailed verification without stirrups

$$V_{\text{Rdc}} := \max(V_{\text{Rdc_nom}}, V_{\text{Rdc_d}}) = 295.74 \text{ kN}$$

4.2 Alternative approach without stirrups

$$V_{\text{Rdc_alt}} := \max(V_{\text{Rdc_alt_nom}}, V_{\text{Rdc_alt_d}}) = 298.82 \text{ kN}$$

4.1 Detailed verification with stirrups

Right angle stirrups

$$V_{\text{Rd_sy}} := \max(V_{\text{Rd_sy_nom}}, V_{\text{Rd_sy_d}}) = (1.13 \cdot 10^3) \text{ kN}$$

B.2 Case study calculations

Calculation is made at the critical sections defined in Section 10.2.1 (edge strip and mid strip, support section)

Shear verification - SS-EN1992-1-1:2005

clause 6.2 - Support section, Edge strip

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$h := 830 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Reinforcement stirrups	$\phi_s := 12 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Concrete cover (top)	$c := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Spacing stirrups transversal	$s_t := 280 \text{ mm}$
Spacing stirrups longitudinal	$s_l := 450 \text{ mm}$
Effective depth, first layer	$d := h - c - \frac{\phi}{2} = 764 \text{ mm}$

1.2 Material

Compressive strength of concrete	$f_{ck} := 35 \text{ MPa}$
Factor concrete	$\alpha_c := 1$
Partial factor concrete	$\gamma_c := 1.5$

Characteristic yield strength reinforcement

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

Partial factor reinforcement

$$\gamma_s := 1.15$$

2 Calculation

2.1 Design material

Design concrete compressive strength

$$f_{cd} := \alpha_c \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

2.2 Sectional parameters

Area of concrete

$$A_c := b \cdot h = 0.83 \text{ m}^2$$

Area of tensile reinforcement at d

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Reinforcement ratio, top layer

$$\rho_d := \frac{A_s}{d \cdot b} = 3.16 \cdot 10^{-3}$$

2.3 Load

Design normal force

$$N_{Ed} := 0 \text{ kN}$$

Design moment (Approximation)

$$M_{Ed} := A_s \cdot 0.9 \cdot d \cdot f_{yd} = 722.46 \text{ kN} \cdot \text{m}$$

2.4 Members without shear reinforcement

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National shear parameter

$$k_1 := 0.15$$

Crdc factor (NDP)

$$C_{Rdc} := \frac{0.18}{\gamma_c} = 0.12$$

k-factor

$$k := \min \left(1 + \sqrt{\frac{200 \text{ mm}}{d}}, 2 \right) = 1.51$$

Reinforcement ratio

$$\rho_1 := \min \left(\frac{A_s}{b \cdot d}, 0.02 \right) = 3.16 \cdot 10^{-3}$$

Axial stress (positive in compression)

$$\sigma_{cp} := \min \left(\frac{-N_{Ed}}{A_c}, 0.2 \cdot f_{cd} \right) = 0 \text{ MPa}$$

Design shear resistance without reinforcement

$$v_{min} := 0.035 \cdot k^{\frac{3}{2}} \cdot \left(\frac{f_{ck}}{\text{MPa}} \right)^{\frac{1}{2}} \cdot \text{MPa} = 0.38 \text{ MPa}$$

Shear sliding capacity

$$V_{Rdc} := \left(C_{Rdc} \cdot k \cdot \left(100 \cdot \rho_1 \cdot \frac{f_{ck}}{\text{MPa}} \right)^{\left(\frac{1}{3} \right)} \cdot \text{MPa} + k_1 \cdot \sigma_{cp} \right) \cdot b \cdot d = 308.88 \text{ kN}$$

Minimum capacity

$$V_{Rdc_min} := (v_{min} + k_1 \cdot \sigma_{cp}) \cdot b \cdot d = 294.02 \text{ kN}$$

Strength reduction factor

$$v := 0.6 \cdot \left(1 - \frac{f_{ck}}{250 \cdot \text{MPa}} \right) = 0.52$$

Shear compression failure

$$V_{Rd_max} := 0.5 \cdot b \cdot d \cdot v \cdot f_{cd} = (4.6 \cdot 10^3) \text{ kN}$$

Design capacity

$$V_{Rdc} := \min \left(\max \left(V_{Rdc}, V_{Rdc_min} \right), V_{Rd_max} \right) = 308.88 \text{ kN}$$

2.5 Members with shear reinforcement

Angle of compression chord

$$\cot \theta := 2.5$$

Angle of stirrups (vertical)

$$\alpha := 70 \text{ deg}$$

Stress state coefficient

$$\alpha_{cw} := 1$$

Area of shear reinforcement

$$A_{sw} := \phi_s^2 \cdot \frac{\pi}{4} \cdot \frac{b}{s_t} = 403.92 \text{ mm}^2$$

Shear reinforcement ratio

$$\rho_w := \frac{A_{sw}}{s_l \cdot b \cdot \sin(\alpha)} = 9.55 \cdot 10^{-4}$$

Check minimum ratio
SS EN 1992-1-1-2005 (9.2.2)

$$\rho_{min} := \frac{\left(0.08 \cdot \sqrt{\frac{f_{ck}}{\text{MPa}}} \right)}{\frac{f_{yk}}{\text{MPa}}} = 9.47 \cdot 10^{-4}$$

$$\rho_w > \rho_{min} = 1$$

Lever arm for shear

$$z := 0.9 \cdot d = 0.69 \text{ m}$$

Shear capacity with stirrups with inclination

$$V_{Rds_a} := \frac{A_{sw}}{s_l} \cdot z \cdot f_{yd} \cdot (\cot \theta + \cot(\alpha)) \cdot \sin(\alpha) = 722.18 \text{ kN}$$

Maximum capacity of inclination stirrups

$$V_{Rds_max_a} := \alpha_{cw} \cdot b \cdot z \cdot v \cdot f_{cd} \cdot \frac{(\cot \theta + \cot(\alpha))}{1 + \cot^2 \theta} = (3.27 \cdot 10^3) \text{ kN}$$

Design shear capacity with inclined stirrups

$$V_{Rds_a} := \min \left(V_{Rds_a}, V_{Rds_max_a} \right) = 722.18 \text{ kN}$$

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clause 8.2 - support section, edge strip

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$h := 830 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Reinforcement stirrups	$\phi_s := 12 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Concrete cover (top)	$c := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Spacing stirrups transversal	$s_t := 280 \text{ mm}$
Spacing stirrups longitudinal	$s_l := 450 \text{ mm}$
Deviation of effective depth	$\Delta d := 15 \text{ mm}$

1.2 Material

Characteristic concrete compressive strength	$f_{ck} := 35 \text{ MPa}$
Partial factor shear for d_d	$\gamma_{vd} := 1.29$
Partial factor shear for d_{nom}	$\gamma_v := 1.4$
Partial factor concrete	$\gamma_c := 1.5$

Characteristic yield strength reinforcement

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

Partial factor steel for d_d

$$\gamma_{sd} := 1.034$$

Reference compressive strength

$$f_{ck_ref} := 40 \text{ MPa}$$

Strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\left(\frac{1}{3} \right)}, 1 \right) = 1$$

Factor for reference time other than 28 days

$$k_{tc} := 1$$

Minimum aggregate diameter for upper sieve size

$$D_{lower} := 20 \text{ mm}$$

Aggregate size parameter

$$d_{dg} := \begin{cases} \text{if } f_{ck} < 60 \text{ MPa} \\ \min(16 \text{ mm} + D_{lower}, 40 \text{ mm}) \\ \text{if } f_{ck} > 60 \text{ MPa} \\ \min \left(16 \text{ mm} + D_{lower} \cdot \left(\frac{60 \text{ MPa}}{f_{ck}} \right)^2, 40 \text{ mm} \right) \end{cases} = 36 \text{ mm}$$

2 Calculation using d_d

2.1 Design material

Design compressive strength concrete

$$f_{cd} := \eta_{cc} \cdot k_{tc} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength rebar

$$f_{yd} := \frac{f_{yk}}{\gamma_{sd}} = 484 \text{ MPa}$$

2.2 Sectional parameters

Effective depth, first layer

$$d_{nom} := h - c - \frac{\phi}{2} = 764 \text{ mm}$$

Design effective depth

$$d_d := d_{nom} - \Delta d = 749 \text{ mm}$$

Area of concrete

$$A_c := b \cdot h = 0.83 \text{ m}^2$$

Area of tensile reinforcement at d_{nom}

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Reinforcement ratio, top layer

$$\rho_d := \frac{A_s}{d_d \cdot b} = 3.23 \cdot 10^{-3}$$

2.3 Minimum shear stress

Minimum shear resistance with d_{nom}

$$\tau_{Rdc_min} := \frac{11}{\gamma_{vd}} \cdot \sqrt{\frac{f_{ck}}{f_{yd}} \cdot \frac{d_{dg}}{d_d}} \cdot \text{MPa} = 0.5 \text{ MPa}$$

2.4 Load

Change these values if needed

Design shear force (assumed to be equal to the minimum capacity of the section)

$$V_{Ed} := 0.9 \cdot d_d \cdot b \cdot \tau_{Rdc_min} = 339.04 \text{ kN}$$

Design bending moment (Approximation)

$$M_{Ed} := A_s \cdot 0.9 \cdot d_d \cdot f_{yd} = 787.74 \text{ kN} \cdot \text{m}$$

Design normal force

$$N_{Ed} := 0 \text{ kN}$$

2.5 Detailed verification without shear reinforcement (8.2.2)

Effective shear span

$$a_{cs} := \max \left(\left| \frac{M_{Ed}}{V_{Ed}} \right|, d_{nom} \right) = (2.32 \cdot 10^3) \text{ mm}$$

Coefficient for axial forces

$$k_{vp} := \max \left(1 + \frac{N_{Ed}}{|V_{Ed}|} \cdot \frac{d_{nom}}{3 \cdot a_{cs}}, 0.1 \right) = 1$$

Mechanical shear span

$$a_v := \begin{cases} \text{if } a_{cs} < 4 \cdot d_{nom} \\ \left\| \sqrt{\frac{a_{cs}}{4} \cdot d_{nom}} \right\| \\ \text{else} \\ \left\| d_{nom} \right\| \end{cases} = 666.17 \text{ mm}$$

Shear stress resistance

$$\tau_{Rdc} := \frac{0.66}{\gamma_v} \cdot \left(100 \cdot \rho_d \cdot \frac{f_{ck}}{\text{MPa}} \cdot \frac{d_{dg}}{k_{vp} \cdot a_v} \right)^{\left(\frac{1}{3} \right)} \cdot \text{MPa} = 0.4 \text{ MPa}$$

Design value of shear stress resistance

$$\tau_{Rdc} := \max (\tau_{Rdc}, \tau_{Rdc_min}) = 0.5 \text{ MPa}$$

Shear stress converted to shear force

$$V_{Rdc} := \tau_{Rdc} \cdot 0.9 \cdot d_d \cdot b = 339.04 \text{ kN}$$

2.6 Detailed verification with reinforcement (8.2.3)

Shear resistance with angled stirrups

Design shear force (assumption)

$$V_{Ed} := 722 \text{ kN}$$

Design shear stress

$$\tau_{Ed} := \frac{V_{Ed}}{b \cdot 0.9 \cdot d_d} = 1.07 \text{ MPa}$$

Angle of compressive field	$\cot\Theta := 2.5$
Angle of stirrups (between 45 and 90 degrees)	$\alpha := 70 \text{ deg}$
Factor (For standard compression field angle)	$v := 0.5$
Area of stirrups	$A_{sw} := \phi_s^2 \cdot \frac{\pi}{4} \cdot \frac{b}{s_t} = 403.92 \text{ mm}^2$
Reinforcement ratio stirrups	$\rho_w := \frac{A_{sw}}{b \cdot s_l} = 8.98 \cdot 10^{-4}$
Minimum shear reinforcement ratio	$\rho_{w_min} := 0.08 \cdot \frac{\sqrt{\frac{f_{ck}}{\text{MPa}}}}{\frac{f_{yk}}{\text{MPa}}} = 9.47 \cdot 10^{-4}$
Check shear reinforcement ratio	$\rho_w > \rho_{w_min} = 0$
Shear resistance reinforcement yielding	$\tau_{Rd_sy_a} := \rho_w \cdot f_{yd} \cdot (\cot\Theta + \cot(\alpha)) \cdot \sin(\alpha) = 1.17 \text{ MPa}$
Stress in compressive field verification	$\sigma_{cd_a} := \tau_{Ed} \cdot \frac{(1 + \cot\Theta^2)}{\cot\Theta + \cot(\alpha)} = 2.71 \text{ MPa}$
	$\sigma_{cd_lim_a} := v \cdot f_{cd} = 11.67 \text{ MPa}$
	$\sigma_{cd_a} < \sigma_{cd_lim_a} = 1$
Angled stirrups, shear stress to shear force	$V_{Rd_sy_a_nom} := \tau_{Rd_sy_a} \cdot 0.9 \cdot d_d \cdot b = 787.43 \text{ kN}$

B.3 Additional graphs

These graphs cover additional loading scenarios than those included in the result of the parametric study of shear force capacity without shear reinforcement. Other input parameters are the same as presented in Section 9.3.

B.3.1 Shear force ($V_{Ed} = V_{Rd}$) and bending moment ($M_{Ed} = 0.25M_{Rd}$)

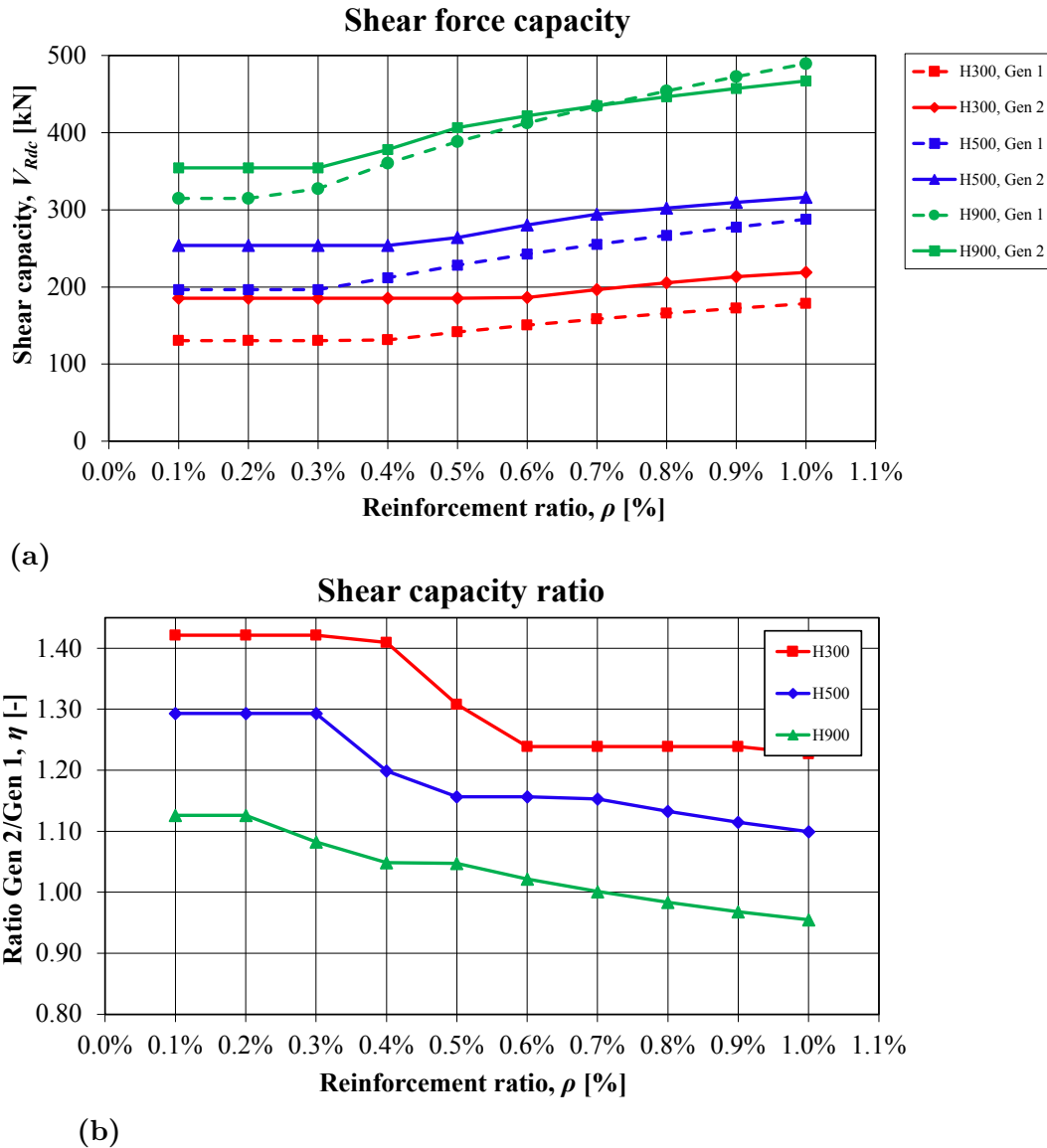
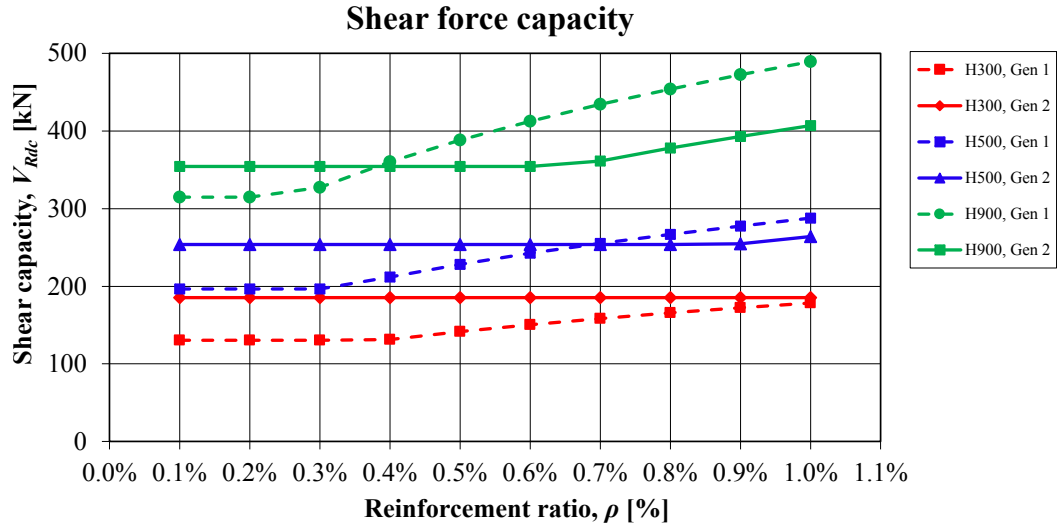
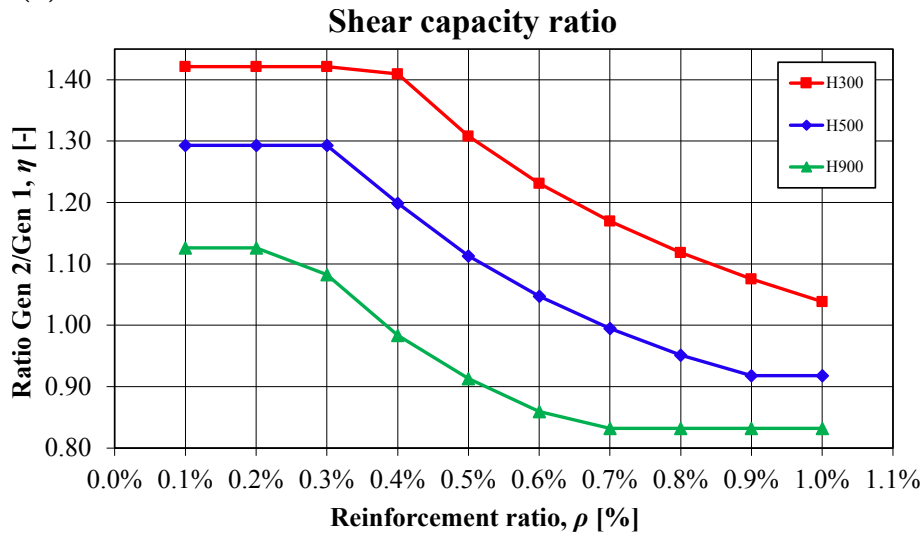


Figure B.1: Loaded with shear $V_{Ed} = V_{Rd,c}$ and moment $M_{Ed} = 0.25 \cdot M_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

B.3.2 Shear force ($V_{Ed} = V_{Rd}$) and bending moment ($M_{Ed} = 0.75M_{Rd}$)



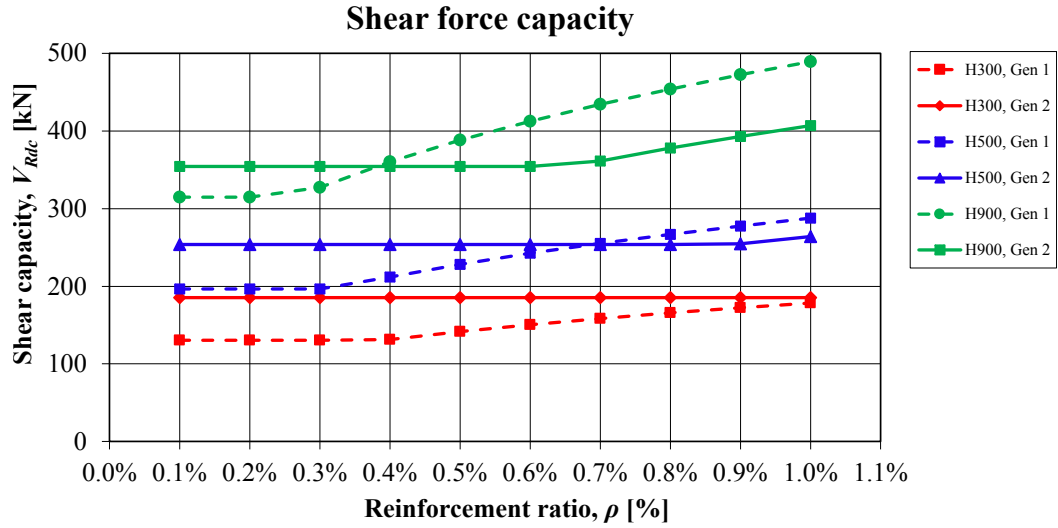
(a)



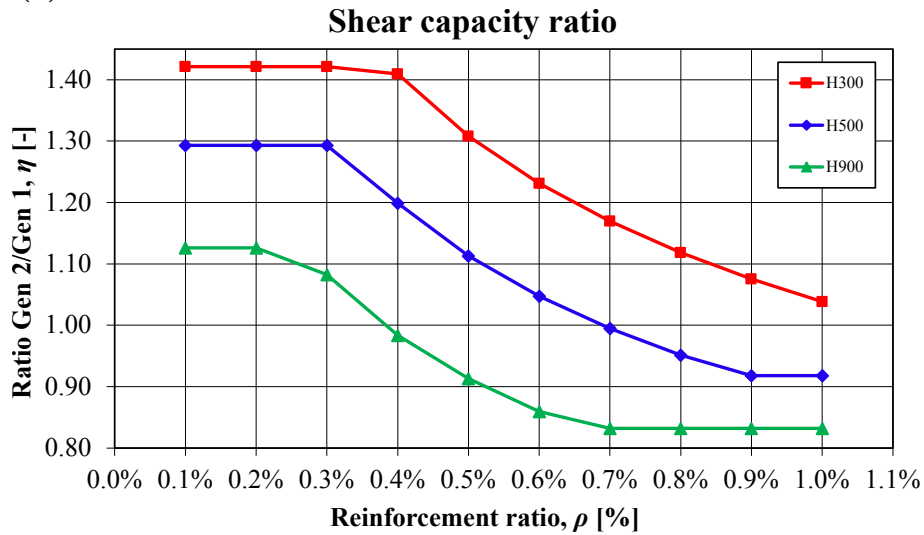
(b)

Figure B.2: Loaded with shear $V_{Ed} = V_{Rd,c}$ and moment $M_{Ed} = 0.75 \cdot M_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

B.3.3 Shear force ($V_{Ed} = V_{Rd}$) and bending moment ($M_{Ed} = 1.0M_{Rd}$)



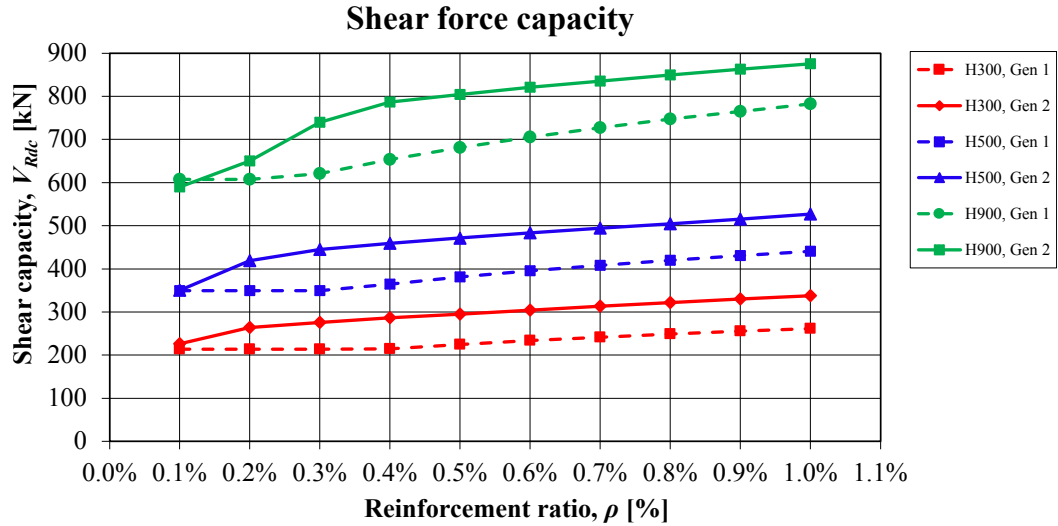
(a)



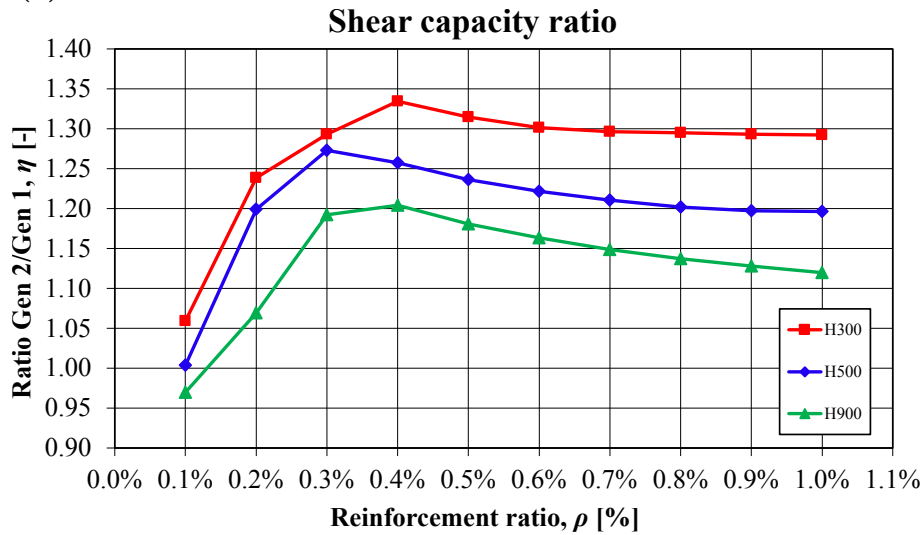
(b)

Figure B.3: Loaded with shear $V_{Ed} = V_{Rd,c}$ and moment $M_{Ed} = 1.0 \cdot M_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

B.3.4 Shear force ($V_{Ed} = V_{Rd}$) and compressive normal force ($N_{Ed} = 0.1N_{Rd}$)



(a)



(b)

Figure B.4: Loaded with shear $V_{Ed} = V_{Rd}$ and normal force $N_{Ed} = 0.1N_{Rd}$ for: a) shear force capacity, and b) shear force capacity ratio, for increasing reinforcement ratio and varying sectional heights.

B.3.5 Shear force ($V_{Ed} = V_{Rd}$) and $D_{lower} = [16; 24]$ mm

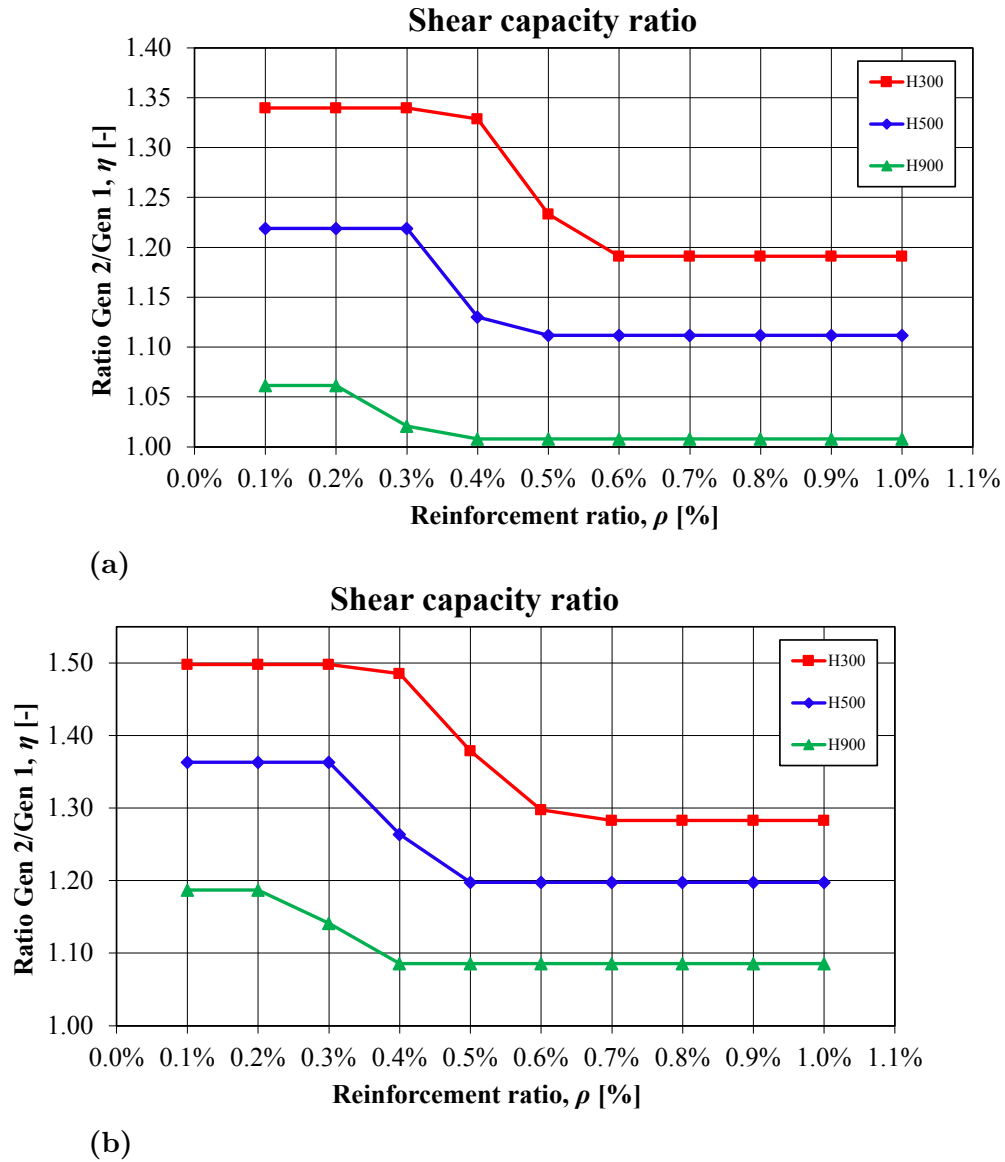


Figure B.5: Loaded with shear $V_{Ed} = V_{Rd}$ for: a) $D_{lower} = 16$ mm b) $D_{lower} = 24$ mm

C

CRACK CONTROL

C.1 Calculation for parametric study

The calculative sheet was used to calculate the crack widths for a variation of scenarios. The input parameters for each specific case are listed in their respective section in Chapter 9.5.

Crack width verification - SS-EN1992-2005

clause 7.3

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$t := 500 \text{ mm}$
Concrete cover reinforcement	$c := 45 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Reinforcement ratio	$\rho := 0.5 \cdot 10^{-2}$

1.2 Material

Modulus steel	$E_s := 200 \text{ GPa}$
Modulus concrete (C35/45)	$E_{cm} := 34 \text{ GPa}$
Tensile strength = mean strength (not cracked before 28 days)	$f_{ct_eff} := 3.2 \text{ MPa}$
Partial factor steel	$\gamma_s := 1.15$
Characteristic strength steel	$f_{yk} := 500 \text{ MPa}$
Design strength steel	$f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.78 \text{ MPa}$

1.3 Long term effects

Creep coefficient	$\phi := 0$
Shrinkage strain	$\epsilon_{cs} := 0$

2 Calculation

2.1 Material properties

Effective modulus

$$E_{c_ef} := \frac{E_{cm}}{(1 + \varphi)} = 34 \text{ GPa}$$

Elasticity ratio

$$\alpha_e := \frac{E_s}{E_{c_ef}} = 5.88$$

2.2 Sectional constants

Effective depth

$$d := t - c - \frac{\phi}{2} = 0.45 \text{ m}$$

Area of reinforcement

$$A_s := d \cdot b \cdot \rho = (2.24 \cdot 10^3) \text{ mm}^2$$

Spacing of reinforcement

$$s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{A_s} = 89.96 \text{ mm}$$

Moment utilization ratio

$$\kappa_M := 0.6$$

Design moment (simplified)

$$M_{Ed} := \kappa_M A_s \cdot 0.9 \cdot d \cdot f_{yd} = 234.56 \text{ kN} \cdot \text{m}$$

2.3 Sectional analysis

A sectional analysis is performed in SLS to determine the height of the compressive zone, and steel stress. The section is assumed to be fully cracked.

Shrinkage force

$$F_{cs} := E_s \cdot \varepsilon_{cs} \cdot A_s = 0 \text{ N}$$

Finding the height of the compressive zone, x

Solve for x

$$\begin{aligned} x &:= 0.1 \cdot t = 0.05 \text{ m} \\ b \cdot \frac{x^2}{2} &= \alpha_e \cdot A_s \cdot (d - x) \\ x &:= \text{find}(x) = 96.06 \text{ mm} \end{aligned}$$

Moment of inertia

$$I := b \cdot \frac{x^3}{12} + b \cdot x \cdot \left(\frac{x}{2}\right)^2 + \alpha_e \cdot A_s \cdot (x - d)^2 = (1.91 \cdot 10^{-3}) \text{ m}^4$$

Effective area of concrete in stage 2

$$A_{c2_ef} := x \cdot b + A_s \cdot \alpha_e = 0.11 \text{ m}^2$$

Concrete stress at steel level

$$\sigma_{cs} := \frac{F_{cs}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x) + M_{Ed})}{I} \cdot (d - x) = 42.99 \text{ MPa}$$

Steel stress

$$\sigma_s := \frac{-F_{cs}}{A_s} + \alpha_e \cdot \sigma_{cs} = 252.9 \text{ MPa}$$

Control: concrete stress at top fibre

$$\sigma_{c_top} := \frac{F_{cs}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (-x) + M_{Ed})}{I} \cdot (-x) = -11.77 \text{ MPa}$$

Note! Make sure steel stress is lower than tensile strength + concrete stress is below limit

2.4 Crack width

Effective height of tensile reinforcement

$$h_{ef} := \min \left(2.5 \cdot (t - d), \frac{(t - x)}{3}, \frac{t}{2} \right) = 132.5 \text{ mm}$$

Effective area of concrete around tensile reinforcement

$$A_{c_eff} := b \cdot h_{ef} = 0.13 \text{ m}^2$$

Effective reinforcement ratio

$$\rho_{eff} := \frac{A_s}{A_{c_eff}} = 0.02$$

Bond quality coefficient

$$k_1 := 0.8$$

Strain distribution coefficient

$$k_2 := 0.5$$

NDP

$$k_3 := 7 \cdot \frac{\phi}{c} = 2.49$$

EC recommended coefficient

$$k_4 := 0.425$$

Maximum crack spacing

$$s_{r_max} := \left| \begin{array}{l} \text{if } s < 5 \cdot \left(c + \frac{\phi}{2} \right) \\ \left\| k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \frac{\phi}{\rho_{eff}} \right\| \\ \text{else} \\ \left\| 1.3 \cdot (t - x) \right\| \end{array} \right| = 273.25 \text{ mm}$$

Load duration factor (long)

$$k_t := 0.4$$

Strain difference concrete and steel

$$\Delta \varepsilon := \max \left(\left(\frac{\left(\sigma_s - k_t \cdot \frac{f_{ct_eff}}{\rho_{eff}} \cdot (1 + \alpha_e \cdot \rho_{eff}) \right)}{E_s} \right), 0.6 \cdot \frac{\sigma_s}{E_s} \right) = 8.47 \cdot 10^{-4}$$

Characteristic crack width

$$w_k := s_{r_max} \cdot \Delta \varepsilon = 0.23 \text{ mm}$$

Crack width verification - SS-EN1992-2023

clause 9.2.3

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$t := 500 \text{ mm}$
Concrete cover reinforcement	$c := 45 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Reinforcement ratio	$\rho := 0.5 \cdot 10^{-2}$

1.2 Material

Tensile strength = mean strength (not cracked before 28 days)	$f_{ct_eff} := 3.2 \text{ MPa}$
Mean compressive strength concrete	$f_{cm} := 43 \text{ MPa}$
Partial factor steel	$\gamma_s := 1.15$
Characteristic strength steel	$f_{yk} := 500 \text{ MPa}$
Design strength steel	$f_{yd} := \frac{f_{yk}}{\gamma_s} = 434.78 \text{ MPa}$
Modulus steel	$E_s := 200 \text{ GPa}$
Aggregate NDP	$K_e := 9500$
Modulus concrete (quartzite aggregate)	$E_{cm} := K_e \cdot \left(\frac{f_{cm}}{\text{MPa}} \right)^{\frac{1}{3}} \cdot \text{MPa} = 33.28 \text{ GPa}$

1.3 Long term effects

Creep coefficient

$$\varphi := 0$$

Shrinkage strain

$$\varepsilon_{cs} := 0$$

2 Calculation

2.1 Material properties

Effective modulus

$$E_{c_ef} := \frac{E_{cm}}{(1 + \varphi)} = 33.28 \text{ GPa}$$

Elasticity ratio

$$\alpha_e := \frac{E_s}{1.05 \cdot E_{c_ef}} = 5.72$$

2.2 Sectional constants

Effective depth

$$d := t - c - \frac{\phi}{2} = 0.45 \text{ m}$$

Distance to tensile reinforcement from the bottom

$$a_y := c + \frac{\phi}{2} = 53 \text{ mm}$$

Area of reinforcement

$$A_s := d \cdot b \cdot \rho = (2.24 \cdot 10^3) \text{ mm}^2$$

Spacing of reinforcement

$$s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{A_s} = 89.96 \text{ mm}$$

Moment utilization ratio

$$\kappa_M := 0.6$$

Design moment (simplified)

$$M_{Ed} := \kappa_M A_s \cdot 0.9 \cdot d \cdot f_{yd} = 234.56 \text{ kN} \cdot \text{m}$$

2.3 Sectional analysis

A sectional analysis is performed in SLS to determine the height of the compressive zone, and steel stress. The section is assumed to be fully cracked.

Shrinkage force

$$F_{cs} := E_s \cdot \varepsilon_{cs} \cdot A_s = 0 \text{ N}$$

Finding the height of the compressive zone, x

Solve for first values

$$\begin{aligned} x &:= 0.1 \cdot t = 0.05 \text{ m} \\ b \cdot \frac{x^2}{2} &= \alpha_e \cdot A_s \cdot (d - x) \\ x &:= \text{find}(x) = 0.09 \text{ m} \end{aligned}$$

Moment of inertia

$$I := b \cdot \frac{x^3}{12} + b \cdot x \cdot \left(\frac{x}{2}\right)^2 + \alpha_e \cdot A_s \cdot (x - d)^2 = (1.87 \cdot 10^{-3}) \text{ m}^4$$

Effective area of concrete in stage 2

$$A_{c2_ef} := x \cdot b + A_s \cdot \alpha_e = 0.11 \text{ m}^2$$

Concrete stress at steel level

$$\sigma_{cs} := \frac{F_{cs}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x) + M_{Ed})}{I} \cdot (d - x) = 44.15 \text{ MPa}$$

Steel stress

$$\sigma_s := \frac{-F_{cs}}{A_s} + \alpha_e \cdot \sigma_{cs} = 252.66 \text{ MPa}$$

Control: concrete stress at top fibre

$$\sigma_{c_top} := \frac{F_{cs}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (-x) + M_{Ed})}{I} \cdot (-x) = -11.9 \text{ MPa}$$

Note! Make sure steel stress is lower than tensile strength + concrete stress is below limit

2.4 Refined calculation, crack width

Effective height of tensile reinforcement

$$h_{ef} := \min \left(a_y + 5 \cdot \phi, 10 \cdot \phi, 3.5 \cdot a_y, t - x, \frac{t}{2} \right) = 0.13 \text{ m}$$

Effective area of concrete around tensile reinforcement

$$A_{c_eff} := b \cdot h_{ef} = 0.13 \text{ m}^2$$

Effective reinforcement ratio

$$\rho_{eff} := \frac{A_s}{A_{c_eff}} = 0.02$$

Factor mean -> characteristic

$$k_w := 1.7$$

Distance to tensile reinforcement from bottom

$$a_{yi} := \frac{\phi}{2} + c = 0.05 \text{ m}$$

Factor curvature

$$k_r := \frac{(t - x)}{t - a_{yi} - x} = 1.15$$

Factor load duration
(long, stabilized cracking)

$$k_t := 0.4$$

Strain difference between concrete and steel

$$\Delta \varepsilon := \max \left(\frac{\left(\sigma_s - k_t \cdot \frac{f_{ct_eff}}{\rho_{eff}} \cdot (1 + \alpha_e \cdot \rho_{eff}) \right)}{E_s}, (1 - k_t) \cdot \frac{\sigma_s}{E_s} \right) = 8.46 \cdot 10^{-4}$$

Factor for rectangular sections in bending

$$k_{fl} := \frac{(t - h_{ef})}{t} = 0.73$$

Factor for bond conditions (assumed good, bottom)

$$k_b := 0.9$$

Mean crack spacing

$$s_{r_m_cal} := \min \left(1.5 \cdot c + k_{fl} \cdot \frac{k_b}{7.2} \cdot \frac{\phi}{\rho_{eff}}, \frac{1.3}{k_w} \cdot (t - x) \right) = 154.86 \text{ mm}$$

Characteristic crack width

$$w_{k_cal} := k_w \cdot k_r \cdot s_{r_m_cal} \cdot \Delta \varepsilon = 0.26 \text{ mm}$$

note: slightly larger than old EC

C.2 Case study - Deriving creep and shrinkage coefficients

The calculation of the long-term effects of the bridge examined in the case study is included in this Appendix.

Creep and shrinkage, SS-EN 1992-1-1-2005

Annex B

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1 Data

1.1 Geometry

Height of section $h := 852 \text{ mm}$

Width of section $b := 1000 \text{ mm}$

1.2 Material

Mean compressive strength $f_{cm} := 43 \text{ MPa}$

Characteristic compressive strength $f_{ck} := 35 \text{ MPa}$

2 Calculation

Age of loading $t_0 := 5 \text{ day}$

Age at the beginning of drying $t_s := 0 \text{ day}$

Lifetime of structure $t := 100 \cdot 365 \text{ day}$

Relative humidity $RH := 0.8$

Perimeter exposed to drying (all sides except top) $u := b = 1 \text{ m}$

Area of concrete $A_c := b \cdot h = 0.85 \text{ m}^2$

Notional size $h_0 := 2 \cdot \frac{A_c}{u} = (1.7 \cdot 10^3) \text{ mm}$

2.1 Creep coefficient Annex B1

Coefficient for cement class (CN, CEM1) $\alpha := 0$

Adjusted age of loading (not adjusted, casting conditions are not known)

$$t_0 := t_0 \cdot \left(\frac{9}{2 + \left(\frac{t_0}{\text{day}} \right)^{1.2}} + 1 \right)^a = 5 \text{ day}$$

Coefficients Strength / RH

$$\alpha_1 := \left(\frac{35 \text{ MPa}}{f_{cm}} \right)^{0.7} = 0.87$$

$$\alpha_2 := \left(\frac{35 \text{ MPa}}{f_{cm}} \right)^{0.2} = 0.96$$

$$\alpha_3 := \left(\frac{35 \text{ MPa}}{f_{cm}} \right)^{0.5} = 0.9$$

Factor Relative Humidity

$$\varphi_{RH} := \begin{cases} \text{if } f_{cm} > 35 \text{ MPa} \\ \left(1 + \frac{(1 - RH)}{0.1 \cdot \sqrt[3]{\frac{h_0}{\text{mm}}}} \cdot \alpha_1 \right) \cdot \alpha_2 \\ \text{else} \\ 1 + \frac{(1 - RH)}{0.1 \cdot \sqrt[3]{\frac{h_0}{\text{mm}}}} \end{cases}$$

Factor strength

$$\beta_{fcm} := \frac{16.8}{\sqrt{\frac{f_{cm}}{\text{MPa}}}} = 2.56$$

Time factor

$$\beta_{t0} := \frac{1}{\left(0.1 + \left(\frac{t_0}{\text{day}} \right)^{0.2} \right)} = 0.68$$

Relative humidity coefficient

$$\beta_H := \begin{cases} \text{if } f_{cm} > 35 \text{ MPa} \\ \min \left(1.5 \cdot \left(1 + (0.012 \cdot RH \cdot 100)^{18} \right) \cdot \frac{h_0}{\text{mm}} + 250 \cdot \alpha_3, \alpha_3 \cdot 1500 \right) \\ \text{else} \\ \min \left(1.5 \cdot \left(1 + (0.012 \cdot RH \cdot 100)^{18} \right) \cdot \frac{h_0}{\text{mm}} + 250, 1500 \right) \end{cases} = 1.35 \cdot 10^3$$

Factor time difference

$$\beta_c := \left(\frac{\frac{(t - t_0)}{\text{day}}}{\beta_H + \frac{(t - t_0)}{\text{day}}} \right)^{0.3} = 0.99$$

Notional creep coefficient

$$\varphi_0 := \varphi_{RH} \cdot \beta_{fcm} \cdot \beta_{t0} = 1.9$$

Total mean creep coefficient

$$\varphi := \varphi_0 \cdot \beta_c = 1.88$$

2.2 Shrinkage (B2)

Early strength coefficient (CEM1, class N)

$$\alpha_{ds2} := 0.12$$

$$\alpha_{ds1} := 4$$

NDPs for shrinkage (EC recommended values)

$$\alpha_{NDP_b} := 1$$

$$\alpha_{NDP_d} := 1$$

Autogenous shrinkage

Long term autogenous strain

$$\varepsilon_{ca_inf} := 2.5 \cdot \left(\frac{f_{ck}}{\text{MPa}} - 10 \right) \cdot 10^{-6} = 6.25 \cdot 10^{-5}$$

Autogenous shrinkage coefficient

$$\beta_{as} := 1 - \exp \left(-0.2 \cdot \left(\frac{t}{\text{day}} \right)^{0.5} \right) = 1$$

Autogenous shrinkage strain

$$\varepsilon_{ca} := \beta_{as} \cdot \varepsilon_{ca_inf} = 6.25 \cdot 10^{-5}$$

Drying shrinkage

Relative humidity reference

$$RH_0 := 1$$

Factor RH

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_0} \right)^3 \right) = 0.76$$

Shrinkage strain from strength

$$\varepsilon_{cd0} := 0.85 \cdot (220 + 110 \cdot \alpha_{ds1}) \cdot \exp \left(-\alpha_{ds2} \cdot \frac{f_{cm}}{10 \cdot \text{MPa}} \right) \cdot 10^{-6} \cdot \beta_{RH} = 2.53 \cdot 10^{-4}$$

Notional size coefficient

$$k_h := 0.7$$

Drying Shrinkage factor

$$\beta_{ds} := \frac{\left(\frac{(t - t_s)}{\text{day}} \right)}{\frac{(t - t_s)}{\text{day}} + 0.04 \cdot \sqrt{\left(\frac{h_0}{\text{mm}} \right)^3}} = 0.93$$

Drying shrinkage strain

$$\varepsilon_{cd} := \beta_{ds} \cdot k_h \cdot \varepsilon_{cd0} = 1.65 \cdot 10^{-4}$$

Total mean shrinkage strain

$$\varepsilon_{cs} := \varepsilon_{ca} + \varepsilon_{cd} = 2.27 \cdot 10^{-4}$$

Creep and shrinkage, SS-EN 1992-1-1-2023

Annex B

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1 Data

1.1 Geometry

Height of section

$$h := 852 \text{ mm}$$

Width of section

$$b := 1000 \text{ mm}$$

1.2 Material

Mean compressive strength

$$f_{cm} := 43 \text{ MPa}$$

2 Calculation

Age of loading

$$t_0 := 5 \text{ day}$$

Age at the beginning of drying (formwork removed)

$$t_s := 0 \text{ day}$$

Lifetime of structure

$$t := 100 \cdot 365 \text{ day}$$

Relative humidity

$$RH := 0.8$$

2.2 Creep coefficient

Coefficient for cement class (CN, CEM1)

$$\alpha_{cs} := 0$$

Adjusted age of loading (not adjusted, casting conditions are not known)

$$t_{0_adj} := t_0 \cdot \left(\frac{9}{2 + \left(\frac{t_0}{\text{day}} \right)^{1.2}} + 1 \right)^{\alpha_{cs}} = 5 \text{ day}$$

Basic creep

Time development function

$$\beta_{bc_t0} := \ln \left(\left(\frac{30}{\frac{t_{0_adj}}{\text{day}}} + 0.035 \right)^2 \cdot \frac{(t - t_0)}{\text{day}} + 1 \right) = 14.1$$

Concrete strength function

$$\beta_{bc_fcm} := \frac{1.8}{\left(\frac{f_{cm}}{\text{MPa}} \right)^{0.7}} = 0.13$$

Basic creep coefficient

$$\varphi_{bc} := \beta_{bc_fcm} \cdot \beta_{bc_t0} = 1.82$$

Drying creep:

Function for strength for drying creep

$$\beta_{dc_fcm} := \frac{412}{\left(\frac{f_{cm}}{\text{MPa}}\right)^{1.4}} = 2.13$$

Perimeter exposed to drying (all sides except top)

$$u := b = 1 \text{ m}$$

Area of concrete

$$A_c := b \cdot h = 0.85 \text{ m}^2$$

Notional size

$$h_n := 2 \cdot \frac{A_c}{u} = (1.7 \cdot 10^3) \text{ mm}$$

Function for relative humidity

$$\beta_{dc_RH} := \frac{(1 - RH)}{\sqrt[3]{0.1 \cdot \frac{h_n}{\text{mm}}}} = 0.17$$

Function for adjusted concrete age

$$\beta_{dc_t0} := \frac{1}{0.1 + \left(\frac{t_{0_adj}}{\text{day}}\right)^{0.2}} = 0.68$$

Strength factor

$$\alpha_{fcm} := \left(\frac{35}{\frac{f_{cm}}{\text{MPa}}}\right)^{0.5} = 0.9$$

Height factor

$$\beta_h := \min\left(1.5 \cdot \frac{h_n}{\text{mm}} + 250 \cdot \alpha_{fcm}, 1500 \cdot \alpha_{fcm}\right) = 1.35 \cdot 10^3$$

Factor adjusted loading time

$$\gamma_{t0_adj} := \frac{1}{\left(2.3 + \frac{3.5}{\sqrt{\frac{t_{0_adj}}{\text{day}}}}\right)} = 0.26$$

Function for time development

$$\beta_{dc_t_t0} := \left(\frac{\frac{(t - t_0)}{\text{day}}}{\beta_h + \frac{(t - t_0)}{\text{day}}}\right)^{\gamma_{t0_adj}} = 0.99$$

Drying creep coefficient

$$\varphi_{dc} := \beta_{dc_fcm} \cdot \beta_{dc_RH} \cdot \beta_{dc_t0} \cdot \beta_{dc_t_t0} = 0.24$$

Total mean creep coefficient

$$\varphi := \varphi_{bc} + \varphi_{dc} = 2.06$$

2.2 Shrinkage (Appendix B.6)

Early strength coefficient (CEM1, high early strength)

$$\alpha_{bs} := 700$$

$$\alpha_{ds} := 4$$

NDPs for shrinkage (EC recommended values)

$$\alpha_{NDP_b} := 1$$

$$\alpha_{NDP_d} := 1$$

Basic shrinkage

Notional basic shrinkage coefficient

$$\varepsilon_{cbs_fcm} := \alpha_{bs} \cdot \left(\frac{f_{cm}}{60 \cdot \text{MPa} + f_{cm}} \right)^{2.5} \cdot 10^{(-6)} = 7.88 \cdot 10^{-5}$$

Factor time for basic shrinkage

$$\beta_{bs_t} := 1 - \exp \left(-0.2 \cdot \sqrt{\frac{t}{\text{day}}} \right) = 1$$

Basic shrinkage (autogenous)

$$\varepsilon_{cbs} := \varepsilon_{cbs_fcm} \cdot \beta_{bs_t} \cdot \alpha_{NDP_b} = 7.88 \cdot 10^{-5}$$

Drying shrinkage

Shrinkage strain from strength

$$\varepsilon_{cds_fcm} := (220 + 110 \cdot \alpha_{ds}) \cdot \exp \left(-0.012 \cdot \frac{f_{cm}}{\text{MPa}} \right) \cdot 10^{-6} = 3.94 \cdot 10^{-4}$$

Relative humidity at equilibrium conditions

$$RH_{eq} := \min \left(0.99, 0.99 \cdot \left(\frac{35 \text{ MPa}}{f_{cm}} \right)^{0.1} \right) = 0.97$$

Factor RH (for 20 % ≤ RH ≤ RH_{eq})

$$\beta_{RH} := 1.55 \cdot \left(1 - \left(\frac{RH}{RH_{eq}} \right)^3 \right) = 0.68$$

Factor time

$$\beta_{ds} := \left(\frac{\frac{(t - t_s)}{\text{day}}}{0.035 \cdot \left(\frac{h_n}{\text{mm}} \right)^2 + \frac{(t - t_s)}{\text{day}}} \right)^{0.5} = 0.51$$

Drying shrinkage

$$\varepsilon_{cds} := \varepsilon_{cds_fcm} \cdot \beta_{RH} \cdot \beta_{ds} \cdot \alpha_{NDP_d} = 1.38 \cdot 10^{-4}$$

Total mean shrinkage strain

$$\varepsilon_{cs} := \varepsilon_{cbs} + \varepsilon_{cds} = 2.17 \cdot 10^{-4}$$

C.3 Case study calculations

Calculation is made at the critical section defined in Section 10.2.1 (edge strip - support section). The same calculation sheet is also used to calculate the shear force resistance with the input parameters defined in Section 10.2.2 for the middle section, located at the edge strip of the bridge deck.

Crack width verification - SS-EN1992-2005

clause 7.3 - support section, edge strip

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$h := 830 \text{ mm}$
Reinforcement diameter, compression	$\phi' := 16 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Monteringsj�rn (Only at the bottom)	$\phi_{MJ} := 12 \text{ mm}$
Concrete cover (top)	$c_{top} := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Concrete cover (bottom)	$c_{bot} := 40 \text{ mm} + \phi_t + \phi_{MJ} = 68 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Bar spacing, compression	$s' := 260 \text{ mm}$
Effective depth, tensile reinforcement	$d := h - c_{top} - \frac{\phi}{2} = 764 \text{ mm}$
Effective depth, to compressive layer	$d' := c_{bot} + \frac{\phi'}{2} = 76 \text{ mm}$
1.2 Material	
Modulus steel	$E_s := 200 \text{ GPa}$

Modulus concrete

$$E_{cm} := 34 \text{ GPa}$$

Tensile strength = mean strength
(not cracked before 28 days)

$$f_{ct_eff} := 3.2 \text{ MPa}$$

1.3 Long term effects

Creep coefficient

$$\varphi := 1.88$$

Shrinkage strain

$$\varepsilon_{cs} := 2.27 \cdot 10^{-4}$$

1.4 Loads

Applied moment

$$M_{Ed} := 305 \text{ kN} \cdot \text{m}$$

2 Calculation

2.1 Material properties

Effective modulus

$$E_{c_ef} := \frac{E_{cm}}{(1 + \varphi)} = 11.81 \text{ GPa}$$

Elasticity ratio

$$\alpha_e := \frac{E_s}{E_{c_ef}} = 16.94$$

2.2 Sectional constants

Area of tensile reinforcement

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Area of compressive reinforcement

$$A_{s'} := \frac{\pi \cdot \phi'^2}{4} \cdot \frac{b}{s'} = 773.32 \text{ mm}^2$$

Reinforcement ratio, tensile

$$\rho := \frac{A_s}{b \cdot d} = 3.16 \cdot 10^{-3}$$

Reinforcement ratio, compressive

$$\rho' := \frac{A_{s'}}{b \cdot d} = 1.01 \cdot 10^{-3}$$

2.3 Sectional analysis

A sectional analysis is performed in SLS to determine the height of the compressive zone, and steel stress. The section is assumed to be fully cracked.

Shrinkage force

$$F_{cs} := E_s \cdot \varepsilon_{cs} \cdot A_s = 109.71 \text{ kN}$$

$$F_{cs'} := E_s \cdot \varepsilon_{cs} \cdot A_{s'} = 35.11 \text{ kN}$$

Assuming the height of the compressive zone, x

$$x := 180 \text{ mm}$$

Effective area of concrete in stage 2 $A_{c2_ef} := x \cdot b + A_s \cdot \alpha_e + A_{s'} \cdot (\alpha_e - 1) = (2.33 \cdot 10^5) \text{ mm}^2$ Centre of gravity from the top $x_{tp} := \frac{\left(b \cdot x \cdot \frac{x}{2} + A_{s'} \cdot d' \cdot (\alpha_e - 1) + A_s \cdot d \cdot \alpha_e\right)}{A_{c2_ef}} = 207.55 \text{ mm}$

Moment of inertia

$$I := b \cdot \frac{x^3}{12} + b \cdot x \cdot \left(\frac{x}{2} - x_{tp}\right)^2 + \alpha_e \cdot A_s \cdot (d - x_{tp})^2 + (\alpha_e - 1) \cdot A_{s'} \cdot (d' - x_{tp})^2 = 0.02 \text{ m}^4$$

Check concrete stress at steel level (should be close to zero)

$$\sigma_x := \frac{F_{cs} + F_{cs'}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x_{tp}) + F_{cs'} \cdot (-x_{tp} + d') + M_{Ed})}{I} \cdot (x - x_{tp}) = -6.91 \cdot 10^{-3} \text{ MPa}$$

Concrete stress at steel level

$$\sigma_{cs} := \frac{F_{cs} + F_{cs'}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x_{tp}) + F_{cs'} \cdot (d' - x_{tp}) + M_{Ed})}{I} \cdot (d - x_{tp}) = 13.3 \text{ MPa}$$

Steel stress

$$\sigma_s := \frac{-F_{cs}}{A_s} + \alpha_e \cdot \sigma_{cs} = 179.9 \text{ MPa}$$

2.4 Crack width

Effective height of tensile reinforcement

$$h_{ef} := \min\left(2.5 \cdot (h - d), \frac{h - x}{3}, \frac{h}{2}\right) = 165 \text{ mm}$$

Effective area of concrete around tensile reinforcement

$$A_{c_eff} := b \cdot h_{ef} = 0.17 \text{ m}^2$$

Effective reinforcement ratio

$$\rho_{eff} := \frac{A_s}{A_{c_eff}} = 0.01$$

factors k1-4 for maximum crack spacing

Factor steel surface (good properties)

$$k_1 := 0.8$$

Factor bending

$$k_2 := 0.5$$

NDP, TSFS

$$k_3 := 7 \cdot \frac{\phi}{c_{top}} = 2.5$$

EC Recommended value

$$k_4 := 0.425$$

Maximum crack spacing

$$s_{r_max} := \begin{cases} \text{if } s < 5 \cdot \left(c_{top} + \frac{\phi}{2} \right) \\ \left\| k_3 \cdot c_{top} + k_1 \cdot k_2 \cdot k_4 \cdot \frac{\phi}{\rho_{eff}} \right\| \\ \text{else} \\ \left\| 1.3 \cdot (h - x) \right\| \end{cases} = 0.37 \text{ m}$$

Load duration factor (long)

$$k_t := 0.4$$

Strain difference concrete and steel

$$\Delta \varepsilon := \max \left(\left(\frac{\left(\sigma_s - k_t \cdot \frac{f_{ct_eff}}{\rho_{eff}} \cdot (1 + \alpha_c \cdot \rho_{eff}) \right)}{E_s} \right), 0.6 \cdot \frac{\sigma_s}{E_s} \right) = 5.4 \cdot 10^{-4}$$

Characteristic crack width

$$w_k := s_{r_max} \cdot \Delta \varepsilon = 0.201 \text{ mm}$$

$$w_{lim_TSFS} := 0.2 \text{ mm}$$

$$M_{Ed} = 305 \text{ kNm}$$

Crack width verification - SS-EN1992-2023

clause 9.2.3 - support section, edge strip

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1 Data

1.1 Geometry

Width	$b := 1000 \text{ mm}$
Thickness	$h := 830 \text{ mm}$
Reinforcement diameter, compression	$\phi' := 16 \text{ mm}$
Reinforcement diameter, tension	$\phi := 20 \text{ mm}$
Reinforcement diameter, monteringsj�rn	$\phi_{MJ} := 12 \text{ mm}$
Transverse reinforcement	$\phi_t := 16 \text{ mm}$
Concrete cover (top)	$c_{top} := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Concrete cover (bottom)	$c_{bot} := 40 \text{ mm} + \phi_t + \phi_{MJ} = 68 \text{ mm}$
Bar spacing, tension	$s := 130 \text{ mm}$
Bar spacing, compression	$s' := 260 \text{ mm}$
Effective depth, first layer	$d := h - c_{top} - \frac{\phi}{2} = 764 \text{ mm}$
Effective depth, to compressive layer	$d' := c_{bot} + \frac{\phi'}{2} = 76 \text{ mm}$

1.2 Material

Tensile strength = mean strength
(not cracked before 28 days)

$$f_{ct_eff} := 3.2 \text{ MPa}$$

Mean compressive strength concrete

$$f_{cm} := 43 \text{ MPa}$$

Modulus steel

$$E_s := 200 \text{ GPa}$$

Aggregate NDP

$$K_c := 9500$$

Modulus concrete (quartzite)

$$E_{cm} := K_c \cdot \left(\frac{f_{cm}}{\text{MPa}} \right)^{\frac{1}{3}} \cdot \text{MPa} = 33.28 \text{ GPa}$$

1.3 Long term effects

Creep coefficient

$$\varphi := 2.06$$

Shrinkage strain

$$\varepsilon_{cs} := 2.17 \cdot 10^{-4}$$

1.4 Loads

Applied moment

$$M_{Ed} := 312 \text{ kN} \cdot \text{m}$$

Applied normal force

$$N_{Ed} := 0 \text{ kN}$$

Används inte nu

2 Calculation

2.1 Material properties

Effective modulus

$$E_{c_ef} := \frac{E_{cm}}{(1 + \varphi)} = 10.88 \text{ GPa}$$

Elasticity ratio

$$\alpha_e := \frac{E_s}{1.05 \cdot E_{c_ef}} = 17.51$$

2.2 Sectional constants

Distance to tensile reinforcement

$$a_y := c_{top} + \frac{\phi}{2} = 66 \text{ mm}$$

Area of tensile reinforcement

$$A_s := \frac{\pi \cdot \phi^2}{4} \cdot \frac{b}{s} = (2.42 \cdot 10^3) \text{ mm}^2$$

Area of compressive reinforcement

$$A_{s'} := \frac{\pi \cdot \phi'^2}{4} \cdot \frac{b}{s'} = 773.32 \text{ mm}^2$$

Reinforcement ratio, tensile

$$\rho := \frac{A_s}{b \cdot d} = 3.16 \cdot 10^{-3}$$

Reinforcement ratio, compressive

$$\rho' := \frac{A_{s'}}{b \cdot d} = 1.01 \cdot 10^{-3}$$

2.3 Sectional analysis

A sectional analysis is performed in SLS to determine the height of the compressive zone, and steel stress. The section is assumed to be fully cracked.

Shrinkage force

$$F_{cs} := E_s \cdot \varepsilon_{cs} \cdot A_s = 104.88 \text{ kN}$$

$$F_{cs'} := E_s \cdot \varepsilon_{cs} \cdot A_{s'} = 33.56 \text{ kN}$$

Assuming the height of the compressive zone, x

$$x := 185 \text{ mm}$$

Effective area of concrete in stage 2

$$A_{c2_ef} := x \cdot b + A_s \cdot \alpha_e + A_{s'} \cdot (\alpha_e - 1) = (2.4 \cdot 10^5) \text{ mm}^2$$

Centre of gravity from the top

$$x_{tp} := \frac{\left(b \cdot x \cdot \frac{x}{2} + A_{s'} \cdot d' \cdot (\alpha_e - 1) + A_s \cdot d \cdot \alpha_e \right)}{A_{c2_ef}} = 209.99 \text{ mm}$$

Moment of inertia

$$I := b \cdot \frac{x^3}{12} + b \cdot x \cdot \left(\frac{x}{2} - x_{tp} \right)^2 + \alpha_e \cdot A_s \cdot (d - x_{tp})^2 + (\alpha_e - 1) \cdot A_{s'} \cdot (d' - x_{tp})^2 = 0.02 \text{ m}^4$$

Check concrete stress at steel level (should be close to zero)

$$\sigma_x := \frac{F_{cs} + F_{cs'}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x_{tp}) + F_{cs'} \cdot (-x_{tp} + d') + M_{Ed})}{I} \cdot (x - x_{tp}) = 0.02 \text{ MPa}$$

Concrete stress at steel level

$$\sigma_{cs} := \frac{F_{cs} + F_{cs'}}{A_{c2_ef}} + \frac{(F_{cs} \cdot (d - x_{tp}) + F_{cs'} \cdot (d' - x_{tp}) + M_{Ed})}{I} \cdot (d - x_{tp}) = 13 \text{ MPa}$$

Steel stress

$$\sigma_s := \frac{-F_{cs}}{A_s} + \alpha_e \cdot \sigma_{cs} = 184.32 \text{ MPa}$$

2.4 Refined calculation, crack width

Effective height of tensile reinforcement

$$h_{ef} := \min \left(a_y + 5 \cdot \phi, 10 \cdot \phi, 3.5 \cdot a_y, h - x, \frac{h}{2} \right) = 0.17 \text{ m}$$

Effective area of concrete around tensile reinforcement

$$A_{c_eff} := b \cdot h_{ef} = 0.17 \text{ m}^2$$

Effective reinforcement ratio

$$\rho_{\text{eff}} := \frac{A_s}{A_{c_eff}} = 0.01$$

Factor mean -> characteristic

$$k_w := 1.7$$

Factor curvature

$$k_r := \frac{(h-x)}{h-a_y-x} = 1.11$$

Factor load duration
(long term loading)

$$k_t := 0.4$$

Strain difference between concrete and steel

$$\Delta \epsilon := \max \left(\frac{\left(\sigma_s - k_t \cdot \frac{f_{ct_eff}}{\rho_{eff}} \cdot (1 + \alpha_e \cdot \rho_{eff}) \right)}{E_s}, (1 - k_t) \cdot \frac{\sigma_s}{E_s} \right) = 5.53 \cdot 10^{-4}$$

Factor for rectangular sections in bending

$$k_{fl} := \frac{(h-h_{ef})}{h} = 0.8$$

Factor for bond conditions (assumed poor at the top)

$$k_b := 1.2$$

Mean crack spacing

$$s_{r_m_cal} := \min \left(1.5 \cdot c_{top} + k_{fl} \cdot \frac{k_b}{7.2} \cdot \frac{\phi}{\rho_{eff}}, \frac{1.3}{k_w} \cdot (h-x) \right) = 267.18 \text{ mm}$$

Actual cover thickness

$$c_{act} := c_{top} = 56 \text{ mm}$$

Minimum cover thickness for durability

$$c_{min_dur} := 30 \text{ mm}$$

Surface coefficient

$$k_{surf} := \min \left(\frac{c_{act}}{(10 \text{ mm} + c_{min_dur})}, 1.5 \right) = 1.4$$

Characteristic crack width

$$w_{k_cal} := k_w \cdot k_r \cdot s_{r_m_cal} \cdot \Delta \epsilon = 0.28 \text{ mm}$$

Limiting crack width

$$w_{lim_EC2} := 0.3 \cdot k_{surf} = 0.42$$

$$w_{lim_TSFS} := 0.2 \cdot k_{surf} = 0.28 \quad M_{Ed} = 312 \text{ kNm}$$

D

ANCHORAGE & LAPS

D.1 Calculation for parametric study

The calculative sheet was used to obtain the design anchorage and lap lengths. The investigated parameters are listed in Section 9.6.1. Gen 1 calculations follow the NDP recommendations defined in TSFS 2018:57 [15]. The NDPs for Gen 2 vary accordingly:

- Gen 2 recommended NDPs (defined in Section 9.6)
- NDPs proposed by TSFS (defined in Section 9.6)
- Modified TSFS-proposed-NDPs (defined in Section 9.6.5)

Anchorage and splicing, SS-EN 1992-1-1-2023

Clauses 11.4 and 11.5

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1 Data

1.1 Geometry

Spacing reinforcement	$s := 150 \text{ mm}$
Spacing of confinement reinforcement (stirrups)	$s_c := 300 \text{ mm}$
Concrete cover reinforcement	$c := 45 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Diameter stirrups	$\phi_c := 12 \text{ mm}$

1.2 Material

Characteristic tensile strength of concrete (C35/45)	$f_{ctk} := 2.2 \text{ MPa}$
Concrete coefficient	$\alpha_c := 1$
Partial factor concrete	$\gamma_c := 1.5$
Characteristic yield strength reinforcement	$f_{yk} := 500 \text{ MPa}$
Partial factor reinforcement	$\gamma_{s1} := 1.15$

2 Calculation

2.1 Design. material

Design tension capacity	$f_{ctd} := \alpha_c \cdot \frac{f_{ctk}}{\gamma_c} = 1.47 \text{ MPa}$
Design yield strength reinforcement	$f_{yd} := \frac{f_{yk}}{\gamma_{s1}} = 435 \text{ MPa}$

2.2 Anchorage length

Design according to 8.4

Design stress at position of steel where anchorage is measured

$$\sigma_{sd} := f_{yd} = 434.78 \text{ MPa}$$

Bond condition coefficient

$$\eta_1 := 0.7 \quad (\text{poor conditions})$$

Bar diameter coefficient

$$\eta_2 := \begin{cases} 1 & \text{if } \phi \leq 32 \text{ mm} \\ \frac{(132 - \phi)}{100} & \text{else} \end{cases} = 1$$

Ultimate bond stress

$$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.31 \text{ MPa}$$

Basic anchorage length

$$l_{b_rqd} := \left(\frac{\phi}{4} \right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}} \right) = 752.87 \text{ mm}$$

Design distance for alpha2 factor

$$c_d := \min \left(\frac{(s - \phi)}{2}, c \right) = 45 \text{ mm}$$

Bar shape factor

$$\alpha_1 := 1$$

Factor straight bars

$$\alpha_2 := \min \left(\max \left(1 - 0.15 \cdot \frac{(c_d - \phi)}{\phi}, 0.7 \right), 1 \right) = 0.73$$

Factor confinement

$$\alpha_3 := 1$$

Factor for welded stirrups

$$\alpha_4 := 1$$

Factor transverse pressure

$$\alpha_5 := 1$$

Control

$$\alpha_{235} := \max (\alpha_2 \cdot \alpha_3 \cdot \alpha_5, 0.7) = 0.73$$

Minimum anchorage length (tension)

$$l_{b_min} := \max (0.3 \cdot l_{b_rqd}, 10 \cdot \phi, 100 \text{ mm}) = 225.86 \text{ mm}$$

Design anchorage length

$$l_{bd} := \max (\alpha_1 \cdot \alpha_4 \cdot \alpha_{235} \cdot l_{b_rqd}, l_{b_min}) = 548.18 \text{ mm}$$

2.3 Anchorage length including confinement effect

Arrangement of transverse reinforcement factor

$$K := 0.1$$

Area of minimum transverse reinforcement (slab)

$$\Sigma A_{st_min} := 0$$

Number of transverse bars along the anchorage length

$$n_t := \frac{l_{bd}}{s_c} = 1.83$$

Sum of area of transverse bars along anchorage length

$$\Sigma A_{st} := n_t \cdot \phi_c^2 \cdot \frac{\pi}{4} = 206.66 \text{ mm}^2$$

Factor confinement

$$\lambda := \frac{(\Sigma A_{st} - 0)}{\phi^2 \cdot \frac{\pi}{4}} = 1.03$$

Updated alpha_3 factor considering confinement:

$$\alpha_{3\text{conf}} := \min(\max(0.7, 1 - K \cdot \lambda), 1) = 0.9$$

Control

$$\alpha_{235} := \max(\alpha_2 \cdot \alpha_{3\text{conf}} \cdot \alpha_5, 0.7) = 0.7$$

Minimum anchorage length (tension)

$$l_{b\text{ min}} := \max(0.3 \cdot l_{b\text{ rqd}}, 10 \cdot \phi, 100 \text{ mm}) = 225.86 \text{ mm}$$

Design anchorage length with confinement

$$l_{bd\text{ conf}} := \max(\alpha_1 \cdot \alpha_4 \cdot \alpha_{235} \cdot l_{b\text{ rqd}}, l_{b\text{ min}}) = 527.01 \text{ mm}$$

2.4 Design lap length in 8.7.3

Alpha coefficient (considering the number of lapped bars in the same section)

$$\alpha_6 := 1.4$$

Minimum lap length

$$l_{0\text{ min}} := \max(0.3 \cdot \alpha_6 \cdot l_{b\text{ rqd}}, 15 \cdot \phi, 200 \text{ mm}) = 316.21 \text{ mm}$$

Design lap length

$$l_0 := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b\text{ rqd}} = 767.46 \text{ mm}$$

Check

$$l_0 > l_{0\text{ min}} = 1 \quad \text{OK!}$$

Anchorage and splicing, SS-EN 1992-1-1-2023

Clauses 11.4 and 11.5

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1 Data

1.1 Geometry

Spacing reinforcement	$s := 150 \text{ mm}$
Spacing confinement reinforcement (stirrups)	$s_c := 300 \text{ mm}$
Transverse spacing of confinement reinforcement legs	$s_t := s = 150 \text{ mm}$
Concrete cover reinforcement	$c := 45 \text{ mm}$
Diameter reinforcement (one row)	$\phi := 16 \text{ mm}$
Diameter transverse reinforcement	$\phi_t := 16 \text{ mm}$
Diameter confinement (stirrups)	$\phi_c := 12 \text{ mm}$
Number of anchored bars at failure surface	$n_b := 1$
Number of legs of confinement reinforcement crossing failure surface	$n_c := 1$
Clear spacing between bars	$c_s := s - \phi = 0.13 \text{ m}$

1.2 Material

Reference concrete strength	$f_{ck_ref} := 40 \text{ MPa}$
Ch. tensile strength of concrete (C35/45)	$f_{ctk} := 2.2 \text{ MPa}$
Ch. Compressive strength concrete	$f_{ck} := 35 \text{ MPa}$
Factor concrete	$\alpha_{ct} := 1$

Partial factor concrete

$$\gamma_c := 1.5$$

Characteristic yield strength reinforcement

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

Partial factor reinforcement

$$\gamma_{s1} := 1.15$$

2 Calculation

Strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\frac{1}{3}}, 1 \right) = 1$$

2.1 Design material

Design tensile strength, concrete

$$f_{ctd} := \alpha_{ct} \cdot \frac{f_{ctk}}{\gamma_c} = 1.47 \text{ MPa}$$

Design compressive strength concrete

$$f_{cd} := \eta_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_{s1}} = 435 \text{ MPa}$$

2.2 Anchorage length, detailed analysis

Nominal cover

$$c_d := \min (0.5 \cdot c_s, c, 3.75 \cdot \phi) = 0.05 \text{ m}$$

Confinement reinforcement ratio

$$\rho_{conf} := n_c \cdot \pi \cdot \frac{\phi_c^2}{4 \cdot n_b \cdot \phi \cdot s_c} = 0.02$$

Confinement effect

$$\Delta c_d := 70 \cdot \rho_{conf} \cdot \phi = 0.03 \text{ m}$$

Cover (including confinement)

$$c_{d_conf} := \min \left(c + 25 \cdot \frac{\phi_t^2}{s_t}, 3.75 \cdot \phi, \frac{c_s}{2} \right) + \Delta c_d = 0.09 \text{ m}$$

Design stress at position of steel where anchorage is measured

$$\sigma_{sd} := f_{yd} = 434.78 \text{ MPa}$$

Design situation NDP (persistent, TSFS)

$$k_{1b} := 50$$

Steel stress NDP (TSFS)

$$n_\sigma := \frac{3}{2}$$

Bond condition coefficient (poor bond)

$$k_{cp} := 1.2$$

Strength factor

$$f_{ck_25} := \max \left(\frac{25 \text{ MPa}}{f_{ck}}, 0.3 \right) = 0.71$$

Diameter factor

$$\phi_{20} := \max \left(\frac{\phi}{20 \text{ mm}}, 0.6 \right) = 0.8$$

Anchorage length without confinement:

$$l_{bd} := \max \left(k_{1b} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435 \text{ MPa}} \right)^{n_{\sigma}} \cdot (f_{ck_25})^{\frac{1}{2}} \cdot (\phi_{20})^{\frac{1}{3}} \cdot \left(1.5 \cdot \frac{\phi}{c_d} \right)^{\frac{1}{2}}, 10 \cdot \phi \right) = 549.64 \text{ mm}$$

Anchorage length with confinement:

$$l_{bd_conf} := \max \left(k_{1b} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435 \text{ MPa}} \right)^{n_{\sigma}} \cdot (f_{ck_25})^{\frac{1}{2}} \cdot (\phi_{20})^{\frac{1}{3}} \cdot \left(1.5 \cdot \frac{\phi}{c_{d_conf}} \right)^{\frac{1}{2}}, 10 \cdot \phi \right) = 396.69 \text{ mm}$$

2.3 Design lap length

Clause 11.5

Lap length NDP

$$k_{ls} := 1.2$$

Design lap length for straight bars

without confinement

$$l_{sd} := k_{ls} \cdot l_{bd} = 659.57 \text{ mm}$$

with confinement

$$l_{sd_conf} := k_{ls} \cdot l_{bd_conf} = 476.03 \text{ mm}$$

Check

$$l_{sd} \geq 15 \cdot \phi = 1$$

OK!

$$l_{sd_conf} \geq 15 \cdot \phi = 1$$

D.2 Case study calculations

Calculation is made at the critical section defined in Section 10.2.1 for the mid strip and the edge strip of the support section of the bridge. The same calculation sheet is also used to calculate the design anchorage and lap length with the input parameters defined in Section 10.2.2 for the edge strip in the mid section of the bridge.

The NDPs recommended by Eurocode, and the preliminary TSFS NDPs used for the calculations for Gen 2, are defined in Section 9.6.

Anchorage and splicing, SS-EN 1992-1-1-2005

Clauses 8.4 and 8.7.3 - support section, mid strip

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2.3 Design of lap length, acc. to 8.7.3	3

1 Data

1.1 Geometry

Diameter reinforcement tension	$\phi := 16 \text{ mm}$
Diameter transverse reinforcement	$\phi_t := 16 \text{ mm}$
Spacing reinforcement, tension	$s := 130 \text{ mm}$
Spacing, transverse reinforcement	$s_t := 300 \text{ mm}$
Concrete cover (top)	$c_{top} := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Clear spacing between bars	$c_s := s - \phi = 114 \text{ mm}$

1.2 Material

Ch. Compressive strength concrete	$f_{ck} := 35 \text{ MPa}$
Factor concrete	$\alpha_c := 1$
Partial factor concrete	$\gamma_c := 1.5$
Characteristic yield strength reinforcement	$f_{yk} := 500 \text{ MPa}$
Partial factor reinforcement	$\gamma_{sl} := 1.15$

2 Calculation

2.1 Design material

Ch. tensile strength of concrete, 5% fractile	$f_{ctk_{0.05}} := 2.2 \text{ MPa}$
---	-------------------------------------

Governing tension capacity

$$f_{ctd} := \alpha_c \cdot \frac{f_{ctk_{0.05}}}{\gamma_c} = 1.47 \text{ MPa}$$

Design yield limit reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_{s1}} = 435 \text{ MPa}$$

2.3 Anchorage of longitudinal reinforcement, acc. to 8.4

Design stress at position of steel
where anchorage is measured

$$\sigma_{sd} := f_{yd} = 434.78 \text{ MPa}$$

Bond condition coefficient

$$\eta_1 := 0.7 \quad (\text{poor conditions})$$

Bar diameter coefficient

$$\eta_2 := \begin{cases} 1 & \text{if } \phi \leq 32 \text{ mm} \\ \frac{(132 - \phi)}{100} & \text{else} \end{cases} = 1$$

Ultimate bond stress

$$f_{bd} := 2.25 \cdot \eta_1 \cdot \eta_2 \cdot f_{ctd} = 2.31 \text{ MPa}$$

Basic anchorage length

$$l_{b_rqd} := \left(\frac{\phi}{4} \right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}} \right) = 752.87 \text{ mm}$$

Design distance for alpha2 factor

$$c_d := \min \left(\frac{(s - \phi)}{2}, c_{top} \right) = 56 \text{ mm}$$

Bar shape factor

$$\alpha_1 := 1$$

Factor straight bars

$$\alpha_2 := \min \left(\max \left(1 - 0.15 \cdot \frac{(c_d - \phi)}{\phi}, 0.7 \right), 1 \right) = 0.7$$

Factor confinement

$$\alpha_3 := 1$$

Factor for welded stirrups

$$\alpha_4 := 1$$

Factor transverse pressure

$$\alpha_5 := 1$$

Control

$$\alpha_{235} := \max (\alpha_2 \cdot \alpha_3 \cdot \alpha_5, 0.7) = 0.7$$

Minimum anchorage length (**tension**)

$$l_{b_min} := \max (0.3 \cdot l_{b_rqd}, 10 \cdot \phi, 100 \text{ mm}) = 225.86 \text{ mm}$$

Design anchorage length

$$l_{bd} := \max (\alpha_1 \cdot \alpha_4 \cdot \alpha_{235} \cdot l_{b_rqd}, l_{b_min}) = 527.01 \text{ mm}$$

Including confinement:

Arrangement of transverse reinforcement factor

$$K := 0.05$$

Area of minimum transverse reinforcement (slab)

$$\Sigma A_{st_min} := 0$$

Number of transverse bars along the anchorage length

$$n_t := \text{round} \left(\frac{l_{bd}}{s_t} \right) = 2$$

Sum of area of transverse bars along anchorage length

$$\Sigma A_{st} := n_t \cdot \phi_t^2 \cdot \frac{\pi}{4} = 402.12 \text{ mm}^2$$

Factor confinement

$$\lambda := \frac{(\Sigma A_{st} - 0)}{\phi^2 \cdot \frac{\pi}{4}} = 2$$

Updated α_3 factor considering confinement

$$\alpha_3 := \min(\max(0.7, 1 - K \cdot \lambda), 1) = 0.9$$

$$\alpha_{235} := \max(\alpha_2 \cdot \alpha_3 \cdot \alpha_5, 0.7) = 0.7$$

Minimum anchorage length

$$l_{b_min} := \max(0.3 \cdot l_{b_rqd}, 10 \cdot \phi, 100 \text{ mm}) = 225.86 \text{ mm}$$

New anchorage length

$$l_{bd} := \max(\alpha_1 \cdot \alpha_4 \cdot \alpha_{235} \cdot l_{b_rqd}, l_{b_min}) = 527.01 \text{ mm}$$

2.4 Design of lap length, acc. to 8.7.3

Alpha coefficient (considering the number of lapped bars in the same section)

$$\alpha_6 := 1.4$$

Minimum lap length

$$l_{0min} := \max(0.3 \cdot \alpha_6 \cdot l_{b_rqd}, 15 \cdot \phi, 200 \text{ mm}) = 316.21 \text{ mm}$$

Design lap length

$$l_0 := \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b_rqd} = 664.03 \text{ mm}$$

Check

$$l_0 > l_{0min} = 1 \quad \text{OK!}$$

Anchorage and splicing, SS-EN 1992-1-1-2023

Clauses 11.4 and 11.5 - support section, mid strip

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1 Data

1.1 Geometry

Diameter reinforcement tension	$\phi := 16 \text{ mm}$
Diameter transverse reinforcement	$\phi_t := 16 \text{ mm}$
Spacing reinforcement, tension	$s := 130 \text{ mm}$
Spacing, transverse reinforcement	$s_t := 300 \text{ mm}$
Concrete cover (top)	$c_{top} := 40 \text{ mm} + \phi_t = 56 \text{ mm}$
Clear spacing between bars	$c_s := s - \phi = 114 \text{ mm}$
Clear cover distance, y-axis	$c_y := c_{top} = 56 \text{ mm}$

1.2 Material

Reference concrete strength	$f_{ck_ref} := 40 \text{ MPa}$
Ch. compressive strength concrete	$f_{ck} := 35 \text{ MPa}$
Factor concrete	$\alpha_{ct} := 1$
Partial factor concrete	$\gamma_c := 1.5$
Characteristic yield strength reinforcement	$f_{yk} := 500 \text{ MPa}$
Partial factor reinforcement	$\gamma_s := 1.15$

Strength reduction factor

$$\eta_{cc} := \min \left(\left(\frac{f_{ck_ref}}{f_{ck}} \right)^{\frac{1}{3}}, 1 \right) = 1$$

2 Calculation

2.1 Design material

Ch. tensile strength of concrete, 5% fractile

$$f_{ctk_{0.05}} := 2.2 \text{ MPa}$$

Design compressive strength concrete

$$f_{cd} := \eta_{cc} \cdot \frac{f_{ck}}{\gamma_c} = 23.33 \text{ MPa}$$

Design yield strength reinforcement

$$f_{yd} := \frac{f_{yk}}{\gamma_s} = 435 \text{ MPa}$$

2.2 Anchorage, detailed analysis

Nominal cover

$$c_d := \min(0.5 \cdot c_s, c_y, 3.75 \cdot \phi) = 56 \text{ mm}$$

Confinement effect (no confinement reinf)

$$\Delta c_d := 0$$

Cover (including confinement by transverse reinf)

$$c_{d_conf} := \min \left(\min \left(c_y + \frac{25 \cdot \phi_t^2}{s_t}, \frac{c_s}{2}, 3.75 \cdot \phi \right) + \Delta c_d, 6 \cdot \phi \right) = 57 \text{ mm}$$

Include confinement effect (due to transv. reinf)

$$c_d := c_{d_conf} = 57 \text{ mm}$$

Design stress at position of steel where anchorage is measured

$$\sigma_{sd} := f_{yd} = 434.78 \text{ MPa}$$

Design situation coefficient (persistent)

$$k_{1b} := 40$$

Unknown factor

$$n_{\sigma} := \frac{2}{2}$$

Bond condition coefficient (poor bond)

$$k_{cp} := 1.2$$

Strength factor

$$f_{ck_25} := \max \left(\frac{25 \text{ MPa}}{f_{ck}}, 0.3 \right) = 0.71$$

Diameter factor

$$\phi_{20} := \max \left(\frac{\phi}{20 \text{ mm}}, 0.6 \right) = 0.8$$

Anchorage length:

$$l_{bd} := \max \left(k_{1b} \cdot k_{cp} \cdot \phi \cdot \left(\frac{\sigma_{sd}}{435 \text{ MPa}} \right)^{n_{\sigma}} \cdot (f_{ck_25})^{\frac{1}{2}} \cdot (\phi_{20})^{\frac{1}{3}} \cdot \left(1.5 \cdot \frac{\phi}{c_d} \right)^{\frac{1}{2}}, 10 \cdot \phi \right) = 390.79 \text{ mm}$$

2.3 Design of lap length

Clause 11.5

Lap length factor

$$k_{ls} := 1.5$$

Design lap length for straight bars

$$l_{sd} := k_{ls} \cdot l_{bd} = 586.19 \text{ mm}$$

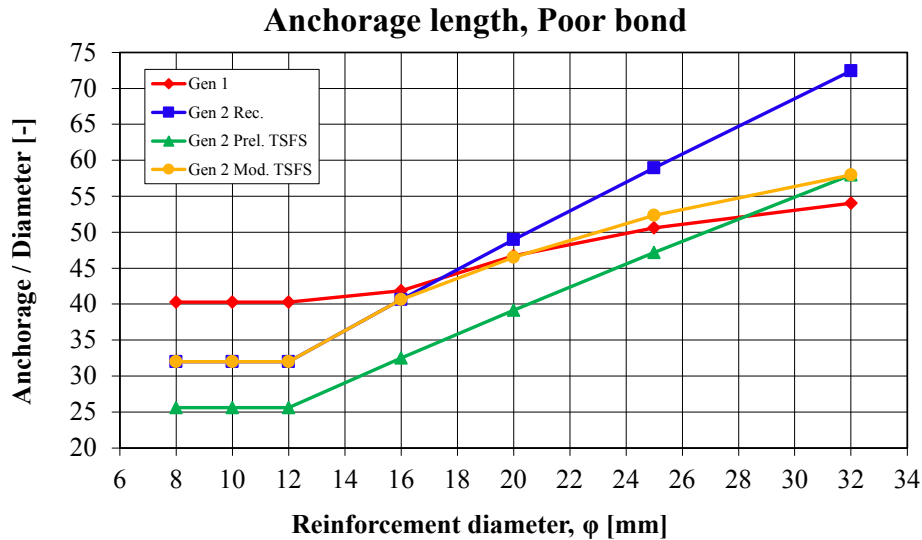
Check

$$l_{sd} \geq 15 \cdot \phi = 1 \quad OK!$$

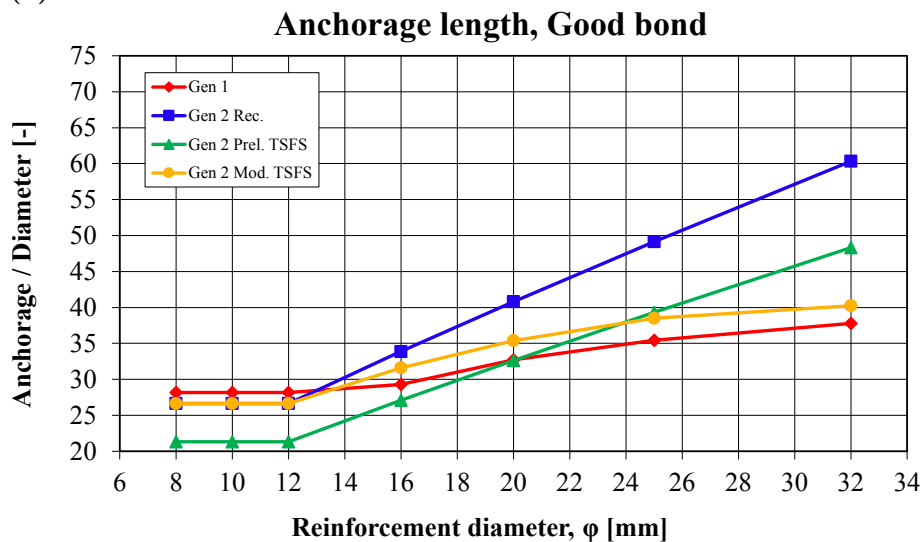
D.3 Additional graphs

These additional graphs cover additional concrete classes than those included in the result of the parametric study for the anchorage and lap length. Other input parameters are the same as presented in Section 9.6.

D.3.1 Anchorage length of straight bars, concrete class C25/30



(a)



(b)

Figure D.1: Design anchorage length/Diameter relationship for concrete class C25/30 with different NDPs using a) poor bond conditions and b) good bond conditions.

D.3.2 Lap length of straight bars, concrete class C25/30

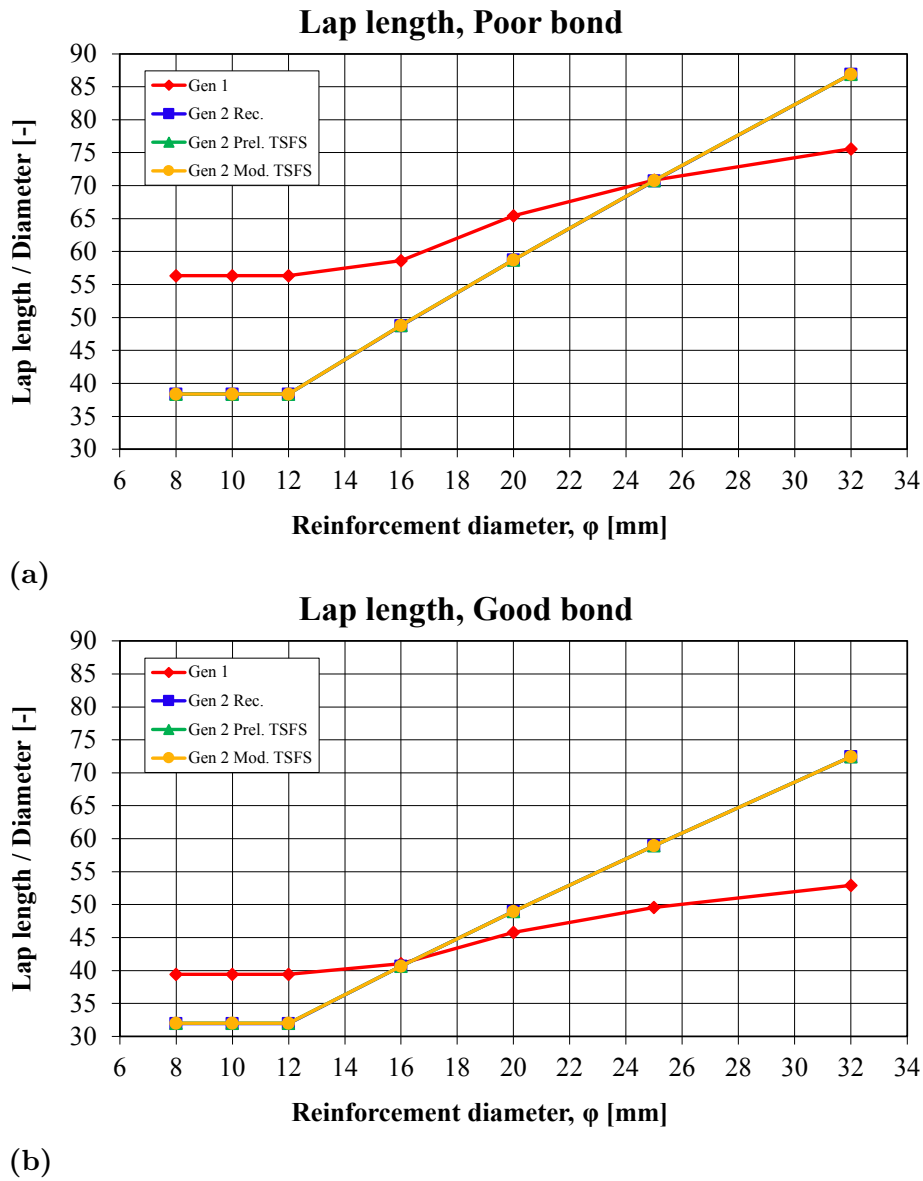
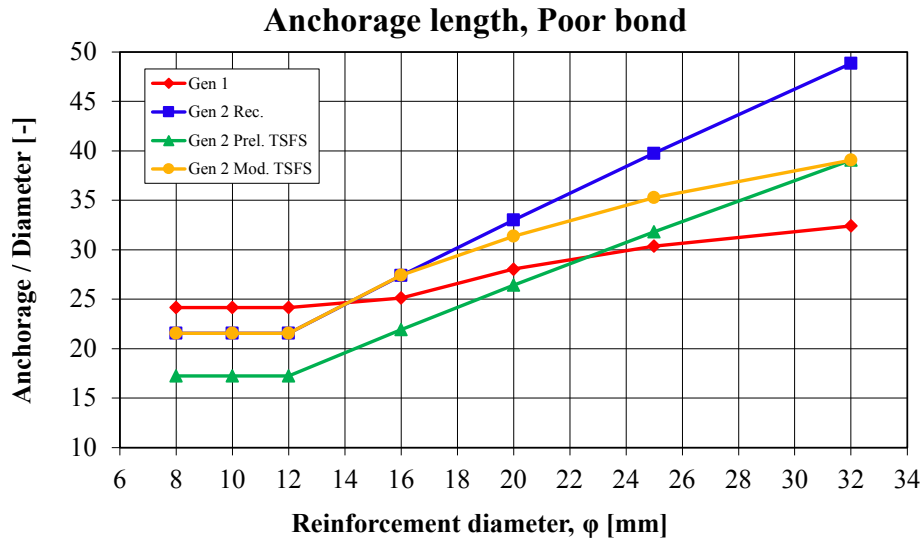
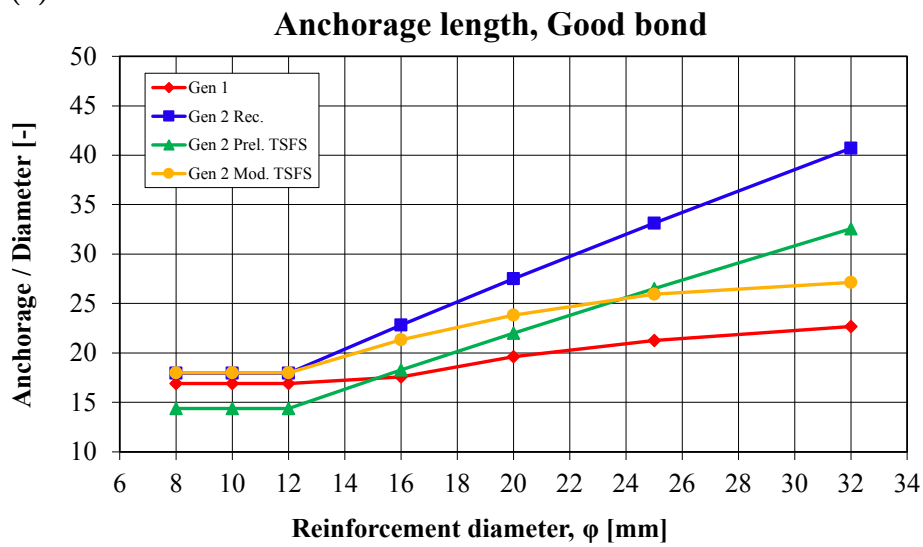


Figure D.2: Design lap length/Diameter relationship for concrete class C25/30 with different NDPs using a) poor bond conditions and b) good bond conditions.

D.3.3 Anchorage length of straight bars, concrete class C55/67



(a)



(b)

Figure D.3: Design anchorage length/Diameter relationship for concrete class C55/67 with different NDPs using a) poor bond conditions and b) good bond conditions.

D.3.4 Lap length of straight bars, concrete class C55/67

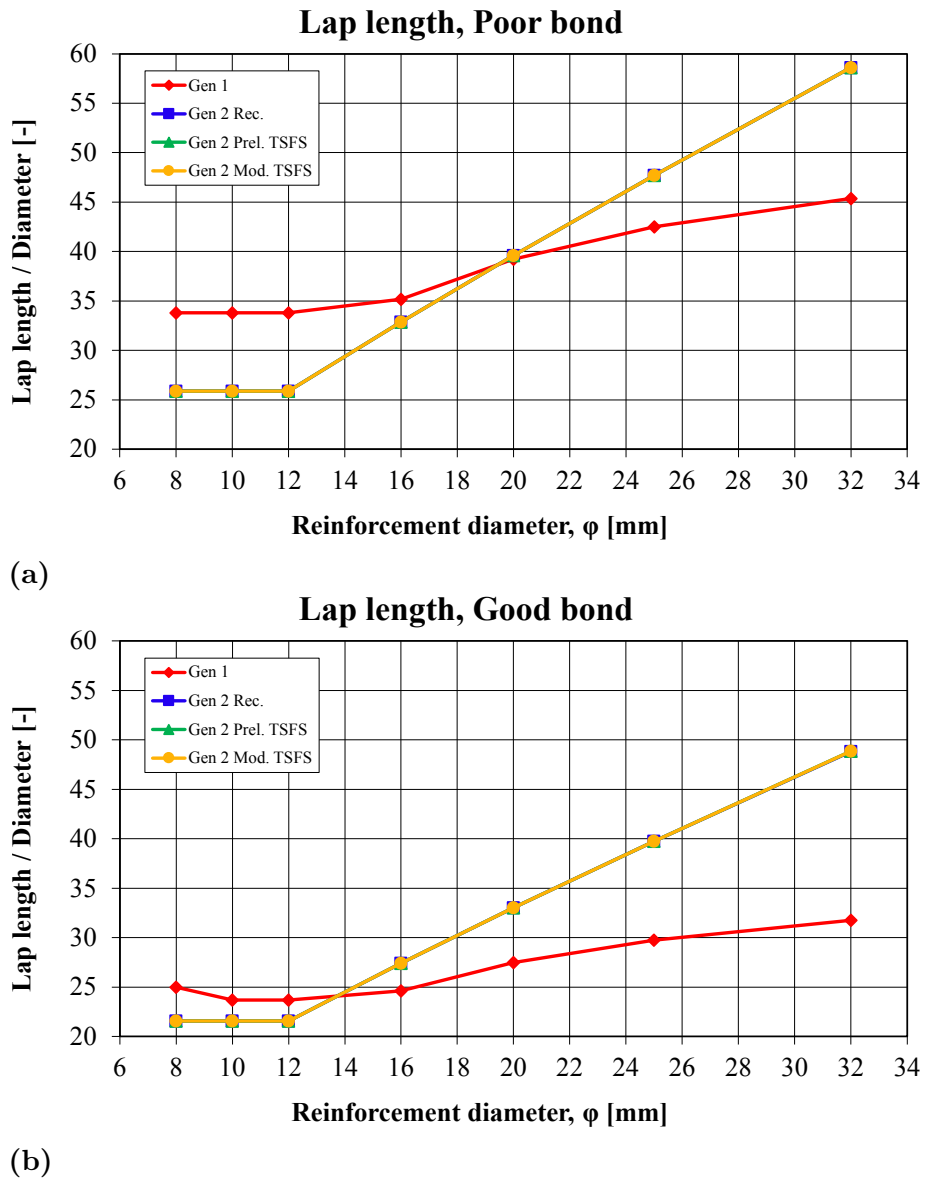


Figure D.4: Design lap length/Diameter relationship for concrete class C55/67 with different NDPs using a) poor bond conditions and b) good bond conditions.

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