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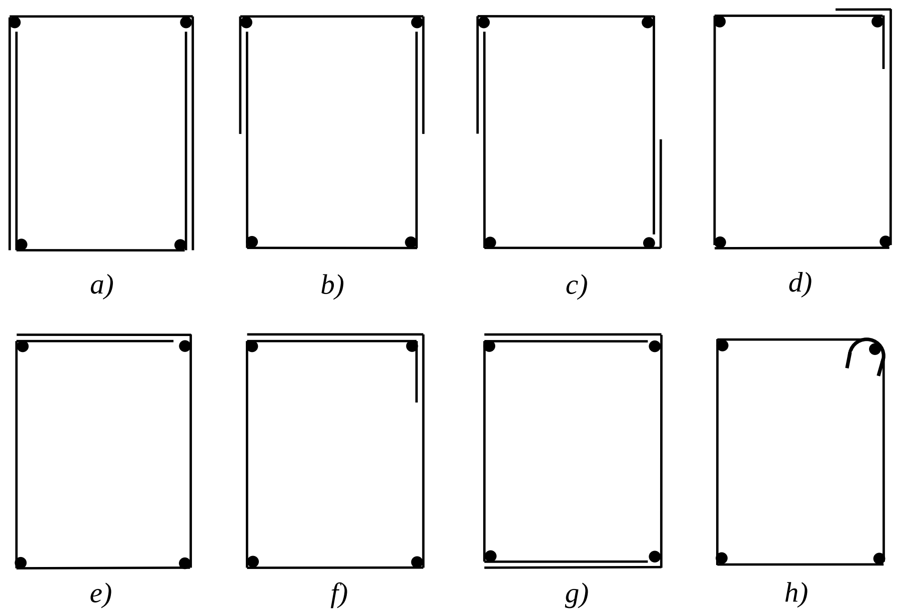

Guidelines and Rules for Detailing of Reinforcement in Concrete Structures

A Compilation and Evaluation of Ambiguities in Eurocode 2

Master of Science Thesis in the Master's Programme Structural Engineering and Building Technology

ANNELI DAHLGREN
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Department of Civil and Environmental Engineering
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Concrete Structures
CHALMERS UNIVERSITY OF TECHNOLOGY
Göteborg, Sweden 2013
Master's Thesis 2013:142

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Cover:
Different configurations of links used as shear or torsional reinforcement in concrete structures.

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ABSTRACT

A proper detailing of reinforcement in concrete structures is very important with regard to structural behaviour, safety and good performance. However, rules in codes for detailing and minimum reinforcement ratios are not always easy to understand and interpret, since there is very limited or no background information explaining the function and providing motivations. The aim of this project was to investigate and explain the background for the rules in the codes, especially Eurocode 2, and give guidelines for good detailing, based on previous research and experience.

Rules and guidelines for detailing of reinforcement that might be difficult to interpret or understand were identified by an initial study of the European Standard. From this study the background to some of the ambiguities were examined more thoroughly in a detailed literature study.

An investigation consisting of interviews and a qualitative survey was performed in order to evaluate how structural engineers interprets and applies the rules provided in Eurocode 2. This was performed also with the intention to illuminate ambiguities to be able to suggest improvements of the new standard.

The literature study showed that there is very limited background information to Eurocode 2 and that it can be difficult to find answer, if something is perceived as unclear in the code. However, knowledge about the fundamental theory and required structural behaviour of reinforced concrete structures can be enough to understand many requirements, why this is provided in this report.

The result from the investigation indicated that there are sections in Eurocode 2 that can be difficult to interpret. The standard needs to be improved or clarified in order to facilitate for the users and in order to know how to apply the rules on cases other than standard cases. Some of the identified problem areas in Eurocode 2 are for instance lapping of longitudinal reinforcement, concrete frame corners subjected to opening moment and limitations of crack widths for concrete members subjected to shear and torsion. In some cases, specific changes of Eurocode 2 have been suggested, while it in some cases is sufficient to add even stronger references between sections or include a short description of the motive for the intended rule, in order to improve the standard. A need for further research has also been identified in order to be able to improve certain parts of Eurocode 2.

Key words: Eurocode 2, EN 1992-1-1, Reinforcement, Reinforced concrete, Detailing

Riktlinjer och regler för detaljutformning av armering i betongkonstruktioner
En sammanställning och utvärdering av oklarheter i Eurokod 2

Examensarbete inom mastersprogrammet Structural Engineering and Building Technology

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SAMMANFATTNING

God detaljutformning av armering i betongkonstruktioner är väldigt viktigt med hänsyn till säkerhet, funktion och verkningssätt. Trots detta är regler för detaljutformning och minimiarmering inte alltid så lätta att förstå och tolka eftersom det finns väldigt begränsad, om ens någon, bakgrundsinformation som förklarar funktionen och ger motiv för reglerna. Målet med projektet var att undersöka och förklara bakgrunden för reglerna i standarderna, särskilt Eurokod 2, och ge riktlinjer för god detaljutformning, baserad på tidigare forskning och erfarenhet

Regler och riktlinjer för detaljutformning av armering som kan vara svåra att tolka eller förstå identifierades i en inledande studie av den europeiska standarden. Utifrån denna studie utforskades därefter bakgrunden till några av svårigheterna i en mer ingående litteraturstudie.

En undersökning innefattande intervjuer och en kvalitativ enkätundersökning utfördes för att utreda hur konstruktörer tolkar och tillämpar regler som finns i Eurokod 2. Detta gjordes också med avsikten att ytterligare kunna belysa problemområden och komma med förslag till förbättringar av den nya standarden.

Litteraturstudien visade på att det finns mycket begränsad bakgrundsinformation till Eurokod 2 och att det kan vara svårt att finna svar om något uppfattas som oklart i normen. Dock kan kunskap om grundläggande teori och erforderligt verkningssätt hos armerade betongkonstruktioner räcka för att förstå många av reglerna, varför detta ges i denna rapport.

Resultatet från undersökningen indikerar att det finns avsnitt i Eurokod 2 som kan vara svåra att tolka. Standarden behöver förbättras eller förtydligas för att underlätta för dess användare och för att veta hur regler ska tillämpas i situationer som inte medför standardlösningar. Några exempel på identifierade problemområden i Eurokod 2 är omlottskarvning av längsgående armering, ramhörn utsatta för öppnande moment samt beräkning av sprickbredd för betongelement som belastas av både tvärkraft och vridning. Konkreta ändringar av Eurokod 2 har föreslagits medan det i andra fall räcker med tydligare hänvisningar mellan avsnitten eller en kort förklaring av motivet för den avsedda regeln. Ett behov av mer ingående forskning har också identifierats för att kunna förbättra vissa delar av Eurokod 2.

Nyckelord: Eurokod 2, EN 1992-1-1, armering, armerad betong, detaljutformning

Contents

Preface

In this project a literature study has been conducted searching for the background to ambiguities encountered in the new European standard, Eurocode 2. The literature study was followed by an investigation consisting of interviews and a quantitative survey, where the ambiguities were further evaluated and discussed. The work has mostly been performed by the authors together. However, when the background to the rules and guidelines in Eurocode 2 were investigated more thoroughly a division of the subjects was made. Continuous discussions and exchange of knowledge between the authors have been made in order to ensure that both authors understand the content of the report.

The project has been carried out during 2013, from February to December at Reinertsen Sverige AB in cooperation with the Department of Civil and Environmental engineering, Division of Structural Engineering, Concrete Structures at Chalmers University of Technology.

Supervisors have been PhD Morgan Johansson from Reinertsen Sweden AB and Prof. Björn Engström at the Division of Structural Engineering at Chalmers University of Technology, who also was the examiner of the master’s thesis. We would like to thank both of them for their friendly approach, persistent and tireless manner and the desire to always answer our questions. The discussions between the supervisors themselves have been extremely educational and inspiring to take part of.

The peoples chosen for the interviews were Ebbe Rosell at Trafikverket, Mikael Hallgren at Tyréns and Bo Westerberg at Bo Westerberg konsult AB. These are all highly regarded structural engineers with long and extensive experience from design of concrete structures. We would like to thank all of them for their fast and yet comprehensive answers and for adding important aspects to the discussions. Their participation and informative answers provided perspectives to the rules and guidelines in Eurocode 2 that otherwise would not have caught the attention of the authors.

During the master’s thesis project we also had the opportunity to come and visit Johan Söderberg, foreman at PEAB, at his working place, the construction site at Perstorp industry in Stenungsund. He provided us with valuable information from a contractor’s point of view. We would like to thank him and his colleague David Eriksson for answering our questions and providing an additional dimension to our project.

We also send our greatest appreciation to the persons that participated in the survey. We got the impression that those who answered took the questions seriously and did their best to bring out their opinion. The comments obtained from the survey have been extremely valuable in the evaluation of the results. Engineers from the following companies were involved in the survey and earn special attention; Chalmers, COWI, ELU, Inhouse Tech, Reinertsen, Skanska, Structor, Sweco, Trafikverket, Vattenfall and WSP.

Finally, we thank our opponents Anna Sandberg and Joanna Klorek for their thoughtful and improving comments.

Göteborg, December 2013

Anneli Dahlgren & Louise Svensson

Abbreviations and translations

Abbreviations

ACI	American Concrete Institute
BABS	Byggnadsstyrelsens anvisningar till byggnadsstadgan (Byggnadsstyrelsen's Instructions to the Building Law)
BBK	Boverkets Handbok om Betongkonstruktioner (Boverket's Handbook on Concrete Structures)
BBR	Boverkets Byggregler (Boverket's Building Regulations)
BFS	Boverkets Författningssamling (Boverket's Statutes)
BKR	Boverkets Konstruktionsregler (Boverket's Regulations for Structural Design)
BSI	British Standards Institute
CEN	Comité Européen de Normalisation European Committee for Standardisation
CEB	Comité Euro-International du Betón (Euro-International Concrete Committee)
EC2	Eurocode 2
EC	Eurocode
EC	European Commission
ECC	European Economic Community
ECP	European Concrete Platform
EN	European Standards
ENV	European Prestandards
EK	Eurokod (Eurocode)
EKS	Europeiska konstruktionsstandarder (European Standards for Structural Design)
EU	European Union
fib	Féderation Internationale du Betón (International Federation for Structural Concrete)
FIP	Fédération Internationale de la Précontrainte (International Federation for Prestressing)
JRC	Joint Research Centre
MC	Model Code
NA	National Annex
NDP	Nationally Determined Parameters

NR	Nybyggnadsregler (Building Regulations)
PBL	Plan- och bygglagen (The Planning and Building Act)
SBN	Svensk Byggnorm (Swedish Building Code)
SIS	Swedish Standards Institute
TK	Teknisk Komitte (Technical Committee)
VVFS	Vägverkets Föreskrifter (Vägverket's Regulations)

Translations

Boverket	Swedish National Board of Housing, Building and Planning
Byggnadsstyrelsen	Swedish National Board of Building
Planverket	Swedish national Board of Planning
Statens Betongkomitté	Swedish National Concrete Committee
Svenska betongföreningen	Swedish Concrete Association
Svensk Byggtjänst	Swedish Building Service
Trafikverket	Swedish Transport Administration
Vägverket	Swedish Road Administration

Notations

Roman upper case letters

A	area
A_c	cross-sectional area of concrete
A_{ct}	area of concrete within the part of the section which is calculated to be in tension just before formation of the first crack
A_{ef}	effective concrete area
A_{eff}	effective flange area
$A_{eff,i}$	effective area of flange i
A_f	part of the compressive zone or the area of the bending reinforcement within the width $b_{eff,1}$
A_k	area enclosed by the centre-lines of the connecting walls, including inner hollow areas
A_s	cross-sectional area of steel bar or dowel
A_{sf}	cross-sectional area of transversal reinforcement in flange

A_{si} area of one reinforcing bar

A_{si} cross-sectional area of one leg of reinforcement

$A_{si,l}$ cross-sectional area of one longitudinal torsional bar

A_{sl} area of longitudinal torsional reinforcement

$A_{sl,tot}$ total area of longitudinal torsional reinforcement in bottom horizontal wall

$A_{s,max}$ maximum area of bending reinforcement

$A_{s,min}$ minimum area of bending reinforcement

A_{sw} cross-sectional area of one shear reinforcement unit

$A_{sw,i}$ cross-sectional area of one torsional link

$A_{sw,max,\,i}$ maximum amount of transversal torsional reinforcement

$A_s^{provided}$ provided reinforcement amount (ACI)

$A_s^{required}$ required reinforcement amount (ACI)

E modulus of elasticity

E_c modulus of elasticity of concrete

E_s modulus of elasticity of reinforcing steel

F force

F_{bt} is the tensile force from ultimate loads in a bar, or group of bars in contact, at start of a bend

F_c force in concrete compressive strut

$F_{c,i}$ force in concrete in part i (flange or web)

F_{cd} design value of compressive force

F_{ct} force taken by the concrete in tension prior to cracking

F_{cw} compressive force due to inclined cracks

$F_{cw,i}$ compressive force due to inclined cracks in one wall, i, of a cross-section

F_E tensile force that needs to be anchored at a support section

F_{td} design value of tensile force

F_s force in reinforcing steel

F_{sl} tensile force that needs to be resisted by longitudinal torsional reinforcement

$F_{sl,i}$ tensile force that needs to be resisted by longitudinal torsional reinforcement in one wall i

F_{sw} force that can be taken by shear reinforcement

$F_{sw,n}$ force that can be taken by one shear reinforcement unit

F_t force in transverse bar

$F_{u,dow}$ ultimate dowel capacity

F_v shear force acting on a dowel

$F_{v,el}$ elastic shear capacity of a dowel

F_{vR} shear capacity of dowel

I_{eq} second moment of area of equivalent concrete cross-section in state I or II

M bending moment

M_{cr} cracking moment

M_E applied bending moment

M_{Ed} design value of the applied bending moment

$M_{Ed,max}$ maximum design value of the applied bending moment

M_{max} maximum bending moment

M_R moment resistance

M_{Rd} design moment resistance

M_u ultimate moment, moment at failure

M_y yield moment

N axial force

N_a longitudinal component due to inclined compressive stress field

N_b longitudinal component due to inclination of the shear reinforcement

N_{cr} axial cracking force

N_{Ed} normal force that should be added or subtracted from the tensile force

P point load

S_{bd} force growth

T torsional moment

T_{Ed} design torsional moment

$T_{Rd,max}$ design value of maximum torsional moment that needs to be sustained by a wall i of the cross-section

V shear force

V_c contribution to shear capacity from concrete

V_{Ed} design value of the applied shear force

V_i shear force i in one wall of a hollow box section

$V_{Rd,max}$ design value of maximum shear force that can be sustained by the member

$V_{Rd,s}$ design value of shear force that can be sustained by the yielding shear reinforcement

V_s contribution to shear capacity from reinforcement

V_T torsional shear force

V_V vertical shear force (V_{Ed})

Q distributed load

Roman lower case letters

a	distance between bars
a	width of compressive strut
a_b	for a given bar (or group of bars in contact) is half of the centre-to-centre distance between bars (or groups of bars) perpendicular to the plane of the bend. For a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus $\phi/2$
a_l	distance for shifting the moment curve sideways
a_v	distance between load and support
b	width of cross-section
b	width of support
b_i	width of the interface
b_{ef}	effective width of the support
b_{eff}	effective width
b_t	mean width of the part of cross-section in tension
b_w	web width
c	cohesion factor, factor which depends on the roughness of the joint interface
c	concrete cover
c_e	coefficient that considers the eccentricity
c_{min}	minimum concrete cover
$c_{min,b}$	minimum cover due to bond requirement
$c_{min,dur}$	minimum cover due to environmental conditions
c_{nom}	nominal concrete cover
c_0	coefficient that considers the bearing strength of concrete
d	effective depth of cross-section
d_g	maximum aggregate size
e	eccentricity
f_{bd}	bond strength
f_c	compressive cylinder strength of concrete at 28 days
f_{cc}	design concrete compressive strength
f_{cd}	design value of concrete compressive strength
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
f_{ct}	tensile strength of concrete
f_{ctd}	design axial tensile strength of concrete
f_{ctk}	characteristic axial tensile strength of concrete
f_{ctd}	concrete design strength in tension

$f_{ct,eff}$ mean value of the tensile strength of the concrete at the time when the cracks may first be expected to occur. This is equal to f_{ctm} or $f_{ctm}(t)$.

f_{cth} $= 1.5f_{ctk}$. High value of the tensile strength of concrete

f_{ctm} mean value of axial tensile strength of concrete

f_u ultimate tensile strength of reinforcing steel

f_v $=0.35f_{ct}$

f_y yield tensile strength of reinforcing steel

f_{yd} design yield tensile strength of reinforcing steel

f_{yk} characteristic yield tensile strength of reinforcing steel

f_{ym} mean value of the yield strength of the reinforcement

f_{ywd} design yield strength of shear reinforcement

h height of cross-section

h_{cr} depth of tensile zone immediately prior to cracking

h_{ef} height of effective concrete area

h_f thickness of the flange at the junctions between web and flange

k coefficient factor

l_a minimum grouting length for a dowel

l_{bd} design anchorage length

$l_{b,max}$ maximum anchorage length

$l_{b,rqd}$ required anchorage length

l_t transmission length

$l_{t,max}$ maximum transmission length

l_0 lap length

$l_{0,min}$ minimum lap length

n number of shear reinforcement units

n number of links that crosses each crack

n_{leg} number of shear reinforcement legs in one of the shear reinforcement units that crosses the crack in one section

r bending radius of reinforcing bar

s shear slip, shear displacement

s spacing of bars

$s_{b,max}$ maximum spacing between bent up bars

s_{el} elastic shear slip

s_f spacing of transversal reinforcement in flange

s_l maximum spacing of torsional links

$s_{l,max}$ maximum spacing between shear reinforcement

s_{max}	maximum shear slip
$s_{r,max}$	maximum crack distance
$s_{r,max,x}$	maximum crack distance in the x-direction
$s_{r,max,y}$	maximum crack distance in the y-direction
$s_{t,max}$	maximum distance between legs of a series of shear legs
$t_{ef,i}$	thickness of a wall *i* in a hollow box section
u	height of the node
u_k	is the perimeter of the area A_k
v_{Ed}	longitudinal shear stress at the junction between one side of a flange and the web
v_{Edi}	design value of shear stress at joint interface
v_{Rdi}	design value of the shear resistance at joint interface
$1/v$	curvature
w	joint separation, crack width
w_{max}	maximum joint separation
x	depth of compression zone
x_u	depth of neutral axis at the ultimate limit state
x_0	distance to maximum bending moment
z	inner lever arm
z	distance from gravity centre
z_i	side length of wall *i* between the intersection points with the adjacent walls
z_s	level of reinforcing steel in relation to gravity centre
q	shear flow
q_c	concrete reaction
w_k	crack width
w_m	mean crack width

Greek letters

α	angle of inclination
α	ratio between E_s and E_c
α	coefficient factor
β	coefficient factor
β	ratio
β	angle
γ_c	partial factor for concrete

γ_n partial factor considering the safety class

γ_s partial factor for reinforcing steel

Δc_{dev} factor that allow for some deviation of concrete cover in design

$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection

$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel

$\Delta c_{dur,\gamma}$ additive safety element

ΔF_c change of normal force in the joint intersection over the length Δx

ΔF_d change of normal force in the flange over the length Δx

ΔF_{td} additional tensile force

Δx length under consideration in shear between web and flanges

$\Delta \sigma_c$ compressive force at joint intersection due to pullout resistance

$\Delta \sigma_s$ tensile force in transverse bar in joint intersection due to pullout resistance

ε strain

ε_c concrete strain

ε_{cc} compressive concrete strain

ε_{cm} mean concrete strain

ε_{ct} tensile concrete strain

ε_{cu} ultimate concrete strain in concrete

ε_{cu2} ultimate compressive strain in concrete, parabolic stress-strain relation

ε_{cu3} ultimate compressive strain in concrete, bi-linear stress-strain relation

ε_{c2} compressive concrete strain at the peak of f_c, parabolic stress-strain relation

ε_{c3} compressive concrete strain at the peak of f_c, bi-linear stress-strain relation

ε_s steel strain

ε_{sm} mean steel strain

ε_{sx} steel strain in x-direction

ε_{sy} steel strain in y-direction

ε_{sy} yield strain of reinforcing steel

ε_{ud} strain limit of reinforcing steel

ε_{uk} characteristic strain of reinforcing steel at maximum load

η_1 coefficient related to the quality of the bond condition and the position of the bar during concreting

η_2 coefficient related to diameter of the reinforcing bar

θ angle between compression strut and the longitudinal axis

θ_f angle between compression strut and the longitudinal axis in the flange

θ_I angle of the tensile principal stress, σ_I

μ	friction coefficient, factor which depends on the roughness of the joint interface
v	strength reduction factor
v_1	strength reduction factor
v_{Edi}	shear stress at a joint interface
v_{Rdi}	shear resistance at a joint interface
ρ	reinforcement ratio
ρ_{max}	maximum reinforcement ratio
ρ_w	shear reinforcement ratio
$\rho_{w,min}$	minimum shear reinforcement ratio
ρ_x	reinforcement ratio in x-direction
ρ_y	reinforcement ratio in y-direction
$\rho_{\rho,ef}$	effective reinforcement ratio
σ	stress
σ_c	concrete stress
σ_{cc}	compressive strength in concrete
$\sigma_{cc,max}$	design compressive strength in concrete
σ_{ct}	tensile strength in concrete
$\sigma_{cw,i}$	compressive stress in one wall, i, due to inclined strut
$\sigma_{cw,t}$	stress acting in the compressive strut
σ_n	stress per unit area caused by the minimum external force across the interface that can act simultaneously with the shear force
σ_n	normal stress
σ_r	radial compressive stress
σ_s	steel stress
σ_{sd}	design steel stress
σ_{sx}	steel stress in x-direction
σ_{sy}	steel stress in y-direction
σ_x	concrete stress in x-direction
σ_y	concrete stress in y-direction
σ_q	transversal component related to bond stress
σ_{I}	tensile principal stress
σ_{II}	compressive principal stress
τ_b	bond stress
τ_c	shear stress in concrete
τ_{max}	maximum shear stress

$\tau_{t,i}$	torsional shear stress in wall i
τ_{xy}	shear stress
ϕ	frictional angle
ϕ	diameter of the reinforcing bar or the dowel
ϕ_m	mandrel diameter
$\phi_{m,min}$	minimum mandrel diameter
ϕ_s	maximum bar diameter
ϕ^*_s	maximum bar diameter given in Table EC2 7.2N
ω_s	mechanical reinforcement amount
ω_s'	mechanical reinforcement amount with regard to the factor v

1 Introduction

1.1 Background

In the beginning of 2011 the European Standard Eurocode became mandatory for design of supporting structures in Sweden, SIS (2013a). Eurocode 2, SIS (2008), is the current standard in Sweden concerning concrete structures and replaced the previous Swedish handbook BBK 04, Boverket (2004).

According to the Swedish standards institute, SIS (2013b), the goal of the establishment of a unified standard is to facilitate the cooperation between structural engineers from different countries all over Europe. A common technical language will increase the opportunity of exchange of knowledge as well as services between countries.

Eurocode 2 provides rules and guidelines in short sentences and equations complemented with informative figures in order to facilitate for the users of the code. However, there is very limited or no background information explaining the mechanical response and providing motivations for the different expressions.

It is essential that the detailing of reinforcement in concrete structures is performed in such a way that the intended response of the structure with regard to safety and good performance is fulfilled. It is also important that the fundamental theory and required structural behaviour of reinforced concrete structures are known in order to reduce the risk for incorrect interpretations that can lead to a great diversity and even unacceptable solutions. Therefore an investigation where the background to Eurocode 2 is determined is of great interest within the building industry to provide sufficient information of how to implement the rules and guidelines in a correct manner. It is also interesting to investigate and evaluate how structural engineers interprets and applies Eurocode 2 in order to illuminate ambiguities and to be able to identify need for improvements of the new standard.

This master's thesis has been carried out in collaboration with engineers in Reinertsen Sverige AB who have identified ambiguities concerning requirements and rules for configuration of reinforcement in concrete structures. A compilation where problems concerning the design and detailing of reinforcement in concrete structures are identified, investigated, exemplified and discussed was therefore desired by the company.

1.2 Aim

The purpose of this master's thesis was to, from a compilation of ambiguities identified in Eurocode 2 concerning reinforcement requirements and configurations in concrete structures, determine and explain the background for the rules and guidelines based on previous research and experience. This should be performed in order to facilitate the use of Eurocode 2, but also to identify lack of information.

As a part of the project an investigation, by means of interviews and a qualitative survey, should also be carried out to evaluate the usage of Eurocode 2 in order to be able to identify the need for and recommend further development of the standard.

1.3 Method

Starting from Eurocode 2 Part 1-1, SIS (2008), different ambiguities concerning reinforcement requirements and configurations in concrete structures were identified. Comparisons to the old Swedish standards were made to detect any notable changes in the new standard. Further, thoughts and reflections were reconciled during interviews and meetings with people who have experience within the industry. Identification of ambiguities in Eurocode 2 has partly been based on already performed studies within the area. However, the studies that were found were at a lower academic level of knowledge and emphasis has therefore been on interviews with experienced structural engineers and discussions with supervisors.

In order to make the extensive amount of compiled ambiguities more manageable a categorisation based primarily on mechanisms to resist certain load effects was made. From that categorisation a number of ambiguities were chosen to be illuminated and investigated even further.

A more detailed literature study was executed where the background to the highlighted areas was looked into further. This was performed to clarify and explain where expressions and requirements descend from. Main references have been European model codes such as *fib* Model Code 2010, *fib* (2012), CEB-FIP Model Code 1990, CEB-FIP (1991), and Model Code for Concrete Structures, CEB-FIP (1978), that laid the base to Eurocode, as well as Svenska betongföreningens handbok till Eurokod 2, Betongföreningen (2010) and Commentary and Worked Examples to Eurocode 2 published by the European Concrete Platform, ECP (2008).

To get further information and increase the understanding codes and related publications from other countries like USA and Great Britain were also used. In contrast to Eurocode 2 and the European model codes the concrete code from the American Concrete Institute, ACI (2007), include many references to publications and articles. However, this has only been used to some extent due to time constraints. When the information in the literature mentioned above was insufficient, a literature search among articles and publications at the library at Chalmers University of Technology was executed.

Illustrative and educational examples of reinforcement solutions within the area were derived from a combination of interviews and meetings as well as information from literature. No practical experiments or numerical modelling using FE-programs have been executed within this project. This was not performed because of the reliability of that there already existed such investigations.

To determine how actors in the industry interpret Eurocode 2 a survey was conducted including multiple choice questions consisting of possible detail solutions of reinforcement configurations. The motivation of the survey was to stress problem areas where it is believed that Eurocode 2 might be interpreted differently. Structural engineers with experience of detailing and design of reinforcement in concrete structures were chosen to participate in the survey. Multiple choice questions were used for the convenience of those who answered the questions but also to make it easy to compare the results.

For additional and more detailed information interviews were performed with experienced structural engineers in order to capture the overall way of thinking and to get the opinion of the persons who have participated in the development and implementation of Eurocode 2 in Sweden. Questions from the survey laid the base for the discussions at these interviews.

In addition to structural engineers production managers from the construction company PEAB were interviewed in order to examine whether the different practices between the actors are compatible and to find reinforcement detailing solutions that are practically applicable at the construction site.

The methodology chosen for this project have resulted in interpretation of answers form the survey, interviews and technical handbooks. It should be emphasised that texts and answers can be interpreted differently depending on who is reading it and in what context it is read. In the worst situation this can result in an interpretation that the creator of the text or answer did not intend. The authors have therefore throughout the report tried to reproduce the content from texts and interviews as concrete and correct as possible. Interpretations that have been made have also been clarified.

In order to make sure that the progress was going in the right direction and that the aim of the master’s thesis was reached, continuous reconciliation with the supervisors, Morgan Johansson, Reinertsen Sverige AB and Björn Engström, Chalmers University of Technology, was carried out throughout the project.

1.4 Limitations

This project was only related to the general rules and requirements of reinforcement in concrete structures found in Eurocode 2, Part 1-1. Some of the questions encountered where chosen to be processed more deeply while some were only brought to the surface. Furthermore, plain-, prestressed- and prefabricated concrete structures should be left out in order to reduce the number of issues into a manageable amount.

The report should primarily concentrate on design and detailing of reinforcement in beams and slabs and less focus should be on columns and walls since many of the rules applicable for beams and slabs also apply for walls and columns.

Dowel action is something that is not treated in Eurocode 2, but was included in the previous Swedish handbook, BBK 04, Boverket (2004). It is important to take this effect into account when designing connections, why this was chosen to be included in this master’s thesis.

The survey, which was performed in the investigating part of the project, should be limited to about 20 structural engineers in order to get a glimpse of how they practice the standard. The need for a larger number of participants was not considered to be as important as the quality of the answers obtained, why a smaller number, but more experienced structural engineers, were selected for the survey.

Due to time constraints focus have been on the theoretical parts of Eurocode 2 and only a small part of the project has been devoted to examining the constructability and practical aspect of the reinforcement configurations recommended in Eurocode 2.

1.5 Outline of the thesis

The first part, Chapter 2, gives background knowledge about the development of codes for design of concrete structures in Europe and in Sweden. It includes a short overview of what have influenced the codes and the reason why a transition to a common European standard for design of concrete structures was agreed upon. This chapter also provides some examples of consequences due to poor detailing as well as a compilation of ambiguities found in Eurocode 2 in order to provide motivation for increased knowledge and further development of the European Standard.

In Chapter 3 the basic theory behind design and detailing of reinforced concrete structures is presented. Material properties of both reinforcing steel and concrete are explained as well as the composite behaviour of reinforced concrete structures. In order to understand how concrete and reinforcement interact with each other at a global level, models describing the overall structural response and different analyse approaches used in design are also described.

Chapter 4-10 are the result of the literature study performed in this project, and are referred to as the main chapters. Each chapter represent different mechanisms to resist certain load effects, in which different ambiguities identified in Eurocode 2 are presented, explained and discussed in each subchapter. Each main chapter begins with a short description of the required structural response and way of modelling it in order to facilitate the understanding of the requirements presented, explained and discussed in the following subchapters.

The investigating part of the project has been compiled in Chapter 11. This chapter contains presentations of the procedures and the results obtained from the interviews and the survey. The result from each question also contains a short discussion.

In Chapter 12 the result from Chapter 11 is compared to, evaluated and analysed together with some of the results from the literature study presented in Chapter 4-10.

A summary of the results obtained and the conclusions drawn in Chapter 12 is presented in Chapter 13, where references to relevant chapters in this report as well as to treated equations or paragraphs in Eurocode 2 are provided.

Finally, more general conclusions and suggestions for further investigations are presented in Chapter 14.

It should also be clarified that abbreviations and Swedish words that are used in the report are compiled and can be found with English translation in the beginning of this report. Throughout the report references are made to sections, paragraphs and equations in Eurocode 2. To clarify that a reference is referring to an item in Eurocode 2 the notation “EC2” has been added to the reference number.

2 Development of standards for design of concrete structures

2.1 Transition to the European Standards

2.1.1 Development of European Standards - Eurocodes

In 1957 an international agreement named the Treaty of Rome was made between the following European countries: Belgium, France, the Federal Republic of Germany, Italy, Luxemburg and the Netherlands, Treaty of Rome (2013). The treaty established a common market and custom union named the European Economic Community, ECC, which was going to be an important part of the European Union, EU, created in 1993.

As a result of the agreement in Rome the European Commission, EC, who is the executive part of the European Union, introduced the work of Eurocodes in 1975, JRC (2013). This was an action program in the field of construction meant to result in a harmonization of technical rules among the member states, EC (2003).

In 1989 the European Commission approved the mandate to the European Committee for Standardisation, CEN, to prepare the Eurocodes, SIS (2013b). By that, the Swedish Standards Institute, SIS, got involved in the development of standardised rules for structural design by being a part of CEN.

The publication of the first European Standards, EN, i.e. Eurocodes, started in the beginning of the 21st century. These standards were based on the released pre-standards, ENV, that were published between 1992 and 1998, EC (2003).

During 2006 a period began where both Eurocodes and other standards were used simultaneously for design of load-bearing structures, JRC (2013). At the start of 2011 it became mandatory in European countries including Sweden.

Eurocode have ten main parts, depending on type of structure, see Table 2.1, which in turn are divided into several parts, SIS (2008). All these parts are regularly revised and updated versions are intended to be released about every five years, Johansson (2013).

Table 2.1 Subdivisions of Eurocode.

EN 1990	Eurocode 0:	Basis of Structural Design
EN 1991	Eurocode 1:	Actions on structures
EN 1992	Eurocode 2:	Design of concrete structures
EN 1993	Eurocode 3:	Design of steel structures
EN 1994	Eurocode 4:	Design of composite steel and concrete structures
EN 1995	Eurocode 5:	Design of timber structures
EN 1996	Eurocode 6:	Design of masonry structures
EN 1997	Eurocode 7:	Geotechnical design
EN 1998	Eurocode 8:	Design of structures for earthquake resistance
EN 1999	Eurocode 9:	Design of aluminium structures

In Sweden SIS has distributed the responsibility of the Eurocodes on a number of technical committees. In order to meet the need for information about the Eurocodes in Sweden a homepage, www.eurokoder.se, and a helpdesk function, eurokoder@sis.se, have been established, SIS (2013c).

2.1.2 Development of Standards for design of concrete - Eurocode 2

Eurocode 2 is the collective name of the European Standards on design of concrete structures, SS-EN 1992, and the code is divided in eight parts presented in Table 2.2, SIS (2013d).

Table 2.2 Subdivision of Eurocode 2.

SS-EN 1992	**Design of concrete structures**
SS-EN 1992-1-1	General rules and rules for buildings
SS-EN 1992-1-2	General rules - Structural fire design
SS-EN 1992-1-3	General rules - Precast element and structures
SS-EN 1992-1-4	General rules - Lightweight aggregate concrete
SS-EN 1992-1-5	General rules - Structures with unbonded and external prestressing tendons
SS-EN 1992-1-6	General rules - Plain concrete structures
SS-EN 1992-2	Concrete bridges - Design and detailing rules
SS-EN 1992-3	Liquid retaining and containment structures

Eurocode 2 part 1-1 containing general rules and rules for buildings is in the scope of this master's project and is in the following recalled simply by Eurocode 2 or EC2.

Eurocode 2 has to a large extent been developed from CEB-FIP Model Code, CEB-FIP (1978) , which was published in 1978 after a longstanding collaboration between the Euro-International Concrete Committee (CEB) and the International Federation for Prestressing (FIP) to produce international recommendations within the area of concrete structures, *fib* (2013). Since 1978 two more editions of Model Code have been published: CEB-FIP Model Code 1990 published in 1991, CEB-FIP (1991), and *fib* Model Code 2010 published in 2012, *fib* (2012). The latest were published by the International Federation for Structural Concrete (*fib*) which is a fusion between CEB and FIP that took place in 1998. At *fib*'s webpage the following purpose of Model Code 2010 is stated: "The objectives of MC2010 are to serve as a basis for future codes for concrete structures, and present new developments with regard to concrete structures, structural materials and new ideas in order to achieve optimum behaviour".

As a complement to Eurocode 2 the European Concrete Platform, ECP, a non-profit association aiming to promote concrete as the material of choice, has published Commentary to Eurocode 2, ECP (2008a) and Worked Examples for Eurocode 2, ECP (2008b). The reason for the development of these publications was to facilitate the transition to the new set of codes that by many were considered to be too general in character and therefore difficult to work with.

Another handbook that can provide additional information to the rules and guidelines in Eurocode 2 is Designer's guide to EN 1992-2, Hendy and Smith (2010). This is published by Thomas Telford Publishing in collaboration with Eurocodes Expert, Eurocodes Expert (2013). The book refers mainly to design of concrete bridges in Eurocode 2, Part 2. However, it also covers the general rules and guidelines in Eurocode 2, Part 1, since Part 2 often refers to the general rules. It should be noted that this book refers to the British version of Eurocode 2.

2.1.3 Codes for design of concrete in Sweden

In Sweden it is Boverket that is responsible for issuing building regulations for housing. Boverket is the successor of Byggnadsstyrelsen and Planverket. The predecessor to Eurocode in Sweden was Boverkets konstruktionsregler, BKR, which became effective in 1994, Boverket (2013). BKR, Boverket (1994), was in turn the successor of a number of building regulations that are presented in Table 2.3.

Table 2.3 Building codes in Sweden from 1947 until today, Boverket (2013).

Standard		**Published /Entry**
BABS	Byggnadsstyrelsens anvisningar till byggnadsstadgan, BABS *Byggnadsstyrelsen's Instructions to the Building Charter*	1946/1947 1950/1950 1960/1960
SBN	Svensk byggnorm, SBN *Swedish Building Code*	1967/1968 1975/1976 1980/1982
PBL[a)]	Plan- och bygglagen, PBL *The Planning and Building Act*	1987/1987 (2010/2011)
NR	Boverkets nybyggnadsregler, NR *Boverket's Rules for New Construction*	1988/1989 1990/1991 1991/1992 1993/1993
BBR[a)]	Boverkets byggregler, BBR *Boverket's Building Rules*	1993/1994
BKR	Boverkets konstruktionsregler, BKR *Boverket's Designing Rules*	1993/1994
EKS[a)]	Europeiska konstruktionsstandarder, Eurokoder, EK *European Standards, Eurocodes, EC*	2010/2011
[a)] Still valid		

The previous Swedish code for design of concrete structures was Boverkets handbok om betongkonstruktioner, BBK 04, Boverket (2004), and is thus the predecessor to Eurocode 2. BBK 04 was published by Boverket in 2004 as a supporting text to the application of regulations and general advice to the law on technical requirements for structures in BKR, Boverket (1993). BBK 04 has in turn two predecessors, BBK 94, Boverket (1994), and BBK 79, Boverket (1979). It can be added that Model Code 78 has influenced the content in BBK 79.

The new European Standard is developed to fit the requirements of its members. However, it has not been possible to fully satisfy the demands of all the countries and a number of nationally determined parameters, NDP, have therefore been introduced to the Eurocodes. These national parameters are in Sweden published by Boverket in BFS 2011:10 – EKS8, Boverket (2011) and by Trafikverket in VVFS 2004:43, Vägverket (2004), SIS (2013b). All relevant national parameters are also complied and attached to each Eurocode in a National Annex, NA, SIS (2008).

It should be emphasised that the new European Standard and the previous Swedish code BKR and handbook BBK 04 treat combinations of loads and application of partial safety factors somewhat differently, resulting in that it is not possible to combine the rules and guidelines provided in the different codes. Examples of the differences can be found in Hammar (2011).

2.1.4 Additional information to the Swedish Standards

In addition to the codes for design of concrete structures used in Sweden a number of different types of publications have been written. The text that lay the basis for BBK 79 was written by Statens Betongkommitté and contains provisions for design of concrete structures with comments (1975). It can be noted that this text is a draft and it was not allowed to be published or referred to.

Another helpful book is Betonghandbok – Konstruktion edited by Svensk Byggtjänst in 1990, Svensk byggtjänst (1990). The book is a compliance of demands, design requirements, calculation methods, diagrams and examples on the basis of building codes in NR 89, Boverket (1989), and BBK 79, Boverket (1979), Svensk Byggtjänst (2013).

Svenska betongföreningen has published a handbook to Eurocode 2, Betongföreningen (2010), which explains, comments and exemplifies rules and guidelines in order to facilitate the use of Eurocode 2 in Sweden.

2.2 Consequences due to poor detailing

2.2.1 General

Poor detailing is something that needs to be avoided and is often coupled with lack of sufficient consideration in design in combination with poor workmanship at the construction site. This combination can lead to an insufficient performance, damages and catastrophic failure. Insufficient design can depend on that the designer has performed detailing of similar structures before and due to experience performs it in the same way, only with small modifications, in the next project. This may result in that important checks might not be performed and a safe structure is not ensured.

If the designer does not know how to interpret the requirements, the development of the code is insignificant. This is why there is a need for further explaining of the background to expressions and paragraphs in Eurocode 2. In Sections 2.2.2 and 2.2.3 two different types of failure due to poor detailing will be briefly described.

2.2.2 Alvik's Bridge and Gröndal's Bridge

The two tramway bridges in Stockholm were finished between the year of 1998 and 1999 and opened for traffic the same year and cost 2 million SEK to build, see Figure 2.1a, Aftonbladet (2002a). They are designed with a beam consisting of a hollow box section and are prestressed in the top and bottom of the cross-section, see Figure 2.1b, Ny Teknik (2002). In order to make the structure consistent and resistant against shear forces and torsional moment the vertical walls are reinforced with vertical shear reinforcement.

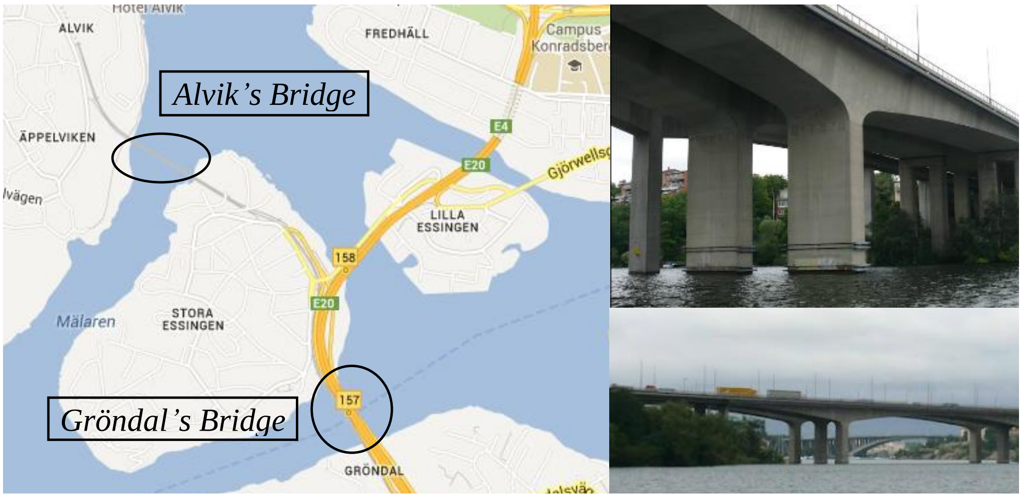


Figure 2.1 Alvik's Bridge and Gröndal's Bridge in Stockholm, a) map showing the locations, b) photo of Gröndal's Bridge taken from below and from the side.

Inclined cracks in the webs of the bridge girder were observed already at the first inspection, Aftonbladet (2002b). However, the cracks of the two bridges became, after three years of use, too large. This is a problem in the service state with regard to corrosion of the reinforcement. Consulted experts are still not sure if it was any danger with regard to resistance in the ultimate limit state. The reason for insufficient crack control depended on insufficient amount of transverse reinforcement. It should be mentioned that no one got injured due to the cracks. According to Håkan Sundqvist, professor in bridge building at KTH, the shear reinforcement should have been three times as large as the amount that was provided, Ny Teknik (2002a).

The designer's that were responsible for the detailing of the bridges state that the current code in Sweden regarding bridges has been followed. The designers argue that the code might be wrong since external consultant companies have controlled the calculations and no errors could be found, Aftonbladet (2002c). However, when comparing to the, then upcoming, European standard Eurocode and German and American codes the requirements in the Swedish code were significantly lower. The difference can depend on that larger effect of prestressing was accounted for differently in the Swedish code, which led to lower need of shear reinforcement. However, the problem was also that it in the Swedish code was no clear instructions for control of crack widths in the service state and in combination with the fact that cracks can occur due to other effects than because of external loads, Engström (2013). A master's thesis performed at Chalmers University of Technology in 2003 implied that the cracks at the two bridges occurred because of restraint stresses caused by temperature changes, Borbolla and Mazzola (2003).

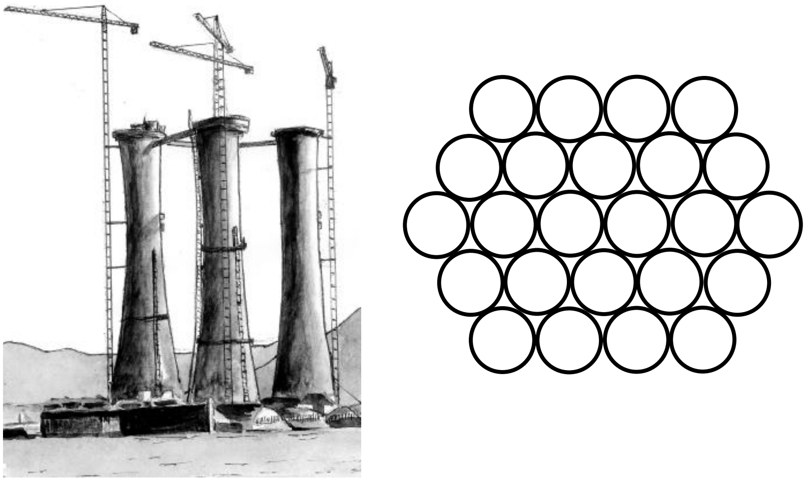
After the cracks were detected and the bridges were closed, the girders were at first preliminary reinforced in order to get traffic moving as quick as possible, Ny Teknik (2002), and in 2002 the final reinforcement of the bridge was performed.

When Vägverket, current Trafikverket, performed calculations later on, they realised that a limitation of the utilised steel stress in the ultimate limit state of 250 MPa would probably be sufficient in order to keep any inclined shear cracks sufficiently small in the service state. This was only a fast and temporary requirement that Vägverket recommended. However, it was removed later on and replaced by the rules in Eurocode.

This problem is mainly related to the rules for checking crack widths of inclined cracks in webs and minimum reinforcement for crack control in the service state.

2.2.3 Sleipner concrete off shore platform

The offshore platform named Sleipner included a large cellular concrete structure below the three towers, see Figure 2.2, Whittle (2013). During the construction the platform was lowered down in the water in order to interfitting the deck. After this the plan was to raise the platform again and tow it to its final position in the oil field. One of the tri-cells failed just before the deck mating took place. Thereafter the structure started to take in water which resulted in sinking and a total collapse at the sea bottom occurred.



a) *b)*

Figure 2.2 The offshore platform Sleipner a) during construction, b) plan section of cell structure. Figure a) is taken from and figure b) is based on Whittle (2013).

The tri-cells were from the beginning designed in order to resist the water pressure, see Figure 2.3b. However, the cylindrically shaped walls were changed to having more sharp edges, see Figure 2.3a. In this case the natural arch action could not be utilised. The modification was made because the formwork became simpler to construct.

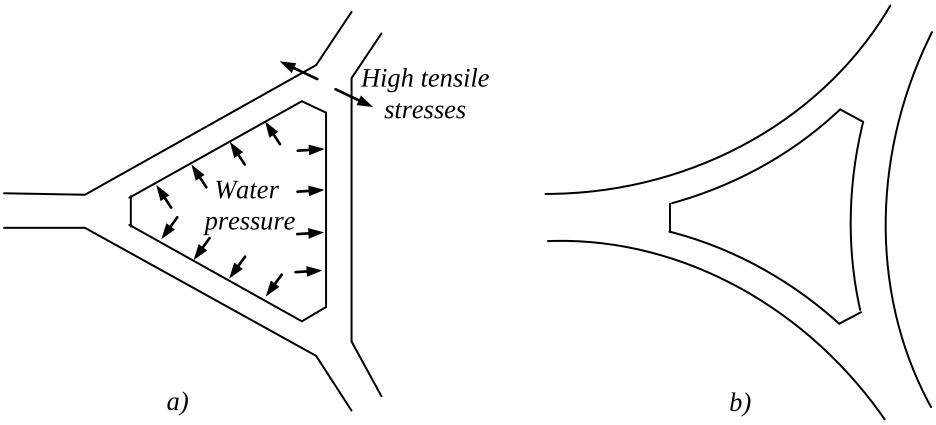


Figure 2.3 Detail of tri-cells with a) sharp edges and b) cylindrical edges. The figure is based on Whittle (2013).

The design of the cell structure was carried out by an analysis using finite element models. However, the quadratical elements used in the analysis did not capture the entire response of the structure and the elements at the edges of the tri-cells became distorted from the ideal square shape. It has been realised after the collapse that the analysis provided values of the shear stresses on the unsafe side.

T-headed bars were used in the critical shear sections, see Figure 2.4. In order to get a sufficient design and capture the whole stress field the bars should extend across the full width of the cross-section. However, they were difficult to anchor through the outer layer of reinforcement why the bars were shortened.

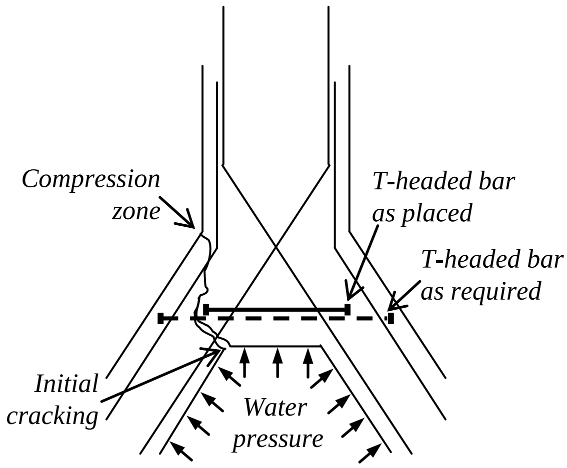


Figure 2.4 Section through the tri-cells where failure occurred. The figure is based on Whittle (2013).

During the submerging of the structure a crack developed at a corner of the cell where the propagation accelerated due to the water pressure. The crack eventually spread to the other end at the compression zone and resulted in a brittle failure.

In linear elastic analysis high tensile stresses were observed in the region where the inclined cell walls meet. However, it was not fully understood how the corresponding tensile force must be resisted in cracked reinforced concrete.

The failure at the corner of the cell could have been prevented and so also the collapse of the whole structure by elongating the transverse reinforcement along the full width of the cross-section. This mistake in detailing can be assumed to be the main reason why the structure failed. Another way to prevent failure could have been to ensure that sufficient arch action can take place by changing the shape of the tri-cells to intersected cylinders.

The element mesh used in the finite element program was too coarse in order to get accurate result and was needed to be made finer. At the time the designer was involved in three other more complicated platforms. This, in combination with that it was a type of structure that was well established, resulted in that few checks of the design and detailing were performed. The rebuilding of the platform was made with cylindrical shaped tri-cells and T-headed bars that were extended to the outer reinforcement.

This problem is mainly related to transformation of calculation results of structural analysis to proper reinforcement detailing that fulfils equilibrium in cracked reinforced concrete in the ultimate state.

2.3 Compilation of ambiguities in Eurocode 2

The European standard, Eurocode 2, summarises information and knowledge concerning design of structural members in concrete and becomes like a tool for the daily work of structural engineers. The code tries to facilitate the design process by presenting the rules and guidelines in and recalling some of the basic theories in a brief way. This can cause problem since the information in the standard is not written in an educational manner and a lot of background information is missing explaining motivations and reasons for the requirements. Therefore it can in several paragraphs be difficult to interpret the different rules and requirements. This risk concerning incorrect interpretations of the standard can lead to insufficiently designed or detailed structures that will not fulfil the demands in the ultimate limit state and serviceability limit state. This is why a careful investigation of the different rules and requirements in the standard was performed in this project in order to highlight problem areas that will be further discussed in this report, see Chapter 4-10. In these chapters the requirements in the code are further explained and clarified. This will hopefully facilitate the interpretation and increase the probability in reaching acceptable design and detailing solutions.

Table 2.4 to Table 2.10 give an overview of different paragraphs and expressions in Eurocode 2 that are treated in this report and each table correspond to one chapter in the report. These tables are based on a larger compilation of ambiguities concerning reinforcement in concrete structures identified in Eurocode 2. This compilation can be found in Appendix A and B. In Appendix A the ambiguities and questions that laid the base for Chapter 4-10 in this report are presented and in Appendix B additional questions found in Eurocode 2 during the initial literature study are stated. It should be noted that all questions presented in Appendix A have not been fully answered in this report, but most of them have been explained or discussed in some way.

Table 2.4 Paragraphs and expressions in Eurocode 2 treated in Chapter 4, Bending.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
9.2.1.1 (1)	(9.1)N	Minimum reinforcement	4.2
9.2.1.1 (3)		Maximum reinforcement	4.3
5.6.2 (2)		Ductility requirements	4.3
5.6.3 (2)		Ductility requirements	4.3
J.2.2 (4), J.2.3 (1), (2)	Fig. J.2, Fig. J.3, J.4	Concrete frame corners	4.4

Table 2.5 Paragraphs and expressions in Eurocode 2 treated in Chapter 5, Shear.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
6.2.3 (3), (4)	(6.8), (6.13)	Needed shear reinforcement	5.2
6.2.3 (3), (4)	(6.9), (6.14), (6.12), (6.15)	Maximum shear reinforcement	5.3
9.2.2 (5)	(9.4), (9.5N)	Minimum shear reinforcement	5.4
9.2.2 (6), (7), (8)	(9.6N), (9.7N), (9.8N)	Spacing of shear reinforcement, beams	5.4
9.3.2 (4), (5)	(9.9), (9.10)	Spacing of shear reinforcement, slabs	5.4
6.2.1 (4), (5)		Minimum shear reinforcement	5.4
6.2.3 (2), (7)	(6.18), (6.7N)	Additional tensile reinforcement due to shear cracks, θ	5.5
9.2.2 (1)		Additional tensile reinforcement due to shear cracks, α	5.5
9.2.2 (2), (4)	Fig. 9.5	Detailing of shear reinforcement, beams	5.6
9.3.2 (2), (3)		Detailing of shear reinforcement, slabs	5.6
6.2.3 (8)	(6.19), Fig. 6.6	Load close to supports	5.7
6.2.2 (6)		Load close to supports	5.7
6.2.1 (8)		Load close to supports	5.7
9.2.5 (1), (2)	Fig. 9.7	Indirect support Suspension reinforcement	5.8
6.2.1 (9)		Suspension reinforcement	5.8
9.2.1.4	Fig. 9.3	Indirect support	5.8

Table 2.6 Paragraphs and expressions in Eurocode 2 treated in Chapter 6, Torsion.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
6.3.2 (1) (3)	(6.28), Fig. 6.11	Longitudinal torsion reinforcement	6.2
9.2.3 (4)		Longitudinal torsion reinforcement	6.2
6.3.2 (1), (4)	(6.26), (6.27), (6.29), (6.30)	Transversal torsion reinforcement	6.3
9.2.3 (1), (2), (3)	Fig. 9.6	Detailing of torsion reinforcement	6.3
9.2.2 (3)		Detailing of torsion reinforcement	6.3
6.3.2 (2), (4)	(6.29)	Combination of shear and torsion	6.4

Table 2.7 Paragraphs and expressions in Eurocode 2 treated in Chapter 7, Shear between web and flanges.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
6.2.4 (3), (6)	(6.20), Fig. 6.7	Shear between web and flanges, longitudinal shear stress	7.2
6.2.4 (4), (5)	(6.21), (6.22)	Shear between web and flanges, transversal reinforcement	7.3

Table 2.8 Paragraphs and expressions in Eurocode 2 treated in Chapter 8, Shear friction and dowel action.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
6.2.5 (1), (3), (4)	(6.23), (6.24), (6.25), Fig. 6.10	Shear at the interface between concrete cast at different times	8.2, 8.3
-		Dowel action	8.4

Table 2.9 Paragraphs and expressions in Eurocode 2 treated in Chapter 9, Bond and anchorage.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
6.2.3 (7)		Curtailment of reinforcement	9.2
6.2.2 (5)		Curtailment of reinforcement	9.2
9.2.1.1 (4)		Curtailment of reinforcement, beams	9.2
9.2.1.3 (1), (2)	(9.2), Fig. 9.2	Curtailment of reinforcement, slabs	9.2
8.4.3 (2)	(8.3)	Basic anchorage length	9.3
8.4.4 (1)	(8.4)	Anchorage length	9.3
8.4.2 (2)	(8.2)	Design ultimate bond stress	9.3
9.2.1.4 (1), (2), (3)	(9.3), Fig 9.3	Anchorage of bottom reinforcement at supports, beams	9.3
9.3.1.2 (1)		Anchorage of bottom reinforcement at supports, slabs	9.3
6.5.4 (7)	Fig. 6.27	Anchorage of reinforcement in compression-tension nodes	9.3
8.7.2 (2), (3), (4)	Fig. 8.7	Lapping of reinforcement	9.4
8.7.3 (1)	(8.10), Tab. 8.3	Lap length	9.4
8.7.4.1		Transversal bars in the lap zone	9.4
4.4.1.2 (1), (2), (3)	(4.2), Tab. 4.2	Concrete cover	9.5
4.4.1.1 (2)		Concrete cover	9.5
4.4.1.3 (1)		Concrete cover	9.5
8.2 (1), (2), (3)		Clear distance between bars	9.5
8.3 (3)	(8.1)	Mandrel diameter	9.6

Table 2.10 Paragraphs and expressions in Eurocode 2 treated in Chapter 10, Crack control.

Paragraph in EC2	**Expressions, figures and tables**	**Subject**	**Section in report**
7.3.2 (1), (2)	(7.1)	Minimum reinforcement for crack control	10.2
7.3.3 (2)	Tab. 7.2N, 7.3N, (7.6N), (7.7N)	Simplified method for crack control	10.2
7.3.1 (2)		Crack control for shear and torsion	10.3
7.3.4 (1), (2), (3)	(7.8), (7.9), (7.10), (7.11), (7.14), (7.15)	Calculation of crack width	10.3

3 Reinforced concrete structures

3.1 Material properties

Concrete has been used as a structural material for thousands of years. Floorings made of concrete discovered in southern Israel have been dated to as early as 7000 B.C., Domone (2010). It is also well known that the old Greek and Roman societies used concrete to build large structures, such as Pantheon in Rome Figure 3.1 that dates back to about 125 A.D, Fazio *et al*. (2008).

Figure 3.1 Pantheon in Rome. Figure is taken from Fazio et al. (2008).

Pantheon is a good example of that the Romans knew the properties of concrete and how to work with such a material. The building is made out of arches and vaults, not to mention the gigantic dome; –structures that transfer forces mainly in compression. In Figure 3.2 a schematic stress-strain relation of concrete is presented, showing that concrete is much stronger in compression than in tension, something that the Romans obviously knew.

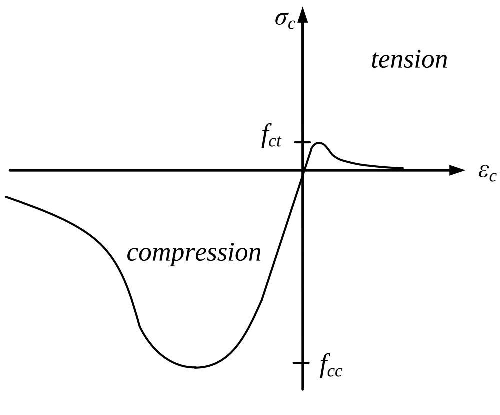


Figure 3.2 Schematic stress-strain relation of concrete.

Due to its small strength in tension concrete is a material with important limitations. During the 19th century the first reinforced concrete structures were introduced, Building Construction (2013), gaining from the advantageous properties of steel in tension, see Figure 3.3. Ordinary reinforcing steel has a tensile strength at yielding, f_y, of about 500 MPa which can be compared to the tensile strength of concrete which is about 1.6-5.0 MPa dependent on concrete class, SIS (2008).

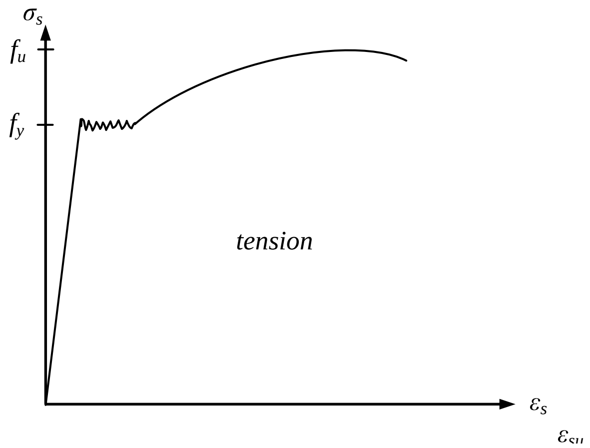


Figure 3.3 Schematic stress-strain relation of reinforcing steel in tension.

In design it is however convenient to use simplified stress strain relations for steel and concrete. Since it is favourable to let the concrete take compressive forces, while the reinforcement should act in tension the simplified relations for concrete in compression and steel in tension are of interest, see Figure 3.4 and Figure 3.5. These figures show both characteristic stress strain relations, denoted with the letter *k*, and the relations used in design situations, denoted with the letter *d*.

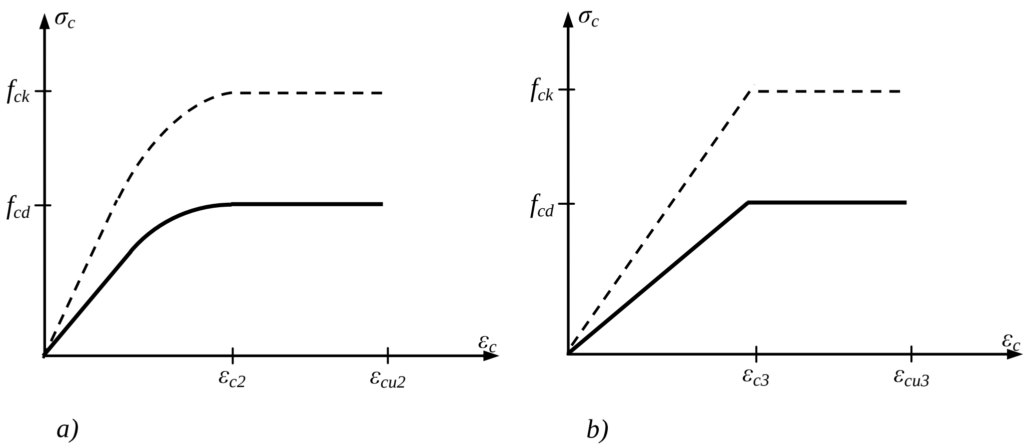


Figure 3.4 Material models for compressed concrete, a) simplified parabolic-rectangular curve. b) simplified bi-linear curve. The figures and notations are based on SIS (2008).

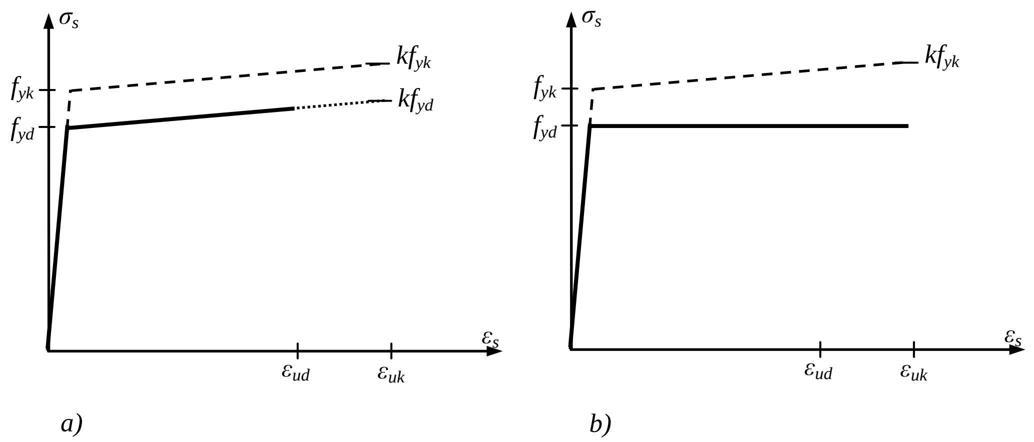


Figure 3.5 Material models for reinforcing steel in tension, a) simplified bi-linear curve with inclined top branch, b) simplified bi-linear curve with horizontal top branch. The figures and notations are based on SIS (2008).

It is a common misunderstanding that reinforcement is used in concrete to avoid cracking. This is however not the case, if not talking about prestressed concrete that is out of the scope of this project. For uncracked concrete the reinforcement has very limited influence and the concrete will crack when its tensile capacity is reached. This can be illustrated by a moment-curvature relation, see Figure 3.6, where the concrete cracks at M_{cr}.

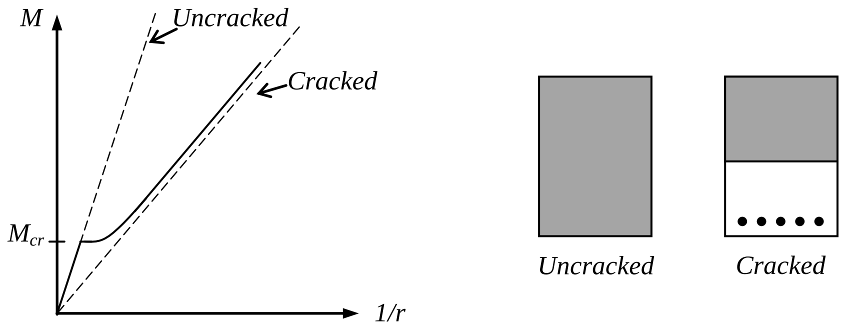


Figure 3.6 Response of reinforced concrete cross-sections before and after cracking.

The reinforcement will nevertheless contribute to the distribution of cracks and hence to the limitation of crack widths after cracking of concrete. This is illustrated in Figure 3.7.

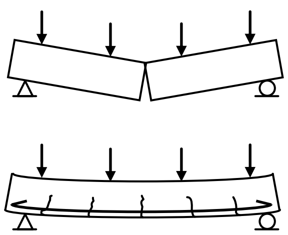


Figure 3.7 Response of plain and reinforced concrete.

Reinforcement will not only contribute to the tensile capacity of reinforced concrete structures, but it will also contribute to the ductility since it enables redistribution of forces in a structure, which is a desirable property. In Eurocode 2 the ductility class of reinforcing steel is defined by characteristic values of the ratio of tensile strength to the yield stress, $k = (f_u / f_y)_k$ and as the strain, ε_{uk}, at maximum tensile force, Betongföreningen (2010a). Different types of reinforcing steel hold different ductility properties and are therefore divided into three classes in Eurocode 2: A, B and C. Type B steel, e.g. B500B, is the most commonly used reinforcing steel in Sweden except for seismic design where the more ductile type of steel, class C, is used, Johansson (2013). It should be noted that the ductility of the reinforcement is not the same as the ductility of the structure. The behaviour of the structure is affected also by other properties as the bond between steel and concrete and the amount of reinforcement in relation to concrete, Betongföreningen (2010a).

It should be noted that Eurocode 2 Part 1-1 is valid for normal strength steel within a yield strength range, f_{yk}, from 400 MPa to 600 MPa and applies to ribbed and weldable reinforcement. Hence, no recommendations are given of how to proceed for plain bars, which is a problem in analysis of old concrete structures containing this type of reinforcement. The previous Swedish handbook BBK 04, Boverket (2004), did provide rules that applied for plain bars as well and can give some guidance of how to handle this problem. This is one example of how the technical development over the years results in uncertainties in how to use the codes in a correct manner.

Over time there have been large changes in material properties of both concrete and reinforcing steel, Whittle (2013). The development of strength properties of both materials has increased significantly during the 20th century. For concrete this is much thanks to the use of admixtures that have become common since the 1980s.

Although this trend is a good thing, making the materials stronger, there is also a backside. Expressions and rules of thumbs used in Eurocode 2 can have been developed a long time ago and might be based on empirical results. It is therefore important to know and understand the background to, and under what conditions, the expressions were developed, in order to know if they are applicable to material properties of today. However, it can be assumed that the expressions provided in the codes are valid for the specified steel and concrete classes. Examples of this will be shown in this report, for instance in Section 4.2 concerning minimum reinforcement requirements in beams.

3.2 Transfer of forces between the materials

3.2.1 Bond between reinforcing steel and concrete

Understanding about how forces are transferred between the materials, steel and surrounding concrete, is necessary when performing structural design. Transmission of forces between the materials depends on the roughness of the bar, i.e. if it is a plain or a deformed bar. The reinforcement normally used in Sweden is heat treated ribbed bar (K500B), hot-rolled ribbed bar (Ks600S) and cold-worked indented bar (Ps500), Engström (2011a).

When a reinforced concrete member is loaded, the transmission of forces is due to bond stresses, τ_b, which are acting along the bar's mantle surface within the transmission length, see Figure 3.8.

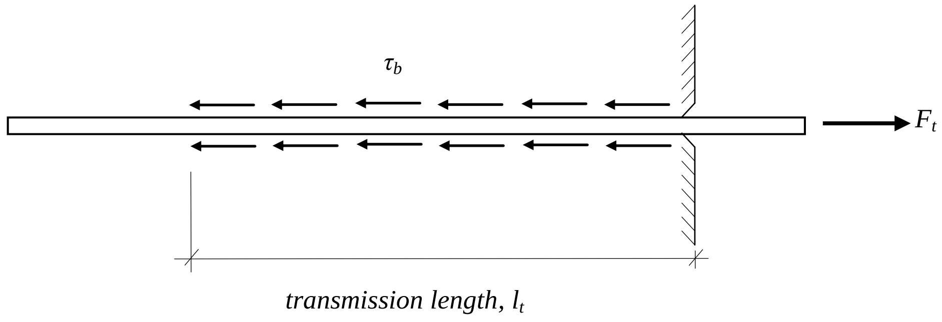


Figure 3.8 *Bond stress acting along the bar's mantle within the transmission length. The figure is based on Engström (2011a).*

The bond stress is largest closest to the loaded end of the bar and decreases successively along the bar. It should be noted that the bond stress is related to a certain deformation, slip, between the surface of the steel and the surrounding concrete. For moderate loads in the service state the slip increases in relation to bond stress, see Figure 3.9.

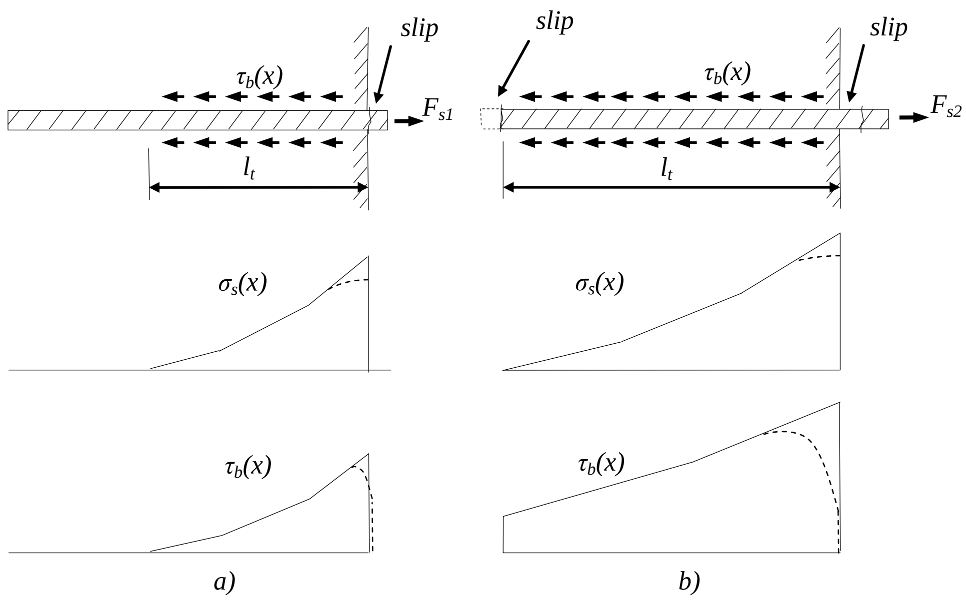


Figure 3.9 *Inclined cracks occur out from the ribs from the reinforcing bar subjected to tension due to transfer of forces between the materials. The figure is based on Engström (2011a).*

Figure 3.9a illustrates how the bond stress varies for a bar subjected to a small tensile force. In this case the bond stress is only generated along a part of the embedded reinforcing bar. Hence, the transmission length, l_t, is smaller than the available bar length. The bar will slip with different values along the transmission length. This depends on the fact that the tensile force decreases along this length, which results in horizontal equilibrium. Hence, the steel strain will also decrease successively along the bar. The maximum slip occurs at the loaded end, while it in the other end of the transmission length will not slide at all.

Figure 3.9b illustrates how the bond stress and the transmission length both have increased when a large tensile force is acting on the bar, which also results in a large steel stress. In the case shown in Figure 3.9b the bond stress is acting along the whole length of the bar, i.e. the transmission length is equal to the available anchorage length. This means that the whole bar is sliding, however, with different magnitudes in different sections.

As can be seen in Figure 3.9b the bond stress will not be zero at the end of the bar since the whole bar is sliding. However, this is not the case for the steel stress which needs to be zero at the end of the bar. It can be noted that for moderate loading a large slip will generate large bond stress.

When the tensile force in a section of the bar is small the bond stresses in this section depend on adhesion. However, when the force becomes larger, the bond stresses depend on the shear key effect that is obtained due to the roughness of the surface. It should be noted that in the ultimate state splitting cracks may occur around the bar resulting in that the bond stresses are evened out and the distribution becomes more uniform along the anchorage length.

When a bar is pulled by a tensile force, shear stresses between the steel and the surrounding concrete give rise to inclined principal compressive stresses and principal tensile stresses in the regions of the concrete closest to the bar. Cracking will occur, with an angle out from the ribs of the bar, if the concrete tensile strength is reached, see Figure 3.10a.

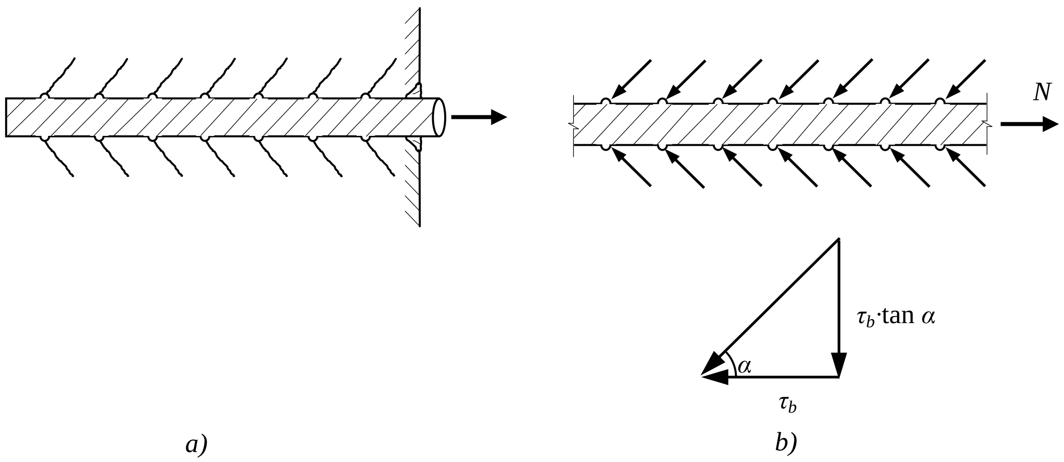


Figure 3.10 Reinforcing bar subjected to tension, a) inclined cracks occur out from the ribs from the reinforcing bar, b) inclined compressive forces due to shear key effect between the surfaces of the reinforcing bar and the surrounding concrete. The figure is based on Engström (2011a).

At this stage it is in general the inclined compressive forces that are anchoring the bar into the concrete, see Figure 3.10b. The longitudinal component of the compressive stress can be described as the bond stress, τ_b. The transversal component can be calculated as $\tau_b \tan\alpha$, where α is the angle between the bar and the inclined compressive force.

Figure 3.11 shows how the compressive force is spread in all directions out from the bar, creating compressed conical shells. These shells only exist between the cracks and start from the ribs of the bar. The compressed conical shell needs to be equalised by tensile forces due to equilibrium. This is achieved by tensile stresses in the concrete in form of tangential stresses formed as a ring at the bottom of the cone.

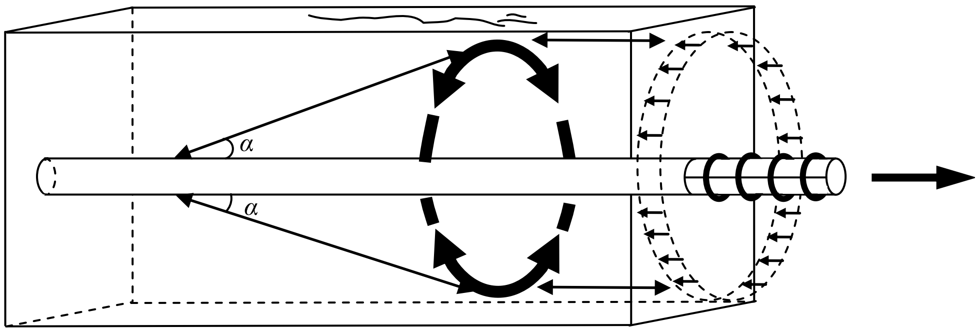


Figure 3.11 Compressive forces that are spread in all direction in a compressed conical shell that is hold together by tensile stresses in the concrete at the outer edge. The figure is based on Engström (2011a) who has borrowed it from Tepfers (1973).

If the concrete cover is small in relation to the bar dimension, splitting cracks may occur, see Figure 3.12. These cracks can be described by looking at the bar as a tube. The tube has a large inner pressure, which results in radial compressive stresses to the surrounding concrete that needs to be equalised by tensile stresses in the tangential direction, see Figure 3.12. The tensile stresses cause splitting cracks when the concrete tensile strength is reached. At this stage the effect of the circle in Figure 3.11 is lost.

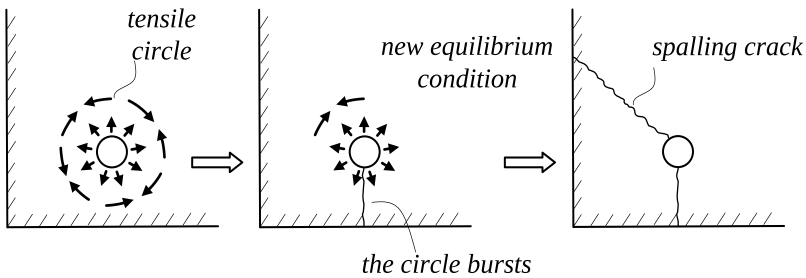


Figure 3.12 Radial compressive stresses are balanced by tension stresses in the tangential direction. If the tensile strength of the concrete is reached splitting cracks through the concrete cover will occur. The figure is based on Engström (2011a).

To counteract a decrease of stiffness in the concrete section when splitting cracks occur transverse reinforcement can be provided within the concrete cover and perpendicular to the anchored bar. If the concrete cover is sufficient, the concrete can equalise the radial component of the inclined compressive force and thereby prevent the splitting cracks. Then it is instead the longitudinal component of the compressive force, see Figure 3.10b, which becomes critical for the failure. The concrete between the ribs of the bar will be crushed and sheared off. When the bar starts to slide significantly, frictional forces develop due to the compressive stresses that are still acting at the mantle surface. It can be explained as the bar will be prevented to slide because of the ribs that are embedded in the concrete.

Three different types of anchorage failure can be distinguished:

- pullout failure in concrete without splitting cracks, see Figure 3.13a
- pullout failure in concrete with splitting cracks
- splitting failure, see Figure 3.13b

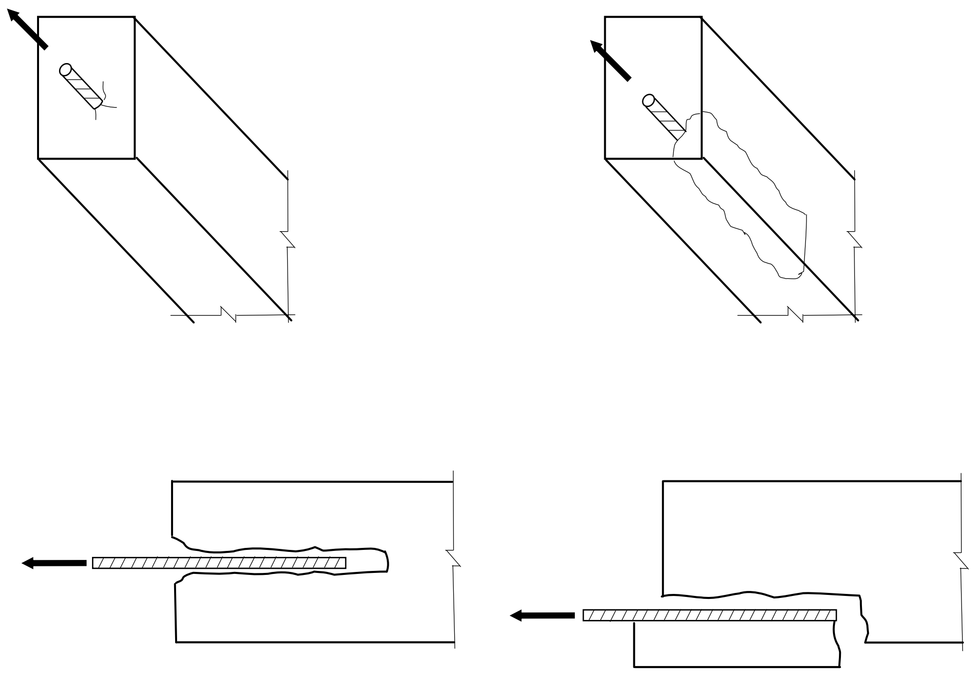
If the concrete cover is sufficient pullout failure will occur without splitting cracking, see Figure 3.13a. In order to get pullout failure without splitting cracks it is recommended to use a concrete cover of 3ϕ, where ϕ is the diameter of the anchored bar and a large distance between adjacent bars. This type of failure will give an upper limit for the anchorage capacity.

If the concrete cover or distance between adjacent bars is insufficient, then splitting cracks will occur through the concrete cover or between the bars. However, if transversal reinforcement is provided the final failure will be pullout failure with splitting cracks, but with a lower anchorage capacity than what is btained when splitting cracks are avoided, since the concrete cover is weakened by cracks in this case. The capacity is affected by the amount of transversal reinforcement.

If the concrete cover is small and no, or an insufficient amount, of transversal reinforcement is provided, splitting cracks might lead to spalling of the concrete so that the reinforcing bar is detached, see Figure 3.13b. This type of splitting failure will have a sudden and brittle nature.

The expected failure when designing beams and slabs with regard to anchorage is pull-out failure with splitting cracks or splitting failure. In order to prevent very brittle failure of the member a certain amount of transversal reinforcement is helpful.

In Chapter 9 more about bond and anchorage of reinforcement in concrete is explained.



a) *b)*

Figure 3.13 Different types of anchorage failure, a) pullout failure without splitting cracks in the case of large concrete cover, b) splitting failure when the concrete cover is small. The figure is based on Engström (2011a).

3.2.2 Friction

The frictional force can in a simple way be defined as the friction coefficient times the normal force, $F_{vR} = \mu N$, and describes the shear resistance when two elements are sliding along an interface. The frictional force acts in the opposite direction to the force, F, that causes the movement, see Figure 3.14a. It develops from the roughness of the surfaces that are sliding against each other. Friction expresses the resistance of movement and cohesion expresses the molecular forces and interlocking effects holding an element together. Note that cohesion also exists when no normal force is acting on the body, see Figure 3.14b.

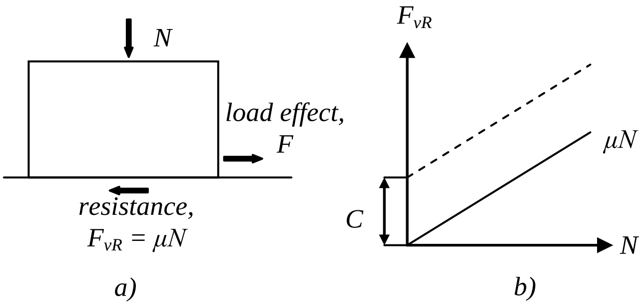


Figure 3.14 Frictional resistance, a) model of the frictional force, b) the relation between the normal force and friction force. Here the cohesion, C, gives a start value for the shear force.

The cohesion, C, gives a start value for the shear force and can be calculated according to Equation (3.1). This can be compared to the cohesion coefficient, c, in Figure 3.15c where it gives a start value for the shear stress, τ. The cohesion factor, c, is part of a general model and can express various effects which results in that the cohesion between concrete elements is difficult to describe.

$$C = c \cdot F_{ct} = c \cdot f_{ct} \cdot A_c \tag{3.1}$$

F_{ct}	tensile force taken by the concrete
c	cohesion coefficient
f_{ct}	tensile strength of concrete
A_c	area of the concrete

$$\tau_R = c + \mu \sigma_n \tag{3.2}$$

σ_n	normal stress
μ	friction coefficient

where

$$\mu = \tan \phi \tag{3.3}$$

ϕ	frictional angle

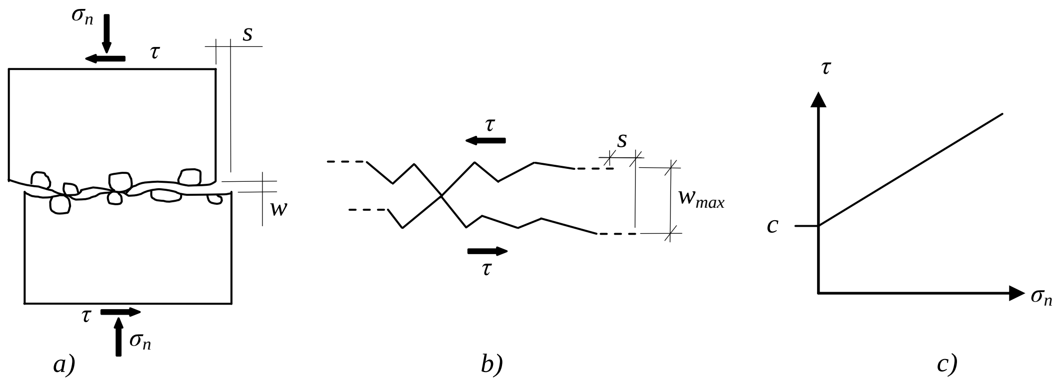


Figure 3.15 Model for shear resistance along an interface, a) shear slip, s, develops which results in a lateral joint separation, w, and b) eventually in w_{max}, c) relationship between the normal stress and the shear stress.

The cohesion also depends on the scale, Engström (2013). When the scale is normal it can be understood as a glue-effect between the interfaces, see Figure 3.16a. However, when the scale is smaller and a microscope is used, see Figure 3.16b, it is the sharp edges of the roughness at the interface that hooks into each other, called interlocking effects. When slip occurs along the interface, contact regions at the irregular joint face, are successively teared off contributing to the shear resistance. This will prevent the shear sliding to develop and that adds to the cohesion factor. Due to this the cohesion can be regained even after cracking has occurred.

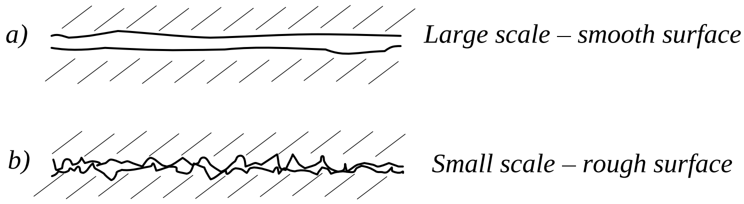


Figure 3.16 Different levels of the scale describe the cohesion in different ways, a) glue-effect holds the joint interface together, b) interlocking effects.

When a joint, consisting of concrete surfaces, is subjected to a shear force along a joint a shear sliding, *s*, develops, *fib* (2008). The frictional resistance that is obtained can be compared to shear transfer in cracks due to aggregate interlock effects and can be determined by Equation (3.2), see Figure 3.15c. The aggregate interlock effect can be described by wedging of the joint interface surface. When the roughness of the joint faces is more distinct, the influence of aggregate interlock will be more significant, i.e. the shear damage of the joint faces will contribute to the shear resistance.

The roughness of the joint interface will generate a joint separation, *w*, when it is subjected to a shear displacement, *s*, see Figure 3.15a, *fib* (2008). The maximum shear slip, w_{max}, is determined by adding the largest tip of the irregularities of each adjoining member's surface, see Figure 3.15b. The shear slip, and hence the joint separation, is decreased if a transverse compression force is acting on the joint interface. This compression force can be an externally imposed load or the pullout resistance of reinforcing bars crossing the interface.

3.2.3 Shear friction at joint interface with transverse reinforcement

One of the basic mechanisms for shear transfer is frictional resistance in joint interfaces, *fib* (2008). It should be noted that this mainly refers to connection in precast members. The frictional resistance is further on referred to as shear friction. When the joint contains transverse reinforcement and is subjected to shear sliding, s, internal compressive forces are generated acting on the concrete, due to the pullout resistance of the transverse reinforcing bar, see Figure 3.17. When the shear slip develops along the joint it will separate because of the roughness of the concrete. This separation causes tensile stresses in the transverse bars and the tensile forces needs to be equalised by a compressive force of the same magnitude acting across the joint. This self-generated compressive force will clamp the adjacent concrete elements together; see Figure 3.17b and Figure 3.17c.

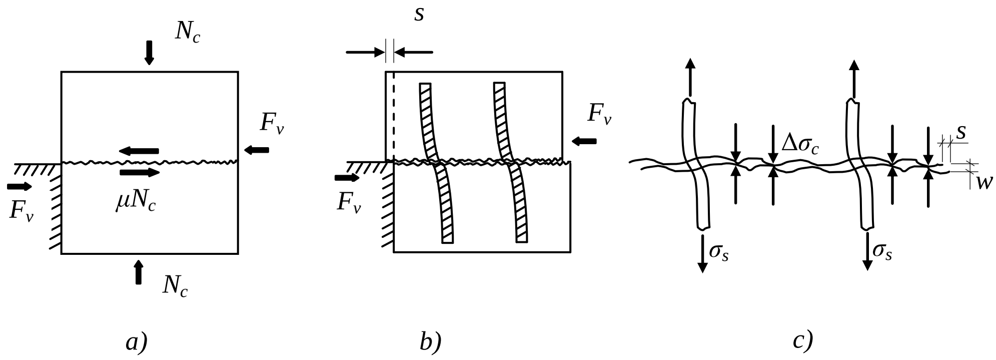


Figure 3.17 Shear transfer at joint, a) external compression across the joint, b) and c) compression is generated by the pullout resistance of transverse bars across the joint. The figure is based on fib (2008).

The shear transfer at a joint interface can be visualised schematically as shown in Figure 3.18, *fib* (2008). Here the saw-tooth geometry expresses the roughness of the joint interfaces and the inclination of each tooth is equal to the frictional angle, ϕ. This is actually a good illustration of how the shear force is transferred. The most pronounced irregularities of the joint face will be loaded first. The shear force creates high concentrated stresses at these spots that eventually will result in local crushing of the irregularities and shear-off of tips and sharp edges. When this has occurred the roughness will be more even and symmetric as shown in Figure 3.18.

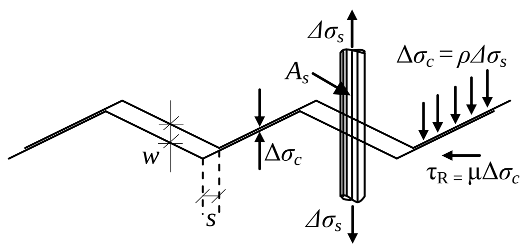


Figure 3.18 Schematic model of how the shear force is transferred by friction due to the pullout resistance of the transversal bars across the joint interface. The figure is based on fib (2008).

If the joint is designed with properly placed transverse reinforcement, the pullout resistance of the bars will increase the shear capacity, since the reinforcement generates a compressive force at the joint interface making friction possible, *fib* (2008). When the bars yield maximum shear resistance is reached. The conditions for this model are:

- rough surfaces that gives a separation, *w*, when shear displacement, *s*, occurs
- sufficient bond resistance between the steel and the concrete that will give large local steel strains for small shear displacement. Here a smaller bar diameter gives better result.

If these two conditions are not fulfilled a different behaviour will develop in form of dowel action, which will give a lower shear capacity, see Section 3.2.4 and Chapter 8.

3.2.4 Dowel action

A basic mechanism of shear resistance in concrete joint is dowel action of a partly embedded steel bar, *fib* (2008). As for shear friction at joint interface with transverse reinforcement this refers mainly to connection in precast members. The dowel action of transverse steel bars, pins and bolts resists the load by shear displacement, *s*, between the joint interfaces. When the dowel is loaded in shear it is supported by the concrete on the opposite side of the dowel where the load is acting. If comparing dowel action and shear friction the steel bar in the former case will fail in bending and the latter in tension. Thus, for the same steel bar and joint, the steel bar is more effectively used in shear friction than in dowel action. The shear resistance in shear friction is greater than the shear resistance in dowel action. Which type is decisive depends on the pull-out resistance of the bar and the roughness of the joint. You don´t need to take dowel action into account when designing a joint, if it is rough and the bars are well anchored.

The shear force capacity of the connection is influenced by many factors, for instance the size of the dowel, the strength of the steel and concrete or the concrete cover of the dowel pin, *fib* (2008). The following failure modes can be distinguished

- concrete splitting failure
- steel shear failure
- steel flexural failure (combined steel/concrete failure)

Splitting cracks in the concrete is one of the failures that can occur in dowel connections between concrete elements, *fib* (2008). When the dowel is loaded in shear, high concentrated compressive forces will be applied to the area surrounding the dowel. When these forces are spread substantial tensile stresses develop in the concrete. Splitting cracks are likely to occur even for small shear forces, if the dimensions of the concrete elements are small or if the concrete cover of the dowel is inadequate. This can limit the shear resistance of the connection by causing a premature brittle failure. It is of importance that the connection is designed correctly so that the concentrated reactions are safely spread and transferred into the element. The designer can prevent splitting failure by providing splitting reinforcement in the shear connection by using suitable strut and tie models, see Figure 3.19. The reinforcement will not prevent splitting cracks from occurring, but makes it possible to keep equilibrium in cracked reinforced concrete. Hence, it is possible for the dowel to reach its full shear capacity governed by one of the two failure modes.

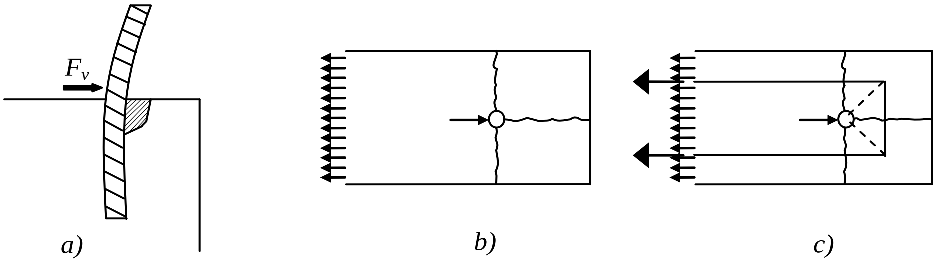


Figure 3.19 Splitting effects around a dowel pin loaded in shear, a) concrete splitting failure occurs at the opposite side of the load, b) possible risk of cracking, c) strut and tie model for design of splitting reinforcement. The figure is based on fib (2008).

It is possible that a weak bar loaded in pure shear placed in an element with high concrete strength and large concrete cover fails by shear of the bar itself, i.e. steel shear failure, *fib* (2008).

If the concrete cover is sufficient or if equilibrium can be achieved in spite of splitting cracks by properly designed splitting reinforcement, the failure of the connection will generally be due to combined concrete/steel flexural failure, *fib* (2008). This will be explained more thoroughly in Section 3.2.4.

The simplest case of dowel action is when a bar embedded at one end is loaded by shear force acting along the joint face or at some distance from the joint face, see Figure 3.20, *fib* (2008). If a section through the dowel pin is studied the stresses in the concrete vary along the pin as indicated in Figure 3.20b and Figure 3.20c and here the theory of beams on elastic foundation gives a good result. The shear force distribution, which the dowel pin is subjected to, will change sign along the dowel pin, and this will result in a bending moment with a maximum value at some distance from the joint face.

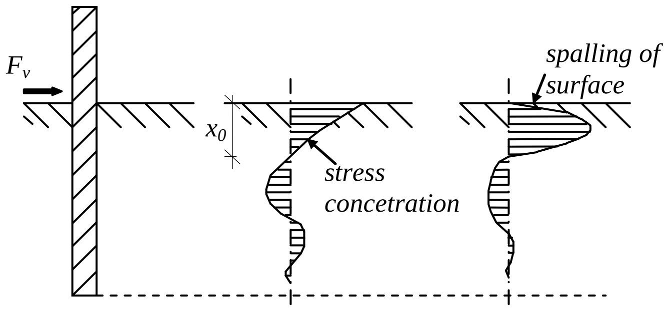


Figure 3.20 Dowel action, a) a one-sided dowel pin is loaded by a shear force and is used for resisting the load at the joint interface, b) bearing stresses in the surrounding concrete in the plane through the dowel pin, c) due to spalling of the concrete at the surface the bearing stresses becomes zero there. The figure is based on fib (2008).

If the concrete cover is such that the resistance of the concrete and dowel are almost equal but the dowel is slightly more prone to go to failure then bending of the dowel and formation of plastic hinges at some distance from the joint interface will occur, i.e. at the section with the maximum bending moment, *fib* (2008).

At the same time major settlement of the dowel pin occurs in the surrounding concrete which means that the concrete will be crushed due to the high compressive stresses, see Figure 3.21. It should be noted that it is not the plastic hinge that causes crushing of the concrete. Plastic redistribution takes place between steel and concrete until both are yielding, then a mechanism occurs. The combined concrete/steel flexural failure will often be the case when using normal dimensions and strengths of the steel and the concrete.

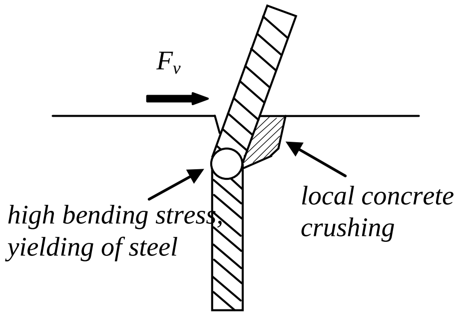


Figure 3.21 Normal failure mechanism in dowel action. The figure is based on fib (2008).

In Section 8.4 the expression concerning the shear capacity for a dowel failing in the combined steel/concrete failure is explained and derived.

3.3 General behaviour of reinforced concrete structures

3.3.1 Response

In order to know how to design a structural member and what analysis approaches that are valid, it is important to understand the typical response of the structural member in reinforced concrete. This response can be divided in four different stages: uncracked stage, cracked stage, yielding and collapse. These four different stages will be presented in the following text which is based on information from Engström (2011b).

In the uncracked state the influence of reinforcement is small meaning that a homogenous material can normally be assumed in analysis. The relation between the load and deformation is linear elastic, meaning that in case of bending the curvature increases linearly with the applied bending moment. The linear elastic analysis results in one unique solution independent of the load; hence it is only the magnitude of the stresses that increases with increased loading. Since the sections are uncracked, the stiffness is constant even when the load increases, but the shape of the stress field remains, which means that the global response also is be linear elastic.

Since the stiffness depends on whether the section is uncracked or cracked, there will be drastic changes of stiffness when cracking occur. In the cracked regions the stiffness is dependent on the reinforcement amount and its arrangement. In this stage the relation between the load and the deformation is no longer linear.

In statically indeterminate structures stiffer regions attract forces. Hence redistribution of moment takes place, stress redistribution due to cracking. The moment thus decreases in the cracked regions and increases in the uncracked regions and influences also the global response. As the load increases cracking of the structural member continue until the member becomes fully cracked. As a result the stress field configuration differs from that in the uncracked state.

Due to cracking and continuous change of the stiffness distribution, reinforced concrete members have a non-linear response in the cracked stage. The ultimate state is reached when one of the materials, concrete or steel, reaches non-linear behaviour, for instance when the reinforcement starts to yield in a region of the member.

Yielding of a region will drastically affect the response of that region as well as the global response. However, this does not mean that the capacity of the member is reached. For statically indeterminate structures, the load can still be increased due to plastic redistribution, even though significant increase of the moment in the yielding section is not possible. The plastic redistribution is due to the plastic deformation of the yielding region, which behaves like a plastic hinge.

When load increases, one or more plastic regions will develop and when the plastic resistance is reached in some critical regions, it determines the resistance of the whole structural member. A collapse mechanism forms, when a critical number of plastic hinges develop. However, a condition is that the needed plastic rotations in the first hinge(s) can develop. Otherwise, a premature failure of a hinge occurs before a collapse mechanism is formed.

To analyse the equilibrium conditions in the collapse stage an ideally plastic behaviour of the sections can be assumed and the force distributions can be solved by means of theory of plasticity.

The response of a concrete member subjected to bending can be described by a moment curvature relationship as in FIG. The different stages have been pointed out in this figure.

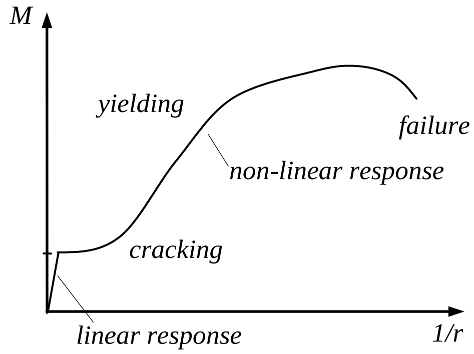


Figure 3.22 Response of a reinforced concrete section subjected to bending.

3.3.2 Modelling

A structural member subjected to shear and bending, in this case a simply supported beam subjected to a uniformly distributed load, will have moment and shear force distributions as in Figure 3.23.

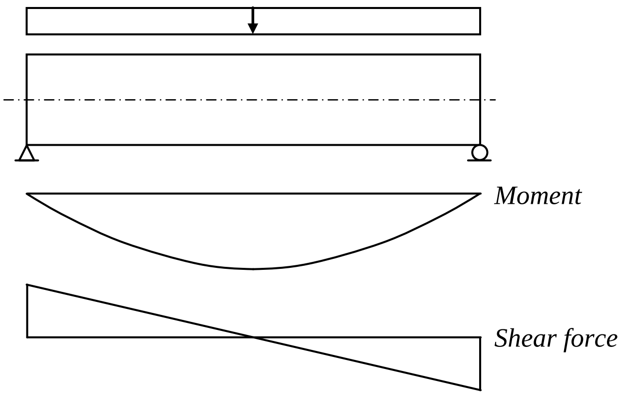


Figure 3.23 Moment and shear force distribution of a simply supported beam subjected to a uniformly distributed load.

The moment will in this example create compression in the top of the beam and tension in the bottom with maximum values at the middle of the beam. The shear force will on the other hand generate large shear stresses at the beam ends, where the supports are, while the shear stress in the middle of the beam will be zero. This is illustrated in Figure 3.24 that shows the distribution of stresses, normal stress, σ_c, generated by the bending moment and shear stress, τ_c, generated by the shear force, along the beam. The figure also shows how the stresses vary over the depth of the beam. It is clear that the normal stresses have a linear distribution with maximum values at the top and bottom of the beam. On the contrary, the shear stresses are maximum at the centroid and zero at the top and bottom of the beam

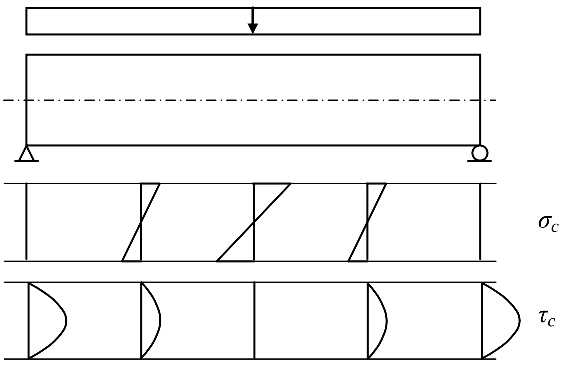


Figure 3.24 Distributions of normal and shear stresses along a simply supported beam subjected to a uniformly distributed load.

A concrete structure cracks, if the principal tensile stress is reaches the tensile strength of the concrete material. To understand how a structural member behaves at cracking the principal stresses are therefore of importance. By using Mohr's circle it can be shown that the principal stresses at the sectional centroid are equal to the maximum shear stress at this section, τ_{max}, with directions shown Figure 3.25a, Engström (2010). However, at the bottom of the beam the principal stresses, σ_{I} and σ_{II}, are equal to the normal stresses both in size and direction, see Figure 3.25b.

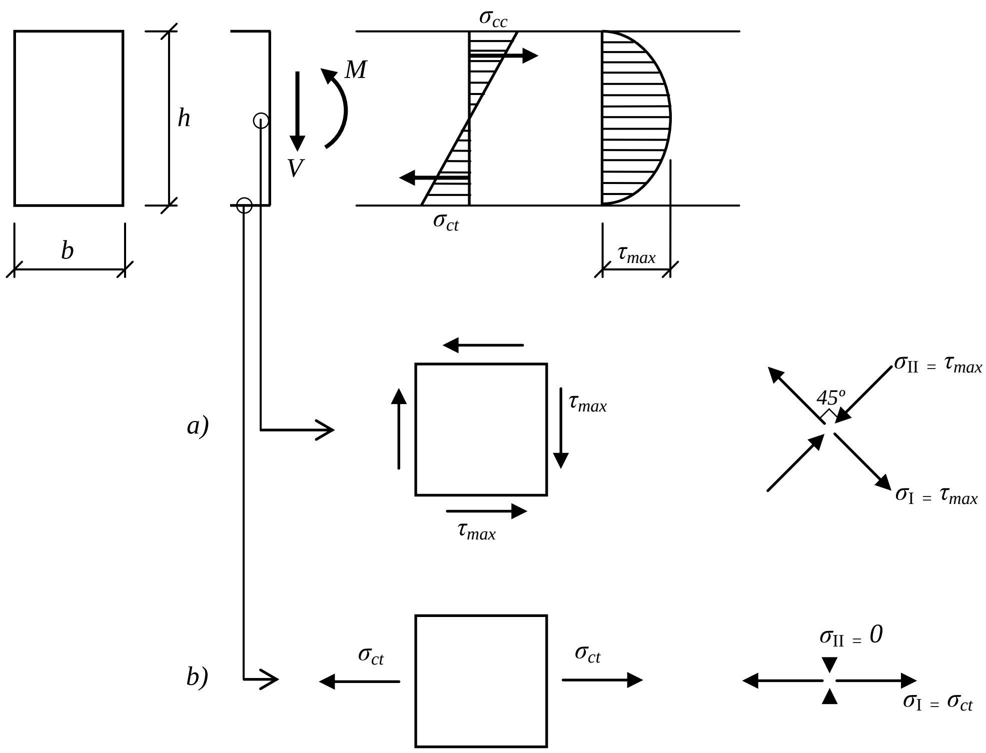


Figure 3.25 Stresses of an arbitrary cross-section, a) at the sectional centroid and b) at the bottom edge. σ_{I} are principal stresses in tension and σ_{II} are in compression.

The variation of principal tensile stresses at different sections along the beam will due to the variation of shear and normal stresses be as shown in Figure 3.26.

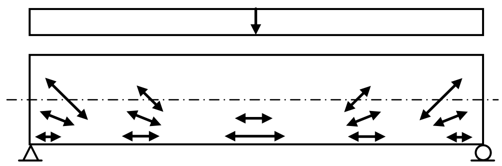


Figure 3.26 Distribution of principal tensile stresses along a simply supported beam subjected to a uniformly distributed load.

This will result in a crack pattern along the beam as shown in Figure 3.27. The cracks originate from the positions where the principal tensile stresses are highest, i.e. at the sectional centroid of beam end sections and at the bottom of the middle part of the beam. Three different types of cracks can be distinguished and are named after their origin and type of load effect, i.e. bending or shear, which caused them. The three crack types are flexural cracks, flexural shear cracks and web shear cracks. All of them are pointed out in Figure 3.27.

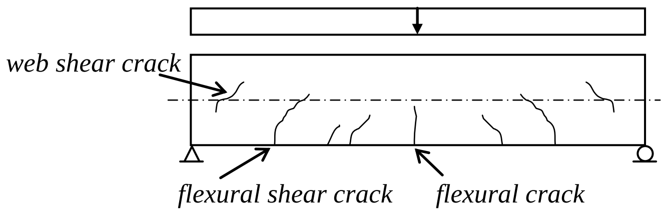


Figure 3.27 Different types of cracks in a beam subjected to bending and shear by a uniformly distributed load.

After cracking the tensile stresses can no longer be resisted by the cracked concrete areas and the load acting on the structure is instead carried by compressed concrete between the cracks and in the top part of the beam. In order to maintain force equilibrium and to transfer the load acting on the beam to the end supports the forces must be resisted across the cracks. This is enabled by friction and shear key effects of the rough crack surfaces and by transverse shear reinforcement at suitable spacing along the beam and longitudinal flexural reinforcement in the bottom of the beam. The load transfer along a beam can therefore be modelled by a truss model as shown in Figure 3.28.

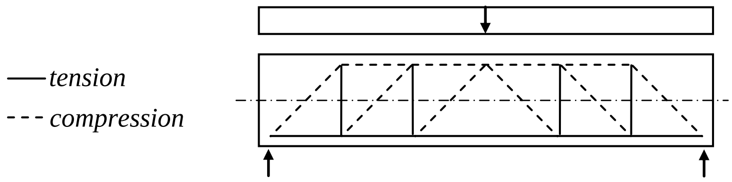


Figure 3.28 Truss model of a cracked reinforced concrete beam subjected to a uniformly distributed load.

From this figure it can be understood that the shear force is mainly resisted by inclined compressive concrete struts balanced by vertical reinforcement ties. The longitudinal struts and ties are mainly taking the compressive and tensile forces caused by bending. The longitudinal ties are however also necessary in order to maintain equilibrium of the inclined struts. How this is modelled is more in detail described in Chapter 4 and 5.

If the structural member at hand is subjected to a load that is displaced from the centre of the beam, the loading result in a torsional moment distribution along the beam, see Figure 3.29.

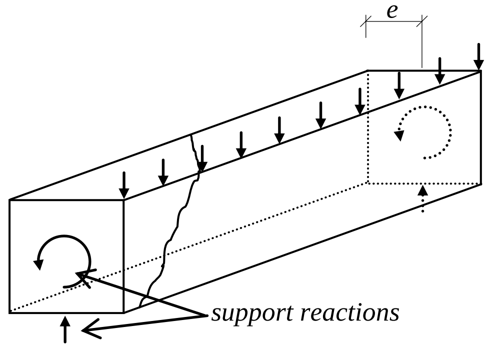


Figure 3.29 Due to an eccentric load torsion can occur in a structural member.

As illustrated in Figure 3.29 torsion in a concrete beam causes inclined cracks that develop around the whole beam, not only on the vertical sides as for shear. Hence, the response in torsion is more of a three dimensional behaviour than centric loading that result in shear and bending moment. This is further shown in Figure 3.30 illustrating how torsion is resisted in a cracked reinforced member by inclined struts and transverse and longitudinal ties.

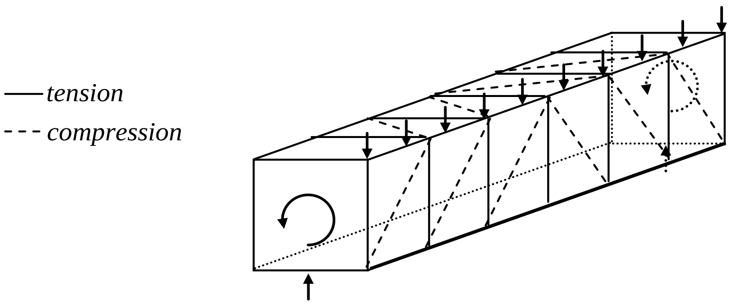


Figure 3.30 Truss model of a cracked reinforced concrete beam subjected to torsion.

It should be noted that the response to vertical shear in reality also is a three dimensional phenomenon but more often it is modelled in two dimensions as shown in Figure 3.31.

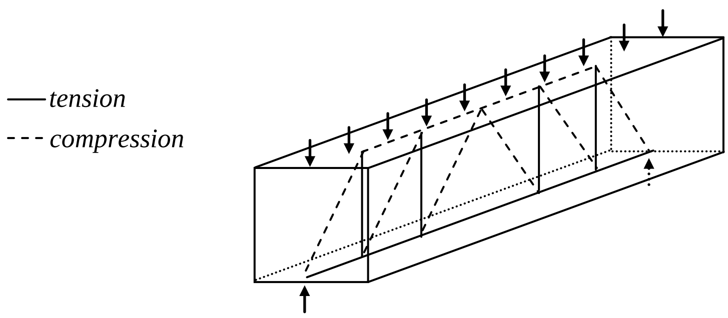


Figure 3.31 Simplified 2D-truss model of a beam subjected to a uniformly distributed centric load.

It can be noted that torsion can occur in two different ways depending on different restraint and loading conditions. The two different types of torsion can be distinguished as, Betongföreningen (2010a):

- Equilibrium torsion:
  The torsional moment is necessary for the equilibrium of the structure. For instance, a single span beam with fixed ends must resist the torsional moment arising from an eccentric load in order not to collapse, see Figure 3.32a.
- Compatibility torsion:
  The torsional moment arises from the restraint of rotation induced by adjoining members. For instance, a secondary beam fixed between two main beams creates torsion in the main beams when it deforms, see Figure 3.32b. In this case torsional cracks in the main beams reduce their torsional stiffness and consequently the torsional moment decreases. This is typically for compatibility torsion.

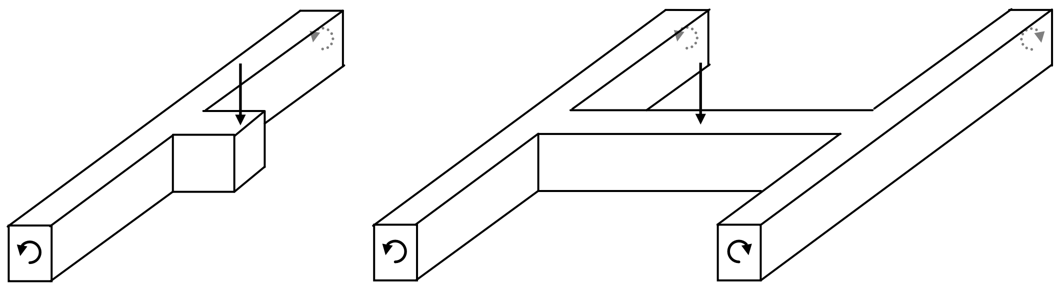


Figure 3.32 Examples of a) equilibrium torsion and, b) compatibility torsion. This figure is based on Svensk Byggtjänst (1990).

More about design for torsion can be found in Chapter 6.

3.4 Design based on different analysis approaches

3.4.1 General

When solving sectional forces for a structure that is statically determinate, e.g. a simply supported beam, this can be performed by means of equilibrium conditions only. On the other hand the sectional moment in a cross-section can be resisted in different ways, since the internal lever arm will be different if the material has elastic or plastic response or by altering the reinforcement amount and the sectional depth of the beam. Hence, even in a statically determinate member, the sectional analysis is a statically indeterminate problem.

When solving the sectional forces in for instance a continuous beam this will be statically indeterminate. The response of statically indeterminate structures normally varies with increasing load because of cracking of concrete, yielding of steel and other non-linear material response. In this case the sectional forces can be solved by means of equilibrium conditions combined with compatibility conditions and constitutive relations on both regional (sectional) and global (structural) levels.

A constitutive relation on local level can be described by means of Hook´s law, see Equation (3.4), where the stress-strain relation is expressed, Engström (2011b). The compatibility condition may express how the concrete strain is related to the steel strain in the most compressed fibre. When combining these two conditions it will result in a constitutive condition on the regional level where for instance the bending moment-curvature relationship of a cross-section is expressed, or the bending moment- average curvature relationship of a segment.

$$\sigma = E \cdot \varepsilon \tag{3.4}$$

σ	stress in the material
ε	strain of the material
E	modulus of elasticity

When the constitutive relationships for the cross-section are achieved, these are combined with the compatibility and equilibrium conditions on the global level, Engström (2011b). The compatibility condition describes for instance how the curvature matches the global deformation with regard to boundary and continuity conditions. The final step is to combine the conditions on global level in order to solve the statically unknowns and thereafter find the moment distribution by equilibrium conditions. Here it is shown that the statically unknowns are related to the response of the structure both on local and global level.

To solve the stress distribution the conditions used can be summarised as:

- Equilibrium condition – describes how the sectional forces are related to the support reactions and load.
- Compatibility condition – describes how the sectional response is related to the global response, for instance between the sectional curvature and support rotation.
- Constitutive condition – describes how the sectional force is related to the sectional response, for instance between moment and curvature.

For some structures, such as for instance a deep beam, or for some parts of a structure denoted as discontinuity regions, the stress field can be classified as statically indeterminate, Engström (2011c). This means that the solution of the statically indeterminate sectional problem cannot be solved by assuming plain sections remain plain and a simple compatibility condition therefore is missing. Design and modelling of discontinuity regions will be further described in Section 3.5

The aim of the design process is to achieve a structural member with sufficient dimensions of the cross-section and proper detailing of the reinforcement in order to fulfil the requirements in the ultimate limit state as well as the serviceability limit state. The design process often starts with design according to requirements in the ultimate limit state. Thereafter the performance in the serviceability limit state is checked and, if required, the design adjusted.

According to theory of plasticity any stress distribution is valid for statically indeterminate members as long as it fulfils equilibrium conditions after plastic redistribution. However, for beams and slabs subjected to bending this requires a certain rotational capacity, which needs to be taken into account and checked.

The analysis approaches possible to use for design in the ultimate limit state are:

- Linear elastic analysis
- Linear elastic analysis with limited redistribution
- Plastic analysis
- Non-linear analysis

The analysis approaches possible to use for design in the service limit state are:

- Linear elastic analysis (approximate)
- Non-linear analysis

The different types of analysis will be described in Sections 3.4.2 to 3.4.5 where the different approaches are exemplified for a concrete member subjected to bending, e.g. a continuous beam, by means of moment-curvature relationships.

3.4.2 Linear elastic analysis

Linear elastic analysis is often carried out in a simplified way and used in the preliminary design, when little information still is available, Engström (2011b). It is valid for design in the uncracked state and in ultimate limit state after plastic redistribution. However, the analysis can also be used to approximately check the behaviour in the service state when the structural member is known.

The moment distribution determined is unique and independent of the load, which means that the proportion of the moment diagram is the same regardless of the magnitude of the load, Engström (2011b). The assumed moment-curvature relationship shows linear elastic response, see Figure 3.33. Needed information for performing linear elastic analysis is the stiffness distribution, where the flexural rigidity often is assumed to be constant. The reinforcement is ignored and the gross concrete section is used, when the analysis is performed in a simplified way. However, a transformed concrete section where the reinforcement is considered can be used for instance when calculating the cracking load.

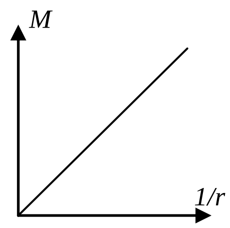


Figure 3.33 Assumed constitutive relation in linear elastic analysis of structural members subjected to bending.

It is assumed that linear elastic analysis only requires small plastic rotation before the moment distribution assumed in the design for the ultimate limit state is reached, Engström (2011b). However, plastic redistribution cannot be avoided.

3.4.3 Linear elastic analysis with limited redistribution

Linear elastic analysis approach with limited redistribution is a simple analysis used in design where the assumed moment-curvature relationship shows plastic response in some part and plastic significant rotation is chosen and taken into account, see Figure 3.34, Engström (2011b). In a first step linear elastic analysis is performed in the same manner as described in Section 5.3.2, but in a second step this moment distribution is redistributed with an amount chosen by the designer. This has to be performed without violating the equilibrium conditions and in relation to the solution achieved in the linear elastic analysis.

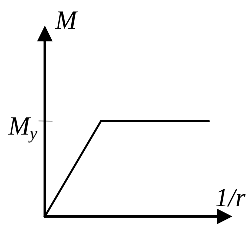


Figure 3.34 Assumed constitutive relation in linear elastic analysis with limited redistribution of structural members subjected to bending.

Needed information for performing linear elastic analysis with limited redistribution is the stiffness distribution. Since significant plastic redistribution is assumed, the calculated force distribution is only valid in the ultimate limit state, Engström (2011b). The model considers the moment redistribution due to yielding in the ultimate state, as it has been chosen by the designer. However, sufficient ductility is assumed to be available, which has to be checked.

3.4.4 Plastic analysis

Plastic analysis is based on theory of plasticity and can be used for preliminary and simplified design, Engström (2011b). Two different approaches can be distinguished: static method (lower bound), where a stress-field in equilibrium is chosen, and kinematic method (upper bound), where a failure mechanism is chosen. Lower bound solutions are preferable since they give a solution on the safe side. The assumed moment-curvature relationship shows ideally plastic response, see Figure 3.35, where each region reaches its yield capacity. Needed information for performing plastic analysis is moment ratios between critical sections in supports and spans or load dividers, i.e. sections where the shear force is zero. Since significant plastic redistribution is assumed the method is only valid in ultimate limit state.

In plastic analysis the need for plastic deformations can be large. Hence verification of sufficient rotational capacity should be performed, Engström (2011b).

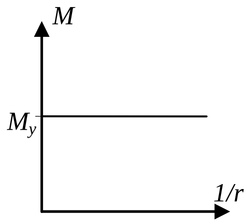


Figure 3.35 Assumed constitutive relation in plastic analysis of structural members subjected to bending.

Strut and tie models are used to simulate stress fields in cracked reinforced concrete in the ultimate limit state, after considerable plastic redistribution and is also based on plastic analysis. This is further described in Section 3.5.

3.4.5 Non-linear analysis

Non-linear analysis cannot be used in preliminary design, since the needed information is not available, Engström (2011b). Instead it can be used as a verification of the simplified approaches, the ones described above. It takes into account the non-linear response of reinforced concrete structures. It is the only analysis that considers the moment redistribution both in the service state, due to cracking, and in the ultimate state, due to plastic rotations. The assumed moment-curvature relationship shows non-linear response after cracking, see Figure 3.36. All information concerning material properties, reinforcement arrangement etc. is required for performing non-linear analysis and the calculated response is valid in both service and ultimate states.

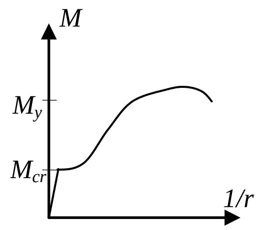


Figure 3.36 Assumed constitutive relation in non-linear elastic analysis of structural members subjected to bending.

A simplified way of performing non-linear analysis in the service state is by step-wise changing the flexural rigidity between the uncracked and cracked state. In both stages a constant rigidity is calculated by transforming the reinforcing steel into a an equivalent concrete area, Engström (2011b). By using FE-methods more advanced non-linear analysis can be performed.

3.5 Strut and tie method

Generally, structural members can be divided into two types of regions: regions where the strain distribution is linear called B-, continuity- or Bernoulli-regions, and regions where the strain distribution is non-linear called D-, Discontinuity- or Disturbed-regions, Engström (2011c). A D-region is where the stress field is disturbed due to changes in geometry (geometric discontinuities) or due to influence of concentrated loads (static discontinuities). Examples of D-regions are shown in Figure 3.37.

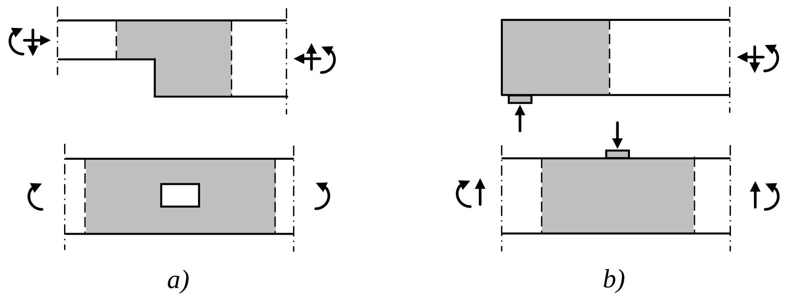


Figure 3.37 Examples of discontinuity regions, a) geometric discontinuities, b) static discontinuities. Adopted from Engström (2011c).

The effect of a discontinuity within a D-region will disperse across the section and the risk for cracking is present, when transverse tensile stresses appear, Engström (2011c). To keep equilibrium after cracking and avoid uncontrolled propagation of crack in these regions proper reinforcement solution need to be designed and provided. Despite the equal importance of B- and D-regions, there is no well-defined theory available for designing D-regions. Instead rules-of-thumb or empirical equations have been used, Nagarajan and Pillai (2008). However, the strut and tie method is today accepted to be an effective tool for design of both B- and D-regions. This method is found in many codes, such as Eurocode, American code, Canadian code, Australian code, New Zeeland code etc.

The strut and tie method is a way to simulate a stress field in cracked reinforced concrete in the ultimate limit state by the use of struts for compression and ties for tension. The method assumes that the chosen stress field can develop by redistribution of stresses and the method is therefore based on theory of plasticity. An appropriate stress field can be chosen by the use of a linear elastic FE-analysis or the load-path method that is a step-by-step method providing guidelines for how to draw the resultants of the stress field in order to obtain a design and detailing that entail good behaviour also in the service state. Figure 3.38 shows how the stress field of a deep beam that can be illustrated by load-paths using the load-path method. Each load-path corresponds to the center-line of the corresponding stress field. As shown by the arrows in Figure 3.38 transverse forces are needed in order to change direction of the load paths.

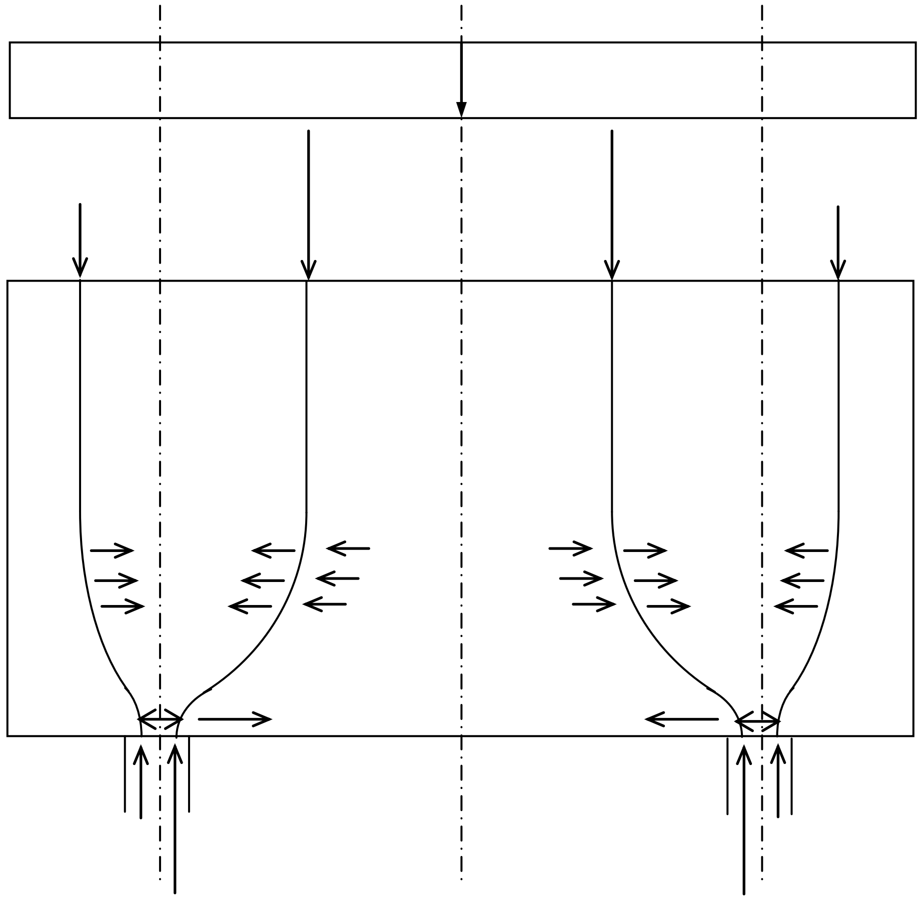


Figure 3.38 Stress field in a deep beam described by the load-path method.

To obtain a strut and tie model from the drawn stress field in Figure 3.38 nodes are inserted at positions where the stress field change direction. Struts and ties are after that inserted between the nodes representing compression and tension. In Figure 3.39 below, a strut-and-tie model based on the stress field in Figure 3.38 is shown.

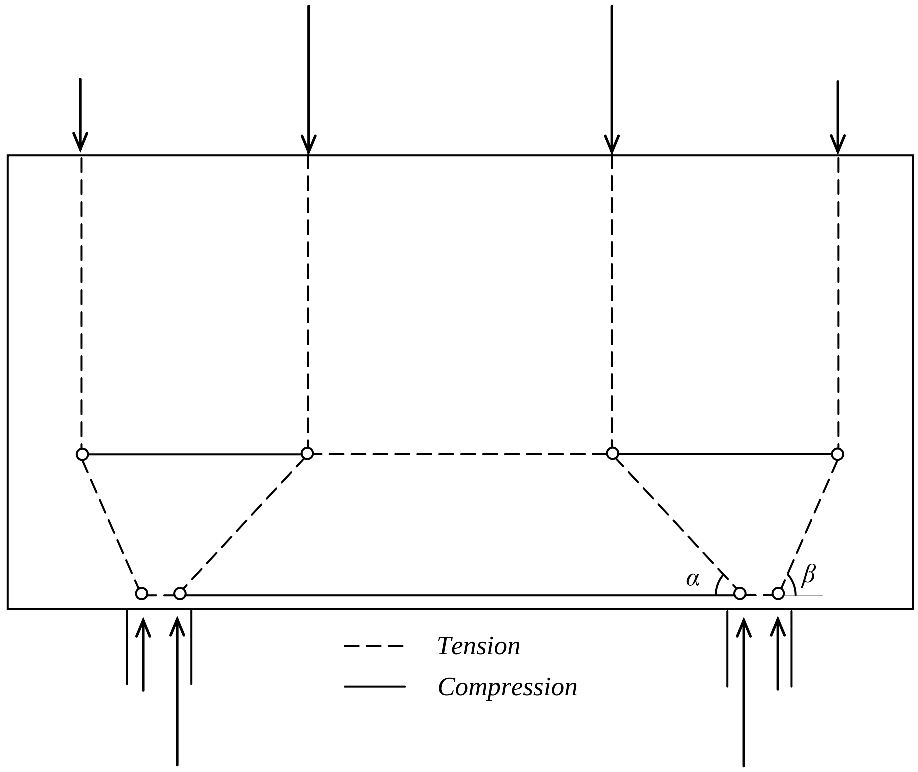


Figure 3.39 Strut and tie model of the deep beam in Figure 3.38.

In order to provide satisfactory behaviour in both ultimate state and service state there are guidelines for how to establish the strut and tie model. Angles between struts and ties should be held within certain ranges depending on the type of node. Depending on the stress field the reinforcement realising the ties should be concentrated to a certain area or, the opposite, be distributed over a large part of the structure. The forces in the struts and ties are calculated and the amount of reinforcement necessary is determined. The stresses within concentrated struts in nodal areas are checked to ensure sufficient capacity with regard to biaxial conditions. The procedure for the strut and tie method as well as the load path method is described more in detail in Engström (2011c) and Betongföreningen (2010).

Figure 3.40 shows a simplified reinforcement sketch that is established from the strut and tie model in Figure 3.38. Even if the load path method and the strut and tie method provide detailed rules in order to obtain a good design, it is still important to bear in mind that the detailing should reflect the actual stress field. It is normally not enough to place the reinforcement between the nodal areas. The reinforcement should cover the entire tensile stress field and be sufficiently anchored.

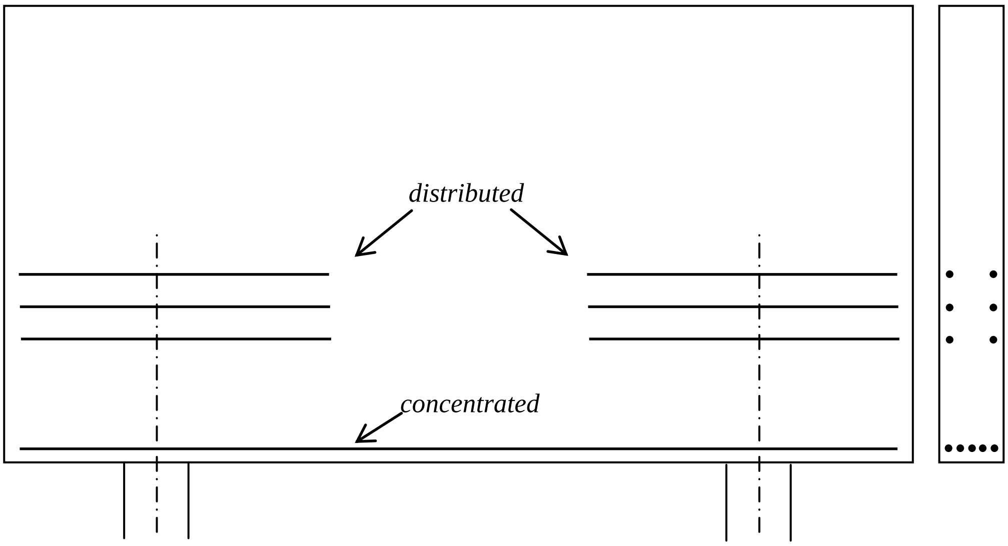


Figure 3.40 Reinforcement sketch of the beam in Figure 3.38.

It should be noted that strut and tie models can be used also for B-regions. In this report this is referred to as a truss model.

4 Design and detailing for bending

4.1 Structural response and modelling

A statically indeterminate structure subjected to bending should be designed to have a ductile response in the ultimate state.

To provide for a ductile response there are a number of things that must be considered in design. The amount of reinforcement is of great importance; it should not be too small or too large. A brittle premature failure can occur, if the moment capacity ensured by tensile reinforcement is low in comparison to the moment taken by the uncracked concrete section, Engström (2010). If the capacity of the steel is too low to catch the force that previously was taken by the tensile zone of the uncracked concrete, the reinforcement will rupture suddenly when the concrete cracks. Besides keeping the amount of reinforcement at a reasonable level for economic reasons, a heavily reinforced structure fails in a brittle way in the ultimate limit state. In order to prevent this type of failure it is of importance to make sure that yielding of reinforcement is reached before the compressed concrete is crushed Betongföreningen (2010a).

A way to describe the desired behaviour of a reinforced concrete section subjected to bending is by means of the relation between bending moment, *M*, and curvature, *1/r*, see Figure 4.1, Engström (2010).

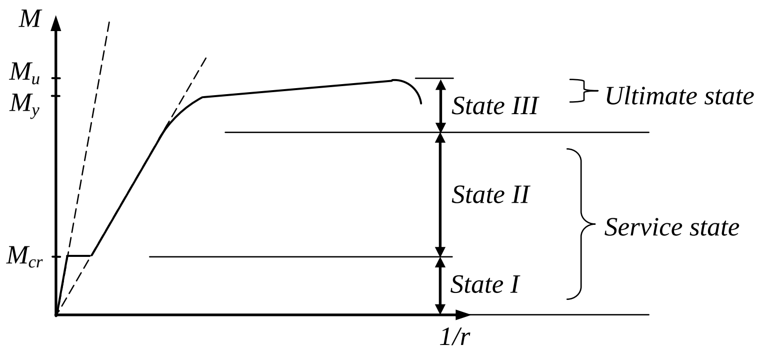


Figure 4.1 Desired response of a reinforced concrete section subjected to bending.

As shown in Figure 4.1 the moment-curvature relationship can be divided in three states:

- State I: Uncracked state.
  The reinforcement has very small influence and the response of the structure is linear elastic and highly dependent of the flexural rigidity of the uncracked concrete section. State I lasts until the cracking moment, M_{cr}, is reached, i.e. when the concrete cracks

- State II: Cracked state
  The concrete has cracked and the behaviour is now dependent on both concrete and reinforcement. Both materials show linear elastic response. In the service state sections of members subjected to bending are designed to be in State I or state II or intermediate.

- State III: Non-linear state
  One or both of the materials have significant non-linear behaviour. For a cracked concrete section the resisted bending moment mainly depends on the reinforcement capacity and its lever arm. When the reinforcement has reached its yielding point, at a moment M_y, the ultimate state starts and lasts until failure, M_u.

The response in bending can for all of these three stages be modelled at a sectional level, by means of local models for material responses, i.e. relations between stress, σ, and strain, ε. The models are illustrated in Figure 4.2 to Figure 4.4.

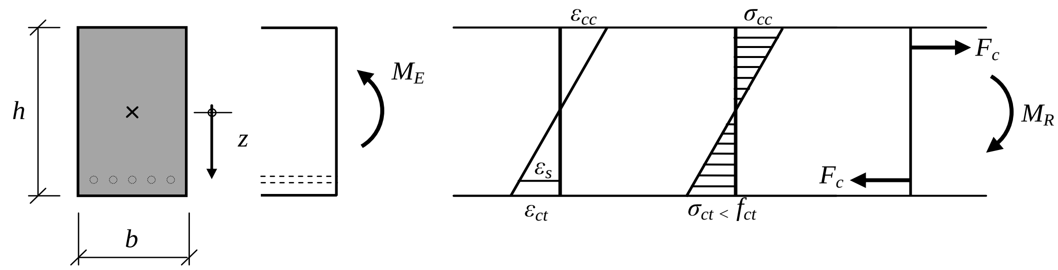


Figure 4.2 Sectional model for bending moment in state I, uncracked state.

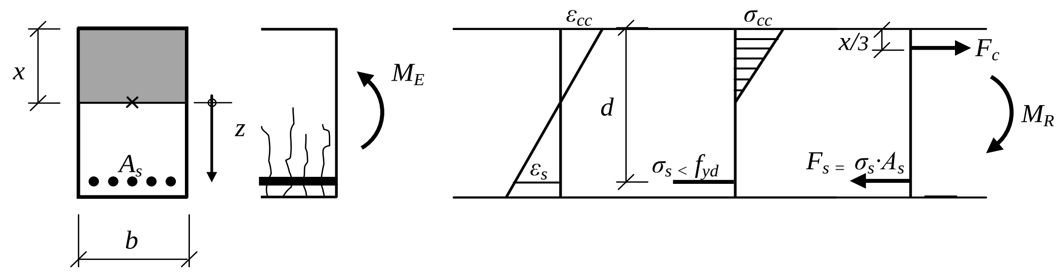


Figure 4.3 Sectional model for bending moment in state II, cracked state.

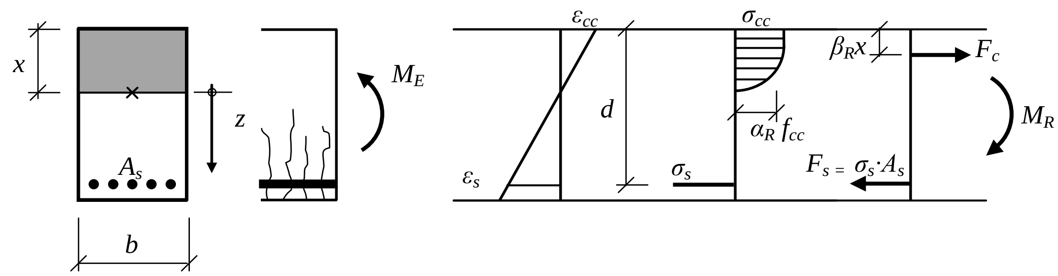


Figure 4.4 Sectional model for bending moment in state III.

From these figures it is clear that the bending moment is resisted by a force couple, one force in tension and one in compression, as presented in Section 3.3.2.

These models are often used to find relations between stresses, strain and bending moment. Such relations can be derived by using the geometry as well as the equilibrium conditions, deformation conditions and constitutive relations given in Figure 4.2 to Figure 4.4 above. The constitutive relations are the same as the simplified stress-strain relations describing the materials' mechanical properties in

Figure 3.4a and Figure 3.5b. The coefficients, the so called stress block factors, α_R and β_R, as can be seen in Figure 4.4, are used to define the size and centroid of the stress block for a rectangular cross-section. Expressions for these factors can be found in Engström (2011b) and are based on expressions for the simplified stress-strain relationship for concrete in Eurocode 2, see Figure 3.4a. For other cross-sectional shapes the stress block factors need to be recalculated.

It should also be noted that some concrete still in tension below the neutral axis, x, in cracked cross-sections in reality can contribute to the resistance. However, according to the definitions of the model in state II and III this effect is ignored, which is on the safe side.

Another way to find relations between stresses and moments in state I and II is by using Navier's formula, see Equation (4.1).

$$\sigma_c(z) = \frac{M}{I_{trans}} z \tag{4.1}$$

$\sigma_c(z)$	concrete stress at level z
M	bending moment
I_{trans}	second moment of area of transformed concrete cross-section in state I or state II
z	distance from centroid of transformed concrete cross-section to the level of interest

In order for Navier's formula to be applicable the material responses must be linear elastic and the reinforcement steel must be transformed into an equivalent concrete area taking into account the higher stiffness of steel in relation to concrete. This is performed by multiplying the reinforcement area with a factor α

$$\alpha = \frac{E_s}{E_c} \tag{4.2}$$

where E_s and E_c are the moduli of elasticity of reinforcing steel and concrete respectively. Note that for state I the reinforcement is often assumed to have a minor effect on the response and is therefore often neglected.

The steel stress in the reinforcement can be derived by using constitutive relations together with the assumption that the steel strain is equal to the strain in the concrete at the level of the reinforcement, i.e. the deformation criterion. Hence, the stress can be derived as

$$\sigma_s = E_s \varepsilon_s = E_s \varepsilon_c(z_s) = E_s \frac{\sigma_c(z_s)}{E_c} = \alpha \frac{M}{I_{trans}} z_s \tag{4.3}$$

E_s	modulus of elasticity of reinforcing steel
ε_s	steel strain
$\varepsilon_c(z_s)$	concrete strain at the level of the steel, z_s
E_c	modulus of elasticity of concrete

α ratio between E_s and E_c, see Equation (4.2)

z_s level of reinforcing steel in relation to centroid of transformed concrete cross-section

When designing reinforcement in a reinforced concrete section the goal is to achieve a moment-curvature relation as is schematically shown in Figure 4.1. As initially described in Section 4.1 it is important to keep the amount of reinforcement within certain limits to ensure the desired type of behaviour. If the contribution from the reinforcement is too low in relation to the moment resistance of the uncracked concrete section the behaviour of the section will be as in Figure 4.5a. Figure 4.5b on the other hand illustrates the consequences from having too much reinforcement in a reinforced concrete section resulting in a brittle failure, since the reinforcement does not reach yielding.

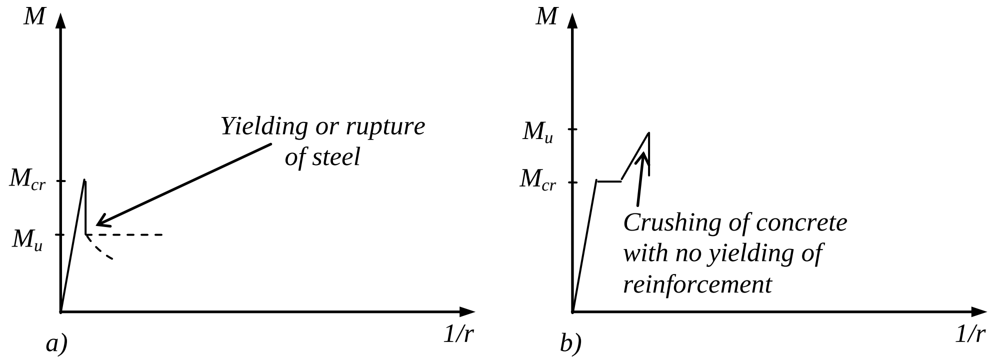


Figure 4.5 Brittle response of a reinforced concrete section with different amounts of reinforcement, a) too little reinforcement, b) too much reinforcement.

4.2 Minimum longitudinal reinforcement

4.2.1 Requirements in Eurocode 2

The rules and guidelines concerning both minimum and maximum reinforcement in reinforced concrete sections are primary stated in Section EC2 9.2, which considers detailing of beams. However, the rules and expressions are in some cases applicable to other types of structural members and Eurocode 2 then refers to the equations in Section EC2 9.2.

Section EC2 9.2.1.1 provides rules and guidelines concerning maximum and minimum longitudinal reinforcement. The lower limit of the amount of reinforcement in beams is set to avoid brittle failure. Expression EC2 (9.1N) for minimum reinforcement is here given in Equation (4.4).

$$A_{s,\min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d \qquad \geq 0.0013\ b_t d \tag{4.4}$$

f_{ctm} mean value of axial tensile strength of concrete

f_{yk} characteristic yield strength of longitudinal reinforcement

b_t mean width of the part of the cross-section in tension *d* effective depth of the cross-section

Derivation of the expression used for calculation of minimum longitudinal reinforcement can be found in Section 4.2.2 and is based on Johansson (2012a).

4.2.2 Explanation and derivation

By ensuring that the moment capacity in the ultimate limit state, M_{Rd}, calculated in state III, is greater than the moment resistance of an uncracked section, M_{cr}, calculated in state I, a brittle failure can be avoided. Figure 4.6 and Figure 4.7 show how the respective resisting resistance moments can be estimated for a rectangular cross-section.

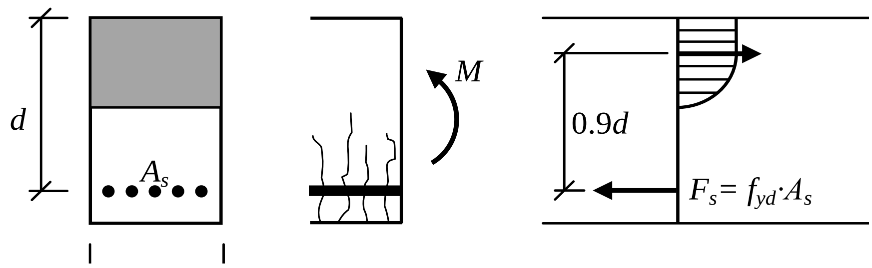


Figure 4.6 Moment capacity of a cross-section in the ultimate limit state.

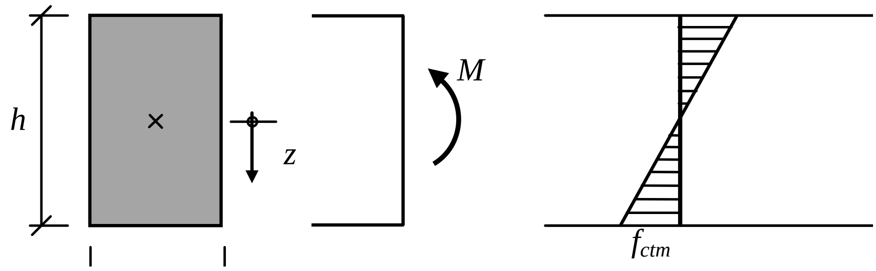


Figure 4.7 Moment resistance of an uncracked cross-section.

Equation (4.4) can be derived by the inequality in expression (4.5)

$$M_{Rd} \geq M_{cr} \tag{4.5}$$

The resisting moments are estimated using moment equilibrium, Equation (4.6), and Navier's formula, Equation (4.7).

$$M_{Rd} \approx f_{yd} \cdot A_s \cdot 0.9d \tag{4.6}$$

$$M_{cr} \approx f_{ctm} \cdot \frac{I_c}{z_{\max}} = f_{ctm} \cdot \frac{bh^3}{12} \cdot \frac{2}{h} = f_{ctm} \cdot \frac{bh^2}{6} \tag{4.7}$$

From this Expression (4.5) can be rewritten as

$$f_{yd} \cdot A_s \cdot 0.9d \geq f_{ctm} \cdot \frac{bh^2}{6} \tag{4.8}$$

The design yield strength of reinforcement, f_{yd}, is defined in Equation (4.9) as a function of the characteristic yield strength of reinforcing steel, f_{yk}.

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{f_{yk}}{1.15} \tag{4.9}$$

γ_s partial factor for reinforcing steel

The reinforcement can be assumed to be placed with an effective depth, d, approximated to 90 % of the height, h, of the concrete cross-section resulting in

$$h = \frac{d}{0.9} \tag{4.10}$$

The inequality in Equation (4.8) can be further developed by inserting Equations (4.9) and (4.10), which gives

$$\frac{f_{yk}}{1.15} \cdot A_s \cdot 0.9d \geq f_{ctm} \cdot \frac{b\left(\frac{d}{0.9}\right)^2}{6} \tag{4.11}$$

Finally, an expression for the reinforcement amount can be expressed as

$$A_s \geq \frac{f_{ctm}}{f_{yk}} \cdot \frac{1.15 \cdot bd}{6 \cdot 0.9^3} = 0.26 \frac{f_{ctm}}{f_{yk}} bd \tag{4.12}$$

which corresponds to the minimum reinforcement requirement given in Eurocode 2.

4.2.3 Discussion

It should be noted that Expression EC2 (9.1N) is based on a rectangular cross-section of width b. For other types of cross-sections Eurocode 2 recommends to use the mean width and for T-beams, in particular, to use the width of the web instead. This is according to Betongföreningen (2010a) an approximation on the safe side, but to get a more accurate amount of minimum reinforcement the actual shape of the cross-section should be considered by detailed calculations.

It should also be observed that the internal lever arm is taken as the estimated value 0.9d. Further, the effective depth, d, is also an estimation to 90 % of the sectional height. This means that the expression in Eurocode 2 in general gives very approximate values and more detailed calculations, using the actual shape of the cross-section, will give a more accurate amount of the needed minimum reinforcement. Consequently, the requirement can accurately and more generally be expressed by the inequality in Equation (4.5).

In Svensk Byggtjänst (1990) the demand for avoiding a brittle failure is expressed in this manner. It is stated that it should be shown that the cracking moment, M_{cr}, is well below the moment capacity, M_{Rd}, for an arbitrary cross-section. This criterion is formulated as

$$M_{Rd} \geq 1.1\gamma_n M_{cr} \tag{4.13}$$

γ_n partial factor considering the safety class

Another way to derive Expression EC2 (9.1N) is found in Hendy and Smith (2010). The same inequality is set up as in Equation (4.5), but the capacity in the ultimate limit state is set to the characteristic yield strength of reinforcing steel and the internal lever arm is instead set to 0.8*d*. This is according to Hendy and Smith (2010) not a likely scenario for rectangular cross-sections, but can occur for other cross-sections. The coefficient in the final expression will in this case be 0.25. The value is thereafter rounded up to 0.26 in order to allow for further cross-sectional shapes. This suggests that the expression for minimum reinforcement in Eurocode 2 is applicable for other types of cross-sections. However, it should be used considering the effect of different shapes of cross-sections.

As can be seen in Equation (4.4) there is a lower limit of the amount of minimum reinforcement area set to $0.0013b_t d$. The logic behind this expression has not been found, but similarities between the expression used in Eurocode 2 and the minimum reinforcement requirement according to the American code, ACI 318-05, ACI (2007), have been identified, see Equation (4.14) below.

$$A_{s,\min} = 0.224\frac{\sqrt{f_c}}{f_y}b_w d \qquad \geq \frac{1.4}{f_y}b_w d \tag{4.14}$$

f_y yield strength of reinforcement specified in ACI (2007)

$\sqrt{f_c}$ square root of compressive strength of concrete specified in ACI (2007)

b_w web width, or diameter of circular section

Until 1995 only the lower limit of this expression was used as a minimum reinforcement requirement. It was then recognised that the amount of steel given from this expression may not be sufficient for high strength concrete with compressive strength over 35 MPa, Subramanian (2010), and the first expression in Equation (4.14) was introduced. It is possible that the European Standard has been developed in a similar way. Numerical investigations performed by Ozbolt and Bruckner (1998) show that the amount of minimum reinforcement is dependent on the brittleness of concrete. A strong, and hence more brittle, concrete releases a larger amount of energy when cracking, which must be taken by the reinforcement. The lower limit of minimum reinforcement given in Eurocode 2 corresponds to a concrete strength class close to C25/30 for reinforcement yield strength of 500 MPa. This means that the minimum amount of reinforcement is dependent on the strength of concrete for all concrete strength classes stronger than C25/30.

It can be noticed that according to both experimental tests and numerical analyses by Ozbolt and Bruckner (1998) the minimum amount of reinforcement should also depend on the size of the beam, concrete to steel bond relationship and type and amount of distributed reinforcement. Small beams show ductile response even with low amount of minimum reinforcement. Larger beams will on the other hand, due to the size effect, exhibit a more brittle behaviour resulting in a need for a larger amount and more distributed minimum reinforcement. As can be seen in Subramanian (2010), and is also stated in Ozbolt and Bruckner (1998), the minimum reinforcement requirements are independent of the beam depth in almost all design codes. Further studies within this subject must be carried out to be able to refine the expressions more.

4.3 Maximum longitudinal reinforcement

4.3.1 Requirements in Eurocode 2

According to Eurocode 2, Paragraph 9.2.1.1(3), the maximum amount of longitudinal reinforcement for reinforced concrete sections in beams can be found in the National Annex. The recommended value according to Eurocode 2 is

$$A_{s,\max} = 0.04 A_c \tag{4.15}$$

A_c cross-sectional area of concrete

However, in Sweden no upper limit is given of the amount of reinforcement that can be placed in a cross-section. The reason why the recommended value in Eurocode 2 is disregarded in Sweden is, according to Betongföreningen (2010a), because other criteria are considered to better determine the maximum amount of reinforcement.

As mentioned in Section 4.1 the amount of reinforcement in a reinforced concrete section should be limited in order to provide for a ductile response in the ultimate state. In Eurocode 2 ductility is considered by the ratio x_u / d. Criteria related to ductility are found in Paragraph EC2 5.6.3(2), concerning plastic rotational capacity of statically indeterminate members designed according to plastic analysis, and are expressed as

$$\frac{x_u}{d} \leq 0.45 \qquad \text{for concrete strength class} \leq \text{C50/60} \tag{4.16}$$

$$\frac{x_u}{d} \leq 0.35 \qquad \text{for concrete strength class} \geq \text{C55/67} \tag{4.17}$$

x_u depth of compression zone in the ultimate limit state

d effective depth

Depending on what type of analysis used in design different criteria concerning ductility should be fulfilled.

According to Eurocode 2 Section EC2 5.6.2(2) there is no need to check the rotational capacity for members designed according to plastic analysis, if the reinforcement is in ductility class B or C and the following conditions are fulfilled

$$\frac{x_u}{d} \leq 0.25 \qquad \text{for concrete strength class} \leq \text{C50/60} \tag{4.18}$$

$$\frac{x_u}{d} \leq 0.15 \qquad \text{for concrete strength class} \geq \text{C55/67} \tag{4.19}$$

For design according to linear elastic analysis with limited redistribution intermediate demands are expressed in Eurocode 2.

Explanation for the recommended maximum reinforcement requirement in Equation (4.15) has not been found. However, derivation of the ductility requirements in Equations (4.16) to (4.19) is presented in Section 4.3.2 and are based on Johansson (2012a).

4.3.2 Explanation and derivation

The rules presented in Equations (4.16) and (4.17) are simplified ways of checking the ductility of a cross-section and can be derived by the relationship between concrete and steel strains seen in Figure 4.8

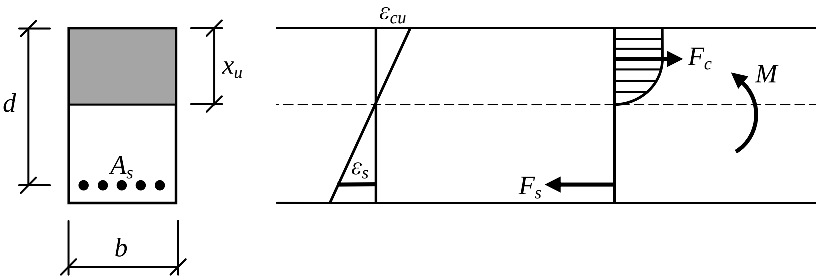


Figure 4.8 Moment capacity of a cross-section in the ultimate limit state.

To make sure that the steel is yielding before the concrete reaches its critical strain the steel strain should be equal to or larger than the yield strain, ε_{sy}, when the concrete has reached its ultimate strain, ε_{cu}. Expression (4.20) and (4.21) shows how the requirement for concrete strength classes equal to or below C50/60, see Equation (4.16), fulfils this condition. For these concrete strength classes the ultimate strain is ε_{cu} = 3.5 ‰. It should be noted that the calculated steel strain is compared to the yield strain of reinforcing steel ε_{sy} = 2.17 ‰, corresponding to a characteristic yield strength of 500 MPa in the ultimate limit state.

$$\varepsilon_s = \frac{d - x_u}{x_u}\varepsilon_{cu} \tag{4.20}$$

$$\frac{x_u}{d} = 0.45 \tag{4.21}$$

$$\left.\begin{matrix} (4.20) \\ (4.21) \end{matrix}\right\} \quad \varepsilon_s = \frac{1-0.45}{0.45}0.0035 = 4.2‰ \qquad > \varepsilon_{sy} = 2.17‰$$

ε_s steel strain

The other ductility requirements in Eurocode 2 can be derived in the same manner. The criteria in Equations (4.16), (4.17), (4.18) and (4.19) can be written as steel strain, ε_s, in relation to the ultimate concrete strain, ε_{cu}, see Table 4.1. Derivations can be found in Appendix C.

Table 4.1 Steel strain corresponding to different ductility requirements.

$\frac{x_u}{d} \leq$	0.45	0.35	0.25	0.15
ε_s [‰]	$1.36\varepsilon_{cu}$	$1.86\varepsilon_{cu}$	$3.00\varepsilon_{cu}$	$5.67\varepsilon_{cu}$

For concrete strength classes equal to or below C50/60 the ultimate concrete strain, ε_{cu}, is constant according to Eurocode 2 but for strength classes higher than C50/60 the ultimate concrete strain decreases, see Table 4.2 and Table 4.3. The steel strain, ε_s, for each criterion must therefore be calculated for each concrete strength class in order to relate it to the yield strain of reinforcement, ε_{sy}. In Table 4.2 and Table 4.3 the requirements in Equations (4.16), (4.17), (4.18) and (4.19) are stated as corresponding steel strains ε_s, see also Appendix C.

Table 4.2 Steel strain corresponding to the ductility requirements $x_u/d=0.45$ and $x_u/d=0.35$.

f_{ck} [MPa]	12-50	55	60	70	80	90
ε_{cu} [‰]	3.5	3.1	2.9	2.7	2.6	2.6
$\frac{x_u}{d}$	0.45	0.35				
ε_s [‰]	4.3	5.8	5.4	5.0	4.8	4.8

Table 4.3 Steel strain corresponding to the ductility requirements $x_u/d=0.25$ and $x_u/d=0.15$.

f_{ck} [MPa]	12-50	55	60	70	80	90
ε_{cu} [‰]	3.5	3.1	2.9	2.7	2.6	2.6
$\frac{x_u}{d}$	0.25	0.15				
ε_s [‰]	10.5	17.6	16.4	15.3	14.7	14.7

4.3.3 Discussion

When designing a member in the ultimate limit state according to linear elastic analysis the solution is valid due to theory of plasticity. It is generally assumed that a linear elastic solution does not require significant plastic deformations that must be checked by direct calculations. It is sufficient to use the simplified rules in Equations (4.16) and (4.17) to make sure that the ductility of the member is adequate, Engström (2011b). However, no requirement of ductility is presented in Section EC2 5.4 concerning linear elastic analysis. It can be argued that Paragraph EC2 5.6.3(2), providing the rules in Equations (4.16) and (4.17), is wrongly placed under Section EC2 5.6 about plastic analysis and should be moved to the section regarding linear elastic analysis instead, Engström (2013). Ductility is always needed for statically indeterminate structures, since reinforced concrete structures show a non-linear response under loading.

Another motivation for this argument is that Model Code 78, CEB-FIP (1978), and Model Code 90, CEB-FIP (1991), state that a solution based on linear analysis in the ultimate limit state cannot always satisfy the conditions of compatibility and should therefore be checked to provide for sufficient plastic rotation to prevent rupture before the designed capacity is attained in every section of the structure. The ductility of the structure can be assumed to be enough, if the criterion in Equation (4.22) or (4.23) is fulfilled.

$$\frac{x_u}{d} \leq 0.45 \qquad \text{for concrete grades C12 to C35} \tag{4.22}$$

$$\frac{x_u}{d} \leq 0.35 \qquad \text{for concrete grades greater than C40} \tag{4.23}$$

If looking in the Prestandard ENV 1992-1-1, CEN(1991), to Eurocode 2 it is also noteworthy that requirements similar to Equations (4.16) and (4.17) are found in ENV Section 2.5.3.4.2, 'Linear analysis with or without redistribution'. The requirements are

$$\frac{x_u}{d} \leq 0.45 \qquad \text{for concrete grades C12/15 to C35/45} \tag{4.24}$$

$$\frac{x_u}{d} \leq 0.35 \qquad \text{for concrete grades greater than C40/45} \tag{4.25}$$

Design according to linear elastic analysis with limited redistribution is also valid in the ultimate limit state due to theory of plasticity, provided that the assumed redistribution of moments is sufficiently small in relation to available ductility. This type of design normally requires larger plastic rotations resulting in higher demands than for a linear elastic solution. The ductility needed for a solution obtained from plastic analysis might require even larger plastic rotations.

The criteria for plastic analysis, see Equations (4.18) and (4.19), require that the reinforcement in the tensile zone should be able to deform even more before the concrete reaches its ultimate strain and are therefore much stricter than the ones that should be used for linear elastic analysis, Equations (4.16) and (4.17). This can be seen in Table 4.1 to Table 4.3. A plastic solution can be chosen more freely and might require larger plastic redistribution before the design load is reached.

It can be mentioned that Table 4.1 also shows that the ductility criteria for higher concrete strength classes are stricter than the ones for ordinary concrete grades. This is because high strength concrete is more brittle than concrete of lower strengths, Johansson (2013).

In Table 4.1 to Table 4.3 it is shown that all simplified ductility requirements result in a steel strain at failure well above the yield strain of reinforcing steel with characteristic yield strength f_{yk} = 500 MPa. The increased brittleness of high strength concrete, i.e. decreasing values of the ultimate strain, ε_{cu}, result in a decrease of the required steel strain when the concrete reaches its ultimate strain. However, the required steel strain for the strongest concrete, C90/105, and hence the most brittle type, is still larger than the steel strain at failure required for concrete strength classes below C50/60.

It has not been possible to find the reason why Eurocode 2 provides a maximum reinforcement amount expressed as $A_{s,max} = 0.04A_c$. To get an understanding of what this criterion means in relation to the simplified ductility requirements it is convenient to express the recommended amount of maximum reinforcement by means of a reinforcement ratio

$$\rho = \frac{A_s}{A_c} = \frac{A_s}{bd} \tag{4.26}$$

The recommended criterion in Equation (4.15) can for a rectangular cross-section be written as

$$\rho_{\max} = 4\% \tag{4.27}$$

This value can be compared to the reinforcement ratio, ρ, corresponding to $x_u / d = 0.45$ for a rectangular cross-section without compression reinforcement, presented in Table 4.4. The corresponding calculations are presented in Appendix C.

Table 4.4 Reinforcement ratio corresponding to the simplified ductility requirements $x_u / d = 0.45$ for different concrete types.

f_{ck} [MPa]	12	16	20	25	30	35	40	45	50
$\frac{x_u}{d}$	0.45								
$\rho = \frac{A_s}{A_c}$ [%]	0.7	0.9	1.1	1.4	1.7	2.0	2.3	2.5	2.8

It can be noticed that none of the calculated reinforcement ratios, ρ, exceeds 4 %. This indicates that if the criterion in Equation (4.16) is fulfilled, and hence, the recommended value given in Paragraph EC2 9.2.1.1(3) of $A_{s,max} = 0.04A_c$ is automatically satisfied, if compression reinforcement is not added. However, if compression reinforcement is added to the section, the height of the compressive zone, x_u, will decrease, and thus, also the ratio x_u / d. This means that it is possible to choose the amount of reinforcement to for instance 4 % and after that add compression reinforcement until the ductility requirement is fulfilled. Hence, there is no immediate dependency between maximum reinforcement amount and the ductility requirements. This implies that the upper limit of longitudinal reinforcement that is recommended in Eurocode 2 is given for other reasons than to provide sufficient ductility.

As mentioned in Section 4.3.1, the recommended maximum reinforcement amount provided in Eurocode 2 is disregarded in Sweden since other criteria are considered to better determine the maximum amount of reinforcement. In Section 4.3.2 this was shown by pointing out the influence of the ductility requirements on the maximum amount of reinforcement. However, it should be noted that there is no reference between Paragraphs EC2 9.2.1.1(3) and EC2 5.6.3(2); instead it is up to the designer to know that the ductility of critical sections should be checked. It is also important to emphasise that since there are no ductility requirements in the section concerning linear elastic analysis in Eurocode 2, i.e. Section EC2 5.4, it can be argued that a real maximum limit of the amount of longitudinal reinforcement is missing in Sweden. However, a general rule is to ensure that the reinforcement is yielding in the ultimate limit state in order to utilise its full capacity, Johansson (2013), and it can perhaps be argued that this together with economical aspects will limit the amount of reinforcement sufficiently. However, the authors of this master's thesis do believe that the ductility requirement in Paragraph EC2 5.6.3(2) is misplaced in Eurocode 2 and should apply also to linear elastic analysis in Section EC2 5.4.

4.4 Reinforcement detailing for concrete frame corners

4.4.1 Requirements in Eurocode 2

Eurocode 2, Annex J.2.3 gives recommendation concerning detailing of concrete frame corners subjected to opening moment. In Figure 4.9 frame corners subjected to moderate opening moment are shown, which also can be found in Figure EC2 J.3.

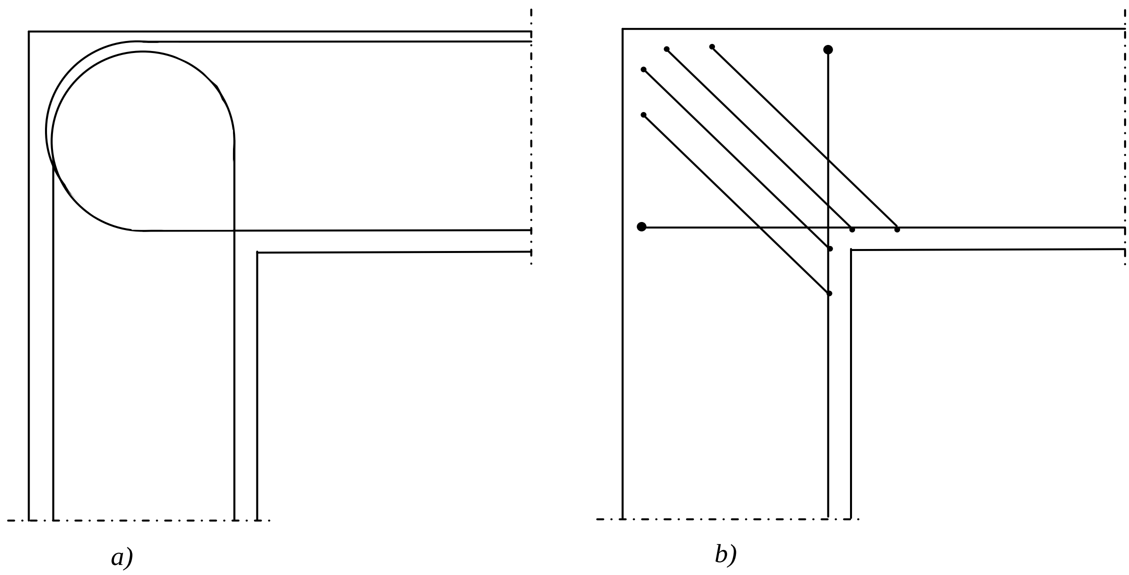


Figure 4.9 Frame corner with moderate opening moment. a) and b) show alternative arrangement of detailing reinforcement. The figure is based on SIS (2008).

The reinforcement amount that is recommended in Eurocode 2 is expressed according to Equation (4.28)

$$\rho = \frac{A_s}{bd} \leq 2\% \tag{4.28}$$

A_s	area of the reinforcement
b	width of the concrete cross-section
d	effective depth of the cross-section.

In Figure EC2 J.4 recommendation regarding detailing of concrete frame corners subjected to large opening moment are illustrated, which also are repeated in Figure 4.10.

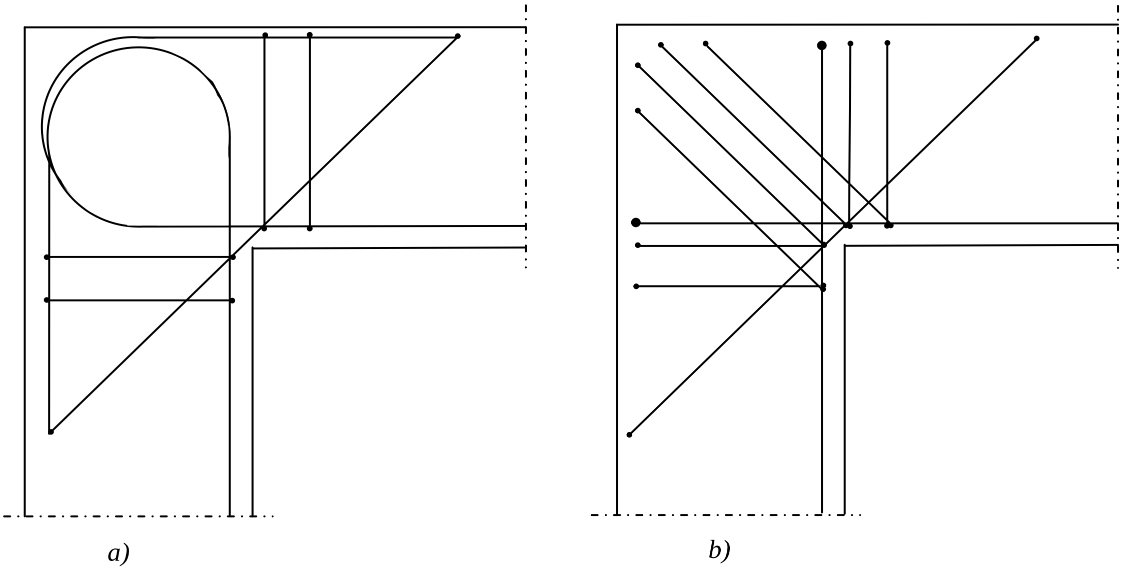


Figure 4.10 Frame corner with large opening moment. a) and b) show alternative arrangement of reinforcement. The figure is based on SIS (2008).

The reinforcement amount that is recommended in Eurocode 2 is expressed according to Equation (4.29)

$$\rho = \frac{A_s}{bd} > 2\% \tag{4.29}$$

According to Eurocode 2, Annex J.2.2, the reinforcement arrangement for a concrete frame corner subjected to a closing moment should be performed according to Figure 4.11 and Figure 4.12. In the former a concrete frame corner subjected to closing moment and with almost equal depths of the beam and column is shown. In the latter a concrete frame corner subjected to closing moment with very different depths of the beam and column is shown.

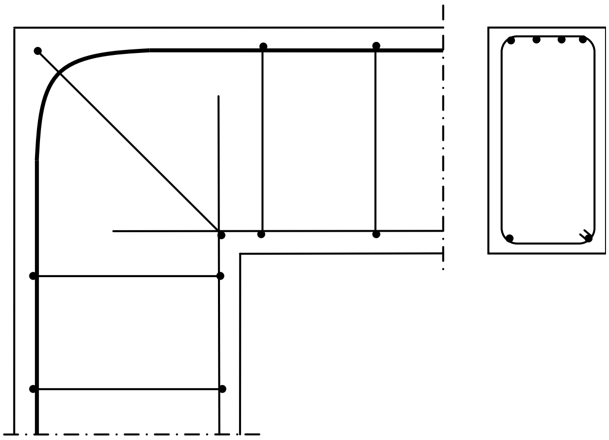


Figure 4.11 Frame corner with closing moment and with almost equal depths of the beam and the column. The figure is based on SIS (2008).

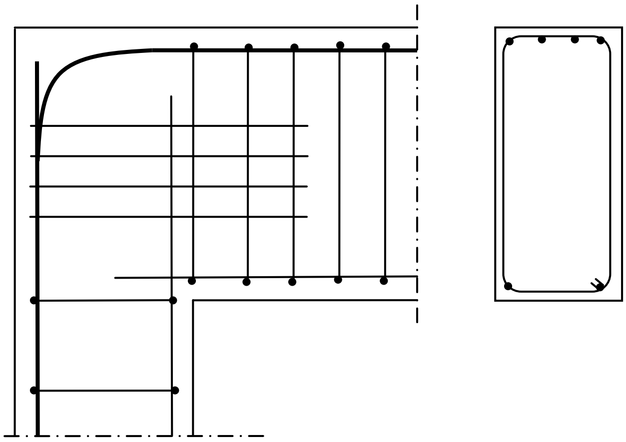


Figure 4.12 Frame corner with closing moments and with very different depths of beam and column. The figure is based on SIS (2008).

4.4.2 Explanation and derivation

One type of structural region that requires special consideration when subjected to bending is concrete frame corners. The structural response of such a region is a bit more complicated than what is described for beams in Section 4.2 and 4.3 and has therefore been explicitly investigated. A concrete frame corner can be considered as a discontinuity region, where the stress field within the concrete is statically indeterminate. Due to this it is difficult to design and arrange the reinforcement appropriately in order to achieve the desired behaviour.

Concerning concrete frame corners there are two separate principal types, the ones subjected to positive or negative moment, see Figure 4.13, Johansson (2000). The difference in structural behaviour can be described as follows:

- in case of an opening (positive) moment the forces tend to split the corner in two by pushing off the outside of the concrete portion
- in case of a closing (negative) moment the inside concrete portion is confined due to the interaction of the tensile and compressive forces

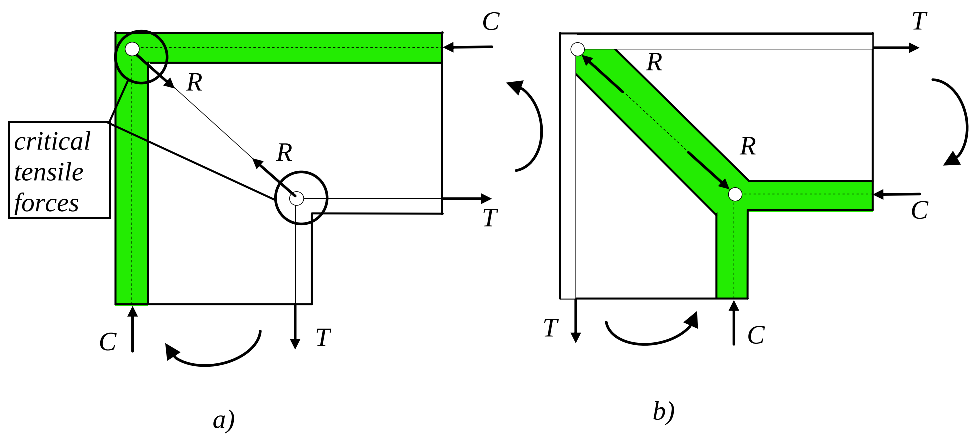


Figure 4.13 Different types of frame corners, a) corner subjected to positive (opening) moment with principal forces and b) negative (closing) moment with principal forces. The figure is based on Johansson (2000).

The response and capacity of corners with opening moment are greatly influenced by cracking due to the diagonal tensile force, *R*, see Figure 4.13a, Johansson (2000). For corners with closing moments the diagonal compressive force, *R*, will create biaxial compressive stresses that will be beneficial for the corner region, see Figure 4.13b. Hence, the concrete frame corners subjected to an opening moment are significantly more difficult to perform in design than those that are subjected to a closing moment and may in the former case potentially present poor performance and a rather brittle behaviour at failure.

In the case of opening moment it is difficult to achieve a reinforcement detailing where the internal forces are balanced after concrete cracking has occurred, Johansson (2000). This is not the case for a closing moment, where it is not as complicated to achieve a force pattern that is in equilibrium after cracking. One of the main purposes with the detailing of frame corners is to control the cracks and delay their propagation. The other main purpose of the detailing is to ensure that the structural region exhibits ductile behaviour and allows redistribution of forces. By making the structure capable of large plastic deformations this behaviour is obtained. One may think that an increased moment capacity, and thereby increased reinforcement amount, will improve the structure. However, this is not always the case, since a too large reinforcement amount might cause a brittle failure.

The moment capacity of the corner should preferably be at least as large as the moment capacity of its adjoining members, Johansson (2000). The corner efficiency describes how well the corner fulfils this requirement and can be determined as follows

$$\textit{corner efficiency} = \frac{\textit{moment capacity of frame corner obtained in tests}}{\textit{theoretical moment capacity of adjoining members}}$$

The moment capacity of the frame corner is determined by the product of the maximum applied load and the lever arm between it and the critical section, see Figure 4.14, Johansson (2000). The critical section is where the expected crack will occur for each case, which is the section between the corner and its adjoining member. Note that the critical section is defined a bit differently for opening and closing moment.

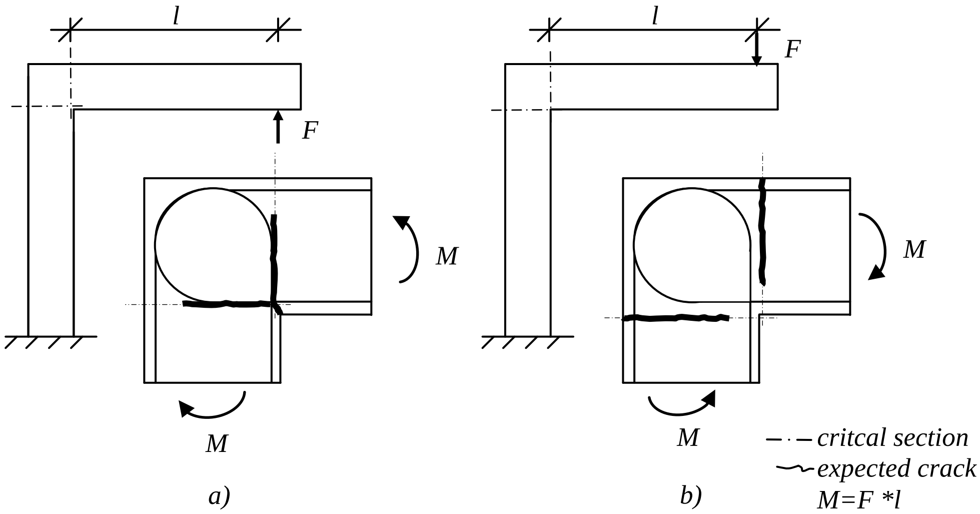


Figure 4.14 Testing of frame corners, a) concrete frame corner subjected to an opening moment, b) concrete frame corner subjected to a closing moment. The thick lines in a) and b) show where the expected critical cracks occur. The figure is based on Johansson (2000).

The corner efficiency is something that applies to analysis and test situations, not in case of design. The corner efficiency is therefore not stated in Eurocode 2. However, in the research program carried out by Johansson (2000) a number of tests were summarised from other research performed on detailing of concrete frame corners similar to the ones recommended in Annex EC2 J.2. Four different types of detailing of concrete frame corners have been included in this masters' thesis taken from Johansson (2000), see Figure 4.15. This research also acts as a base for many arguments that are presented in this chapter. In Section 4.4.3 the different detail solutions are evaluated. To learn more about the four detail solutions in Figure 4.15, see Appendix D.

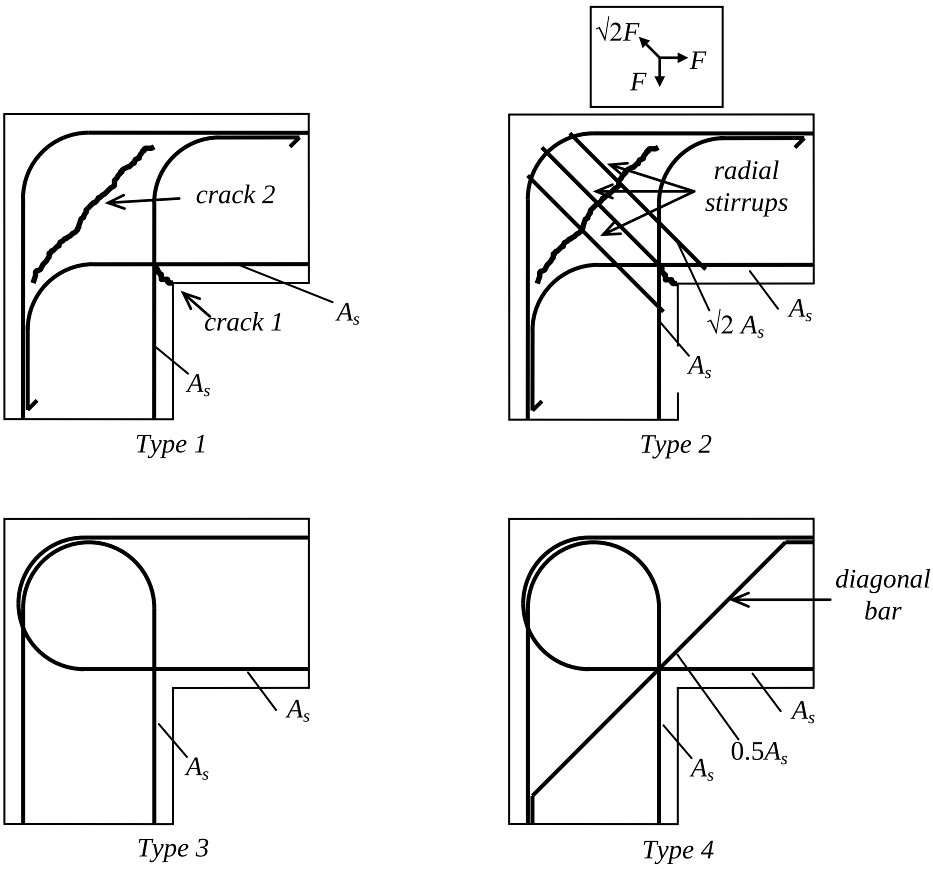


Figure 4.15 The four different types of reinforcement configurations investigated by Johansson (2000). Note that crack 2 will not propagate exactly as the figure shows, i.e. it is only a schematic illustration and indicates how the tensile forces are acting in the region.

In Figure 4.15, for Type 1, crack 1 illustrates where the highest risk for cracking is in a concrete frame corner subjected to opening moment, Johansson (2000). Crack 2 shows where there is a potential risk for cracking in case of opening moment. The first crack is expected to form at the inside of the corner and thereafter a second inclined crack might occur. If the diagonal tensile force, see Figure 4.13a, is not resisted by a correctly designed reinforcement, the outside of the concrete corner will be pushed off.

If the two adjacent members are of equal sizes, the maximum reinforcement amount that is always possible to be used, while still reaching yielding in the reinforcement, can be determined according to Equation (4.30), Johansson (2000). This reinforcement amount, A_s, concerns the reinforcement configuration in detail solution Type 1 in Figure 4.15. The equation will result in that the concrete tensile strength is decisive and that the expected corner efficiency can be reached if, A_s is small. For the derivation of the critical reinforcement amount see Appendix D.

$$\rho = \frac{0.45 f_{ct}}{f_y} \tag{4.30}$$

Although it can be assumed that frame corners subjected to closing moment have larger capacity than their adjoining members, due to the larger lever arm within the corner, see Figure 4.14b, this might not always be the case, Johansson (2000). It has been shown in tests that concrete frame corners with closing moment in some cases fail before yielding of the reinforcement and therefore result in a capacity lower than expected. Three different reasons for this have been identified by Stroband and Kolpa (1983), where spalling of the side concrete cover is of special interest, since it is the most critical failure mode. Crushing failure will occur for concrete frame corners subjected to closing moment if the compressive strength of the region is insufficient and for large values of the mechanical reinforcement ratio, ω_s. In this case it is risk of crushing of the diagonal compressive strut within the corner or of the concrete at the inside of the corner, Johansson (2000).

$$\omega_s = \frac{A_s f_y}{bdf_c} = \rho \frac{f_y}{f_c} \tag{4.31}$$

f_c	cylinder compressive strength
f_y	yield strength of the reinforcement
A_s	area of the reinforcement
b	width of the concrete cross-section
d	effective depth of the cross-section

Spalling of the side concrete cover is a risk for both opening and closing moments, Johansson (2000). Due to this it is, for a corner subjected to closing moment, problems with reaching the assumed load capacity before failure has occurred. However, other reasons will probably be decisive for failure in case of concrete frame corners subjected to opening moment and spalling of the side concrete cover will therefore not be the primary design cause.

In Figure 4.16 the result from tests on concrete frame corners performed by a number of researchers is summarised by Johansson (2000). The frame corners of Types 1-4, see Figure 4.15, were all subjected to opening moments. In the tests no extra moment capacity in the adjoining members, due to the diagonal reinforcement in Type 4, was accounted for when the corner capacity was calculated[1]. The mechanical reinforcement ratio, ω_s, used varied between 0.38-3.02 %.The comparison demonstrates the inefficiency of Types 1 and 2 where the former is the least effective detail solution. Types 3 and 4 work better and where there is a significant advantage using Type 4. For a complete compilation of the test results see Johansson (2000). When comparing Type 2 and Type 3 it can be shown that the former is less effective. However, full capacity is not obtained for either of them.

In Figure 4.17 the efficiency of frame corners Type 1 and Type 3 is compared for corners subjected to closing moment, Johansson (2000). The two detailing solutions are comparable and seem to give similar moment capacity. Both Type 1 and Type 3 seem to be two possible solutions for concrete frame corners subjected to closing moment.

[1] This was a decision made by Johansson (2000) in order to better explain the background to the conclusions drawn by Nilsson and others.

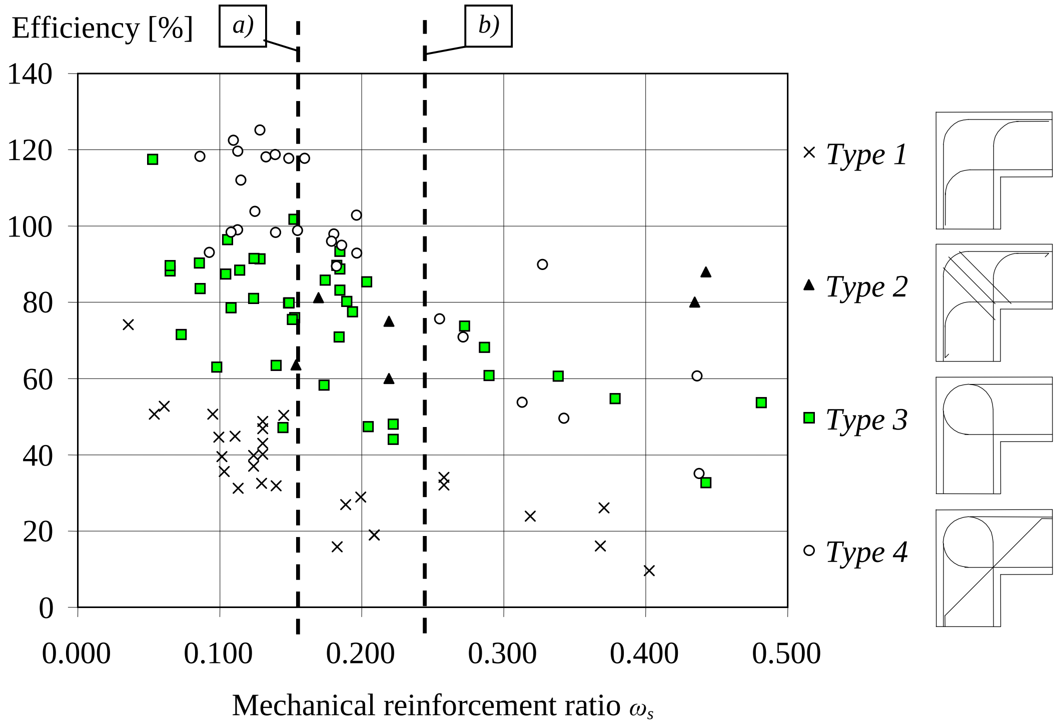


Figure 4.16 *Efficiency of different reinforcement arrangements in frame corners subjecting to opening moment. Line a) shows $\omega_{s.max}$ for sufficient efficiency according to the authors and line b) shows what $\omega_{s.max}$ would have been if using ρ=2 %. The figure is taken from Johansson (2000).*

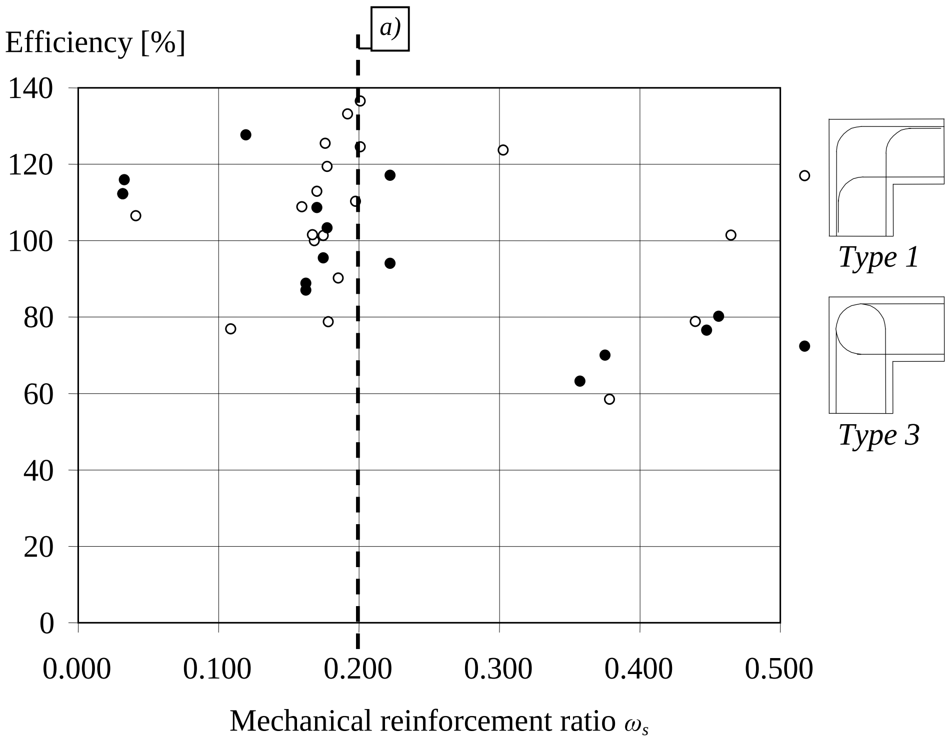


Figure 4.17 *Efficiency of different reinforcement arrangements in frame corners subjected to closing moment. Line a) shows $\omega_{s.max}$ for sufficient efficiency according to Johansson (2000). The figure is taken from Johansson (2000).*

4.4.3 Discussion

The recommended solutions of frame corners subjected to opening moment in Eurocode 2, presented in Figure 4.9 and Figure 4.10, all have reinforcement that crosses crack 1 in Figure 4.15. However, it is a bit more complicated to ensure an appropriate capacity with regard to crack 2.

The solution of Type 1 was shown to be a poor detail when the frame corner with opening moment is evaluated since crack 2 is left without control. This type of solution is not recommended in Eurocode 2, which seems reasonable. For the solution of Type 2, which can be compared to Figure 4.9b, radial stirrups are added to the design of Type 1 in order to control crack 2. According to the strut and tie model it can be expected to be a proper detail solution. However, the radial stirrups were shown to not contribute sufficiently to the moment capacity and the needed capacity was not reached. According to Johansson (2000) Type 2 is therefore not a good design and consequently the recommended solutions in Figure 4.9b and Figure 4.10b can be questioned. Solutions of Type 3 and Type 4 were both shown to be much better designs. By using 180º bent bars crack 2 was delayed. The solution of Type 4 was shown to be even better than Type 3, since the inclined bar also delayed crack 1. The solutions of Type 3 and Type 4 that showed best performance are also represented in Eurocode 2, see Figure 4.9a and Figure 4.10a.

The solutions in Figure 4.11 and Figure 4.12 are both possible solutions for concrete frame corners subjected to closing moment. In Johansson (2000) no tests of such designs were performed. However, solutions Type 1 and Type 3 in Figure 4.15 were tested with regard to closing moments. Both of these were shown to be good details, provided that a sufficient side concrete cover was applied.

The requirements regarding minimum diameter to which the bar is bent, see Section 9.6, and minimum concrete cover, see Section 9.5, should be followed in order to avoid brittle failure. This concerns concrete frame corners subjected to opening or closing moment. However, as was described in Section 4.3.2, concrete frame corners subjected to closing moment are more sensitive with regard to spalling of the side concrete cover, i.e. a large concrete cover is important.

In the recommended solution presented it should be remembered that when using reinforcement loops and inclined bars, it is important to keep the splicing of the loops and straight bars just outside the corner region.

It is interesting that only one solution of a concrete frame corner subjected to an opening moment is presented in BBK 04, Boverket (2004). The recommended solution is the same as the one in Figure 4.10a which was the one that showed the best results according to the research in Johansson (2000). BBK 04 does not provide any recommended solution for frame corners subjected to closing moment. The reason for this may be that a corner subjected to a closing moment is much less sensitive, and thus less difficult to design, than frame corners subjected to an opening moment.

The European Standard gives a recommendation on a strut and tie model when the solutions shown in Figure 4.10 should be designed. However, it is up to the designer to solve the strut and tie model and no guidance is given regarding the amount of the diagonal reinforcement that should be provided to the solution. However, according to Karlsson (1999) the solution in Figure 4.10a is an accepted detailing in many countries. Karlsson (1999) recommends to provide diagonal bars with an amount of about one-half of the main reinforcement, A_s. Karlsson (1999) also states that to improve the design even further, radial stirrups, similar to the ones in Type 2, can be provided in addition to the loops, compare to Figure 4.10. However, whether these stirrups can be sufficiently anchored or not can be questioned with regard to the results shown in Johansson (2000).

According to Johansson (2013) the recommended limits of the reinforcement ratio, ρ, in Annex EC2 J.2.3, concerning detailing of the reinforcement layout in frames subjected to an opening moment, are not appropriate, see Figure 4.9 and Figure 4.10 and Equation (4.28) and Equation (4.29), respectively. The high limit of the reinforcement amount can result in that the desired response will not always occur. Hence the full corner efficiency will not always be reached and the failure might be brittle, see Figure D.3 in Appendix D. In order to investigate this further it is of interest to see the value of the critical reinforcement amount, ρ, see Equation (4.30), when using different strength values of concrete (f_{ctm}, f_{ctk} and f_{ctd}) and reinforcing steel (f_{ym}, f_{yk} and f_{yd}) in order to compare the result with the recommended limit of 2 % in Eurocode 2. This is shown in Equations (4.32)-(4.34), where the used concrete strength class is C40 and characteristic steel strength is 500 MPa.

$$\rho = \frac{0.45 f_{ctm}}{f_{ym}} = \frac{0.45 \cdot 3.5 \cdot 10^6}{500 \cdot 10^6 \cdot 1.10} = 0.29\ \% \tag{4.32}$$

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

f_{ym} is the mean value of the yield strength of reinforcing steel

$$\rho = \frac{0.45 f_{ctk}}{f_{yk}} = \frac{0.45 \cdot 2.5 \cdot 10^6}{500 \cdot 10^6} = 0.23\,\% \tag{4.33}$$

f_{ctk} is the characteristic axial tensile strength of concrete

f_{yk} is the characteristic yield strength of reinforcing steel

$$\rho = \frac{0.45\, {f_{ctk}}/{\gamma_C}}{{f_{sy}}/{\gamma_S}} = \frac{0.45 f_{ctd}}{f_{yd}} = \frac{0.45 \cdot 1.67 \cdot 10^6}{435 \cdot 10^6} = 0.17\ \% \tag{4.34}$$

f_{ctd} is the design axial tensile strength of concrete

f_{yd} is the design yield strength of reinforcing steel

If calculating the critical reinforcement amount according to BBK 04 the design axial tensile strength of concrete should also be divided by two and ρ is then equal to

$$\rho = \frac{0.17\ \%}{2} = 0.085\% \tag{4.35}$$

Equations (4.32)-(4.35) indicate that a limit of the reinforcement amount of about 0.2 % would be more adequate in order to avoid crack 2 in the corner and hence give a more effective designed concrete frame corner subjected to opening moment.

When comparing the corner efficiency of different solutions it is preferable to use the mechanical reinforcement ratio, ω_s, instead of the reinforcement ratio, ρ, since the concrete and steel strengths, which are of high importance, also are incorporated into the parameter, Johansson (2000). The corner efficiency for ω_s less than 0.160 is insufficient for opening moment according to the authors; see line a) in Figure 4.16. To confirm that the limit of the reinforcement ratio, ρ, of 2 % is too high, the corresponding mechanical reinforcement ratio, ω_s, is calculated in Equation (4.36) using this value. For this compilation a concrete strength of $f_{ck} = 40$ MPa and a reinforcement strength $f_{yk} = 500$ MPa are used.

$$\omega_s = \frac{f_{yk}}{f_{ck}}\rho = \frac{500\cdot 10^6}{40\cdot 10^6}\rho = 12.5\rho = 12.5\cdot 0.02 = 0.250 \tag{4.36}$$

If the design strength of the concrete and the reinforcement is used in the calculation it will result in an even higher mechanical reinforcement ratio, see Equation (4.37).

$$\omega_s = \frac{f_{yk}/\gamma_C}{f_{ck}/\gamma_S}\rho = \frac{f_{yd}}{f_{cd}}\rho = \frac{435\cdot 10^6}{27\cdot 10^6}\rho = 16.11\cdot 0.02 = 0.322 \tag{4.37}$$

When comparing to Figure 4.16 it can be seen that a value of $\omega_s = 0.250$ is too high. Hence, the frame corners will not always reach full corner efficiency and might develop a brittle failure. The reinforcement amounts, ρ, that are calculated in Equation (4.32)-(4.35) are based on the detail solution in Type 1 in Figure 4.15. In Equation (4.32)-(4.35) ρ varies between 0.085-0.29 %. This implies that it should be on the safe side to limit the recommendation for frame corners subjected to moderate opening moment in Figure 4.9a to $\rho < 0.2$ % instead, since Type 3 reaches higher corner efficiency than Type 1. However, it should be noted that this is a very conservative limit of the reinforcement amount since the derivation is based on a poorly designed reinforced concrete frame corner where it can be assumed that it is almost only the concrete that resist the load.

It has already been discussed that solution Type 2, that can be compared to Figure 4.9b, shows poor corner efficiency why the authors does not give any new recommendations for ρ in this case.

Eurocode 2 does not give any recommended limitation of ρ for frame corners subjected to large opening moment, see Figure 4.10. If the reinforcement amount is calculated for ω_s equal to 0.160,see line a) in Figure 4.16, ρ would be equal to about 1.2 %, see Equation (4.38). This is why the recommendation for the detail solution shown in Figure 4.10a should be changed to $\rho < 1.2$ % instead. No lower limit is determined, i.e. the authors thinks that the solution in Figure 4.10a should be allowed to be used for the same ρ recommended in Figure 4.9. It should be noted that no tests have been found regarding the solution in Figure 4.10b why the authors cannot give any recommendations for ρ in this case.

$$\rho = \omega_s \frac{f_{ck}}{f_{yk}} = 0.160\frac{40}{500} \approx 1.2\% \tag{4.38}$$

It should be noted that the mechanical reinforcement amount ω_s can be coupled with x / d, see Equation (4.39). In order to see the definition for α_R see Figure 4.4.

$$\frac{x}{d} = \frac{A_s f_{yk}}{\alpha_R f_{ck} bd} = \frac{1}{\alpha_R} \omega_s \tag{4.39}$$

x_u depth of neutral axis at the ultimate limit state, after redistribution

d effective depth

The corner efficiency of Type 4 is very high. However, these results are a bit misleading, since the theoretical moment capacity was calculated without consideration of the diagonal reinforcing bar. This was also pointed out by Johansson (2000), who performed a second comparison, where the diagonal reinforcing bar was considered in the theoretical moment capacity, see Figure 4.18. Figure 4.18 shows that the total moment capacity of Type 4 is lower than Type 3 and this indicates that the diagonal bar actually does not have any positive effect on the corner efficiency.

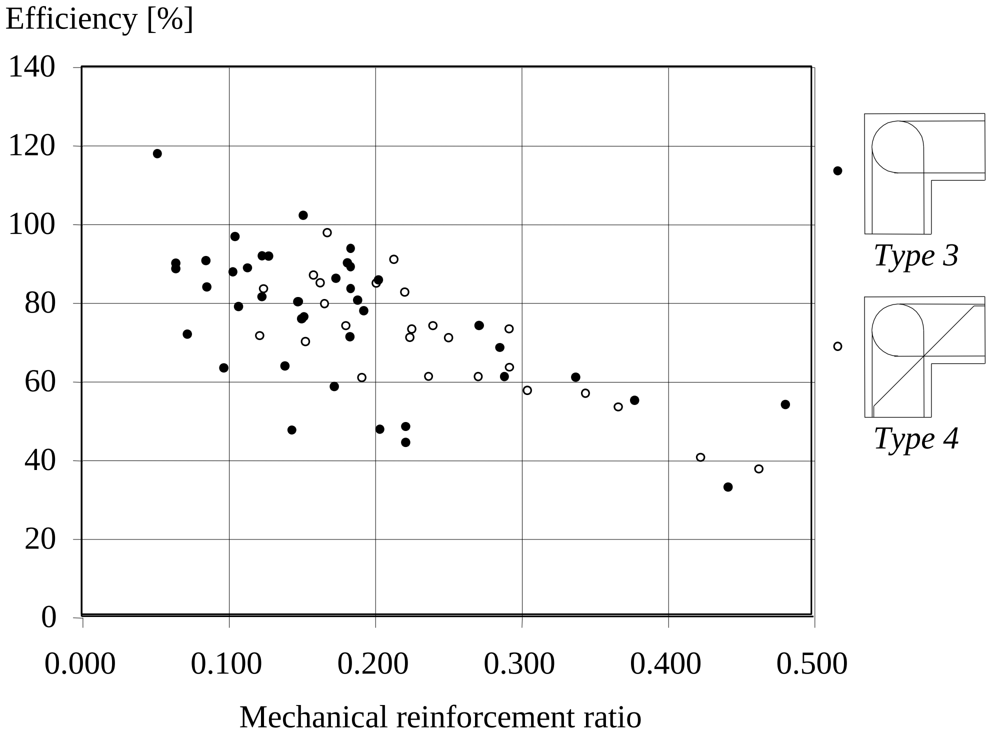


Figure 4.18 Comparison of solutions of Type 3 and Type 4 when the diagonal bar is considered in the theoretical moment capacity. The figure is taken from Johansson (2000).

Tests performed by Campana *et al*. (2013), where the efficiency of solutions similar to those used in Johansson (2000) have been investigated. It should be noted that the reinforcement amount used was $\rho = 0.7$ %, and the corresponding mechanical reinforcement ratio used was about $\omega_s \approx 0.0875$[2]. The test were performed for corners with a nodal angle of 125º, which means that the tensile forces created within the corner is less critical, see Figure 4.13a. The nodal angle used in the tests in Johansson (2000) was 90º. Note that none of the detailing solutions results in sufficient efficiency, unless the mechanical reinforcement ratio is very low, according to Campana *et al*. (2013).

[2] Different values of ω_s were used for the different test specimens.

Campana *et al*. (2013) means that a strut and tie model is an adequate model to design and arrange the reinforcement in concrete frame corners subjected to opening moment. However, Johansson (2000) argues that a strut and tie model does not fully capture the real response of concrete frame corners with a nodal angle of 90º.

The result from the tests that are summarized in Johansson (2000) indicates that the solutions of Type 1 and Type 2 show poor performance. However, Campana *et al.* (2013) chose to investigate these details further to see if the corner efficiency could be effectively improved by adding radial stirrups to the solution of Type 1. This can be compared to the solution of Type 2 in Figure 4.15 and the recommended solution in Eurocode 2 shown in Figure 4.9b. One of the reason why this solution was interesting to test was that it is easy to perform at the construction site. The result showed that the behaviour was effectively improved. However, Johansson (2000) argues that the corner efficiency increases insignificantly by adding radial stirrups.

The comparison between Johansson (2000) and Campana *et al.* (2013) shows that the design and detailing of concrete frame corners subjected to opening moment is difficult and the tests that have been executed show inconsistent results. However, here the research performed by Johansson (2000) has been compared to the one performed by Campana *et al*. (2013) using different nodal angles on the concrete frame corner. Hence the conclusions drawn above, regarding whether the strut and tie model gives an accurate solution of the design and arrangement of the reinforcement or not and whether radial stirrups improve the corner efficiency or not, may not be correct. Since a concrete frame corner with a larger nodal angle will give a frame corner that is less sensitive, Johansson (2013).

For frame corners with closing moment the efficiency is insufficient for ω_s less than 0.200 according to Johansson (2000), see line a) in Figure 4.17. Here it is shown that the corner efficiency for solutions of Type 1 and Type 3 is comparable. In Johansson (2000) there is also a suggestion by Stroband and Kolpa (1983) to limit ω_s to 0.240 to avoid crushing of the diagonal compressive concrete strut.

5 Design and detailing for shear

5.1 Structural response and modelling

The distribution of normal- and shear stresses over an uncracked rectangular concrete cross-section of a structural member subjected to bending moment, *M*, and shear force, *V*, can be seen in Figure 5.1, Engström (2010). The shear stress, τ, is zero at the outermost fibres and has its maximum value, τ_{max}, at the centroid where the normal stress, σ, due to bending is zero.

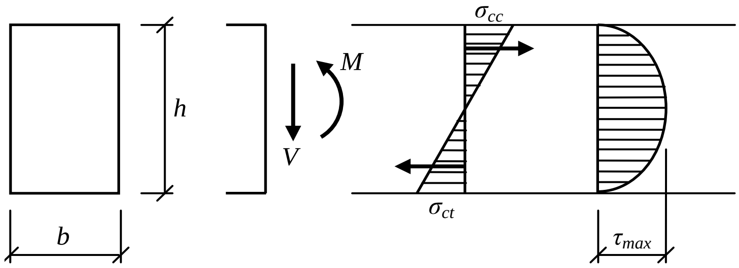


Figure 5.1 Distribution of normal- and shear stresses over an uncracked rectangular cross-section.

The principal stresses, σ_{II} and σ_{I}, at the centroid can be shown to be equal to τ_{max} with the directions shown in Figure 5.2.

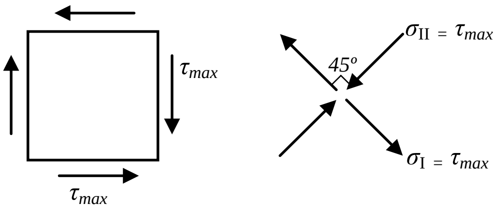


Figure 5.2 Stresses at the centroid of an arbitrary cross-section. Principal stresses in tension, σ_{I} and in compression, σ_{II}.

The principal tensile stresses, σ_{I}, and principal compressive stresses, σ_{II}, that are acting in the web of an arbitrary beam are before cracking equal in size, Engström (2010). When a shear crack occurs due to the tension field, σ_{I}, a redistribution of stresses takes place. The principal tensile stresses can no longer be resisted by the cracked concrete area and the whole shear force, *V*, is instead resisted by the vertical component of the principal compressive stresses acting in the inclined struts formed between the cracks, see Figure 5.3.

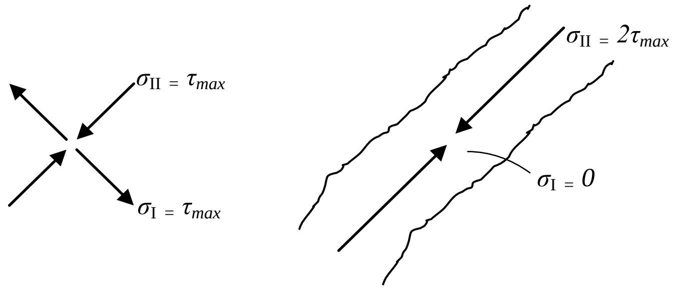


Figure 5.3 *Principal stresses at the centroid of an arbitrary cross-section before and after formation of shear cracks.*

In order to resist shear force along a structure after cracking has occurred the concrete in the inclined struts between the cracks must be strong enough to resist the compressive stresses. Moreover, the load path to the supports, as illustrated in Figure 3.28, Section 3.3.2, is achieved by lifting the shear force up to a new set of inclined struts, either by shear reinforcement or, if possible, by friction and interlocking effects in the concrete, Engström (2010), see Figure 5.4.

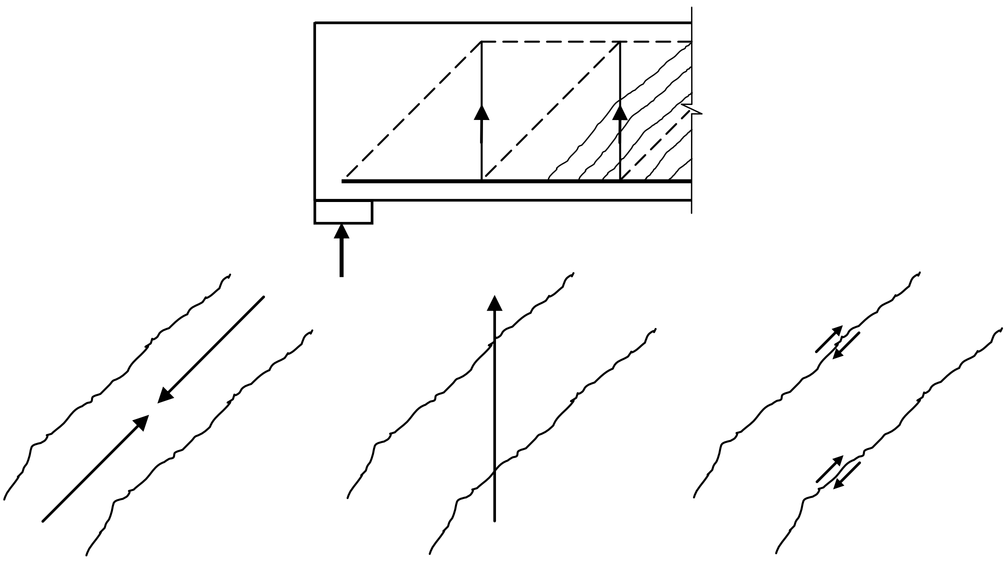


Figure 5.4 *Load path to the supports after cracking, a) compression is resisted by concrete between cracks, b) forces are lifted over the cracks by reinforcement or c) by friction and interlocking effects.*

Due to this behaviour there are two types of shear failures that must be designed for in shear reinforced concrete members, Engström (2010). The shear reinforcement must be able to lift the shear force over the cracks without causing yielding of the reinforcing steel. If yielding occurs this is called shear sliding failure. The other type of failure is called web shear compression failure and is developed when the concrete in the struts between the inclined cracks is crushed due to the high compressive stress.

In order to calculate the stresses that must be taken by the shear reinforcement and the compressed concrete between the inclined cracks a truss model as the one in Figure 5.5 is used. The shear force, V_{Ed}, acting in the section, must be resisted by an inclined compressive force, F_{cw}, corresponding to the inclined stress field over the cross-section. The vertical component, F_{sw}, of this compressive force must be lifted by the shear reinforcement that crosses the inclined cracks of the considered region.

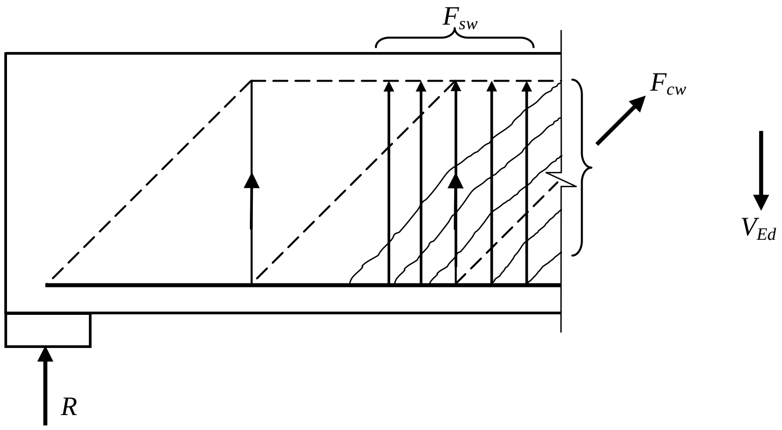


Figure 5.5 Truss model describing the load path for shear force along a concrete member cracked in shear. The angle of the stirrups is α = 90º.

To maintain equilibrium after cracking the longitudinal component, N, of the inclined compressive force, F_{cw}, must be balanced. Since F_{cw} is a force representing an inclined stress field, its longitudinal component, N, also corresponds to a stress field acting over the whole cross-section. In members subjected to bending it is convenient to divide this stress field in two and place the two resulting forces at the positions of the force couple F_s and F_c that resists the bending moment, see Figure 5.6. F_c is the resulting compressive force in the compressive zone due to bending and F_s is the tensile force resisted by the longitudinal bending reinforcement. These positions can also be recalled as tension and compression chords.

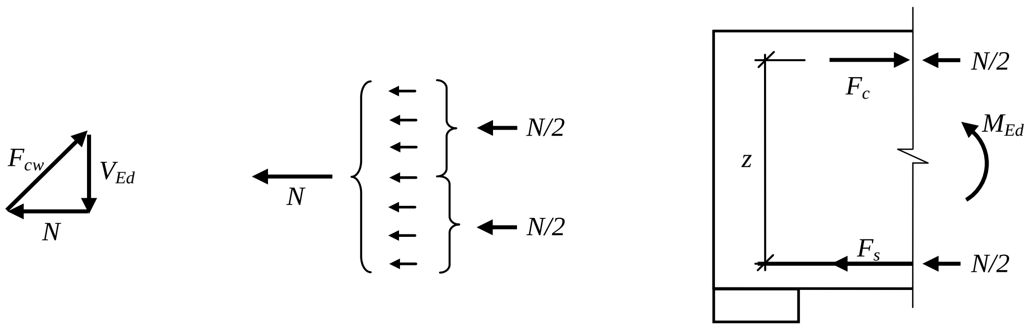


Figure 5.6 The longitudinal component, N, of F_{cw} due to shear cracking is distributed to the positions of the force couple, F_c and F_s, that resists the bending moment.

As illustrated in Figure 5.2 and Figure 5.3 a shear crack will occur in the web with an angle, θ, of about 45º. However, during loading a redistribution of stresses can occur resulting in shear cracks with smaller inclination, θ. This is something that can be used in design, since it affects the ratio between shear and longitudinal reinforcement amounts. It should be noted that the shear reinforcement not necessarily must be vertically placed, but can be placed with an inclination, α, that also will affect the overall design. More about this will be discussed further in Section 5.5. The definition of α and θ can be seen in Figure 5.7.

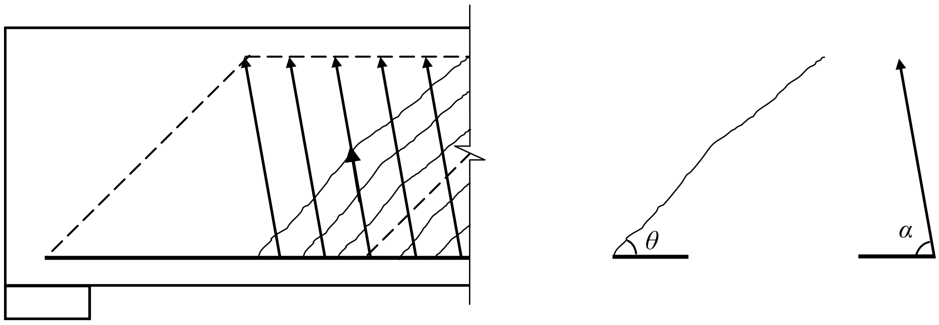


Figure 5.7 Definition of the inclination of compressive stress field, θ, and the inclination of shear reinforcement, α.

5.2 Shear sliding failure

5.2.1 Requirements in Eurocode 2

The design value of the shear force which can be lifted by the yielding shear reinforcement is given by Expressions EC2 (6.8) and EC2 (6.13) in Eurocode 2, see Equation (5.1) and (5.2), respectively. Equation (5.2) is a more general expression considering inclined shear reinforcement, while Equation (5.1) is valid only for vertical reinforcement that has an angle $\alpha = 90^{\circ}$ in relation to the longitudinal axis of the concrete member.

$$V_{Rd,s} = \frac{A_{sw}}{s} z \cdot f_{ywd} \cot\theta \tag{5.1}$$

$$V_{Rd,s} = \frac{A_{sw}}{s} z \cdot f_{ywd} (\cot\theta + \cot\alpha)\sin\alpha \tag{5.2}$$

A_{sw}	cross-sectional area of one shear reinforcement unit
s	spacing of shear reinforcement units
f_{ywd}	design yield strength of the shear reinforcement
θ	angle between compression strut and the longitudinal axis
α	angle between shear reinforcement and the longitudinal axis

The expressions for shear sliding failure provided by Eurocode 2 are derived in Section 5.2.2. The derivation is based on Engström (2010), however, a similar derivation can be found in Betongföreningen (2010a).

5.2.2 Explanation and derivation

The expressions provided in Eurocode 2 for calculation of the shear force which can be lifted by the shear reinforcement, can be derived by the truss models illustrated in Figure 5.8 for vertical stirrups and in Figure 5.9 for inclined shear reinforcement, Engström (2010).

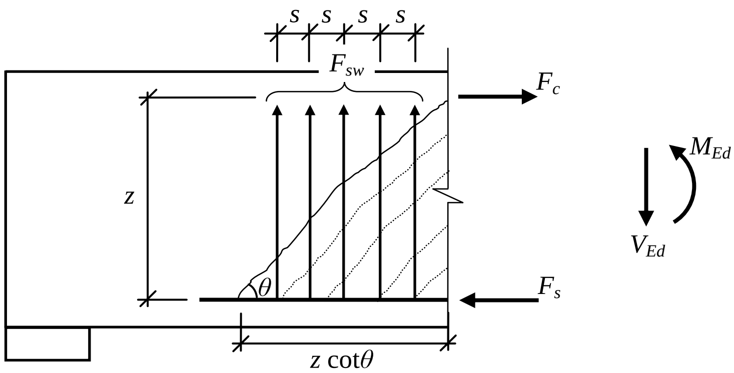


Figure 5.8 Truss model for derivation of the design value of shear force which can be lifted by vertical shear reinforcement.

The number of shear reinforcement units, n, that crosses one inclined crack can be expressed as

$$n = \frac{z \cot\theta}{s} \tag{5.3}$$

The force that can be taken by one shear reinforcement unit, $F_{sw,n}$, can be calculated by the design yield strength, f_{ywd} and the cross-sectional area, A_{sw}, of such reinforcement unit, see Equation (5.4).

$$F_{sw,n} = f_{ywd} \cdot A_{sw} \tag{5.4}$$

Hence, the shear force which can be sustained by vertical shear reinforcement can be calculated as

$$V_{Rd,s} = F_{sw} = nF_{sw,n} = \frac{z \cot\theta}{s} f_{ywd} A_{sw} \tag{5.5}$$

Equation (5.2) that considers the effect of inclined shear reinforcement is derived in a similar way taking into account that the inclined shear reinforcement units cross the critical crack somewhat differently, see Figure 5.9.

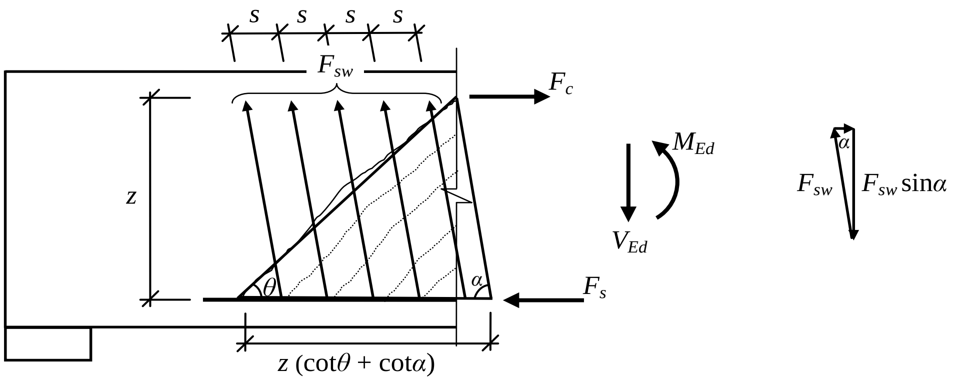


Figure 5.9 Truss model for derivation of the design value of shear force which can be lifted by inclined shear reinforcement.

The number of shear reinforcement units that crosses the inclined crack can therefore be calculated as

$$n = \frac{z\ (\cot\theta + \cot\alpha)}{s} \tag{5.6}$$

which leads to the expression given in Eurocode 2, see Equation (5.7).

$$V_{Rd,s} = F_{sw}\sin\alpha = nF_{sw,n}\sin\alpha = \frac{z\ (\cot\theta + \cot\alpha)}{s}\ f_{ywd}A_{sw}\sin\alpha \tag{5.7}$$

5.2.3 Discussion

Firstly, it should be noted that the area, A_{sw}, of one shear reinforcement unit is not only one bar as can be perceived in the figures above. It is the total cross-sectional area of all reinforcement legs of each reinforcement unit that crosses the crack in each section. See the relation between A_{sw} and the bar diameter, ϕ, in Equations (5.8) and (5.9).

$$A_{sw} = n_{leg} \cdot A_{si} \tag{5.8}$$

n_{leg} number of shear reinforcement legs in one of the shear reinforcement units that cross the critical crack

A_{si} cross-sectional area of one leg of reinforcement, see Equation (5.9)

$$A_{si} = \frac{\pi\phi^2}{4} \tag{5.9}$$

For beams stirrups are often used as shear reinforcement resulting in two legs, see Figure 5.10. However, in case of several stirrups in the same section the number of legs is increased.

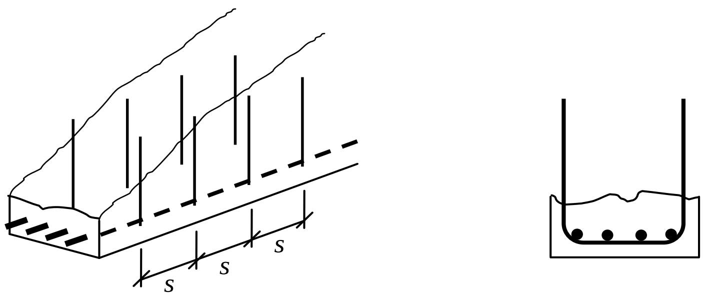


Figure 5.10 Legs that cross the shear crack in a beam in case of single stirrups. Figure is based on Engström (2010).

It should be noted that for concrete structures without shear reinforcement there is still some possibility to lift shear forces across shear cracks due to friction and interlocking effects in the concrete as mentioned in Section 5.1. However, according to Eurocode 2 these effects are not allowed to be utilised together with the capacity of shear reinforcement. In contrast to Eurocode 2 there are two different methods for design of shear reinforcement in BBK 04, Boverket (2004). According to the first of them, the original method also used in BBK 79 and BBK 94, it is allowed to add the contribution to shear capacity from concrete to the contribution of shear reinforcement. The second method, recalled as an alternative method, is the same as the one in Eurocode 2, i.e. based on a truss model, where it is not allowed to superimpose the two effects. However, it should be noted that especially for small values of the strut inclination, θ, the effect of friction is not reliable, Engström (2013).

Another difference between Eurocode 2 and BBK 04 is the implementation of the angle, θ, of the inclined struts. In Eurocode 2 it is allowed to choose the inclination, θ, within certain limits, more about this is explained in Section 5.5. In the original method in BBK 04 the inclination of the inclined struts is always set to 45º. The methodology in Eurocode 2 provides the opportunity to choose among many different configurations and the designer is therefore given a greater ability to optimise the design and thereby the behaviour of the structural member. It should also be noted that the close relation between shear force and torsion, as is further described in Chapter 6, becomes much clearer in Eurocode 2 compared to BBK 04 thanks to the introduction of the variable strut inclination, θ. According to BBK 04 the designer can choose an angle of the inclined struts when designing for torsion but, as mentioned, not when designing for shear force.

According to Eurocode 2 the amount of shear reinforcement is limited in order to ensure a ductile behaviour of the structural member. The lower limit is presented and further discussed in Section 5.4. The amount of shear reinforcement that is allowed to be utilised in a structure is limited by the maximum shear resistance $V_{Rd,max}$ that is governed by the compressive capacity of the inclined concrete struts between the shear cracks. More about this can be found in Section 5.3.

5.3 Web shear compression failure

5.3.1 Requirements in Eurocode 2

The shear force that can be resisted by a reinforced concrete member is limited by crushing of the concrete in the inclined compression struts between the shear cracks. The design value of the shear force that can be resisted by the compressed concrete is, just as the design value of shear force taken by the reinforcement in Section 5.2, given by two equations in Eurocode 2 depending on if the shear reinforcement is inclined or vertical with regard to the longitudinal axis of the concrete member, SIS (2008).

Equation (5.10) corresponds to Expression EC2 (6.9) that considers vertical shear reinforcement

$$V_{Rd,\max} = \frac{\alpha_{cw} \nu_1 f_{cd} b_w z}{\cot\theta + \tan\theta} \tag{5.10}$$

Expression EC2 (6.14) is a more general expression where the inclination of the shear reinforcement is considered, see Equation (5.11).

$$V_{Rd,\max} = \alpha_{cw} \nu_1 f_{cd} b_w z \frac{\cot\theta + \cot\alpha}{1 + \cot^2\theta} \tag{5.11}$$

α_{cw}	coefficient taking account of the state of stress in the web
ν_1	strength reduction factor for concrete cracked in shear
f_{cd}	design compressive strength of concrete
b_w	minimum width of the web between the longitudinal compression and tension chords
z	inner lever arm

The derivations of the expressions for design with regard to web shear compression failure are, as the expressions for design with regard to shear sliding failure, mainly based on Engström (2010) and Betongföreningen (2010a).

5.3.2 Explanation and derivation

Figure 5.11 shows how the compressive stress σ_{cw}, acting in the inclined struts in the web, due to the shear force, V_{Ed}, can be calculated by the help of a truss model, Engström (2010).

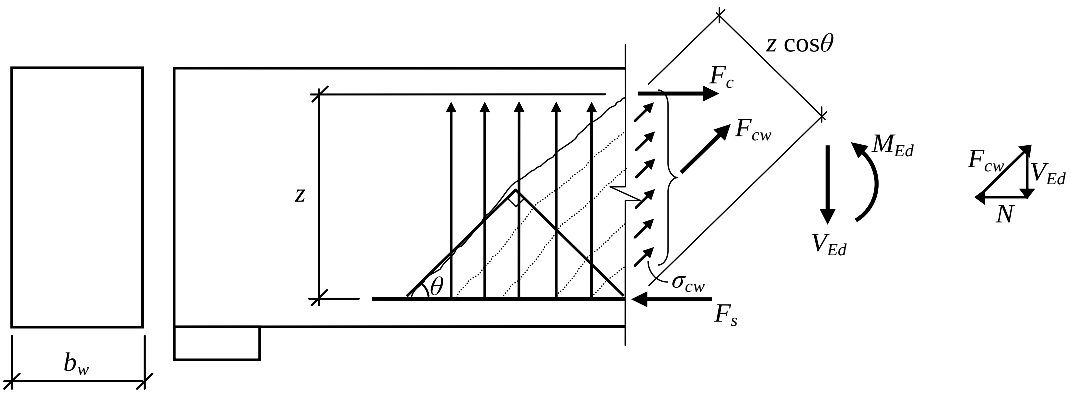


Figure 5.11 Truss model for derivation of the design value of shear force which can be resisted by the concrete within the compressed struts when using vertical shear reinforcement, α = 90º.

The compressive stresses in the web section can be summarised to an inclined compressive force, F_{cw}, as

$$F_{cw} = \sigma_{cw} b_w z \cos\theta \tag{5.12}$$

By equilibrium the shear force corresponding to the inclined component can thereby be written

$$V_{Ed} = F_{cw} \sin\theta = \sigma_{cw} b_w z \cos\theta \sin\theta \tag{5.13}$$

Comparing Equation (5.13) to the expression given in Eurocode 2, Equation (5.10), there are two differences. Firstly, Equation (5.10) has two extra terms, α_{cw} and v_1. The two factors α_{cw} and v_1 are in Eurocode 2 used together with the design concrete strength, f_{cd}, to express the compressive resistance of concrete.

$$\sigma_{cw,\max} = \alpha_{cw} \nu_1 f_{cd} \tag{5.14}$$

The reduction factor v_1 takes into account that the shear reinforcement creates an unfavourable tension field across the struts that decreases the load bearing capacity of the compressed concrete, Engström (2010). Betongföreningen (2010a) adds that the model assumes ideally plastic behaviour.

The factor α_{cw}, on the other hand, considers the effect of any compressive normal force on the section. Very high compressive normal stresses have a bad effect on the load bearing capacity with regard to web shear compression failure. However, moderate stresses slightly increase the load bearing capacity due to a biaxial state of stresses. More information about the two coefficients can be found in Betongföreningen (2010a).

When the derived Expression (5.13) and the one in Eurocode 2, see Equation (5.10), are compared, there is a difference in how the trigonometric transformation is written. Eurocode 2 writes

$$\frac{1}{\cot\theta + \tan\theta} \tag{5.15}$$

The corresponding expression in the derived Equation (5.13) is

$$\cos\theta \sin\theta \tag{5.16}$$

It can be shown that these two expressions are the same

$$\frac{1}{\cot\theta + \tan\theta} = \frac{1}{\dfrac{\cos\theta}{\sin\theta} + \dfrac{\sin\theta}{\cos\theta}} = \frac{\cos\theta \sin\theta}{\cos^2\theta + \sin^2\theta} = \cos\theta \sin\theta \tag{5.17}$$

Similarly, Equation (6.14) in Eurocode 2 can be derived by the truss model in Figure 5.12.

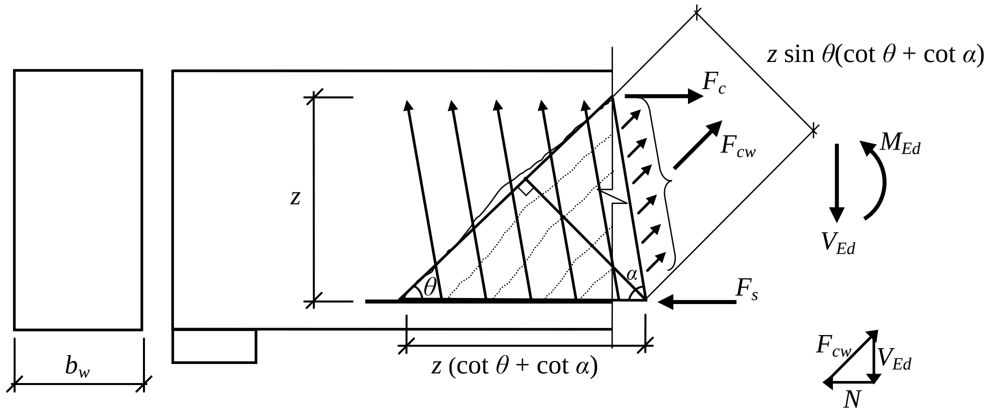


Figure 5.12 Truss model for derivation of the design value of shear force which can be resisted by the concrete within the compressed struts when using inclined shear reinforcement, α < 90º.

The area where the inclined compressive stress σ_{cw} is acting is now also dependent on the inclination of the shear reinforcement. The compressive force F_{cw} is therefore calculated as

$$F_{cw} = \sigma_{cw} b_w z \sin\theta(\cos\theta + \cot\theta) \tag{5.18}$$

the shear force can be written as

$$V_{Ed} = F_{cw}\sin\theta = \sigma_{cw} b_w z \sin^2\theta(\cos\theta + \cot\theta) \tag{5.19}$$

As in previous derivation it can be shown that the trigonometric expressions are the same, i.e.

$$\frac{1}{1+\cot^2\theta} = \frac{1}{1+\dfrac{\cos^2\theta}{\sin^2\theta}} = \frac{\sin^2\theta}{\sin^2\theta + \cos^2\theta} = \sin^2\theta \tag{5.20}$$

5.3.3 Discussion

As mentioned in Section 5.2.3 the expressions of $V_{Rd,max}$ provides an upper limit of the shear reinforcement that can be utilised in a reinforced concrete structure. The amount of reinforcement corresponding to Expressions (5.10) and (5.11) can be derived by equating Expression (5.10) with Equation (5.1) and Equation (5.11) with Equation (5.2), respectively. This is the same as letting the design resistance governed by the shear reinforcement, $V_{Rd,s}$ be equal to the resistance governed by web shear compression failure, $V_{Rd,max}$.

$$V_{Rd,s} = V_{Rd,\max} \tag{5.21}$$

Inserting the expressions from Equations (5.2) and (5.11) results in

$$\frac{A_{sw,\max}}{s} z\ f_{ywd}\left(\cot\theta + \cot\alpha\right)\sin\alpha = \alpha_{cw} \nu_1 f_{cd} b_w z \frac{\cot\theta + \cot\alpha}{1+\cot^2\theta} \tag{5.22}$$

By deleting the terms that cancel each other out and rearranging the other terms an expression including the corresponding amount of shear reinforcement, A_{sw}, is derived.

$$\frac{A_{sw,\max} f_{ywd}}{s b_w} = \frac{\alpha_{cw} \nu_1 f_{cd}}{\sin\alpha\left(1+\cot^2\theta\right)} \tag{5.23}$$

The derived expression for maximum reinforcement amount can be found in Eurocode 2 Equations EC2 (6.12) and EC2 (6.15), but only for cot θ =1, i.e. for an assumed crack inclination, θ, equal to 45º. For vertical shear reinforcement α is equal to 90º. Hence, sin α is equal to 1. Inserting this into Equation (5.23) gives

$$\frac{A_{sw,\max} f_{ywd}}{s b_w} = \frac{\alpha_{cw} \nu_1 f_{cd}}{\left(1+\cot^2\theta\right)} \tag{5.24}$$

By letting cot θ be equal to 1 the expressions in Eurocode 2 are obtained, see Equation (5.25) and (5.26).

$$\frac{A_{sw,\max} f_{ywd}}{s b_w} = \frac{\alpha_{cw} \nu_1 f_{cd}}{2} \tag{5.25}$$

$$\frac{A_{sw,\max} f_{ywd}}{s b_w} = \frac{\alpha_{cw} \nu_1 f_{cd}}{2\sin\alpha} \tag{5.26}$$

It should be emphasised that the derivations of the expressions for design with regard to web shear compression failure are based on a cross-section of width b_w. To be more precise it is the web section that should be considered, i.e. the area between the tension and compression chords due to bending, see F_s and F_c in Figure 5.11 and Figure 5.12. In order to obtain a conservative design Eurocode 2 states that the smallest width of the web section should be used in calculations. This is due to the fact that a smaller concrete area can resist lower forces than a larger concrete area.

Eurocode 2 Commentary, ECP(2008a), implies that the factor α_{cw} is based on empirical test results. It is there stated that Nielsen (1990) proposed an initial expression of α_{cw}, since he realised that prestressing had a positive effect on the shear capacity of beams. A comparison of 93 tests showed that this expression wasn't conservative enough and another proposal was therefore given by Fouré (2000). However, Fouré's expression is not the same as the one used in Eurocode 2, which is even more moderate. To learn more about this, see ECP (2008a). It can be added that according to Eurocode 2 α_{cw} should be chosen as 1 for non-prestressed structures. However, it should be noted that this is not true for columns that need to be designed for shear.

5.4 Minimum shear reinforcement

5.4.1 Requirements in Eurocode 2

The minimum amount of shear reinforcement that should be placed in a reinforced concrete member, in this case a beam, is given by a minimum shear reinforcement ratio in Section EC2 9.2.2, Expression EC2 (9.5N), see Equation (5.27). For design of bridges there are some additional demands that can be found in the Swedish National Annex

$$\rho_{w,\min} = \frac{0.08\sqrt{f_{ck}}}{f_{yk}} \tag{5.27}$$

The shear reinforcement ratio is described as the relation between the shear reinforcement area and the concrete area of the influenced section perpendicular to a shear reinforcement unit, Engström (2013), and is given by Expression EC2 (9.4) in Eurocode 2, see Equation (5.28).

$$\rho_w = \frac{A_{sw}}{s \cdot b_w \cdot \sin\alpha} \tag{5.28}$$

α angle between shear reinforcement and longitudinal axis

The minimum amount of shear reinforcement should, according to Eurocode 2, Paragraph EC2 6.2.1(4) concerning shear force, be placed in members even if shear reinforcement is not necessary according to design calculations. For structural members such as slabs, where transverse redistribution of loads is possible, and in members of minor importance the minimum amount of shear reinforcement may nevertheless be omitted.

In addition to the minimum amount of shear reinforcement that must be placed in a reinforced concrete section there are limits regarding maximum spacing between bars. Eurocode 2 gives as recommendation two different expressions depending on if the shear reinforcement is made out of shear assemblies, $s_{l,max}$, or bent-up bars, $s_{b,max}$. In Sweden on the other hand these two expressions are set to be the same according to the National Annex, see Equation (5.29).

$$s_{l,\max} = s_{b,\max} = 0.75d(1+\cot\alpha) \tag{5.29}$$

The recommended value for $s_{b,max}$ is in Eurocode 2 set to

$$s_{b,\max} = 0.6d(1+\cot\alpha) \tag{5.30}$$

The transversal distance between the legs of a series of shear reinforcement units should also be limited according to Eurocode 2, Expression EC2 (9.8N).

$$s_{t,\max} = 0.75d \qquad \leq 600 \text{ mm} \tag{5.31}$$

According to Eurocode 2 Section EC2 9.3.2 the rules that apply for beams also apply for slabs, but with some exceptions:

- if shear reinforcement is needed, the slab thickness should be at least 200 mm
- the maximum longitudinal distance between shear reinforcement units, $s_{l,max}$, is the same as in Equation (5.29)
- the maximum distance between bent up bars, $s_{b,max}$, should be increased to d
- the maximum transverse distance between shear reinforcement legs, $s_{t,max}$, should be increased to 1.5*d*.

5.4.2 Explanation and derivation

A minimum amount of shear reinforcement should be placed in a structure for the same reason as minimum longitudinal reinforcement, i.e. to avoid a brittle premature failure, Engström (2010). Equation (5.27) will provide sufficient capacity of the shear reinforcement to prevent it from yielding or to be torn apart when forces suddenly are transferred from the cracking concrete to the reinforcing steel. This means that the failure load is higher than the cracking load, *fib* (2010). The same explanation for the expression of minimum shear reinforcement is found in Hendy and Smith (2010). However, no derivation of the expression has been found in the literature studied in this master's thesis project.

The expression for maximum longitudinal distance between shear reinforcement in beams, $s_{l,max}$ and $s_{b,max}$, can be derived. Equation (5.29) ensures that no cracks can propagate between the legs of the shear reinforcement. Each crack should be crossed by at least one shear reinforcement unit so that the shear force can be lifted up over the crack to maintain the load path along the beam, *fib* (2010). The derivation comes from a truss model where the angle of the inclined shear cracks is set to 45º, see Figure 5.13.

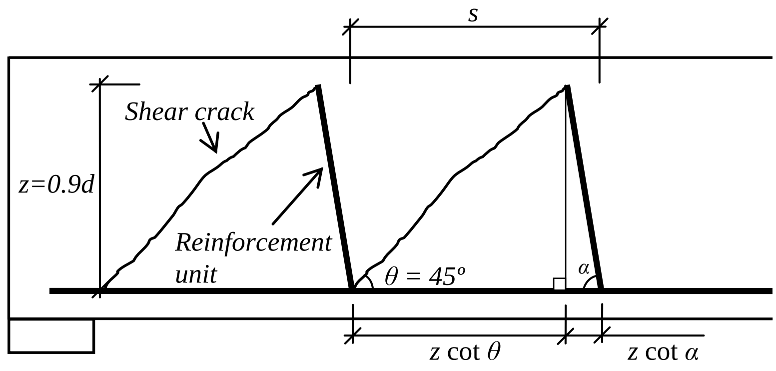


Figure 5.13 Truss model used in the derivation of maximum longitudinal distance between shear reinforcement units.

As can be seen in Figure 5.13 the theoretical spacing between bars should not be larger than the expression in Equation (5.32) in order to avoid that the inclined shear cracks propagate between the shear reinforcement units without being crossed by at least one of them.

$$s = z(\cot\theta + \cot\alpha) \tag{5.32}$$

The inner lever arm is normally estimated to 0.9*d* resulting in

$$s = 0.9d(\cot\theta + \cot\alpha) \tag{5.33}$$

However, shear cracks will not form with an angle of exactly 45º and in order to be on the safe side, a smaller distance between bars should be used in design. A certain overlap is required in a conservative design. This is illustrated in Figure 5.14

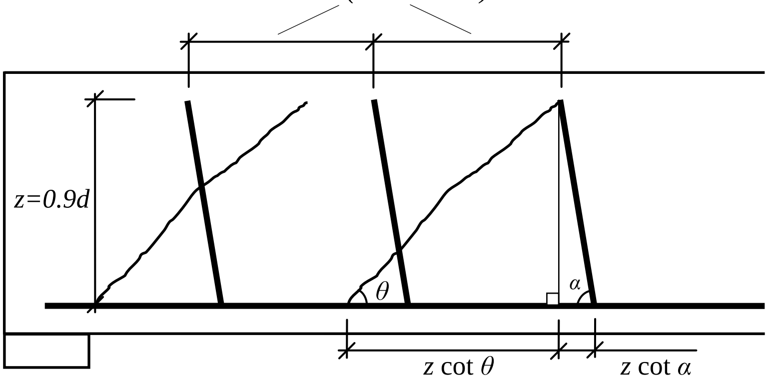


Figure 5.14 The spacing must be smaller than the maximum distance between two cracks in order to provide a certain overlap.

The maximum longitudinal spacing between reinforcing bars can thereby be derived as

$$s = 0.75d(\cot\theta + \cot\alpha) \tag{5.34}$$

By assuming an inclination of the shear cracks θ = 45º the expression will be

$$s = 0.75d(1 + \cot\alpha) \tag{5.35}$$

which is the same expression as in Eurocode 2, see Equation (5.29).

It should be noted that the recommended value of $s_{b,max}$ differs from that used in Sweden. The reason for this has not been found. However, the recommended value in Equation (5.30) is more conservative.

The reason why there is a maximum limit of the distance between shear reinforcement legs in the transverse direction is to ensure that the forces are evenly distributed also in this direction. This will enable an even transfer of forces through the structure, Johansson (2013).

5.4.3 Discussion

According to Paragraph EC2 6.2.1(4) shear reinforcement must always be placed in a structural member, even if the reinforced concrete without shear reinforcement has a shear capacity, $V_{Rd,c}$, that is larger than the shear force acting on the critical section, V_{Ed}. According to Betongföreningen (2010a) this is an important difference from the previous Swedish handbook BBK 04, where no such requirements exist for beams in buildings. It should be noted that the original suggestion to the National Annex entailed that minimum reinforcement should only be placed where shear reinforcement was necessary according to paragraph EC2 6.2.1(5), i.e. where

$V_{Rd,c} < V_{Ed}$. However, Boverket did not include this suggestion into their regulations, why the minimum amount of reinforcement applies without restriction for beams, Betongföreningen (2010a).

However, as stated in Section 5.4.1 it is not necessary to place shear reinforcement in slabs. According to Hendy & Smith (2010) this is simply because of practical reasons. For instance it is not always convenient to place links in slabs due to the lack of space, see more about this in Section 5.6.3. However, in order to be allowed to not place shear reinforcement in a slab transverse redistribution of loads must be possible. This implies that the reason why shear reinforcement can be omitted in slabs may be because slabs in comparison to beams have one additional direction in which forces can be redistributed. Hence, slabs have larger possibility to spread the loads on different parts of the structure resulting in a more favourable load situation.

In Section 5.4.2 the reason for the minimum shear reinforcement requirement was explained. However, no derivation of the expression has been found. It might be possible to find the minimum reinforcement requirement by equating the expression for the shear force capacity of shear reinforcement, $V_{Rd,s}$, see Section 5.2.1, and the shear force corresponding to the shear capacity of uncracked concrete. According to Paragraph EC2 12.6.3(3) a structural concrete member is assumed to be uncracked if the absolute value of the principal stress, σ_{I}, is smaller than f_{ctd}, i.e. the design tensile strength of concrete. Perhaps this is the key to find the relation. Such a relation between the shear capacity of uncracked concrete and shear reinforcement has been investigated by Johansson (2013). However, the expression for the minimum requirement provided in Eurocode 2, Equation (5.27), could not be derived. According to Johansson (2013) it rather seems like the requirement might have been based on expressions found in the previous Swedish handbook BBK 04, Boverket (2004).

The reason why the inclination of the cracks is set to 45º in the derivation of the maximum longitudinal distance between shear reinforcement units is that when a shear crack forms, it will initially have a direction equal to this angle, see Figure 5.2 and Figure 5.3. Due to plastic redistribution the inclination of the crack will successively change into the one chosen in design, when the beam approaches its ultimate capacity. A crack with an angle of 45º represents the smallest value of the maximum spacing of stirrups, since smaller strut inclinations will result in longer cracks.

The maximum longitudinal spacing of shear reinforcement units allowed in Sweden is, as implied in Section 5.4.1, the same for links as for bent up bars. The most probable reason for this is simply that it according to previous Swedish requirements does not matter if the shear reinforcement comprises of links or bent up bars, see more about this in Section 5.6. What may appear as strange is that it for slabs are different requirements for the two different types of shear reinforcement. The longitudinal distance between bent up bars is in slabs allowed to be larger than the distance between links and much larger compared to the recommended value for beams. It can be questioned if this means that the angle of the crack inclination is considered differently in slabs than in beams and if so, why only for bent up bars? Compare also to the reasoning in Section 9.2 where slabs and beams with shear reinforcement are treated differently with regard to crack inclination for curtailment of longitudinal reinforcement. The reason why the requirements differ is not clear from Eurocode 2 and it is therefore desirable to know if there are other motives for the rules than the ones provided in Section 5.4.2.

The background to the maximum transverse distance between shear reinforcement legs is also limited. Since the entire Chapter EC2 9 concerning detailing of reinforcement has been left out from Commentary to Eurocode 2, it is difficult to say what is behind the spacing requirements. However, it is assumed that they have been empirically derived from tests. From Adams *et al*. (2009) it can be deduced that Leonhardt & Walther (1964) suggested a limitation of the transverse spacing equal to *d* in case of low shear stresses and 200 mm in case of high shear stresses. The reason for this was to adequately anchor and suspend the diagonal compression struts associated with the truss model used for shear reinforcement design. This means that the diagonal compression force must flow toward the stirrup legs, see Figure 5.15.

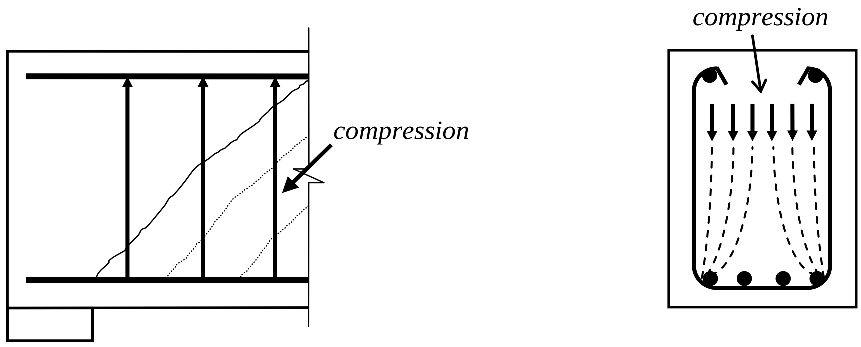


Figure 5.15 The diagonal compression force flows towards the stirrups legs. Figure is based on Adams et al. (2009).

In the publication written by Adams *et al*. (2009) it is also mentioned that according to Anderson and Ramirez (1989) detailing rules for strut and tie models can serve as a guide for transverse spacing of shear reinforcement legs. Their suggestion was a maximum distance equal to 12 times the bar diameter of the longitudinal reinforcement.

However, Adams *et al*. (2009) also performed tests on their own where they saw that, if the shear reinforcement was placed with a large transverse distance between the vertical legs, the increase of shear capacity in comparison to a member without shear reinforcement was very small. They also came to the conclusion that a recommended value of maximum spacing between transverse bars should be equal to *d*, but not larger than 600 mm. It was also suggested that the distance between the reinforcement legs should be reduced by 50 % in case of high nominal shear stresses.

The test results obtained by Adams *et al*. (2009) are in quite good agreement with the requirements provided for transverse spacing of shear reinforcement in beams, presented in Eurocode 2. However, when it comes to slabs the transverse distance between shear reinforcement legs should be increased according to Eurocode 2. The reason for this is unknown, but is further discussed in Section 5.6.3.

Finally, it should be noted that with respect to the three different publications mentioned some configurations of stirrups in wide beams have been compiled and discussed in ACI-CRSI (2010). These configurations are presented and discussed more in Section 5.6.3.

5.5 Additional tensile force due to inclined cracks

5.5.1 Requirements in Eurocode 2

Due to the inclined cracks in the web and corresponding loss of principal stresses in reinforced concrete members subjected to shear force there will be an additional tensile force in the longitudinal reinforcement. The additional tensile force is calculated according to Expression EC2 (6.18) in Eurocode 2, see Equation (5.36).

$$\Delta F_{td} = 0.5 \cdot V_{Ed} (\cos\theta - \cot\theta) \tag{5.36}$$

The inclination, θ, of the compressed struts caused by shear cracks in the concrete can be chosen by the designer, SIS (2008). However, the angle should according to Eurocode 2 be limited within the following intervals for reinforced concrete structures. It should be noted that there are other rules for prestressed structures.

$$1 \leq \cot\theta \leq 2.5 \tag{5.37}$$

corresponding to

$$22° \leq \theta \leq 45° \tag{5.38}$$

It should be noted that cot 22º = 2.5 and cot 45º = 1.

Derivation of the expression for this additional shear force in Equation (5.36) can be found in Section 5.5.2 and is based on Engström (2010) and (2011a).

5.5.2 Explanation and derivation

The expression in Equation (5.36) can derived from equilibrium conditions and is dependent on the inclination of both the compression field and the shear reinforcement, see Figure 5.16. For definitions of F_{cw} and F_{sw} see Sections 5.2 and 5.3.

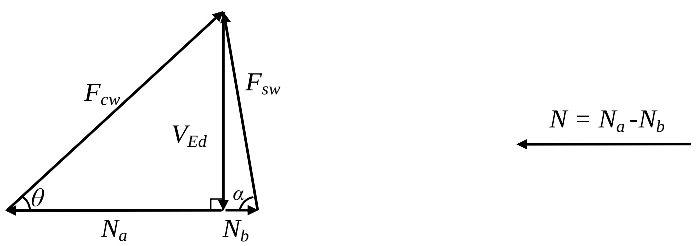


Figure 5.16 Force equilibrium for derivation of design additional tensile force due to inclined shear cracks.

The longitudinal component N_a, due to the inclined compressive stress field in the web, can be written as

$$N_a = \frac{V_{Ed}}{\tan\theta} = V_{Ed} \cot\theta \tag{5.39}$$

Furthermore, the longitudinal component N_b, due to the inclination of the shear reinforcement, is

$$N_b = \frac{V_{Ed}}{\tan\alpha} = V_{Ed}\cot\alpha \tag{5.40}$$

Since the two components have opposite directions, the total longitudinal component, N, is calculated from

$$N = N_a - N_b = V_{Ed}\left(\cot\theta - \cot\alpha\right) \tag{5.41}$$

In Sections 5.2 and 5.3 it was explained that both F_{sw} and F_{cw} are resultants corresponding to stress fields, see Figure 5.9 and Figure 5.12. The resulting component, N, thereby corresponds to a normal stress over the cross-section. As explained in Section 5.1 it is for members subjected to bending convenient to divide the resulting longitudinal component, N, in two and place each normal force at the positions of the force couple F_s and F_c that resists the bending moment. The normal stress is thereby balanced by the forces in the truss model shown in for instance Figure 5.12, by an increase of the tensile force, F_s, resisted by the longitudinal reinforcement, and a decrease of the compressive force, F_c, see Figure 5.17.

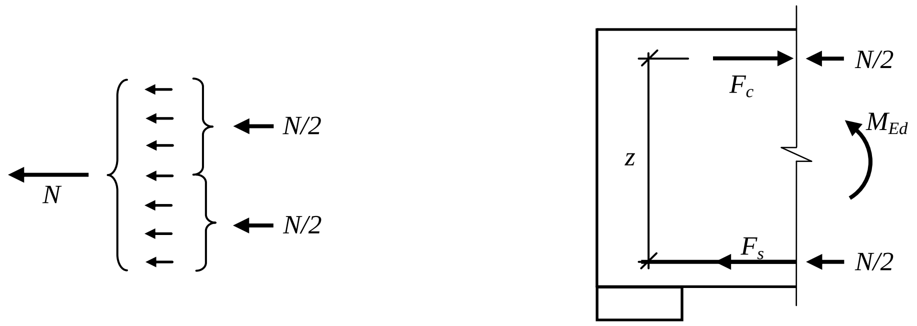


Figure 5.17 The longitudinal force component, N, of F_{cw} due to shear results in an increase of the longitudinal tensile force in the bending reinforcement and a reduction of the compressive force acting in the compressive zone due to bending.

The additional force that should be taken by the longitudinal reinforcement can therefore be derived as

$$\frac{N}{2} = \frac{V_{Ed}}{2}\left(\cot\theta - \cot\alpha\right) \tag{5.42}$$

This is the same expression as for ΔF_{td} in Equation (5.36).

The reason why the angle, θ, can be arbitrary chosen within certain intervals is because the requirements in Eurocode 2, presented in Section 5.2, 5.3 and 5.5, are all based on equilibrium conditions of a truss model and result in solutions that, regardless of the chosen inclination of the compressive struts, are in equilibrium. According to theory of plasticity all designs in equilibrium are valid solutions as long as ductility requirements are fulfilled, *fib* (2010).

The difference between solutions with different assumed angles is the ratio between shear reinforcement and additional longitudinal tensile reinforcement. Hence, the stiffness properties of the designed members will deviate, *fib* (2010). A smaller angle will reduce the amount of shear reinforcement needed, but the risk for web shear compression failure will increase, Engström (2010). The additional tensile force $\Delta F_{td} = N/2$ due to shear will also increase with a small angle of the compressive stress field, see Figure 5.18, which leads to higher need for longitudinal reinforcement.

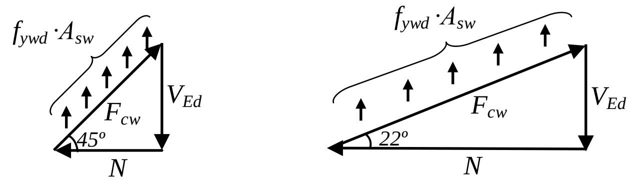


Figure 5.18 Effect of choice of inclination of the compressive stress field due to shear.

The lower limit of $\cot\theta = 1$, corresponding to a large angle θ = 45°, gives an upper limit of the maximum shear force, $V_{Rd,max}$, that can be resisted by the inclined concrete struts between shear cracks. The upper limit of cot θ = 2.5, θ = 22°, will on the other hand act as an upper limit for the shear resistance, $V_{Rd,s}$, that can be provided by shear reinforcement of certain intensity (or spacing), which increases with an increasing value of cot θ, Betongföreningen (2010a).

5.5.3 Discussion

In Eurocode 2 it is stated that the longitudinal reinforcement should be designed with regard to an additional force due to the inclined cracks. However, it should be noted that the inclination, α, of the shear reinforcement also have an effect on the required longitudinal reinforcement. This is shown in Figure 5.16. The additional tensile force due to inclined cracks becomes smaller if the shear reinforcement is placed with an inclination. Paragraph EC2 9.2.2(1) provides limitation for the angle α; between 45° and 90°. If the inclination of the shear reinforcement and the inclination of the compressive struts are both chosen to 45°, the longitudinal force due to shear cracks will be zero.

As is shown in Figure 5.18 the chosen inclination of the compressive struts affects a number of things; the longitudinal bending reinforcement, the amount of shear reinforcement and the compressive stress in the inclined struts between cracks. In Engström (2011b) it is shown how the additional tensile force, ΔF_{td}, varies with the inclination of the compressive struts, see Figure 5.19. It can be noted that an inclination of 45° means that the additional tensile force is equal to half the applied shear force.

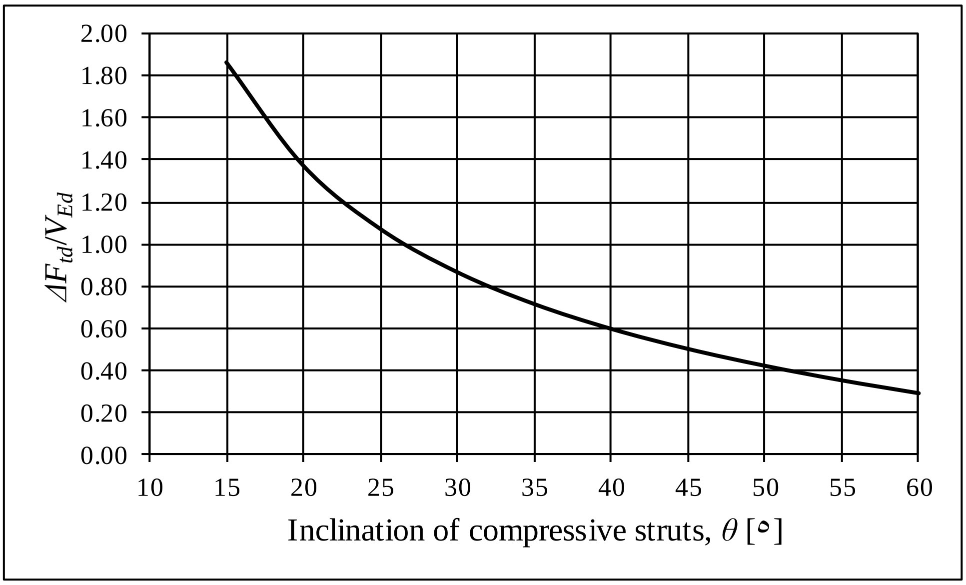


Figure 5.19 Additional tensile force due to inclined shear cracks as a function of the angle, θ. (α = 90º).

By inserting the limiting values of the inclination of the shear cracks into the expression for ΔF_{td}, i.e. Equation (5.36), the effect of the inclination on the additional required longitudinal tensile capacity can be described. The limits of θ are given in Equations (5.37) and (5.38), and the corresponding values of ΔF_{td}, are

$$0.5V_{Ed}(1-\cot\alpha) \leq \Delta F_{td} \leq 1.25V_{Ed} \tag{5.43}$$

In case of vertical shear reinforcement, i.e. $\alpha = 90^{\circ}$, the expression will be

$$0.5V_{Ed} \leq \Delta F_{td} \leq 1.25V_{Ed} \tag{5.44}$$

In addition to what have been mentioned regarding the effect of different shear crack inclinations, θ, Eurocode 2 Commentary states that a smaller angle will activate a larger number of bars and thereby increase the shear capacity, ECP (2008a). This is the same as have been explained in Figure 5.18, only from another point of view.

ECP (2008a) also states that choosing the lowest value of the angle, θ, often results in the most economic design. However, it cannot be derived from the text, if the additional amount of longitudinal reinforcement has been considered or not. It should also be added that such designs, with a small value of the strut inclination θ, requires large redistribution of stresses, since the cracks should change from their initial inclination of 45º and rotate all the way down to 22º.

The effects of different strut inclinations are also explained by Mosley *et al*. (2007). It is explained that for a decreasing angle, θ, the amount of shear reinforcement is reduced. However, when less shear reinforcement is required this is compensated by an increase of the longitudinal reinforcement. It can also be deduced that the maximum shear capacity of the section, based on compressive failure in the inclined struts, is reduced for values of the strut inclination greater or smaller than 45º.

Since the inclination of the compressive struts affects the amount of longitudinal shear reinforcement, it will also affect the curtailment process that is further described in Section 9.2.

In Section 5.2.3 it is mentioned that BBK 04 presents another method for design of shear reinforcement where the inclination of the compressive struts is automatically set to 45º. Structural engineers that are used to the method in BBK 04 might therefore have problem to understand or fully see the new possibilities provided in Eurocode 2. It is important to see the relation between the chosen inclination, amount of shear reinforcement, curtailment of longitudinal reinforcement and torsion design. As an example it is not clarified in BBK 04 that the angle, θ, applies for both torsion and shear. More about this is explained in Chapter 6.

One of the reasons why the angle, θ, must be held within certain limits is to limit the required redistribution of stresses. By letting the lower limit be equal to 22º extreme design solutions are avoided, Engström (2013) and Johansson (2013). To choose an angle larger than 45º is inappropriate, since a shear crack will initially have a direction equal to this angle. In ECP (2008a) the combination of the expressions for design with regard to web shear compression failure, see Section 5.2.1, and shear sliding failure, see Section 5.3.1, with the limiting value of the angle, θ, equal to 22º, i.e. $\cot \theta = 2.5$, is verified with test results. The plot shown in ECP (2008a) is reproduced in Figure 5.20.

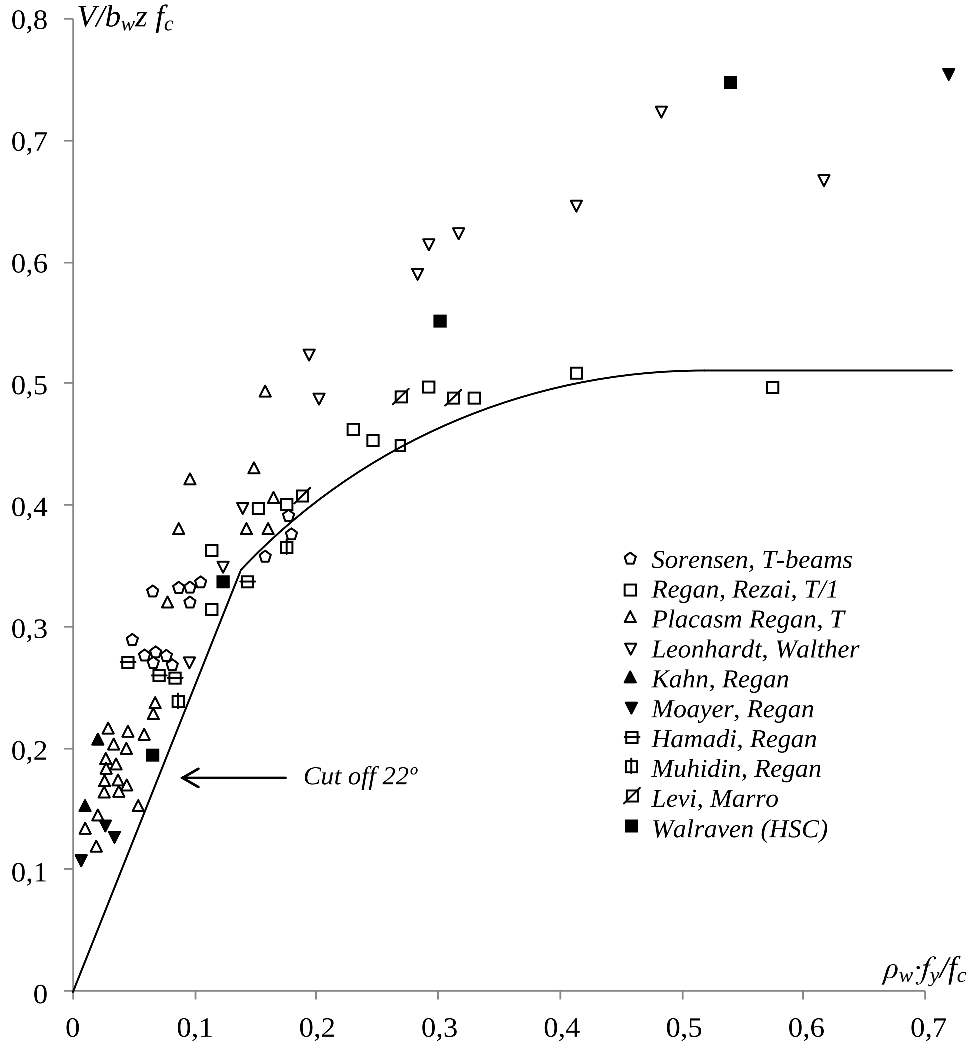


Figure 5.20 Verification of $V_{Rd,max}$ and $V_{Rd,s}$ for the lower limit $\theta = 22^o$, i.e. cot $\theta = 2.5$*, for the strut inclination. Figure is reproduced from ECP (2008a).*

As a final remark it should be noted that shear and torsion can be superimposed under the condition that the inclination, θ, is chosen to the same value. This is further explained in Section 6.4 where similarities between shear and torsion are described and discussed.

5.6 Configurations of shear reinforcement

5.6.1 Requirements in EC2

Depending on how the shear reinforcement is arranged, i.e. what type of shear reinforcement units that is used, different requirements are valid. According to Eurocode 2, Section 9.2.2, the shear reinforcement may consist of a combination of

- links enclosing the longitudinal tension reinforcement and the compression zone, see Figure 5.21
- bent up bars

- cages, ladders, etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones, SIS (2008).

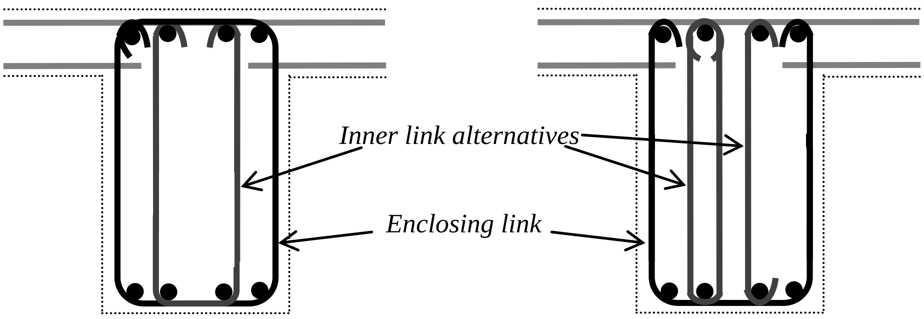


Figure 5.21 Examples of shear reinforcement links. The figure is based on SIS (2008).

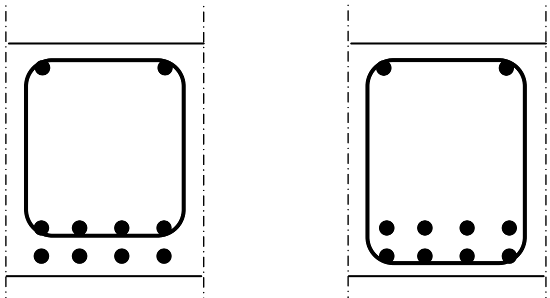
Paragraph EC2 9.2.2(4) recommends that at least 50 % of the necessary shear reinforcement should be in the form of links. However, if the shear reinforcement consists of bent up bars the required percentage of links can be taken as zero, according to the Swedish National Annex.

These rules apply also for slabs but with one exception. In Paragraph EC2 9.3.2(3) it is stated that the shear reinforcement may consist entirely of bent-up bars or of other shear reinforcement assemblies, if the following condition is fulfilled

$$|V_{Ed}| \leq \frac{V_{Rd,\max}}{3} \tag{5.45}$$

5.6.2 Explanation and derivation

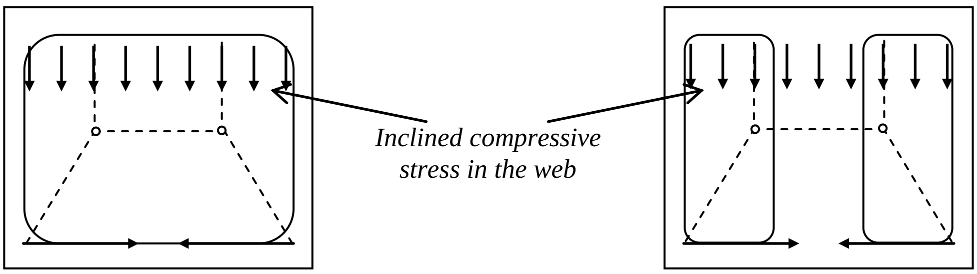
The reason why links should enclose the longitudinal reinforcement is that it, according to the general principle in Eurocode 2, is not allowed to rely on the tensile strength of concrete, Engström (2013). The general approach of Eurocode 2 is to place reinforcement in all areas where tensile stresses might occur. In order to transfer forces along a structure according to the truss model presented in Figure 3.28, Section 3.3.2, a certain amount of the vertical legs of the shear reinforcement must be connected at a joint to the longitudinal reinforcement. This is established by the use of links or by bent up bars. If other shear reinforcement assemblies or if links are placed in such a way that the longitudinal reinforcement is placed at the outside of the shear reinforcement, the tensile strength of concrete must be utilised. This is illustrated in Figure 5.22 where the difference between utilising the tensile strength of concrete and the use of links is illustrated.



a) b)

Figure 5.22 Different shear reinforcement configurations, a) the concrete must resist tensile stresses in the concrete, b) links enclosing all of the longitudinal reinforcement.

Enclosing links, as the ones shown in Figure 5.21, also resist forces in the transverse direction, i.e. across the width of the structural member. If a shear force is acting on, for instance, a wide beam, tensile stresses will occur in the transverse direction, as shown in Figure 5.23. If the shear reinforcement does not enclose the whole section, there will be a gap in which tensile forces must be resisted by plain concrete, see Figure 5.23b.



a) b)

Figure 5.23 Different shear reinforcement configurations, a) shear reinforcement enclosing the whole section and resisting the transverse tensile component, b) shear reinforcement does not enclose the whole section and the concrete must consequently resist the transverse tensile component.

It should also be added that reinforcement that encloses the longitudinal reinforcement also reduces the risk for splitting failure of anchorage, see Section 3.2.1.

5.6.3 Discussion

According to the Eurocode 2 a certain amount of the shear reinforcement should consist of links. However, in Sweden it is sufficient to only use bent up bars. It is not entirely clear from Eurocode 2 whether the links must enclose the longitudinal reinforcement in such a way that there will be a horizontal leg below the longitudinal bars. However, this is most likely the case due to the reasoning in Section 5.6.2 illustrated by Figure 5.23. This raises questions about why it in Sweden is allowed to use only bent up bars. There is no additional demand of transverse reinforcement across the width of the cross-section in case of bent up bars, which implies that the tensile strength of concrete must be utilised to resist the transverse tensile force as illustrated in Figure 5.23b. According to the previous Swedish handbook BBK 04 it is allowed to utilise some of the concrete tensile strength, Engström (2013) and Johansson (2013), which might be the reason for the deviation from the international guidelines. As explained in Section 5.4 the transverse distance between shear reinforcement legs is limited, which ensures that the stresses are evenly distributed so that high concentrated tensile forces are avoided. It can also be noted that no problems have so far occurred with structures that are designed with bent up bars as shear reinforcement without additional enclosing links, Engström (2013). It can therefore be argued that tradition and experience speaks in favour for the Swedish requirements.

In Section 5.4 it is also noted that the maximum transverse distance, $s_{t,max}$, between shear reinforcing bars is larger for slabs than for beams. This can probably also be explained based on the reasoning presented in relation to Figure 5.23. In a reinforced concrete slab the transversal distance between shear reinforcing bars can be increased, since there will always be secondary longitudinal reinforcement in the transversal direction that can resist the transverse tensile component, see Figure 5.24. This also means that it is not as important to use enclosing links in a slab, something that also is implied by Paragraph EC2 9.3.2(3), see Section 5.6.1.

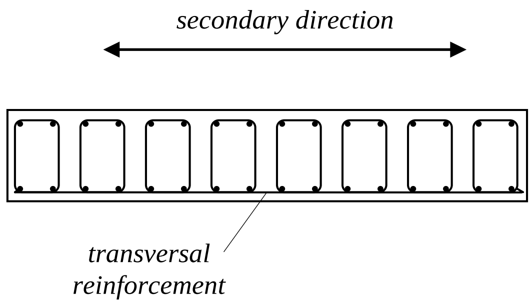


Figure 5.24 Common design of shear reinforcement in a concrete slab where the transversal reinforcement may resist the transverse tensile component created by the shear reinforcement arrangement.

The detailing of shear reinforcement in concrete structures that require large amounts of reinforcement in several directions can often be experienced as difficult. This is for instance quite common in slabs that have a limited depth and a secondary layer of bending reinforcement. This can result in difficulties in making room for shear reinforcement in form of links and at the same time ensure adequate concrete cover and spacing between layers of bars.

A question that concerns the problem of enclosing the longitudinal reinforcement in slabs has been raised in Westerberg (1995), where the shear reinforcement in a slab supported on columns is discussed. The question is if it is acceptable to design the reinforcement in the way that is illustrated in Figure 5.25a, or if the rules in Eurocode 2 require that the stirrups need to be turned 90º, as in Figure 5.25b, so that the longitudinal bending reinforcement can be enclosed?

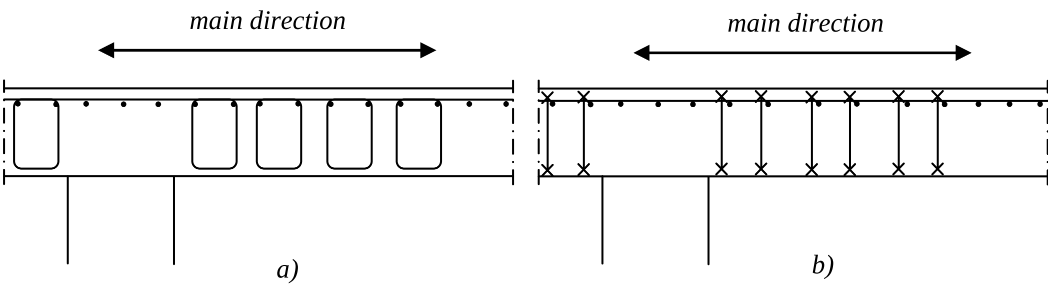


Figure 5.25 *Different shear reinforcement configurations in a flat slab, a) the longitudinal bending reinforcement is not enclosed, b) the longitudinal bending reinforcement is enclosed. The figure is based on Westerberg (1995).*

If the alternative in Figure 5.25a can be used, the horizontal legs of the shear reinforcement links will be at the same level as the main longitudinal reinforcement and the designer will thereby gain some space within the structure. It should be noted that the discussion held by Westerberg (1995) concerns flat slabs and might differ from a situation where a slab is supported on beams or walls.

A slab supported on columns with shear reinforcement should in principle act as a number of radial truss models, each and one acting approximately as a beam, Westerberg (1995). One stirrup that coincides with a radial plane then act as Figure 5.26 illustrates. The inclined compressive struts, with the location according to Figure 5.26a, cannot be taken directly by the closest stirrup. Instead it is transferred indirectly by tensile stresses in the concrete and dowel action in the longitudinal reinforcement. This will, according to Westerberg (1995), create a poorly designed node in the truss model where the full utilisation of the capacity is not enabled. However, the force located according to Figure 5.26b is captured in an efficient way. Here the node is well defined and appropriately designed.

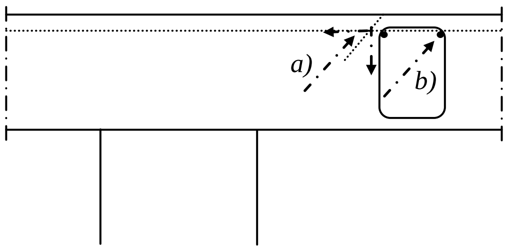


Figure 5.26 *Forces acting in a slab supported on columns, a) the inclined compressive forces cannot be taken by the stirrups, b) the inclined compressive forces can effectively taken by the stirrup. The figure is based on Westerberg (1995).*

The stirrups in a flat slab cannot, according to Westerberg (1995), be placed as in Figure 5.25a without letting the stirrup enclose the outermost reinforcement layer. This will result in that the top reinforcement needs to be moved down one bar diameter of the stirrup or increasing the thickness of the slab with the same size.

The conclusion of this is that closed stirrups located close to a radial plane of a column support is not entirely suitable as shear reinforcement, Westerberg (1995).

An additional way to design the shear reinforcement in a slab in order to gain some vertical space is according to Westerbeg (1995) to change the shape of the shear reinforcement into G-bars, so called Z-stirrups, see Figure 5.27.

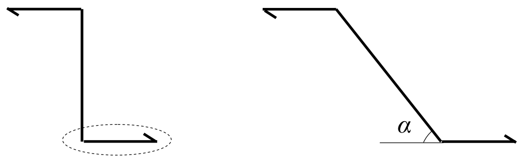


Figure 5.27 The feature of G-bars or "Z-stirrups" in case of shear reinforcement.

The detailing of this type of shear reinforcement is of high importance in order to lift the shear force in a proper way. In Figure 5.28 two different alternatives are illustrated where *Alternative 1*, according to Westerberg (1995), is the correct solution, since the compressive strut is enclosed by the reinforcement. The design in *Alternative 2* cannot capture the inclined strut. The compression force is therefore balanced by tensile stresses just as described in Figure 5.26a.

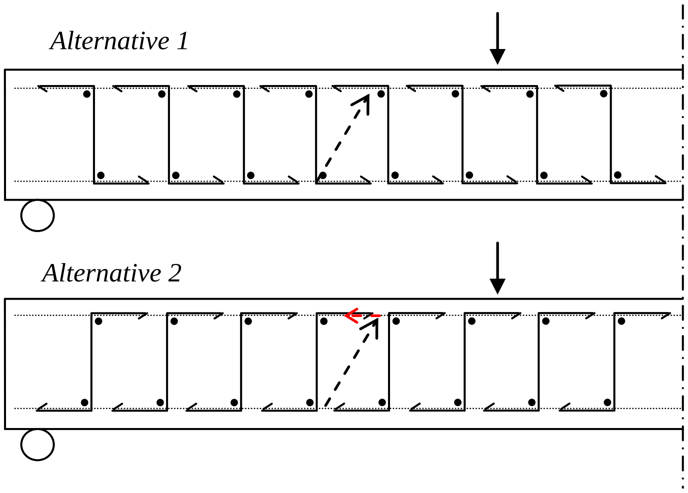


Figure 5.28 Two different solutions of how to place G-bars as shear reinforcement in a slab where Alternative 1, according to Westerberg (1995), is the correct design. The force cannot safely be balanced by tension in the next G-bar in Alternative 2.

It can be argued that it for beams is not enough to use only G-bars for shear reinforcement, since they cannot, all by themselves, take care of the resulting transverse tensile force. Hence, enclosing stirrups are required in addition, Engström (2013). However, it can also be argued that if the horizontal leg of the G-bar, see marking in Figure 5.27, is properly anchored, it is reasonable to consider the G-bars as bent up bars that can be used without any additional enclosing shear reinforcement. In this case it can also be questioned whether the horizontal leg should be spliced to the longitudinal bending reinforcement by means of lapping or if it is sufficient to use the anchorage length, l_{bd}. The difference between anchorage length, l_{bd}, and lap length, l_0, is presented in Chapter 9.

When detailing shear reinforcement in structural members it is important to also consider the buildability. This means that it should be easy and fast to place the reinforcement. Another thing that is important to consider is the risk for deviations at the construction site. It is important that the concrete cover and distance between bars are as stated on the drawings in order not to jeopardise sufficient bond conditions and anchorage capacity of the reinforcement. Such risks can be reduced by a well thought through detail design. Examples of good designs for wide beams are shown and discussed in relation to Figure 5.29 and Figure 5.30. It should be noted that these examples are taken from ACI-CRSI (2010).

Figure 5.29 shows a common configuration for wide beams. This configuration is convenient for the designer, since three closed stirrups of equal size are used. However, this will cause some difficulties for the person that place the reinforcement at the construction site. Since no stirrup surrounds the whole cross-section the overall width of the stirrup set must be measured and the construction worker must make sure that the stirrups are properly assembled to maintain the necessary total width. Since, closed stirrups are used, the longitudinal reinforcement must be inserted from the short edges of the beam, unless the shear reinforcement can be closed after placing the longitudinal reinforcement. Closed stirrups are convenient to use, if the reinforcement can be preassembled and lifted into the form work by a crane. However, there is a risk that the net width of the reinforcement cage might change, if it is lifted which can result in inadequate concrete cover.

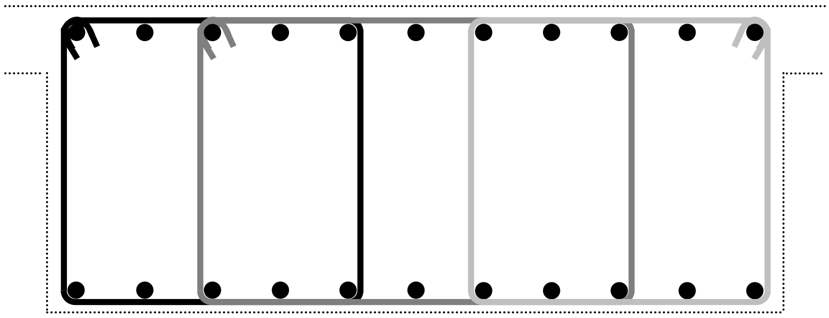
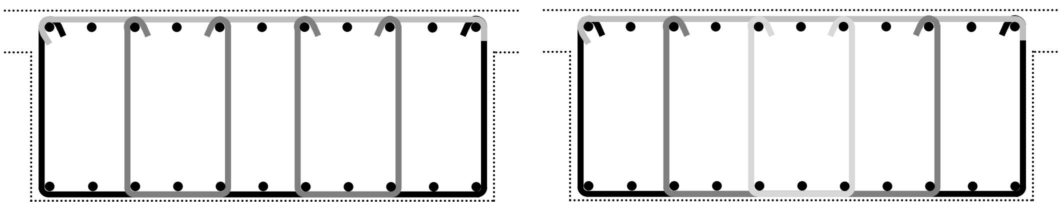


Figure 5.29 Common configuration of stirrups in wide beams according to ACI-CRSI (2010).

Figure 5.30 presents two reinforcement configurations that are advantageous in comparison to the configuration in Figure 5.29 because of two reasons. Firstly, the entire cross-section is enclosed by one stirrup which will maintain the concrete cover. Secondly, the open top enables an easier placing of the longitudinal reinforcement since it can be placed from above. The internal stirrups can be formed according to any of the two configurations in Figure 5.30. However, in Figure 5.30a the same type of stirrups are used which can facilitate the work at the construction site.



a) *b)*

Figure 5.30 Alternate configurations of beam stirrups according to ACI-CRSI (2010).

If the shear reinforcement for some reason has to be closed, this can be achieved by an additional cap bar. It can be questioned whether these two examples are allowed to be used in a beam that is subjected to a torsional moment. According to ACI-CRSI (2010) they can. However, the bends that anchor the bars should then probably be larger than 90º, read more about this in Chapter 6.

5.7 Load close to supports

5.7.1 Requirements in Eurocode 2

In Section EC2 6.2.3, concerning members requiring design shear reinforcement, there are some additional requirements for structural members with loads applied close to supports. According to Paragraph EC2 6.2.3(8) it is allowed to reduce the contribution to the design shear force, V_{Ed}, from a load that is placed on the upper side of a member within a distance a_v to the support section. The contribution to the shear force of a load that is acting on the structural member may be reduced by a factor β defined in Equation (5.46).

$$\beta = \frac{a_v}{2d} \qquad (5.46)$$

a_v	clear distance between load and support, see Figure 5.31
d	effective depth with regard to the longitudinal bending reinforcement

This requirement is only valid if the load is placed within certain distances to the support section. This can be expressed as

$$0.5d \leq a_v \leq 2d \qquad (5.47)$$

It is also stated in Eurocode 2 that if a_v is smaller than $0.5d$ the value on a_v should be set to $0.5d$, which implies that the reduction should be made with a factor $\beta = 0.25$.

The reduced shear force should satisfy the condition in Expression EC2 (6.19), see Equation (5.48).

$$V_{Ed} \leq A_{sw} f_{ywd} \sin\alpha \qquad (5.48)$$

The expression $A_{sw} \cdot f_{ywd}$ is in this case the resistance of the shear reinforcement within the central $0.75a_v$ that also crosses a possible inclined shear crack between the edge of load application and edge of support, see Figure 5.31.

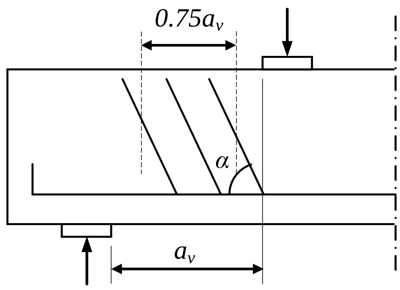


Figure 5.31 The contribution to the shear force from a load might be reduced, if this load is acting within a distance, a_v, from the support. The shear reinforcement should be placed centered between the edge of the support and edge of load application. The figure is based on SIS (2008).

The total shear force, V_{Ed}, calculated without reduction should always be smaller than resistance of web shear compression failure, $V_{Rd,max}$, see Section 5.3.1.

5.7.2 Explanation and derivation

The reduction of the shear force can be determined as illustrated in Figure 5.32 by using a fictitiously reduced load q_{red} within a distance of $2d$ from the support edge, Betongföreningen (2010a).

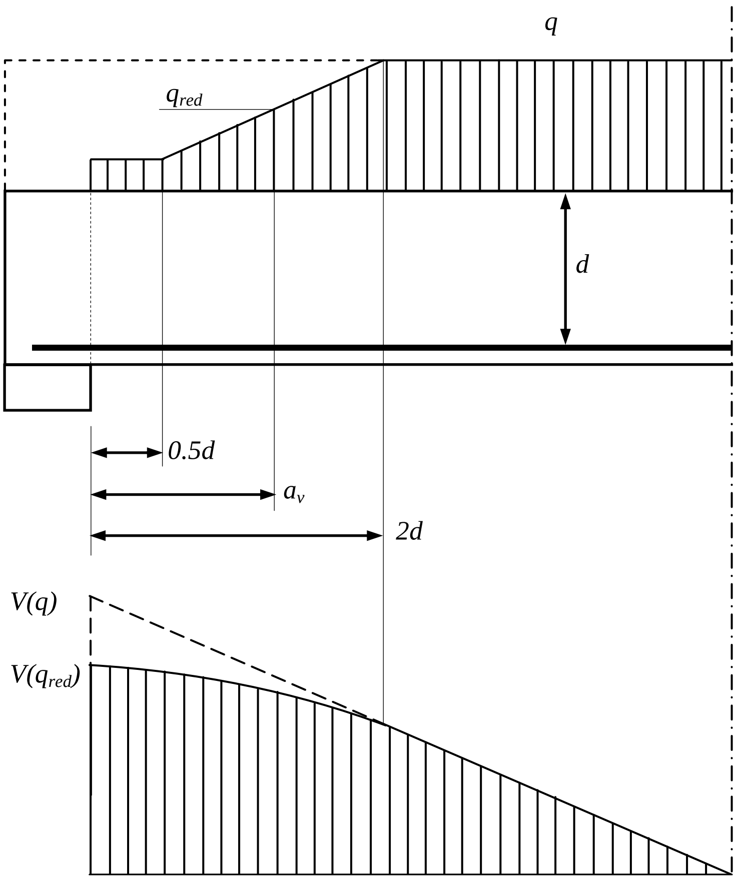


Figure 5.32 Illustration of the reduction of the shear force due to load application close to supports. The figure is based on Betongföreningen (2010a).

The reason why the shear force from loads near the support, can be reduced is because of the possibility for the load to be resisted in two ways when the point of load application is sufficiently close to the support, Engström (2013). One part of the load can go directly to the support in a compressed strut being part of a tied arch, see Figure 5.33a. The remaining part will be resisted by means of a truss model, as explained in Section 5.2, where the vertical shear reinforcement, illustrated by the vertical tie in Figure 5.33b, therefore not have to be designed for the entire shear force V_{Ed}. If the load is applied at a distance larger than $2d$ from the support section, it cannot be considered to go directly to the support in arch action and the shear reinforcement must therefore be designed for the entire shear force.

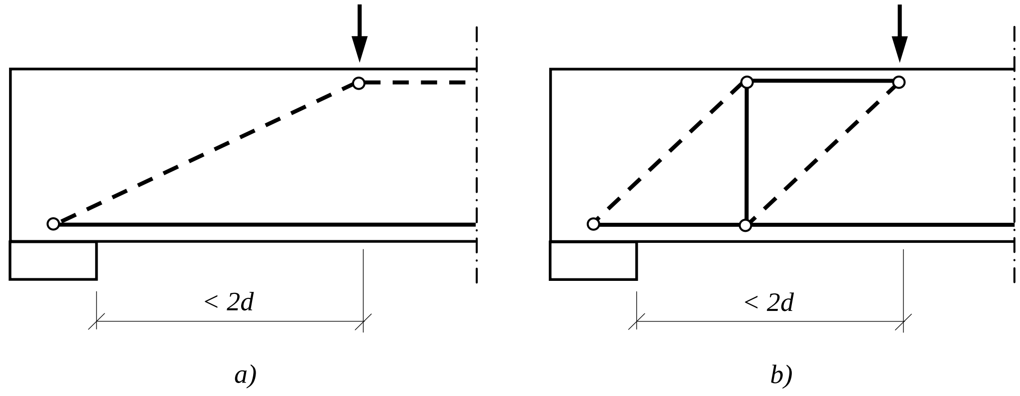


Figure 5.33 When the load is applied at a distance closer to the support than 2d the load can be resisted in two different ways simultaneously, a) one part of the load is resisted by arch action, b) the remaining part is resisted by means of a truss model with shear reinforcement.

The load path can be seen as a combination of a truss and a tied arch model, ECP (2008a), which is illustrated in Figure 5.34.

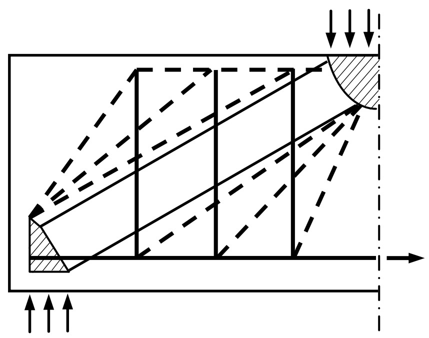


Figure 5.34 The load path can be described as a combination of a truss- and tied arch model. The figure is based on ECP (2008a).

According to ECP (2008a) the factor β, in Equation (5.46) is derived from a comparison of different test results. Regan (1998) concluded that the value used in Eurocode 2 today is sufficient for most structural members. However, it should be noted that the factor is obtained for structures without any shear reinforcement. Nevertheless, is seems reasonable that the load that can be tied arch action should be the same also for reinforced concrete structures with shear reinforcement. This is also implied from the fact that the expression $\beta = a_v / 2d$ should be used also for design of structures without shear reinforcement according to Paragraph EC2 6.2.2(6).

A reason why only the reinforcement in the central part between the edge of load application and edge of support should be considered can be found in ECP (2008a). According to ECP (2008a) measurements made by Asin (2000), on shear reinforcement close to supports, showed that the reinforcement just adjacent to loads and supports could not be considered to be effective, since it did not reach yielding.

This can perhaps be illustrated by a strut and tie model that describes the actual stress field in a better way, Engström (2013), see Figure 5.35. In order for the stress field to change direction vertical ties representing shear reinforcement are needed in the central part between the edge of load application and edge of support.

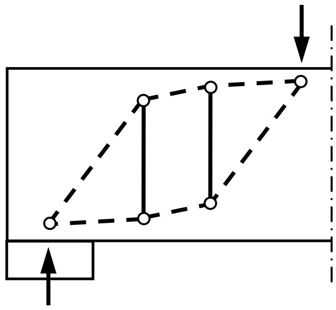


Figure 5.35 The stress field illustrated by a strut and tie model needs vertical ties in order to change direction.

Another reason why it is a good thing to place the reinforcement in the central part of the distance a_v is that there might be a risk that the shear reinforcing bar misses the shear crack, if the vertical bar is placed too close to the edge of the support, Johansson (2013), see Figure 5.36.

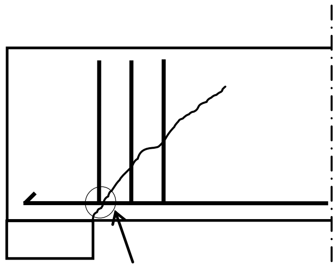


Figure 5.36 If shear reinforcement is placed too close to the support, it might miss the crack and therefore be inefficient.

However, it should be noted that transverse reinforcement might be needed in the support region for other reasons, such as for enclosing the longitudinal reinforcement in order to provide favourable anchorage conditions.

5.7.3 Discussion

In Eurocode 2 nothing is mentioned about what type of load that the reduction of design shear force applies to. According to Betongföreningen (2010a) the figures in Eurocode 2 implies that it is a concentrated load that is considered, see Figure 5.33. However, it can be argued that the rules presented in Section 5.7.1 applies for all types of loads, Johansson (2013). The illustration of the reduction of the shear force from Betongföreningen (2010a) in Figure 5.32 also implies this.

It is unclear if the rules concerning the placement of the reinforcement applies also to distributed loads. Much speaks for that placing the shear reinforcement within a distance of $0.75a_v$ in the central parts between the applied load and the edge of support only is required for concentrated loads. This is implied from the explanation of the requirement in Section 5.7.2.

Something that is not clear from Eurocode 2 is why the lower limit of a_v is set to $0.5d$. A load placed closer to the support than a distance d does not have to be lifted over any shear cracks, since shear cracks forms with an inclination of approximately 45º and cannot be steeper than this, see Figure 5.37. It could therefore be argued that when a load is applied closer than a distance d from the support, the entire load can be resisted by arch action as in Figure 5.33a.

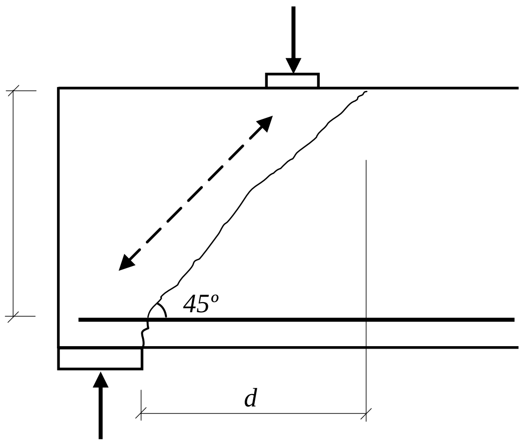


Figure 5.37 It can be argued that loads placed closer to the support than a distance d can be fully resisted by tied-arch action.

It can be argued that loads placed closer to supports than a distance d should be designed for by strut and tie models rather than by truss analogy. The lower limit of a_v equal to $0.5d$ therefore becomes misleading and should perhaps be changed to d instead. This argument is also supported by Paragraph EC2 6.2.1(8) in Eurocode 2 that states that beams subjected to predominantly uniformly distributed loads does not need to be checked at a distance less than d from the face of the support. However, a concentrated load might affect the stress field in some way that cannot be properly described by a simple tied arch model. It is always a possibility that test results have provided a foundation to the requirements in Eurocode 2, even though no such information has been found. Consequently, this needs to be further investigated in order to know if the rules can be altered or not.

In Betongföreningen (2010a) the case when the distance a_v is smaller than $0.5d$ is discussed. From Eurocode 2 it is not clear if reinforcement should be placed for such situations or not. According to Betongföreningen (2010a) it is better to place horizontal reinforcement instead of vertical shear reinforcement for loads applied so close to the support. This is illustrated by a model in Figure 5.38.

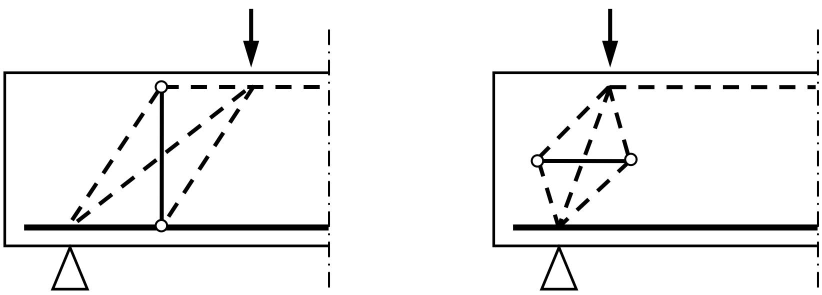


Figure 5.38 Betongföreningen (2010a) suggests that horizontal reinforcement should be placed instead of vertical stirrups, if the load is applied closer than 0.5d to the support.

It could be argued that beam ends where concentrated loads act close to supports should be considered as discontinuity regions and therefore should be designed by means of strut and tie models. However, according to ECP (2008a) there are many arguments in favour for the formulation in Eurocode 2. From the figures presented in Section 5.7.2 it can also be argued that the rules presented in Eurocode 2 to some extent does consider the strut and tie model.

5.8 Suspension reinforcement

5.8.1 Requirements in Eurocode 2

Special attention is made for detailing of reinforcement at indirect supports in Eurocode 2, Section EC2 9.2.5. In Paragraph EC2 9.2.5(1) it is stated that when a beam is supported by another beam instead of a wall or column, extra reinforcement, in addition to that required for other reasons, should be placed to resist the mutual reaction, SIS (2008).

In the same section, EC2 9.2.5, but in the second paragraph, it is stated that the supporting reinforcement between two beams should consist of links surrounding the principal reinforcement of the supporting member. Some of these links may be distributed outside the volume of the concrete, which is common to the two beams. This type of reinforcement is also exemplified in Figure EC2 9.7, which is reproduced here in Figure 5.39.

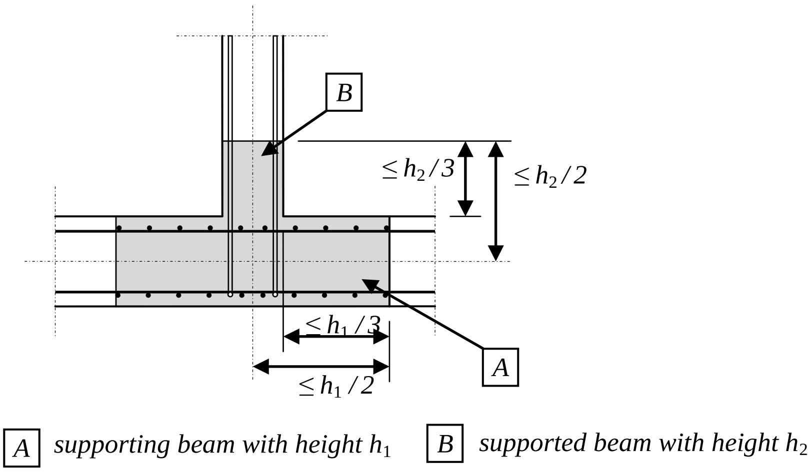


Figure 5.39 Placing of supporting reinforcement in the intersection zone of two beams (plan view). The figure is based on SIS (2008).

In addition to the two paragraphs presented above, Paragraph EC2 6.2.1(9) states that where a load is applied near the bottom of a section, sufficient vertical reinforcement to carry the load to the top of the section should be provided in addition to any reinforcement required to resist shear. This actually concerns the same matter, but no references between the three paragraphs are made in Eurocode 2.

5.8.2 Explanation and derivation

In order to understand the meaning of the two paragraphs in Section EC2 9.2.5 it is necessary to give the definition of an indirect support. An indirect support is when a member is supported by hanging it into another member; i.e. the supported beam is not placed on top of a column or a wall. Instead it is for instance supported by another beam, the supporting beam, which is on the same vertical level as the supported beam. In Eurocode 2 Section EC2 9.2.1.4 Figure EC2 9.3 shows the difference between a direct and an indirect support, see Figure 5.40.

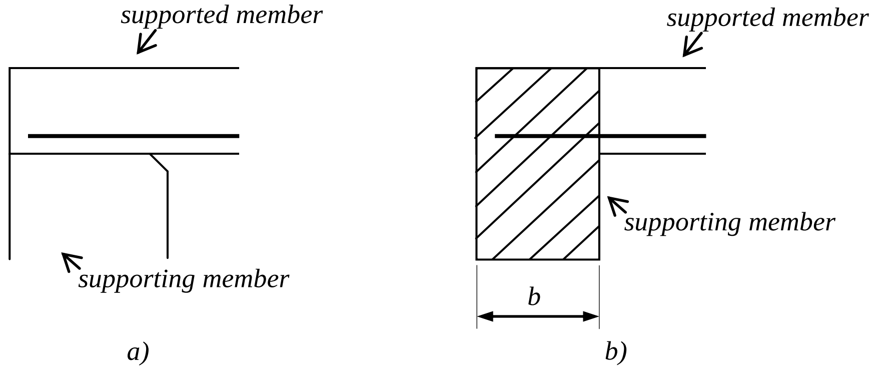


Figure 5.40 Difference between direct and indirect support according to SIS (2008), a) direct support, e.g. a beam supported by wall or column, b) indirect support, e.g. a beam intersecting another supporting beam.

The additional reinforcement that must be placed in case of indirect supports are in fact the supporting reinforcement, or suspension reinforcement, that is described in Paragraph EC2 9.2.5(2), see Section 5.8.1. Figure EC2 9.7, see Figure 5.39, shows a plan view of the intersection between two beams that are connected to each other as in Figure 5.40b. The suspension reinforcement is the links shown in the supporting beam, A, illustrated in Figure 5.39.

The reason why suspension or supporting reinforcement must be placed at indirect supports can be explained by a truss model. Figure 5.41 illustrates the load path for a member that is supported on top of another member, e.g. a direct support. When the load comes in from above it can be resisted according to truss analogy.

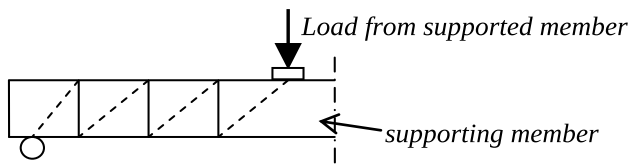


Figure 5.41 Load path in case of a direct support. The load from the supported member acts from above on the supported member and can be resisted by truss action.

However, in case of an indirect support the load is applied at the bottom or at the side of the supporting member, see Figure 5.42. In such a situation the load from the supported member needs to be lifted in order to enable the load path along the supporting member. In case of a load actually hanging from a member it often comes naturally to add additional suspension reinforcement, but for indirect supports this is not always as clear.

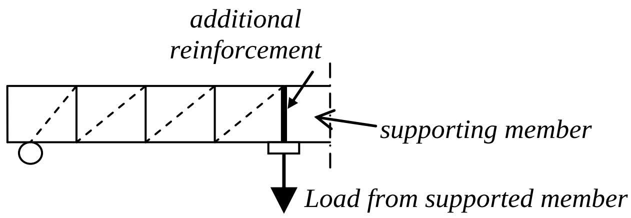


Figure 5.42 Load path in case of an indirect support. The load from the supported member acts from below on the supported member and needs to be lifted to the top of the supporting member to enable a load path to the end support.

In order to better understand Figure 5.39 an additional figure from FIP (1999) has been reproduced here in Figure 5.43b, showing the sectional view of the intersection between the two beams. Figure 5.43a shows a global view of the two beams. Figure 5.43b also shows how the suspension reinforcement is designed in the intersection and how the support reaction of beam B is lifted to the top of member A.

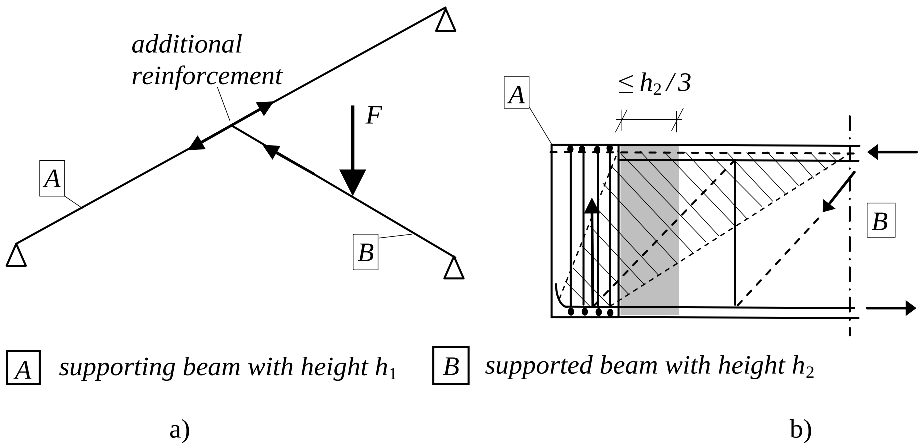


Figure 5.43 Additional reinforcement with the purpose to lift up the support reaction coming from below from the secondary beam, a) global view of the two beams in Figure 5.39, b) sectional view of the two beams in Figure 5.39. Figure b) is based on FIP (1999).

It should be noted that the suspension reinforcement should be placed in addition to any other reinforcement. The reason for this is that the reinforcement that is placed within the section for other reasons may already be utilised as for instance shear reinforcement.

5.8.3 Discussion

In reality it can be difficult to identify indirect supports. An indirect support does not necessarily have to be between two beams. It can for instance be at the intersection of a beam and a slab, if the slab is not supported on top of the beam, SIS (2008). However, if references between Paragraphs EC2 9.2.5(1), EC2 9.2.5(2) and EC2 6.2.1(9), see Section 5.8.1, are added in Eurocode 2, it would probably be easier for the designer to understand the reason for the requirements and facilitate the application of the rules also for other situations.

In order to increase the understanding and provide more knowledge for when it is necessary to design suspension reinforcement, two possible situations are presented below.

In Figure 5.44 a plan view of a part of a structure is illustrated. The load path from the slab to the secondary beam (the supported beam) to the main beam (the supporting beam) and further down to the supports is shown. The connection between the secondary beam and the main beam is an indirect support where the former is supported by hanging into the other.

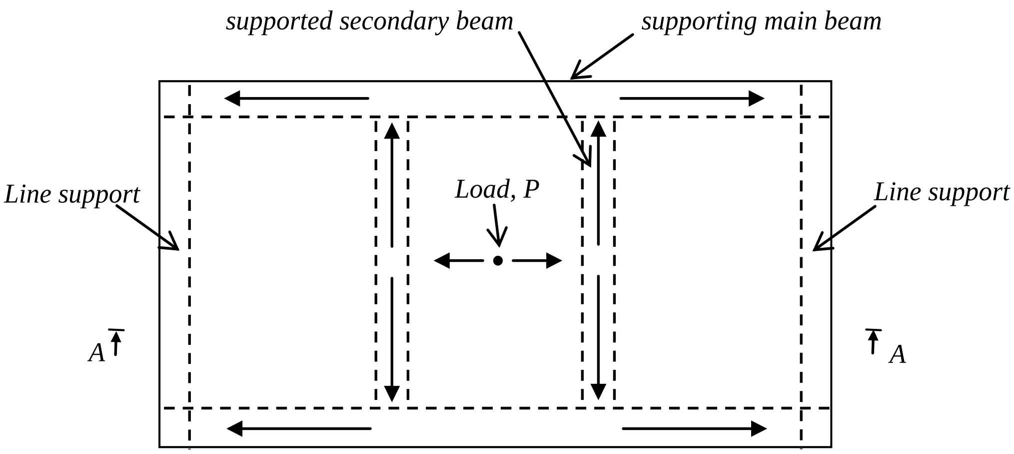


Figure 5.44 Plan of a concrete structure. The slab carries the load in the transverse direction to the supported secondary beam to the supporting main beam and further down to the supports.

BBK 04, Boverket (2004), describes the need for suspension reinforcement in a more pedagogic way than Eurocode 2. In Section BBK 04 3.7.1 an example of a structure with indirect supports is shown, see Figure 5.45. Figure 5.45a can be seen as a sectional view, A-A, of the structure in Figure 5.44. Figure 5.45b on the other hand shows the connection between the main and secondary beam.

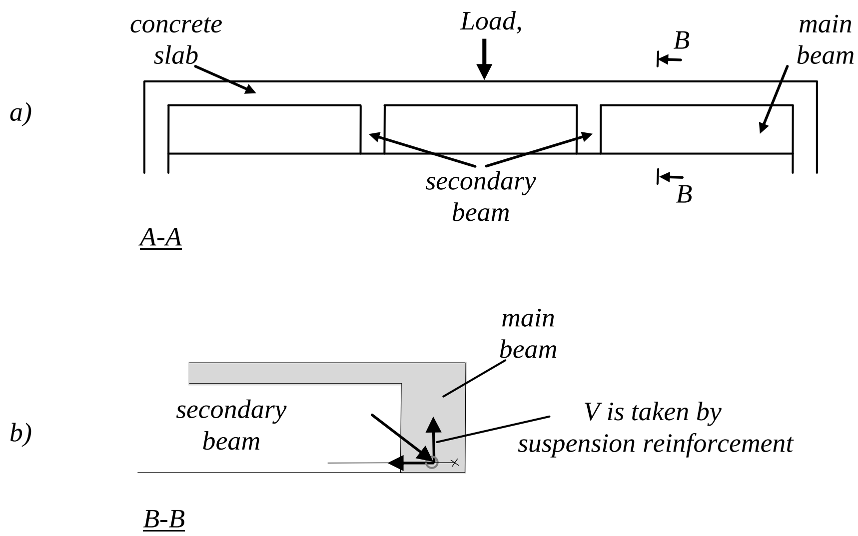


Figure 5.45 Example of a case where load is applied at the bottom of a beam, a) section of the main beam loaded by two secondary beams, b) schematical view of connection where the load should be presumed to act on the main beam. The figure is based on Boverket (2004).

Another example where there have been problems with insufficient, if any, suspension reinforcement is in trough bridges, Johansson (2013). A trough bridge is often used for railways. It is designed with two main beams on each side of a slab, which here is called secondary member, see Figure 5.46.

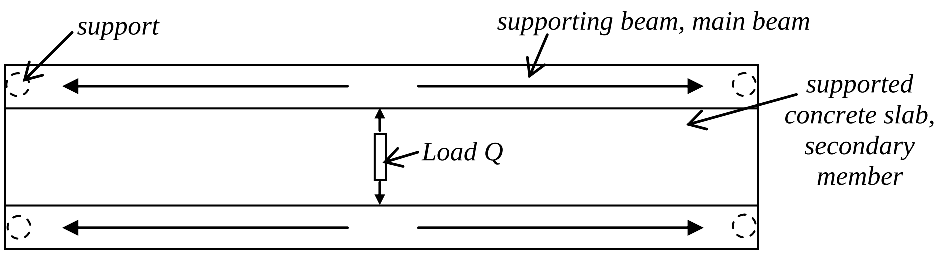


Figure 5.46 Plan of a trough bridge where suspension reinforcement is required to lift the load from the slab to the supporting main beam.

The secondary member, i.e. the slab, is supported by the main beams. Hence the main beam is loaded from the bottom, why the load needs to be lifted up and suspension reinforcement is required, see Figure 5.47.

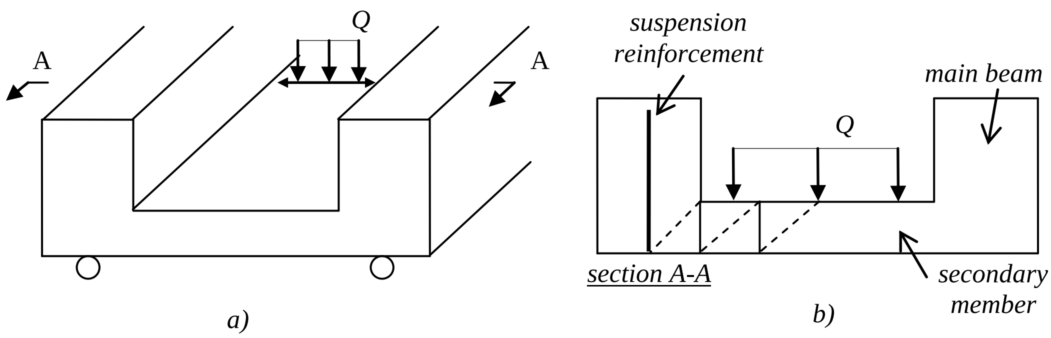


Figure 5.47 Through bridge loaded by a distributed load, Q, a) overview, b) section A-A illustrates how the load is resisted in the secondary member and lifted to the main beam where the load path continues to the supports.

The suspension reinforcement should be designed according to Figure 5.48a where the load Q is lifted up in the main beam in a correct manner. The only difference between Figure 5.48a and Figure 5.48b is the horizontal leg of the suspension reinforcement. In Figure 5.48b the load cannot be lifted all the way to the top of the main beam by the suspension reinforcement, since a part of the reinforcement must be used to anchor the steel bar to the concrete. Only the lower part of the main beam will therefore contribute to the load bearing capacity. In Figure 5.48a the suspension reinforcement is instead anchored by the horizontal leg and the load can be lifted to the top of the beam. Hence, this small modification of the reinforcement, by adding about 50 cm extra steel, will contribute to a much more effective bridge with an increased bearing capacity.

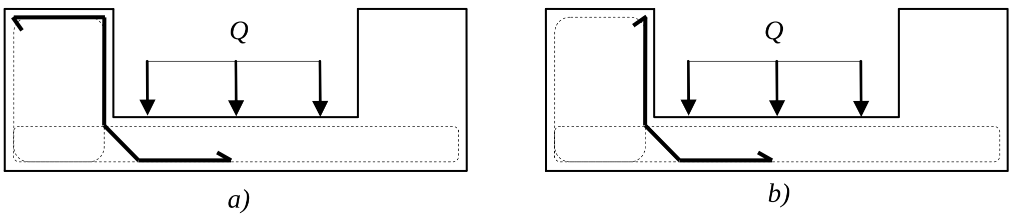


Figure 5.48 Suspension reinforcement in a through bridge, a) correct design where the horizontal leg provides sufficient anchorage, b) not correct design.

It should be noted that if the trough bridge is loaded in a section where the main beam is supported, no additional suspension reinforcement is necessary, since the load will in such a case be resisted directly by the support, see Figure 5.49.

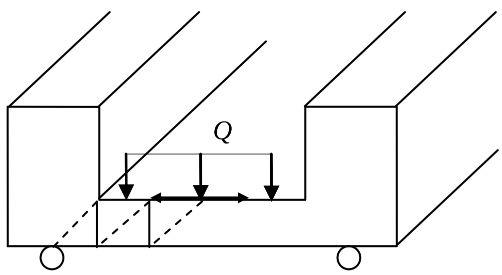


Figure 5.49 When the load is applied close to the support of the main beam no additional reinforcement is required.

It should be emphasised that suspension reinforcement should be placed in addition to the ordinary shear reinforcement and that it should be anchored in such a way that the load can be lifted to the top of the supporting member. However, this it is not apparent from Eurocode 2 since Paragraph EC2 9.2.5(1) does not use the term suspension or supporting reinforcement. The relationship between the two paragraphs in Section EC2 9.2.5 can be more clarified in order to help the designer. A reference to Figure EC2 9.3 in association with Figure EC2 9.7 may also provide some additional understanding.

6 Design and detailing for torsion

6.1 Structural response and modelling

Two different types of torsion can be distinguished; circulatory (or St Venant) and warping torsion. In this chapter only the former will be presented where the torque is resisted by a closed flow of shear around the cross-section, *fib* (2010). In case of warping torsion the cross-section is open, see Figure 6.1.

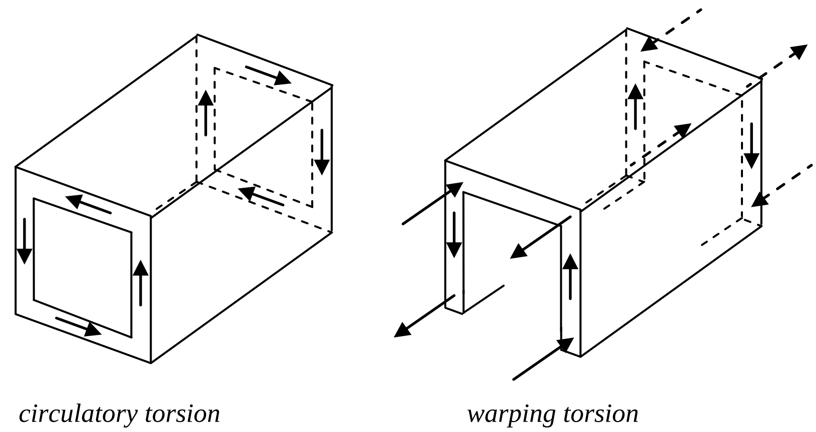


Figure 6.1 Circulatory and warping torsion. The figure is based on fib (2010).

Sections of members that act efficiently in torsion are for instance solid circular and rectangular sections, *fib* (2010). Closed box sections have especially good torsional stiffness and act primarily in circulatory torsion. Even if the section of the member is actually solid, the torsional resistance will be calculated on the basis of a thin-walled closed section, meaning that the inner part of the concrete core is considered as basically inactive when the section is subjected to torsional moment.

When a reinforced concrete member is subjected to a torsional moment, shear stresses will be created that will result in principal tensile stresses at an angle of approximately 45° to the longitudinal axis, Mosley *et al.* (2007). When the tensile stresses reach the tensile strength of concrete, diagonal cracks will develop like a spiral around the member, see Figure 6.2.

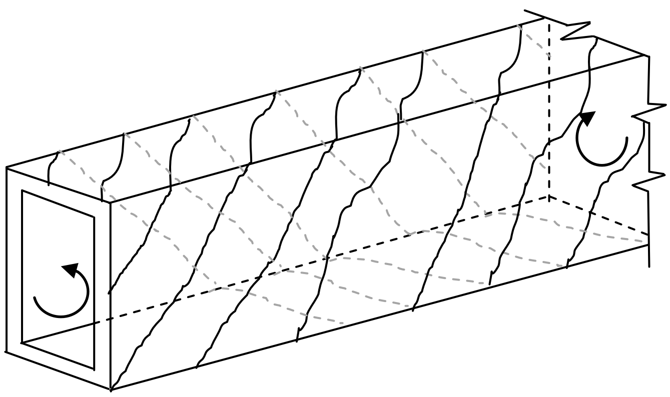


Figure 6.2 Diagonal cracks will develop like a spiral around a concrete member subjected to torsional moment.

Response in torsion reminds to response in shear but the difference is that due to torsional moment, inclined cracks occur also in the top and bottom of the member, not only on the vertical sides. Another difference is also that the shear cracks has different directions in the two vertical sides. Response in torsion is usually modelled using a hollow box cross-section where each box wall can be seen as the cross-section of a beam subjected to shear force. In Figure 6.3 a rectangular concrete cross-section subjected to a torsional moment, T_{Ed}, is modelled by a hollow box section of thickness t_{ef}. Each wall resists a part of the torsional moment in form of a shear force, V_i, acting in the plane of each wall. The torsional moment can also be described as a shear flow, q, which also can be described as the shear force per unit length of the circumference, u_k, of the box section. This is explained more in Section 6.6.2.

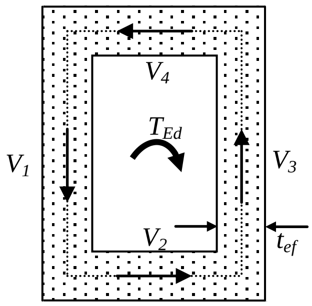


Figure 6.3 Rectangular concrete cross-section subjected to torsional moment resisted by a shear flow in an external shell, or box, with thickness, t_{ef}.

Due to the similarities to shear force the desired behaviour of a member subjected to torsional moment is the same as for a member subjected to shear force, i.e. the compressed concrete between the inclined cracks should not be crushed and the reinforcement must be able to balance the compressed struts in a safe load path to the supports. It is important to notice that torsion is more of a 3D problem, while shear can be considered in 2D only. Torsional reinforcement therefore consists of transverse reinforcement, in form of closed links surrounding the cross-section, and longitudinal reinforcement. In the chapter concerning shear, see Section 5.1, it is described how the reinforcement in combination with the concrete resist the external load by truss-action, which is also applicable in the case for a reinforced concrete member loaded in torsion, but with a few modifications, see Figure 6.4.

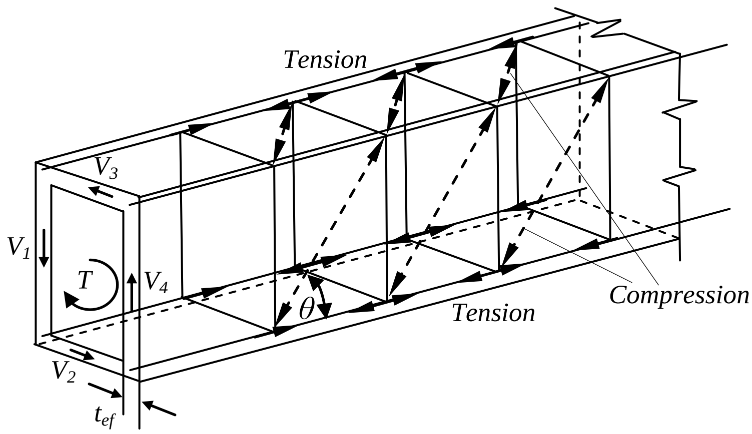


Figure 6.4 Structural model of a reinforced concrete member subjected to torsion. The figure is based on Mosley et al. (2007).

It should be noted that transverse reinforcement in form of closed links, in a section where the torsional shear stress is large enough to cause cracking, must be provided to resist the full torsional moment, just as the shear reinforcement in case of shear force, Mosley *et al.* (2007). However, it should be noted that shear reinforcement is not always required.

A difference to design for shear force is that the resulting longitudinal tensile force due to inclined cracks, that is explained and derived in Section 5.5, cannot be divided in two parts in case of torsional moment, but should be uniformly distributed around the section. The longitudinal reinforcement that resists the resulting tensile force should in case of torsional moment be distributed along each side of the cross-section. Figure 6.5 shows how the resulting tensile force is divided on *n* reinforcing bars along the height of the wall as shown in Figure 6.4.

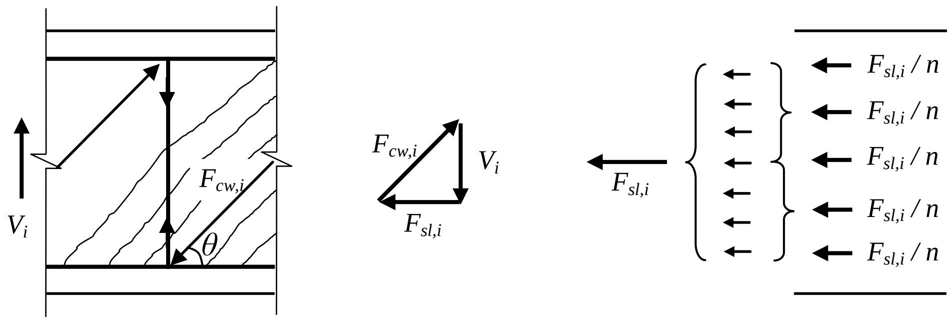


Figure 6.5 The compressive force $F_{cw,i}$, representing the inclined compressive stress field between cracks, will cause a resulting tensile force, $F_{sl,i}$, that is distributed on n reinforcing bars along wall i of the cross-section.

6.2 Longitudinal torsional reinforcement

6.2.1 Requirements in Eurocode 2

The expression that is used to design the longitudinal reinforcement, with regard to the influence of torsional moment only, is described by Expression EC2 (6.28) in Eurocode 2, see Equation (6.1) and Figure 6.6.

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta \tag{6.1}$$

A_k area enclosed by the centre-lines of the connecting walls, including inner hollow areas

u_k perimeter of the area A_k

f_{yd} design yield strength of the longitudinal reinforcement A_{sl}

θ angle of compression strut

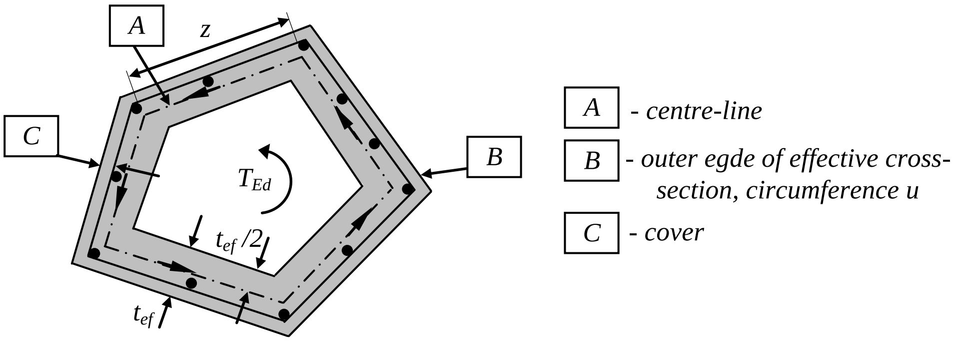


Figure 6.6 Notations and definitions used in Section EC2 6.3. The figure is based on SIS (2008).

According to Paragraph EC2 6.3.2(3) may the longitudinal reinforcement for smaller sections be concentrated to the ends of each side length, z_i, of a cross-section. However, it is stated in Eurocode 2 that the longitudinal reinforcement in general should be distributed along each side length. In Paragraph EC2 9.2.3(4) this requirement is further developed. It is stated that the longitudinal reinforcement should be evenly distributed with a maximum spacing of 350 mm under the condition that the requirement of one reinforcing bar at each corner is fulfilled.

Paragraph EC2 6.3.2(3) states that the longitudinal reinforcement needed to resist torsion in compressive chords may be reduced with the available compressive force. The additional longitudinal reinforcement that is required to resist torsional moment should in the same manner be added to the other longitudinal reinforcement in tensile chords.

6.2.2 Explanation and derivation

In order to derive the expression that determines the amount of longitudinal torsion reinforcement, A_{sl}, in Eurocode 2, see Equation (6.1), an expression for the shear flow, q, in relation to the torsional moment, T, must be determined. The rectangular cross-section in Figure 6.7 is used in the derivation.

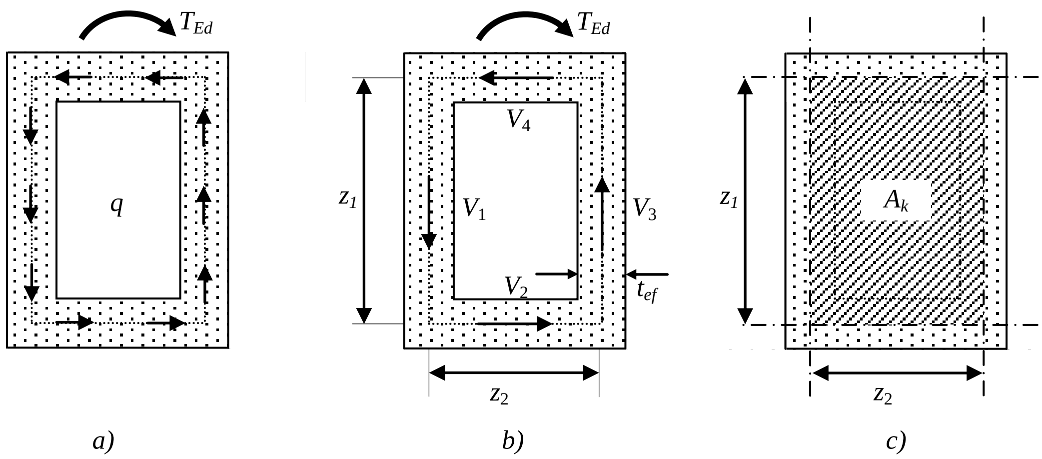


Figure 6.7 Rectangular cross-section where the shear flow, q, due to torsion can be seen as shear forces V_1, V_2, V_3 and V_4 acting in each wall of a hollow box section.

In Section 6.1 it was explained that the torsional moment can be modelled as shear forces, V_i, acting in the plane of all the walls, i, of a hollow box section. The hollow box section is inserted in the original solid cross-section, in this case a rectangular cross-section. The shear force created by the torsional moment will in the following be referred to as torsional shear force. The torsional shear force that is acting on each wall of the hollow box section can be calculated by multiplying the shear flow, q, with the length, l_i, of the corresponding wall.

$$V_i = q \cdot l_i \tag{6.2}$$

Due to symmetry it can be said that the vertical torsional shear forces, V_1 and V_3, are equal in size and that the horizontal torsional shear forces, V_2 and V_4 also are equal. The lengths and lever arms can be written by means of z_1 and z_2 defined in Figure 6.7b.

$$\begin{aligned} V_1 &= V_3 = q \cdot z_1 \\ V_2 &= V_4 = q \cdot z_2 \end{aligned} \tag{6.3}$$

The torsional moment, T, is the sum of all torsional shear forces multiplied with the lever arm, e_i, from the centre of the cross-section to the position of the torsional shear force and can be calculated as

$$T = \Sigma(V_i \cdot e_i) \tag{6.4}$$

Combined with Equation (6.2) the torsion, T, can be described as

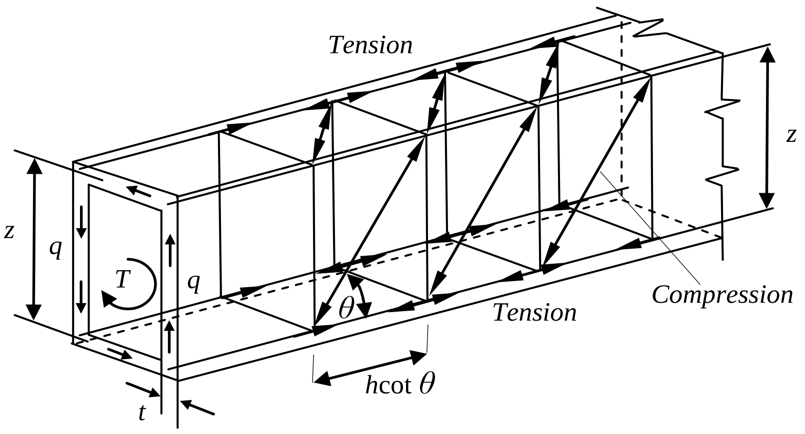
$$T = \Sigma(q_i \cdot l_i \cdot e_i) = q(2\frac{z_1}{2}z_2 + 2z_1\frac{z_2}{2}) = q(z_1 z_2 + z_2 z_1) = 2qA_k \tag{6.5}$$

A_k area enclosed by the centrelines of the walls including the hollow area, see Figure 6.7c.

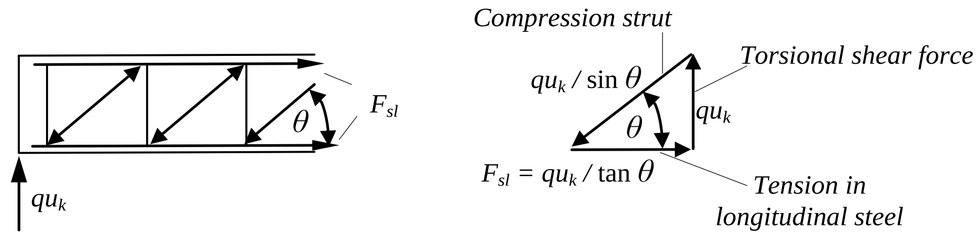
which result in the torsional shear flow, q

$$q = \frac{T}{2A_k} \tag{6.6}$$

Equation (6.1) is derived in the following text, based on Mosley *et al.* (2007). The shear force is in each section resisted by inclined compression. The outward normal force, which is the longitudinal component of the inclined compression, must be balanced by reinforcement. This longitudinal tensile force is named, F_{sl}, see Figure 6.8b.



a)



b)

Figure 6.8 *Structural model of a reinforced concrete member subjected to torsion, a) overview of truss model, b) forces acting on the body (one face shown representative of all four faces). The figure is based on Mosley et al. (2007).*

The torsional shear flow, q, can also be described as the torsional shear force per unit length of the circumference, u_k, of the hollow box section. Hence, the force, $q \cdot u_k$, will be taken by the transverse component of the compressed concrete between the inclined cracks. Just as for shear, see Section 5.5, the longitudinal tensile component of the force resisted by the inclined concrete struts between the cracks, must be balanced and, in this case, by longitudinal reinforcement. This component can be derived from the triangle of forces in Figure 6.8b.

$$F_{sl} = \frac{qu_k}{\tan\theta} \tag{6.7}$$

Inserting the expression for q derived in equation (6.6) the longitudinal tensile force is expressed as

$$F_{sl} = \frac{Tu_k}{2A_k \tan\theta} \tag{6.8}$$

The area, A_{sl}, of the longitudinal reinforcement can carry a force equal to

$$F_{sl} = A_{sl} f_{yd} \tag{6.9}$$

f_{yd} design yield strength of reinforcement

The required area, A_{sl}, of the longitudinal reinforcement for the design load is hence given by

$$A_{sl} f_{yd} = \frac{Tu_k}{2A_k \tan\theta} = \frac{Tu_k \cot\theta}{2A_k} \tag{6.10}$$

If the torsional moment, T, in Equation (6.10) is set to the design value of the load effect, T_{Ed}, the required amount of reinforcement needed to balance the outward longitudinal normal force can be determined from expression (6.11), which is the same expression as in Eurocode 2, see Equation (6.1).

$$\frac{A_{sl} f_{yd}}{u_k} = \frac{T_{Ed} \cot\theta}{2A_k} \tag{6.11}$$

6.2.3 Discussion

The area, A_{sl}, derived in Equation (6.11), is the total amount of longitudinal reinforcement that should be distributed in the whole cross-section in order to sustain the design torsional moment, T_{Ed}. This is also the reason why it is expressed as ΣA_{sl} in Expression EC2 (6.28), see Equation (6.1).

The most important difference between torsional moment and vertical shear force is that the torsional shear force will create a lever arm from each box wall that needs to be taken into account. The lever arm is included in A_k as shown in Equation (6.5).

The longitudinal reinforcement that is needed in design for vertical shear force is in Eurocode 2 expressed as an increase of the tensile force that should be taken by the longitudinal bending reinforcement. However, this additional tensile force is not transformed directly into an additional reinforcement amount but is instead considered in the curtailment of the longitudinal bending reinforcement, see Sections 5.5 and 9.2. In comparison, the longitudinal reinforcement needed in design for torsional moment is calculated directly as a required reinforcement amount. Consequently, the additional longitudinal reinforcement that is needed because of inclined cracks is treated somewhat differently in Eurocode 2 depending on if the cracks are created from torsional moment or vertical shear force. Situations when torsional moment and shear force are acting simultaneously on a structure are treated more extensively in Section 6.4.

Eurocode 2 states that the longitudinal reinforcement needed in design for torsional moment, normally should be evenly distributed along each side of the cross-section. This is also an important difference to shear force design. The requirement in Paragraph EC2 9.3.2(4), see Section 6.2.1, of a maximum distance between longitudinal bars implies that all sides in the hollow box section with side length, z_i, longer than 350 mm, should have distributed reinforcement. However, it is not entirely clear from Eurocode 2, if it always is required to evenly distribute the longitudinal torsion reinforcement also for situations when a structure is subjected to torsional moment, shear force and bending moment simultaneously.

According to Hendy and Smith (2010) tests have been carried out by Chalioris (2003) that showed that longitudinal reinforcement, which was not properly distributed around the perimeter of the cross-section, resulted in a reduced resistance for torsional moment. However, it is not stated what actually caused the reduced capacity and what type of tests that were performed. It is therefore difficult to draw any conclusions whether this is true only for plain torsional moment or if the situation is changed when shear force and torsional moment act simultaneously on a structure.

Something that also raises questions is Paragraph EC2 6.3.2(3), see Section 6.2.1, that states that the required amount of longitudinal torsion reinforcement should be added to the other reinforcement in tensile chords and may be reduced in compressive chords. No advice is given on how to decrease the amount of longitudinal torsion reinforcement in the compression zones. If for instance a rectangular concrete cross-section is subjected to a bending moment, there will be one flexural compressive zone and one flexural tensile zone. Detailed information about how to add and reduce the longitudinal reinforcement in case of mutual torsion and bending moment has been difficult to find in the literature. However, according to Mosley *et al.* (2007) no longitudinal reinforcement is necessary in the flexural compressive zone, if the longitudinal force in the compressive zone due to torsion is smaller than the concrete compressive force due to flexure.

There is also a recommendation in Hendy and Smith (2010), which refers to the previous British Standard, BS 5400 Part 4, that states that the depth of the compressive zone, which should be considered for the reduction of the longitudinal reinforcement, should be equal to twice the concrete cover to the torsion links considered, BSI (1990). This implies that it is only the longitudinal reinforcement in the upper horizontal part of the hollow box section that should be reduced, not the reinforcement along the vertical sides, even if this reinforcement is placed within the compressive zone, see Figure 6.9. It should be noted that this recommendation presumes that the compressive zone is larger than twice the concrete cover to the torsion links.

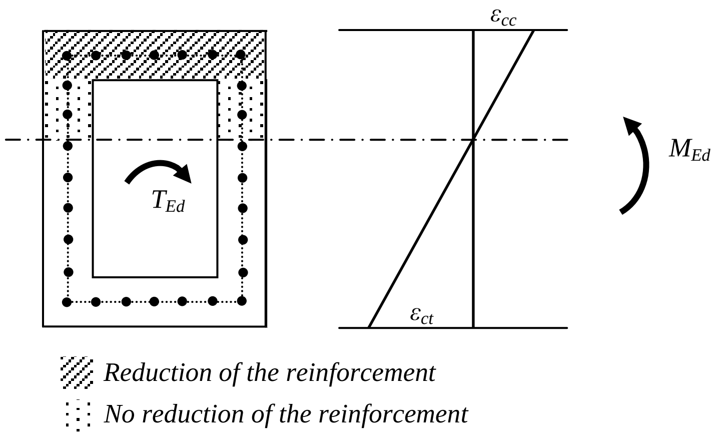


Figure 6.9 Reduction of longitudinal torsion reinforcement within the compressive zone, according to Hendy and Smith (2010).

6.3 Transversal torsional reinforcement

6.3.1 Requirements in Eurocode 2

Expression EC2 (6.26) determines the shear flow in a wall of a section that is subjected to pure torsional moment and is stated as

$$\tau_{t,i} t_{ef,i} = \frac{T_{Ed}}{2A_k} \tag{6.12}$$

$\tau_{t,i}$ torsional shear stress in wall i

$t_{ef,i}$ effective wall thickness

A_k area enclosed by the centre-lines of the connecting walls, including inner hollow areas

T_{Ed} applied design torsional moment

The torsional links that surround the cross-section should be able to lift the torsional shear force, $V_{Ed,i}$, in the compressed inclined struts in each wall of the cross-section. The torsional shear force, $V_{Ed,i}$, in a wall, i, due to torsion is given by Expression EC2 (6.27), see Equation (6.13).

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i \tag{6.13}$$

z_i side length of wall, i, defined by the distance between the intersection points with the adjacent walls

The amount of transverse reinforcement is limited to an upper value in order to avoid concrete compressive failure in the struts formed between the inclined cracks in the wall. This criterion is stated in Eurocode 2 for the combined effect of shear and torsion in Expression EC2 (6.29), see Equation (6.14).

$$\frac{T_{Ed}}{T_{Rd,\max}} + \frac{V_{Ed}}{V_{Rd,\max}} \leq 1.0 \tag{6.14}$$

For the case of plain torsion this criterion could be altered to the one in Equation (6.15) by letting the shear force V_{Ed} be equal to zero.

$$T_{Ed} \leq T_{Rd,\max} \tag{6.15}$$

The torsional resistance moment in strut compressive failure of a wall, i, of the cross-section is

$$T_{Rd,\max} = \frac{2 v_1 \alpha_{cw} f_{cd} t_{ef,i} A_k}{\cot\theta + \tan\theta} \tag{6.16}$$

When it comes to detailing of torsional reinforcement, Section EC2 9.2.3 gives guidelines regarding transverse torsional reinforcement for beams. In Paragraph EC2 9.2.3(3) there is a recommendation that states that the longitudinal distance between torsional links should not exceed $u_k / 8$ or the lesser dimension of the cross-section, SIS (2008), where u_k is the perimeter of the area A_k. Paragraph EC2 9.2.3(2) states that in order to determine the required minimum amount of transverse torsional reinforcement the rules concerning shear reinforcement, Paragraph EC2 9.2.2(5) and (6), are generally applicable, see Section 5.4.

6.3.2 Explanation and derivation

The expression for the shear force, $V_{Ed,i}$, acting on each wall, i, of the cross-section due to torsional moment, can be derived with help from the derivations in Section 6.2.2. The torsional shear force, q, which was derived in Equation (6.6), is constant along the perimeter of the section, Hendy and Smith (2010), and can be written as the product of the torsional shear stress, $\tau_{t,i}$, and the effective thickness, $t_{ef,i}$, of each wall.

$$\tau_{t,i} t_{ef,i} = \frac{T_{Ed}}{2A_k} \tag{6.17}$$

This equation can be found in Expression EC2 (6.26), see Equation (6.12).

The torsional shear force, $V_{Ed,i}$, in wall, i, can be written as the shear stress, $\tau_{t,i}$, multiplied by the area it is acting on. The cross-sectional area of the wall is equal to the effective thickness, $t_{ef,i}$, multiplied by the wall length, z_i, see Equation (6.13). It can be shown that the shear force, $V_{Ed,i}$, in Equation (6.13) is the same as the expression defined in Equation (6.3) used for the derivation of the torsional shear flow, q.

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i = q \cdot z_i \tag{6.18}$$

Since there is no information about how to determine the required transverse torsional reinforcement in Eurocode 2 such an expression is derived here based on Mosley *et al.* (2007). When calculating the required amount of transversal reinforcement, $A_{sw,i} / s$, one face of the box section is regarded, as shown in Figure 6.10. The reinforcing steel area is acting at its design yield strength, f_{yd}, and should be able to lift the torsional shear force acting on one wall, $q{\cdot}z$, in Figure 6.10.

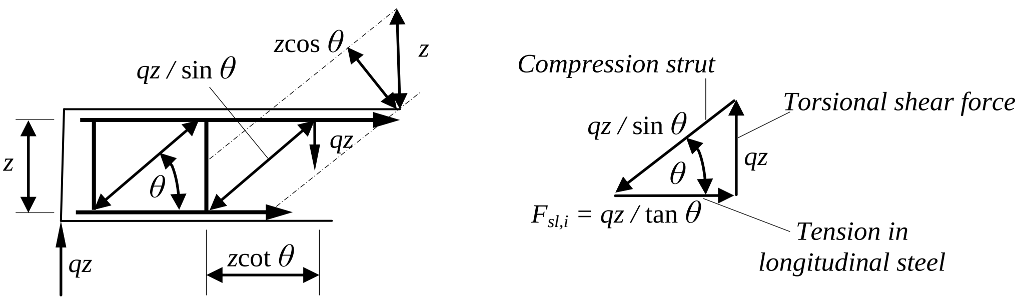


Figure 6.10 Forces acting on one face of the section. The figure is based on Mosley et al. (2007).

In Figure 6.11 it is shown how the transverse torsional reinforcement lifts the force $q{\cdot}z$ across the inclined crack that extends a distance of $z{\cdot}\cos\theta$ along the beam axis.

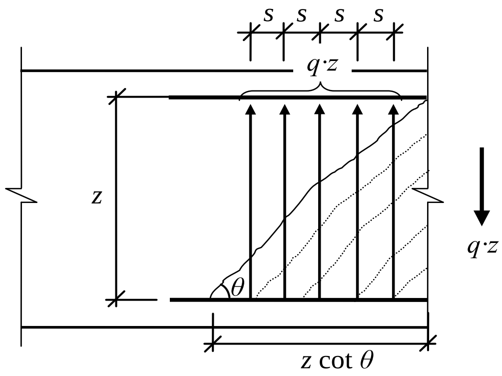


Figure 6.11 Truss model for derivation of needed amount of transverse torsional reinforcement.

If the links are evenly distributed with a spacing, *s*, along the length of the beam, the number of links, *n*, that crosses each critical crack will be

$$n = \frac{z \cot \theta}{s} \tag{6.19}$$

The transverse bars, each with an area of $A_{sw,i}$, that cross the crack should lift the torsional shear force that is acting on one wall.

$$nA_{sw,i} f_{yd} = qz \tag{6.20}$$

By combining Equation (6.19) and Equation (6.20) the following expression is obtained

$$A_{sw,i} f_{yd} \frac{z \cot \theta}{s} = \frac{T_{Ed}}{2A_k} z \tag{6.21}$$

By rearranging this expression, the required amount of transverse reinforcement needed to balance the compressed struts that resist the torsional moment, T_{Ed}, can be calculated as

$$\frac{A_{sw,i}}{s} = \frac{T_{Ed}}{2A_k f_{yd} \cot \theta} \tag{6.22}$$

In order to see the similarity between the design for torsional moment and design for vertical shear force, see Section 5.2, the derivation of the required amount of transverse torsional reinforcement can be altered. This derivation is based on Hendy and Smith (2010).

By combining Equation (6.13) for torsional shear force, $V_{Ed,i}$, and Equation (6.12) for torsional shear flow, *q*, the following expression is obtained

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i = \frac{T_{Ed}}{2A_k} z_i \tag{6.23}$$

The resistance of the vertical shear reinforcement is determined according to Expression EC2 (6.8), see Section 5.5. By equating Equations (6.23) and (5.1) the following expression is derived

$$\frac{T_{Ed}}{2A_k} z_i = \frac{A_{sw}}{s} z_i f_{yd} \cot\theta \tag{6.24}$$

It should be noted that the internal lever arms z_i for torsional moment and z for shear force are not exactly the same but in Equation (6.24) these are treated as equivalent. Equation (6.24) leads to the same expression for transversal torsional reinforcement as in Equation (6.22).

The maximum resistance of a member subjected to plain torsion, stated in Equation (6.16), is derived in Mosley *et al.* (2007). With reference to Figure 6.8 the torsional moment resistance with regard to strut compressive failure, defined as $T_{Rd,max}$, can be determined from the following derivation.

In order to relate the maximum torsional moment resistance to the shear force, only the force in one of the walls of the cross-section is of interest. Based on Figure 6.10 the compressive force in the inclined strut, resisting the torsional shear flow, can be written as

$$F_{cw,i} = \frac{qz}{\sin\theta} \tag{6.25}$$

The expression for the width of the compressive strut, $z \cdot \cos\theta$, can also be derived from Figure 6.10. In order to derive the stress acting in the compressive strut, $\sigma_{cw,t}$, the compressive force, $F_{cw,i}$, must be divided with the area of the strut, i.e. the width of the strut, $z \cdot \cos\theta$, multiplied with the effective thickness of the wall, $t_{ef,i}$.

$$\sigma_{cw,i} = \frac{F_{cw,i}}{t_{ef,i}\, z \cos\theta} = \frac{q}{t_{ef,i} \sin\theta \cos\theta} \tag{6.26}$$

This stress should be smaller than the effective design compressive strength of concrete $\sigma_{Rd,max}$, in the wall in order to avoid crushing of concrete within the inclined strut. By combining the expression for the torsional shear flow in Equation (6.6) with Equation (6.26) the following equation is obtained

$$\frac{T_{Ed}}{2A_k t_{ef,i} \sin\theta \cos\theta} \leq \sigma_{Rd,\max} \tag{6.27}$$

or by rearranging

$$T_{Ed} \leq 2\sigma_{Rd,\max} t_{ef,i} A_k \sin\theta \cos\theta \tag{6.28}$$

This can also be expressed as

$$T_{Ed} \leq \frac{2\sigma_{Rd,\max} t_{ef,i} A_k}{\cot\theta + \tan\theta} \tag{6.29}$$

where

$$\sigma_{Rd,\max} = \upsilon_1 \alpha_{cw} f_{cd}$$

The transformation of sin $\theta \cdot$ cos θ into 1 / (cot θ + tan θ) can be seen in Section 5.3.2. Expression EC2 (6.30), see Equation (6.16), is obtained by adding the factors v_1 and α_{cw}. The factor v_1 is the strength reduction factor for concrete cracked in shear and α_{cw} is a coefficient taking into account the state of stress in the compressed and cracked wall.

6.3.3 Discussion

The amount of transverse reinforcement necessary in design for torsional moment, T_{Ed}, is stated in Equation (6.22). It should be emphasised that the $A_{sw,i}$ in this expression is the area of one unit of the transverse torsional reinforcement needed in one wall of the cross-section. Note that when calculating the required shear reinforcement according to Expression EC2 (6.8), see Equation (5.1), for vertical shear force, the area, A_{sw}, includes all legs of one shear reinforcement unit. The maximum amount of transversal torsional reinforcement $A_{sw,max,i}$ with regard to compressive failure of a wall can be found by inserting $T_{Rd,max}$, see Equation (6.16), into Equation (6.22).

The minimum requirements for transversal torsional reinforcement in Eurocode 2 are the same as for vertical shear reinforcement in Paragraph EC2 9.2.2(5) and (6). The reinforcement ratio, ρ_w, i.e. the amount of transverse reinforcement that is required in order to avoid brittle failure, is limited as well as the maximum spacing between links, s_l. In Section 5.4.2 it is shown that the maximum distance, $s_{l,max}$, ensures that all inclined cracks will be crossed by at least one shear reinforcement unit, which also applies for the transversal torsional reinforcement.

However, here is one additional criterion that applies for the transversal torsional reinforcement. The maximum distance between links should be the smaller value of $u_k / 8$ and the shortest dimension of the cross-section. The reason why the shortest distance of the cross-section should limit the maximum spacing of links is because torsional cracks occur in all sides of the cross-section. If, for a rectangular cross-section, the width, b, of the section is smaller than the height, h, the cracks that occur in the top and bottom of the cross-section with an angle of 45° will reach over a shorter length along the beam section, see Figure 6.12.

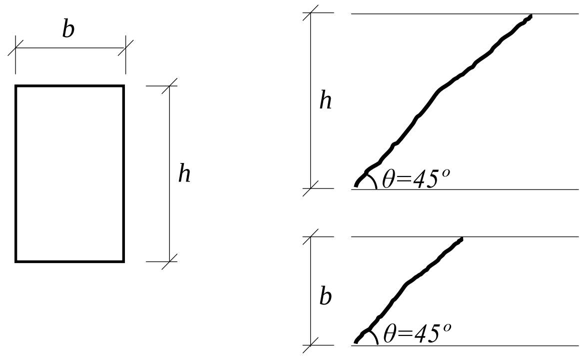


Figure 6.12 Schematic figure illustrating how the extension of a crack varies along the sides of a beam subjected to torsional moment.

The links must in such a case be placed closer together than if only the vertical sides of width *h* where considered. It should also be noted that for vertical shear it is assumed that a crack extends between the compression and tension zone, hence over a distance approximately $z = 0.9d$. This means that design of transversal torsional reinforcement is not consistent with the rules provided for design with regard to shear force. It should be more reasonable to use the internal lever arm *z* for the shortest side dimension also for torsional reinforcement. Thus, the maximum distance between transversal reinforcement should be determined in order to capture the crack, which is the case for vertical shear, see Section 5.4. This requirement would be fulfilled if using $s \leq 0.75 \cdot \min(d_h,\ d_b)$. The notations *h* and *b* describes height and width of the cross-section.

The reason for the limit of $u_k / 8$ is not as obvious. However, in Hendy and Smith (2010) it is stated that this limit will prevent premature spalling of concrete in the corners of the section under the action of the spiralling compression struts. However, such a relation between the spacing of transverse reinforcement and the stresses that occurs in the concrete corners of the cross-section is not easy to understand. It can be mentioned that for a square cross-section this criterion will be governing the maximum distance between the links, see Figure 6.13. This is also the case for a rectangular cross-section where two parallel sides are more than three times as long as the other two sides.

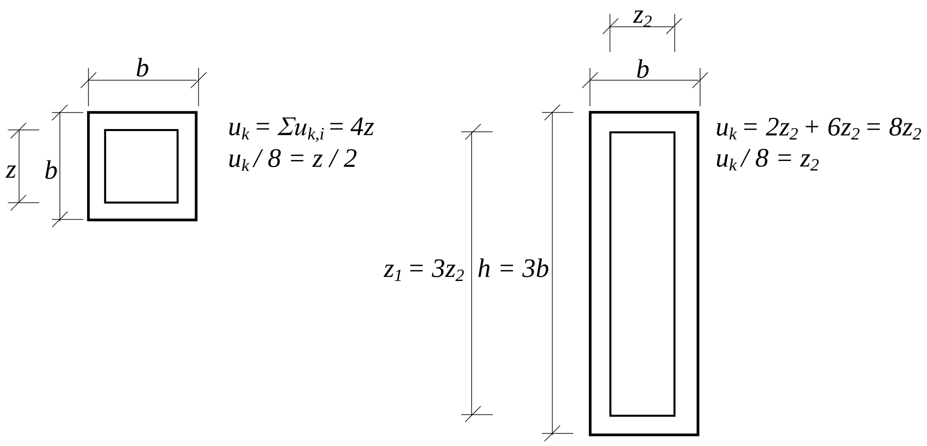


Figure 6.13 Identification of $u_k / 8$ for rectangular cross-sections with different relations between side lengths.

6.4 Combination of torsional moment and shear force

6.4.1 Requirements in Eurocode 2

According to Eurocode 2, Paragraph EC2 6.3.2(2), it is allowed to superimpose the influences of torsional moment and shear force. This means that the torsional shear force is calculated for each wall of the corresponding box section according to Equation (6.30), and is then added to the vertical shear force, Betongföreningen (2010a). However, when adding the torsional moment and shear force together it is important to choose the same angle, θ, of the inclined compressive struts for both cases, SIS (2008). The variables in Equation (6.30) are defined in Equation (6.13).

$$V_{Ed,i} = \tau_{t,i} t_{ef,i} z_i \tag{6.30}$$

Eurocode 2 provides an upper limit for combination of torsional moment and shear force according to Expression EC2 (6.29), see Equation (6.31). This expression is formulated to check that there are no excessive compressive stresses that will crush the concrete in the inclined compressive struts. It can be noted that the effects of torsional moment and shear force are treated separately in this equation.

$$\frac{T_{Ed}}{T_{Rd,\max}} + \frac{V_{Ed}}{V_{Rd,\max}} \leq 1.0 \tag{6.31}$$

Expressions for $T_{Rd,max}$ and $V_{Rd,max}$ can be found in Section 6.3 and Section 5.3 respectively.

6.4.2 Explanation and derivation

Design of the reinforcement with regard to combined torsional moment and shear force is performed according to a truss model, Betongföreningen (2010a). From Eurocode 2 it is not clear how to superimpose the two load effects. However, some guidance can be found in Betongföreningen (2010a), which is presented in the following text.

The combination of torsional moment and shear force can be illustrated by Figure 6.14. This figure shows that the design for superimposed torsional moment and shear force can be performed for one box wall with shear force $V_{Ed,V+T} = V_V + V_T$.

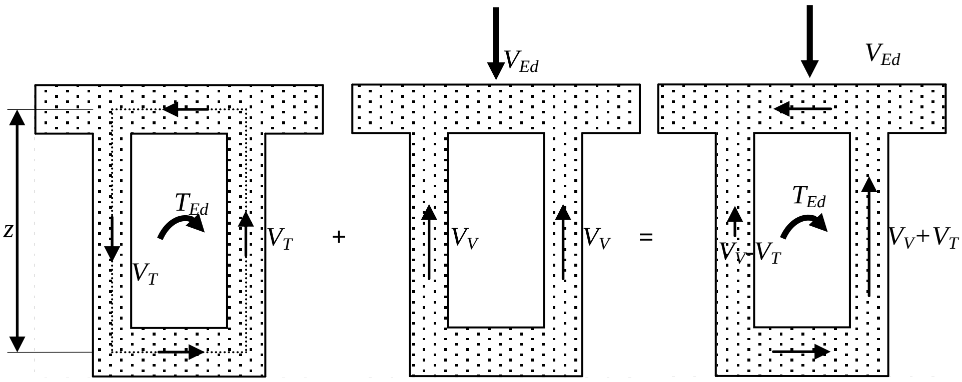


Figure 6.14 Example of the combined effects of torsional moment and shear force. The figure is based on Betongföreningen (2010).

The torsional shear force, V_T, is expressed according to Equation (6.13) and the force V_V due to shear is simply half the shear force, V_{Ed}, acting in the considered section.

$$V_V = \frac{V_{Ed}}{2} \tag{6.32}$$

If the design is performed for the superimposed effects of torsional moment and shear force, as is shown in Figure 6.14, the required longitudinal reinforcement is determined in the same way as for vertical shear force by an increase of the longitudinal tensile force according to Expression EC2 (6.18), see Equation (6.33), Betongföreningen (2010a).

$$\Delta F_{td} = 0{,}5 V_{Ed,V+T} (\cot\theta - \cot\alpha) \tag{6.33}$$

Table 6.1 describes different ways of how to determine the required longitudinal reinforcement with respect to torsional moment and shear force, depending on if the load effects are treated separately or combined.

Table 6.1 Required longitudinal reinforcement due to torsional moment and shear force. The table is based on Betongföreningen (2010a).

Torsional moment and shear force are treated	
Separately	Combined
Additional bending reinforcement for force contribution due to shear force according to Equation (6.33)	Additional bending reinforcement due to the total contribution of shear force and torsional moment according to Equation (6.33)
Additional longitudinal reinforcement due to torsional moment according to Equation (6.1)	Additional longitudinal reinforcement due to torsional reinforcement according to Equation (6.1), but only for the walls that are not designed according to the above
Reinforcement according to Equation (6.1) can be reduced in the compression zone by available compressive force	

The amount of transverse reinforcement is limited by Equation (6.31). However, since it in Section 6.3.2 was shown that the maximum torsional resistance $T_{Rd,max}$ is derived in the same way as the maximum shear resistance $V_{Rd,max}$, it is sufficient to check the corresponding shear force $V_{Ed,V+T}$ for the limit $V_{Rd,max}$, if shear force and torsional moment are superimposed. Both $T_{Rd,max}$ and $V_{Rd,max}$ express the resistance with regard to compressive failure in the inclined compressive struts. The two maximum resistances are for simplicity shown in Equation (6.34) and (6.35) respectively. The most important difference between the two expressions is the width of the compressive strut. For torsional moment the effective wall thickness, t_{ef}, is included in the resistance and for shear force the concrete compressive resistance is calculated based on the width of the web of the cross section. When the two load effects are combined it is important to calculate the concrete compressive resistance based on the smallest concrete area, hence, the smallest width.

$$T_{Rd,\max} = \frac{2 v_1 \alpha_{cw} f_{cd} t_{ef} A_k}{\cot\theta + \tan\theta} \tag{6.34}$$

$$V_{Rd,\max} = \frac{v_1 \alpha_{cw} b_w z f_{cd}}{\cot\theta + \tan\theta} \tag{6.35}$$

6.4.3 Discussion

Equation (6.33) can be shown to describe the same thing as Equation (6.1), which is the longitudinal component of the inclined compression force in the truss model Betongföreningen (2010a). For further information about this, see Section 6.2. The difference is, as mentioned before, that Equation (6.33) implies that the resulting tensile force is divided in two and is resisted by the compressive stress block and an increase of the longitudinal reinforcement needed to resist bending moment. Equation (6.1) does not provide any information about how the reinforcement should be placed, only how large the reinforcement area should be.

It should be noted that if the effects from shear force and torsional moment are superimposed and the required longitudinal reinforcement amount is calculated with Equation (6.33), the flexural reinforcement will be increased and all reinforcement will be gathered in the tensile part of the cross-section. The requirement of evenly distributed longitudinal torsion reinforcement, discussed in Section 6.2, is thereby not fulfilled.

It should also be noted that when using Equation (6.33) the horizontal contribution from the torsional moment is not included, see Figure 6.14, and must be designed for separately, Betongföreningen (2010a). It can be mentioned that this is a problem also when calculating according to the recommendations for torsional design provided in the Swedish handbook BBK 04. The reinforcement amount for each wall is calculated by replacing u_k to z in Equation (6.1).

It should also be emphasised that if the additional tensile force due to inclined cracks, ΔF_{td}, is calculated for $V_{Ed,V+T} = V_T + V_V$, this calculation concerns only half the cross-section and the total amount of longitudinal reinforcement will be twice the calculated value, see Equation (6.36). In Figure 6.15 account is taken for the reduced shear force at the left side wall of the hollow box section. However, generally symmetrical reinforcement is used, i.e. $A_{s,l}$ is provided also on the left side of the tensile zone. This is the reinforcement amount that Equation (6.36) provides.

$$A_{s,l,tot} = 2 \cdot A_{s,l} \tag{6.36}$$

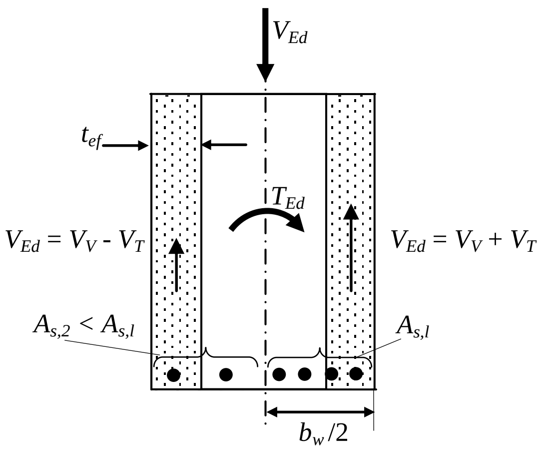


Figure 6.15 Required longitudinal torsional reinforcement when combining shear and torsion according to Equation (6.33). Generally no account is taken for that $A_{s,2} < A_{s,1}$, hence $A_{s,1}$ is provided at the left side as well.

As implied in Section 6.4.2 Equation (6.31) can be shown to give the same result as $V_{Ed,V+T} \leq V_{Rd,max}$. However, it is important to remember that if the transversal reinforcement is calculated by superimposing shear force and torsional moment, i.e. adding V_T and V_V, the upper limit $V_{Rd,max}$ must be calculated for a width b_w equal to t_{ef}, since only one wall of the cross-section is considered. Since only a thin-walled part of the total available solid section is used when calculating the concrete compressive resistance, this method becomes conservative. However, the full capacity of the concrete cross-section is not utilised as it normally is when designing for plain shear force.

It should be noted that the assumption, that only a hollow box section of thickness, $t_{ef,i}$, is involved in the torsional resistance, is conservative also for a cross-section subjected to plain torsion, since the inner part of the solid cross-section to some extent will contribute in reality. To learn more about this see, Hendy and Smith (2010). If the effects of shear force and torsional moment are calculated separately by Equation (6.31) the correct concrete area for resistance of shear force and torsional moment, respectively is considered.

As have been discussed there are a number of things that are affected and treated slightly differently depending on if the design is performed for a combination of torsional moment and shear force, or if the two load effects are treated separately. A list where advantages and disadvantages of the two different approaches are gathered is shown in Table 6.2 in order to summarise what is discussed in this section.

Table 6.2 Advantages and disadvantages depending on if torsional moment and shear force are combined or treated separately in design.

Torsional moment and shear force are treated			
separately		combined	
−	The effect of combined torsional moment and shear force is not utilised	+	Possible to utilise that V_V and V_T can counteract each other in a wall.
−	The flexural compressive zone is not utilised as simple and effective for reduction of the longitudinal torsion reinforcement as it is for additional longitudinal reinforcement in shear design.	+	Decreased amount of longitudinal torsional reinforcement due to simple and effective utilisation of the flexural compressive zone.
+	Distribution of longitudinal torsion reinforcement is fulfilled	−	Distribution of longitudinal torsion reinforcement is not fulfilled
+	Torsional reinforcement is calculated simultaneously for all walls of the cross section	−	Torsional reinforcement in the horizontal walls of the cross section must be calculated separately.
−	$V_{Rd,\,max}$ and $T_{Ed,\,max}$ must be calculated separately.	+	Only necessary to check $V_{Rd,max}$ for $V_{Ed,V+T} = V_V + V_T$
+	The shear resistance in concrete compressive failure is calculated by means of the correct concrete area for both shear force and torsional moment. (b_w and t_{ef} are used respectively).	−	The concrete compressive failure resistance in shear with regard to shear force is underestimated since the concrete area used in the calculations is smaller than in reality. (t_{ef} is used instead of b_w).

As a final remark it is important to choose the same angle, θ, of the inclined struts for a combination of shear force and torsional moment. The effect of different inclinations of the compressed concrete between cracks has been discussed in Section 5.5. It should be noted that the inclination of the cracks along a structural member needs to be consistent, i.e. the angle cannot differ at different sections. The same arguments are applicable also for torsion. Since transversal torsional reinforcement should be placed with an angle $\alpha = 90°$ in relation to the longitudinal axis the same must apply for shear reinforcement as well as for superimposed shear force and torsional moment.

6.5 Configuration of transversal torsional reinforcement

6.5.1 Requirements in Eurocode 2

Section EC2 9.2.2 states requirements regarding shear reinforcement in beams. In this section it is described that it is not permitted to have a lap joint in a leg near the surface of the web, if the link should be used to resist torsion. According to Section EC2 9.2.3 the torsion links should be closed and anchored by means of laps or hooked ends, see Figure 6.16. It is also stated that transversal torsional reinforcement should be placed with an angle of 90º in relation to the longitudinal axis.

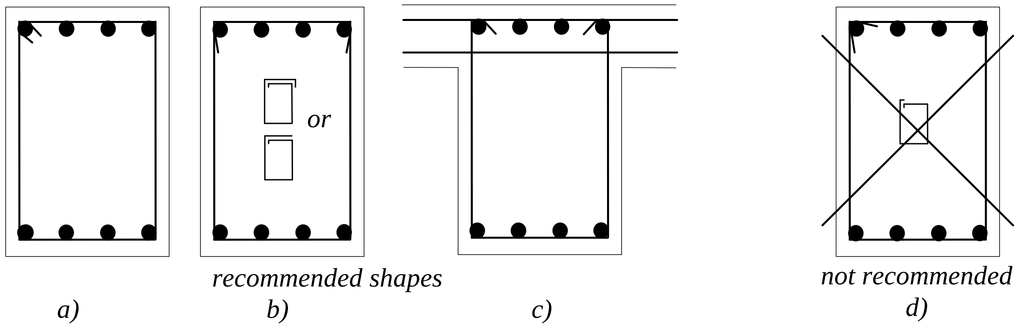


Figure 6.16 Anchorage of torsional links where different shapes are shown. The figure is based on SIS (2008).

6.5.2 Explanation and derivation

The requirements for the design of transversal torsional reinforcement are equivalent to those applicable for design of shear reinforcement. However, the requirements for torsional reinforcement are tougher with regard to anchorage of splices. Why this additional requirement exists is unknown to the authors.

Transverse torsional reinforcement should according to the requirements in Eurocode 2 consist of closed links that surrounds the cross-section. This is because torsional shear cracks occur in all longitudinal sides of a reinforced concrete member and transversal reinforcement must therefore be placed accordingly in order to transfer forces over the cracks.

6.5.3 Discussion

The torsional links should be properly anchored, as has been stated in Section 6.5.1. It is not entirely clear what the recommendations in Figure 6.16 means. However, in Paragraph EC2 9.2.3(1) it is stated that hooks or laps may be used, but nothing about bends, i.e. hooks of 90º, is mentioned. In Eurocode 2 a hook is defined as a bar that is bent with an angle larger than 150º. It can therefore be argued that Figure 6.16d implies that it is not recommended to anchor transversal torsion reinforcement by means of bends.

According to Figure 6.16a and Figure 6.16c hooks are allowed to be used in the design, as long as they are placed in the top flange. It should be noted that it may be difficult to make room in a reinforced concrete structure for hooks with angles larger than 150º. Figure 6.16b probably illustrates the possibility of anchorage by a lap splice. However, it is not clear what the different bends of the two smaller shapes within Figure 6.16b represent. It seems like one or both ends of the link must be bent with an angle of at least 90º around a bar in case of lapping.

Eurocode 2 also provides information that states that it is not allowed to lap torsional links in the web surface of a cross-section. As mentioned in Section 6.5.1, the reason for this rule has not been found. Several questions about what are allowed in detailing of transversal torsional reinforcement are raised due to the requirements in Eurocode 2. Some of these questions are gathered here in Table 6.3.

Table 6.3 Questions concerning detailing of torsional links.

Number	Question	Additional question
1	Is it allowed to anchor the torsional reinforcement by laps in the horizontal legs without bending the ends?	Is it enough to bend the bars with an angle of 90º?
2	If the width of the cross-section is shorter than the required lap length, l_0, is it then sufficient to bend the bars with 90º to provide anchorage?	Should this be regarded as lapping in the web, and therefore not be allowed?
3	Is it allowed to splice torsional links in more than one place?	Do lap splices need to be staggered to each other?
4	Why is it not allowed to splice torsional reinforcement in the web surface	
5	Is it staggering or lap splices?	

The questions that have been raised in Table 6.3 can be illustrated by different configurations of transversal torsional reinforcement, see Figure 6.17. The question is which of these that are allowed to be used and also if they are practical to use.

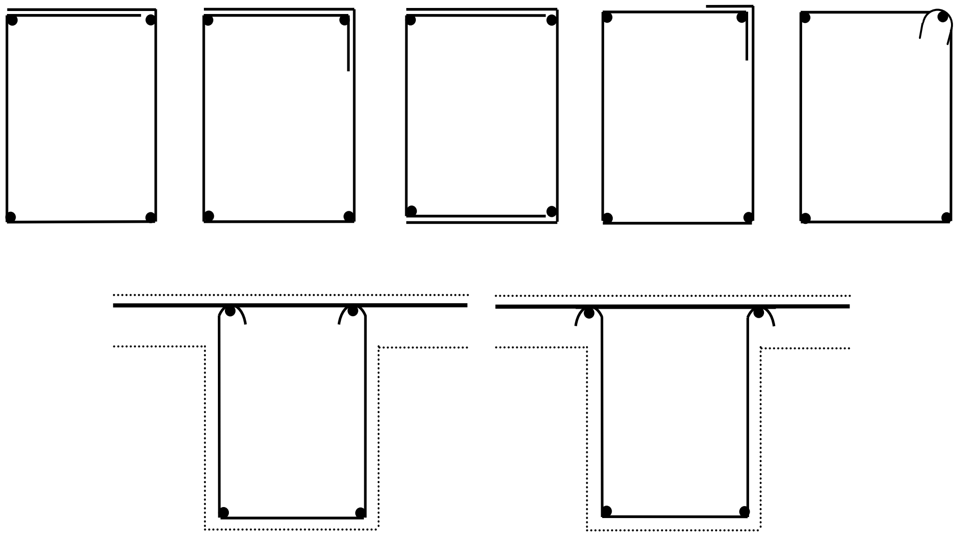


Figure 6.17 Suggestion on configurations of transversal torsional reinforcement.

In addition to what have been discussed above it should be emphasised that the requirements stated that it is not allowed to have a lap joint in the leg near the surface of the web for torsional reinforcement is described in Section EC2 9.2.2 regarding shear reinforcement. It can be argued that this requirement is misplaced and should be placed, or at least be referred to, in Section EC2 9.2.3 concerning design or torsional reinforcement. The general impression of the authors is also that it is common to use shear reinforcement as shown in Figure 6.18, where laps are placed in the web sections.

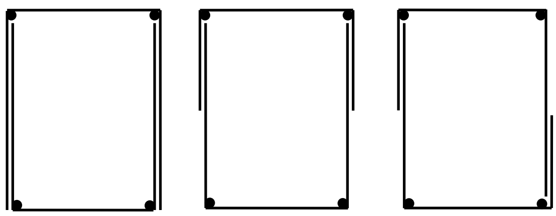


Figure 6.18 Common shear reinforcement configurations.

According to Johansson (2012a) questions on how the recommended design should be interpreted have been raised to the Swedish Standard Institute by a Workshop, Brosamverkan Väst (2012). Two of the questions that the Workshop resulted in are illustrated in Figure 6.19 where the required staggering of lapped splices provided in Eurocode 2 have been considered. See Section 9.4 in the report for more information regarding staggering of lapped splices. The questions that were raised are if it is allowed to splice torsional reinforcement in the web sections, if sufficient staggering of the lap splices is ensured, see Figure 6.19a, and if it at all is allowed to lap torsional links in more than one place as long as the required lap length, l_0, is fulfilled, see Figure 6.19b. It should be noted that it might be difficult to make room for required lap length and distance between the lap ends in these configurations.

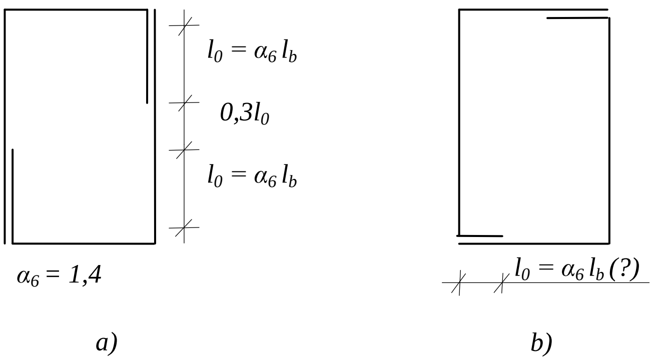


Figure 6.19 Torsional reinforcement configurations suggested by Johansson (2012a).

7 Shear between web and flanges

7.1 Structural response and modelling

Figure 7.1 illustrates the forces in a reinforced concrete beam with a T-section subjected to bending. The reinforcement in the bottom of the web acts in tension with a force, F_s, and the concrete in the flange acts in compression with a force, F_c, which will result in inclined compressive struts. The compressive force, F_c, can be split into F_{c1}, F_{c2} and F_{c3} and since F_{c2} is located in the web, it is the less critical force to design for. In order to resist the forces F_{c1} and F_{c3} in the corresponding flanges a load path between the web and flange must be ensured by transversal reinforcement that is distributed along a certain length of the beam.

It should be noted that the heading of Section EC2 6.2.4 in Eurocode 2 refers to shear between web and flanges of T-sections, which can be misrepresentative since the requirements are applicable also for other types of cross-sections, like hollow box sections, on the basis of the shear per wall of the hollow box section, Betongföreningen (2010a).

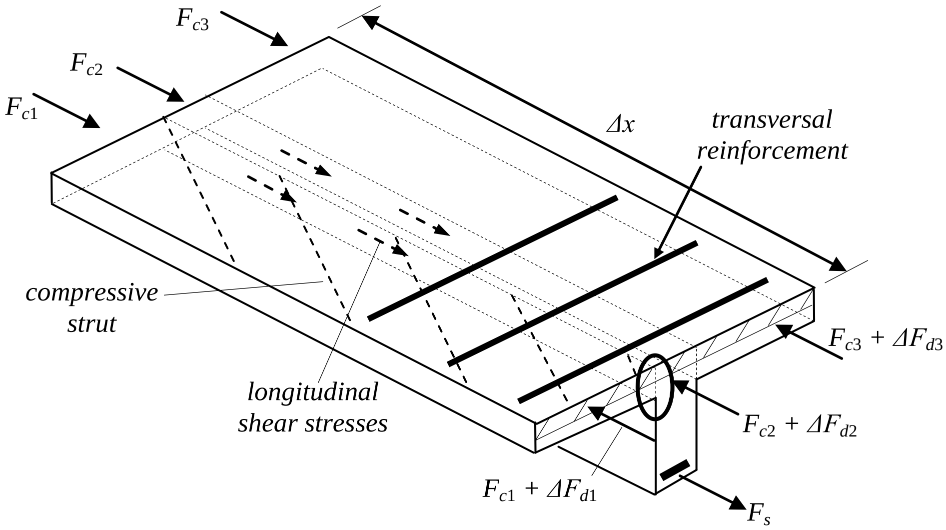


Figure 7.1 Forces acting in a reinforced concrete T-beam subjected to bending moment.

When designing the reinforcement in a concrete beam with a T-section, the vertical shear force is assumed to be taken by the web. Otherwise, the calculations are the same as for a beam with a rectangular section subjected to a uniformly distributed load, q, and the force pattern can be modelled by a truss model, see Figure 7.2, Mosley *et al.* (2007). Complementary longitudinal shear stresses occur at the interface between the web and flange as shown in Figure 7.1. The longitudinal shear stresses are resisted by inclined compression struts in the flanges. To balance the inclined struts transversal reinforcement that acts as ties across the flange is necessary. It is important to check the risk of failure by crushing of the concrete struts and to avoid tensile failure of the ties by providing sufficient steel area.

In Figure 7.2a a flange in compression is shown where the inclined compressive struts in the web are illustrated with dotted lines. The compressive struts have the angle, θ, defined in relation to the longitudinal axis. The compressive forces in the inclined struts are balanced by tension in the vertical stirrups that lift the force through the web up to the flange. Longitudinal reinforcement is provided in the tensile zone in the web. In Figure 7.2b it is shown how a part of the compressive force in the web stays in equilibrium with inclined compressive struts in the flange, now with an angle, θ_f. In order to balance the force in the compressive struts in the flange transversal reinforcement in the flange needs to be provided, see Figure 7.2b. Note that the reinforcement should not be provided at the same section as the face of the support, due to the inclined compression struts in the web. In Figure 7.2c the longitudinal reinforcement is instead provided in the flange, since the tensile zone is located there. Otherwise, the transversal reinforcement in the flange is acting in the same manner as for a flange in compression.

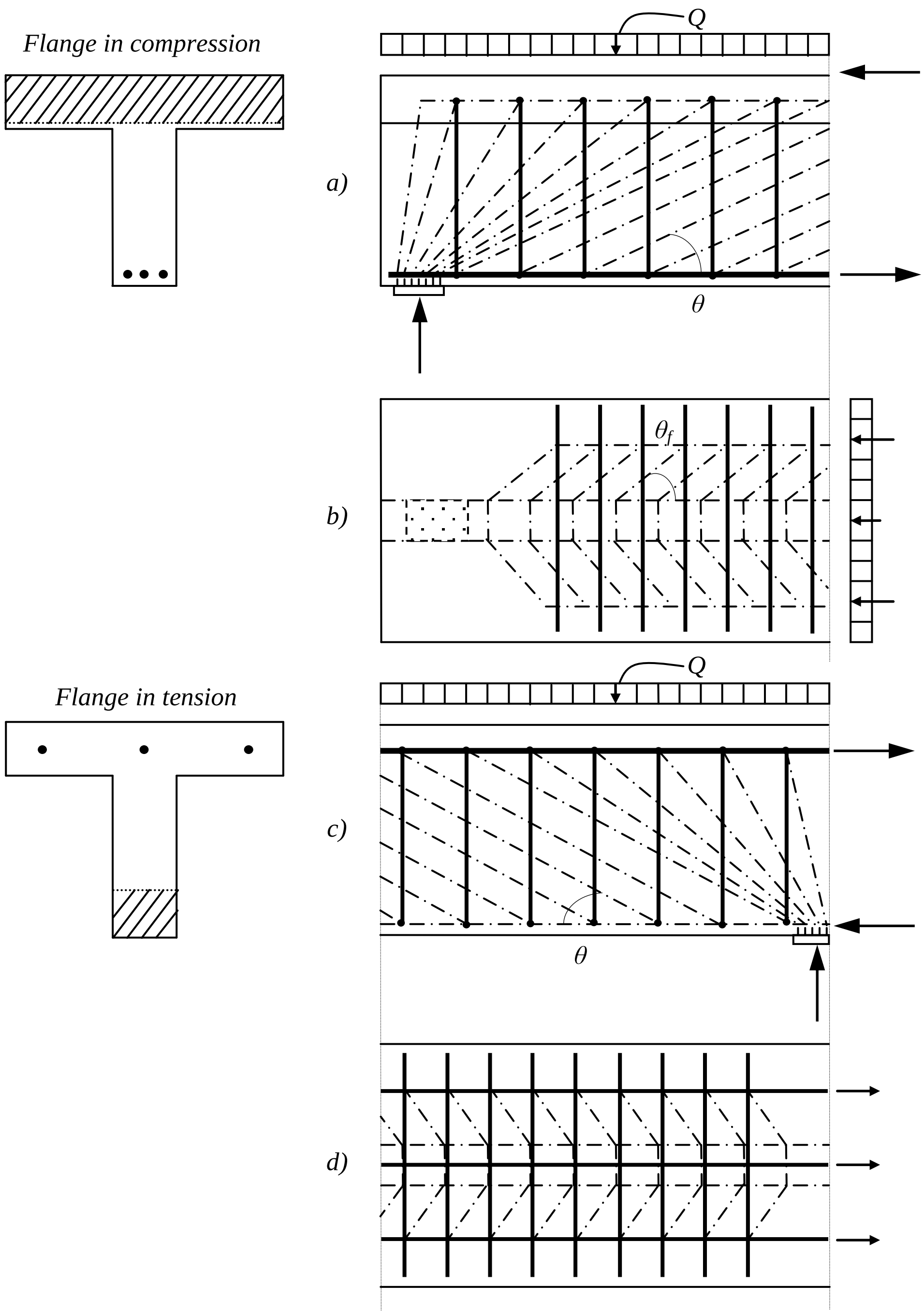


Figure 7.2 Truss model in web and flange and how this determines the position of the transversal reinforcement. The dotted lines represent compressive forces and the solid lines tensile forces. The figure is based on Betongföreningen (2010a).

To avoid creating a truss model for each possible load situation a simplification can be made where the average force increase per meter may be calculated over a longer length, Δx, see Figure 7.1. The length, Δx, should not be taken greater than half the distance between the zero and maximum moment sections according to Eurocode 2. However, where there are point loads acting on the flanges of the beam, as can be the case for a bridge loaded by vehicle axles, the length, Δx, should not be taken greater than the distance between concentrated loads. This is done in order to avoid incorrect estimation of the rate of change of flange force.

7.2 Longitudinal shear stress

7.2.1 Requirements in Eurocode 2

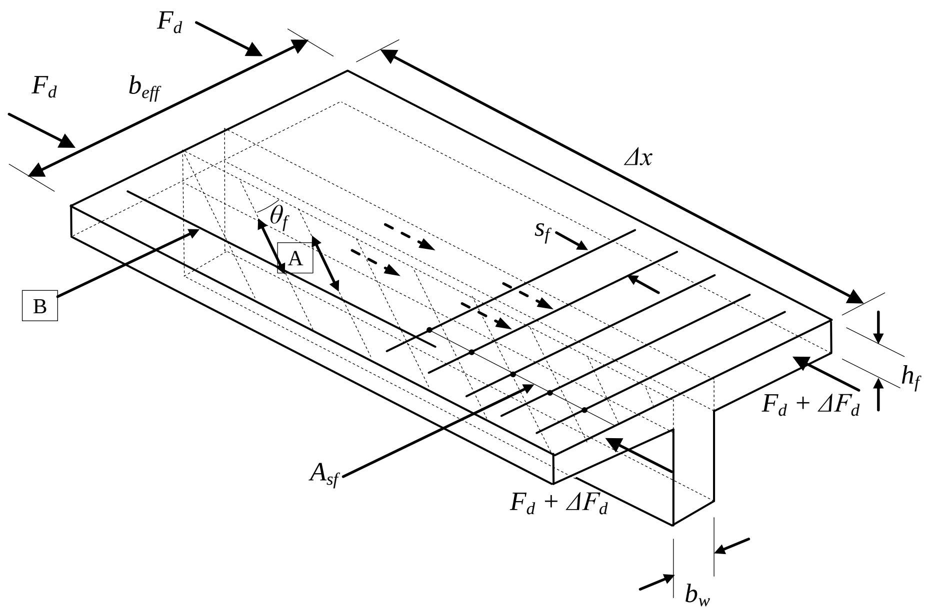
The longitudinal shear stress, v_{Ed}, at the web-flange intersection is determined by the change of the normal (longitudinal) force, ΔF_d, in the part of the flange considered, according to Expression EC2 (6.20), see Equation (7.1). The simplified description of v_{Ed} is average shear force per unit length over the length Δx and is expressed as

$$v_{Ed} = \frac{\Delta F_d}{h_f \cdot \Delta x} \tag{7.1}$$

h_f thickness of the flange at the intersection

Δx length under consideration, see Figure 7.3

ΔF_d change of the normal force in the flange over the length Δx, acting on one side of the web, see Figure 7.3



A *compressive struts* B *longitudinal bar anchored beyond this projected point (see 6.2.4(7))*

Figure 7.3 Notations for the connection between flange and web. The figure is based on SIS (2008).

No additional transversal reinforcement, above that for transverse bending moment of the flange, is according to SIS (2008) required if the shear stress is less than 40 % of the design tensile strength, f_{ctd}, see also Section 7.3. Then the concrete is deemed by itself to take care of the longitudinal shear force.

7.2.2 Explanation and derivation

The requirement regarding longitudinal shear stresses in Eurocode 2 is unclear. The standard does not give any clear guidance on how to calculate the change of the compressive force, ΔF_d. In the following text it is therefore shown how this force can be determined, Johansson (2012a).

The compressive forces $F_{c1} + \Delta F_{c1}$ and $F_{c3} + \Delta F_{c3}$, that are described in Section 7.1, see Figure 7.1, are according to Eurocode 2 defined as $F_d + \Delta F_d$ acting at each side of the web, see Figure 7.3, SIS (2008). The total change of the compressive force, $\Delta F_{c,tot}$, is calculated as

$$\Delta F_{c,tot} = \sum_{i=1}^{3} \Delta F_{c,i} \tag{7.2}$$

where the compressive force in each flange (part 1 and 3) can be calculated as

$$\Delta F_{c,i} = \beta_i' \cdot \Delta F_{c,tot} = \frac{A_i}{A_{tot}} \Delta F_{c,tot} \tag{7.3}$$

β_i' ratio between the compressive force in the flange and the whole compressive force acting on either the compression or tension zone for the section considered

A_i area that takes the force in each part. When the flange is in compression this corresponds to $A_{c,i}$, see Figure 7.4a, and when the flange is in tension this corresponds to $A_{s,i}$,, see Figure 7.4b

A_{tot} total area that takes the force (compression or tension)

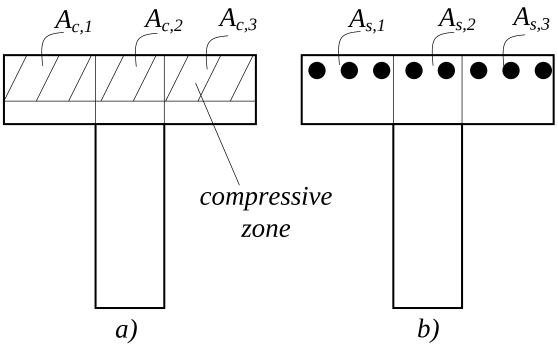


Figure 7.4 Definitions of effective area for a flange in a) compression and b) tension.

If the flange is in compression and the neutral axis lies within the flange height, see Figure 7.5 and Figure 7.6, the shear force at the web-flange intersection can also be expressed by means of flange width, see Equation (7.4).

$$\beta_i' = \frac{A_{i,eff}}{A_{tot,eff}} = \frac{b_{i,eff}}{b_{tot,eff}} \tag{7.4}$$

b_i width of one part of the flange, see Figure 7.5

b_{tot} width of the whole flange, see Figure 7.5

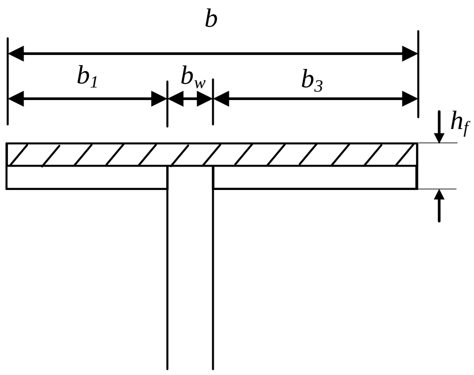


Figure 7.5 Definitions of effective widths.

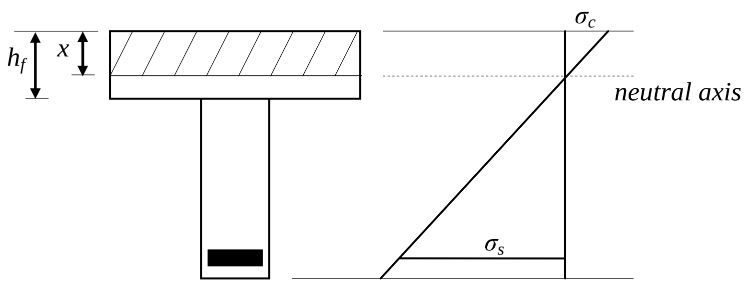


Figure 7.6 Cross-section of the T-beam where the neutral axis lies at the web-flange junction.

In the following text an alternative expression to Equation (7.1) is derived. However, in this case the shear stress is expressed by means of the design value of the sectional shear force, V_{Ed}. The change of compressive force, $\Delta F_{c,i}$, over a length Δx can be found by dividing the change of the bending moment, ΔM_{Ed}, with the internal lever arm, z.

$$\Delta F_{c,i} = \beta_i' \frac{\Delta M_{Ed}}{z} \tag{7.5}$$

z internal lever arm of the cross-section

When the change of the compressive force in a flange is calculated, the shear stress at the web/flange intersection can be determined according to Equation (7.6). The change of the compressive force in a flange is in equilibrium with a shear force that should be resisted in the critical section between the flange and the web.

$$v_{Ed,i} = \frac{\Delta F_{c,i}}{h_f \Delta x} = \frac{1}{h_f \Delta x} \beta_i' \frac{\Delta M_{Ed}}{z} \tag{7.6}$$

The change of the compressive force, $\Delta F_{c,i}$, can also be written by means of the sectional shear force, V_{Ed}, which can be seen as the derivative of M_{Ed}

$$\frac{\Delta F_{c,i}}{\Delta x} = \beta_i' \frac{1}{z} \frac{\Delta M_{Ed}}{\Delta x} = \beta_i' \frac{V_{Ed}}{z} \qquad \text{when } \Delta x \to 0 \tag{7.7}$$

This results in the following expression for calculating the shear stress, v_{Ed}

$$v_{Ed} = \frac{V_{Ed}}{zh_f} \qquad (7.8)$$

V_{Ed} design shear force when $\Delta x \to 0$

7.2.3 Discussion

In order to increase the understanding of how to calculate ΔF_d the requirement in Eurocode 2 can be complemented with a figure similar to Figure 7.4.

The method that is used when calculating the required transversal shear reinforcement is based on the assumption that it is possible to smear the shear force out over a length Δx, see Section 7.2.1. This means that, according to Eurocode 2, it is not necessary to have sufficient capacity in every section, if the capacity of the reinforcement in total is sufficient. The part of the shear force that is not designed for is shown in Figure 7.7.

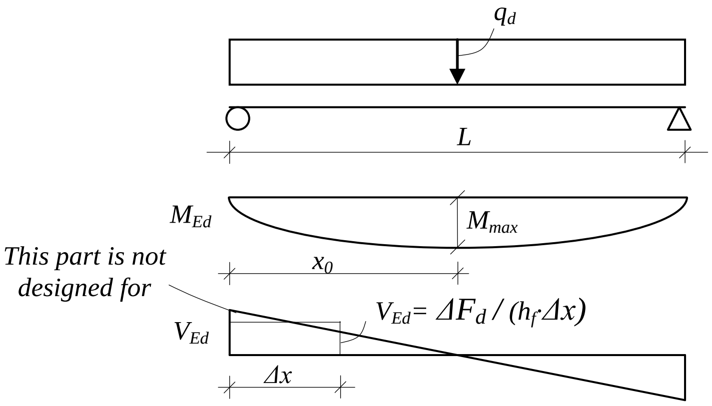


Figure 7.7 When Δx is used as length to determine the transversal reinforcement, this relies on plastic redistribution. This is however questionable especially when the calculated shear force per unit length is less than or close to the shear capacity without transversal reinforcement.

As shown in Equation (7.8) in Section 7.2.2 the shear stress can be calculated by means of the design shear force, V_{Ed}. This means that the worst case for designing the shear force per unit length, v_{Ed}, is by taking the highest design shear force, V_{Ed}, which in most cases occurs at the support.

If comparing Section 6.2.4.2 in BBK 04 with Equation (7.1) it is found that the design shear force, V_{Ed}, is used in the Swedish handbook, see Equation (7.9), Boverket (2004). This corresponds to a length $\Delta x \to 0$. The reason why the requirements in the two texts differ is unknown to the authors. It might be reasonable to smear the shear force over the length Δx, if plastic redistribution is possible, i.e. in the presence of transversal reinforcement. However, it can be perceived as unsafe to use the same approach in regions where the transversal reinforcement is not needed, Johansson (2013).

$$A_t = \left(\frac{A_f}{A} \cdot \frac{V_{Ed}}{z} - h_f f_v \right) \frac{1}{f_{yd}} \qquad (7.9)$$

A total area of the compressive zone if the actual flange is compressed or the total area of the bending reinforcement if it is in tension

A_f area of the part of the compressive zone or the area of the bending reinforcement within the area $A_{eff,i}$ or the width $b_{eff,i}$

f_v $=0.35 f_{ct}$

f_{ct} design tensile strength of concrete

f_{yd} design yield strength of reinforcement

In Section 7.2.1 the condition regarding that no transversal reinforcement is required when the shear stress, v_{Ed}, is less or equal to $0.4f_{ctd}$ was described. This condition is more easily fulfilled when an average value of the shear force, $\Delta F_d / \Delta x$, is used. According to Engström (2013) it may seem illogical to not provide transversal reinforcement for the highest shear force since without transversal reinforcement the redistribution of forces would not be as efficient. According to Eurocode 2 the ordinary shear capacity of a concrete member that does not require shear reinforcement, $V_{Rd,c}$, can be determined by Expression EC2 (6.2), i.e. the cracked reinforced concrete itself can take care of the shear force. This is because, in a member with inclined cracks the shear force resisted by inclined struts can be lifted not only by the shear reinforcement but also by friction and interlocking effects in the cracks. This is also described in Section 5.1. It is probably a similar theory behind the reason for not designing the shear reinforcement for the highest value of the shear force in the flange of a T-beam. However, in this case Equation (7.1) does not say anything about how much of the shear force that can be resisted by the concrete, Johansson (2013). A more reasonable method would be to include the effect of the concrete resistance by providing it a reasonable value.

7.3 Transversal reinforcement

7.3.1 Requirements in Eurocode 2

Paragraph EC2 6.2.4(4) describes the required transverse reinforcement in the flange per unit length, A_{sf} / s_f, Hendy and Smith (2010), see Figure 7.3 for definitions. Here the struts and ties in the truss model in Figure 7.2 are smeared out instead of being discrete. The transverse reinforcement can be calculated as

$$\frac{A_{sf} f_{yd}}{s_f} \geq \frac{v_{Ed} \cdot h_f}{\cot \theta} \tag{7.10}$$

The condition with regard to crushing of the compression struts, illustrated in Figure 7.1, is expressed in Expression EC2 (6.22) as

$$v_{Ed} \leq v f_{cd} \sin \theta_f \cos \theta_f \tag{7.11}$$

The recommended range of $\cot \theta_f$ is:

$$1 \leq \cot \theta_f \leq 2.0 \quad \text{for compression flanges} \quad 45° \geq \theta_f \geq 26.5° \tag{7.12}$$

$$1 \le \cot\theta_f \le 1.25 \qquad \text{for tension flanges}$$

$$45° \ge \theta_f \ge 38.6° \tag{7.13}$$

Flanges forming deck slabs are subjected to both shear force and transversal bending moment from dead and live loads. In this case the transversal reinforcement needs to be checked for its ability to resist both in-plane shear and transverse bending. According to Paragraph EC2 6.2.4(5) the total reinforcement amount is sufficient if the area of the transversal reinforcing steel is greater than the value determined by Equation (7.10) or half that value added to that required for transverse bending.

7.3.2 Explanation and derivation

The derivation of Equation (7.10) that follows in this text is based on Hendy and Smith (2010). A plan of an area ABCD of a concrete flange is shown in Figure 7.8. The flange is assumed to be in compression along the longitudinal direction resulting in an average shear stress, v_{Ed}. The flange is provided with transverse reinforcement, A_{sf}, at a spacing, s_f. The shear force per transverse bar acting on side AB of the rectangle, shown in Figure 7.8, is

$$F_v = v_{Ed} h_f s_f \tag{7.14}$$

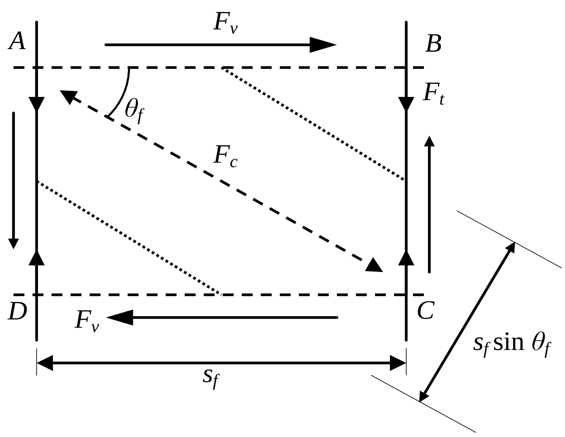


Figure 7.8 Forces acting on a flange element (plan view). The figure is based on Hendy and Smith (2010).

The shear force is acting on side AB and is in equilibrium with an inclined concrete strut at an angle, θ_f. The edges of each strut pass through the mid-points of AB, CB, etc. and it has the width $s_f \sin\theta_f$. To achieve equilibrium in A the force in the strut is determined by

$$F_c = \frac{F_v}{\cos\theta_f} \tag{7.15}$$

By inserting Equation (7.14) into Equation (7.15) F_c can be expressed as

$$F_c = \frac{v_{Ed} h_f s_f}{\cos\theta_f} \tag{7.16}$$

The limiting stress of the compressed concrete in the strut is νf_{cd} and can be inserted as a limiting force according to

$$\frac{v_{Ed} h_f s_f}{\cos\theta_f} \leq h_f s_f \sin\theta_f \nu f_{cd} \tag{7.17}$$

where

$$\nu = 0.6\left(1 - \frac{f_{ck}}{250}\right) \tag{7.18}$$

This leads to the expression in Eurocode 2, see Equation (7.11), in order to prevent crushing of the compression struts in the flange

$$v_{Ed} \leq \nu f_{cd} \sin\theta_f \cos\theta_f \tag{7.19}$$

To achieve equilibrium in point C, the force in the transverse bar, is determined by

$$F_t = F_c \sin\theta_f = F_v \tan\theta_f \tag{7.20}$$

By substituting F_v with Equation (7.14) F_t can be expressed as

$$F_t = v_{Ed} h_f s_f \tan\theta_f \tag{7.21}$$

The maximum force in the transversal steel depends on its design strength f_{yd}

$$F_{t,\max} = A_{sf} f_{yd} \tag{7.22}$$

The needed transversal reinforcement per unit length A_{sf} / s_f can then be determined as

$$\frac{A_{sf} f_{yd}}{s_f} \geq \frac{v_{Ed} \cdot h_f}{\cot\theta} \tag{7.23}$$

7.3.3 Discussion

Paragraph EC2 6.2.4(5) gives guidelines on how to perform the design when a flange is subjected to both longitudinal shear force and transversal bending moment. The total reinforcement amount is considered to be sufficient, if the steel area is greater than that determined by Equation (7.10) or half that given by Equation (7.10) in addition to that required for transverse bending. This can in a simpler way be expressed as

$$A_s = \max\left(A_{sf}, \frac{A_{sf}}{2} + A_{s,bendning}\right) \tag{7.24}$$

where the required amount of transversal reinforcement with regard to longitudinal shear is

$$A_{sf} \geq \frac{s_f \cdot v_{Ed} \cdot h_f}{\cot\theta} \frac{1}{f_{yd}} \tag{7.25}$$

This expression can be compared to Equation (6.2.4.2) in BBK 04 that is stated in Equation (7.9).The design approach in the previous Swedish handbook do to some extent also rely on the concrete tensile strength, Boverket (2004). This is clear from Equation (7.9), since the tensile strength of the concrete, f_{ct}, is included in the expression. It can also be noted that another difference is that the angle, θ, of the compressive strut is not included in Equation (7.9). This is because it in BBK 04 is assumed that the inclination of the compressive strut is equal to 45º and cannot be chosen by the designer.

The shear force per unit length, ΔF_d, acting in the considered zone is for Equation (7.25) and Equation (7.9) considered respectively as

$$\Delta F_d = v_{Ed} h_f \tag{7.26}$$

$$\Delta F_d = \frac{A_f}{A} \cdot \frac{V_{Ed}}{z} \tag{7.27}$$

In Equation (7.9) it is shown that the shear force per unit length, ΔF_d, in BBK 04 is reduced with the shear capacity of the concrete, where the resulting shear force per unit length becomes

$$\Delta F_{d,red} = \left(\frac{A_f}{A} \cdot \frac{V_{Ed}}{z} - h_f f_v\right) \tag{7.28}$$

$$f_v = 0.35 f_{ct}$$

The tensile strength of the concrete is not included in the expression in Eurocode 2, see Equation (7.25). However, the guidelines in Eurocode 2 still relies on the tensile strength of the concrete, since the design of reinforcement is not performed for the whole load acting, i.e. the reinforcement is not designed for $A_{sf} + A_{s,bending}$. This can be perceived as contradictory to the basic design philosophy in Eurocode 2 that assumes that the concrete in general should not resist any tensile stresses, only the reinforcement. The fact that A_{sf} is not needed when $v_{Ed} < 0.40 f_{ctd}$ also means that the concrete tensile strength is utilized.

8 Shear friction and dowel action

8.1 Structural response and modelling

When a joint, which is provided with transversal reinforcement crossing the interface, is designed for shear resistance two basic mechanism can be distinguished; shear friction and dowel action, *fib* (2008). In the case where the shear force is resisted by friction ensured by pullout resistance of the transverse reinforcement bars, it is in this report denoted as "shear friction". Chapter 8 is mainly based on information taken from *fib* (2008).

The significant description of dowel action is that the dowel is allowed to slide inside the concrete while it is mainly subjected to bending when shear slip takes place at the joint interface, *fib* (2008). When a plain dowel is placed across a joint or when there is low friction between the surfaces of the interface shear resistance over a joint will be accomplished by dowel action. No axial stresses will be created within the steel bar and failure will be due to bending of the bar. However, if the bar is restrained with for instance end-anchors or by ribs at the bar surface, as in case of ordinary reinforcement, axial stresses will be created in the bar. In this case friction ensured by the pullout resistance of the bars will also contribute to the shear resistance. When the shear transfer is enabled by shear friction, the bar is not subjected to significant flexure, but mainly axial stresses. The bar will only be strained in the region close to the joint interface and the joint will be clamped together by the pullout resistance of the bar.

Depending on the roughness of the joint faces and the bond and anchorage of the bar, the contribution to the shear resistance of a joint will vary due to the combination of shear friction and dowel action, *fib* (2008). In case of rough joint face and ribbed bars a large pullout resistance will be developed. Hence, the major contribution to shear resistance will be due to shear friction. In case of smooth joint faces and plain bars the pullout resistance will be small and dowel action will dominate the shear resistance. Note that the maximum shear force in case of dowel action will occur for a larger shear slip. For the same steel bars the shear capacity in dowel action is less than in shear friction. To get more information concerning combination of shear friction and dowel action see Appendix E.

In case of shear friction the shear capacity will increase with increased amount of transverse steel area, *fib* (2008). However, an increased number of bars will also result in an increased self-generated compressive force acting on the joint interfaces, which might result in crushing of the concrete. The self-generated compressive stresses in the ultimate limit state are schematically shown in Figure 8.1. Because of this an upper limit of the shear resistance can be determined, which in turn results in an upper limit of transversal reinforcement amount. This will be derived in Section 8.3. Note that the shear capacity in case of dowel action is also increased with increased amount of steel area. However, it is the bending capacity of the dowel that is increased and not the pullout resistance.

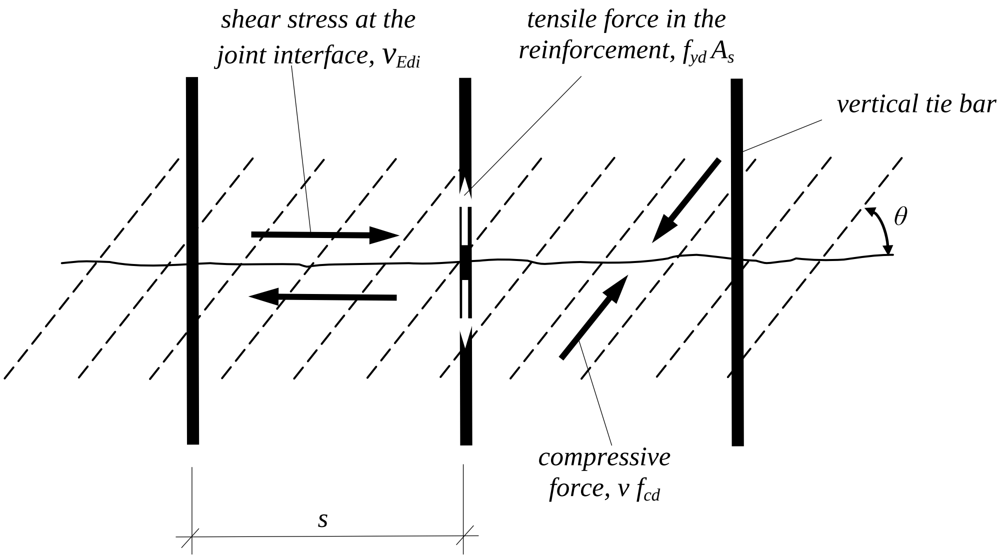


Figure 8.1 Schematic illustration of how the shear force is resisted at the joint interface due to shear friction. The figure is based on fib (2008) who adopted it from Nielsen (1984).

It can be concluded that in most cases where a transversal steel bar is placed across a joint, the resistance against shear will be influenced by a combination of shear friction and dowel action, which depends on the roughness of the joint faces and the bond and anchorage of the bar. The combination of the two shear resisting mechanism is illustrated in Figure 8.2, *fib* (2008).

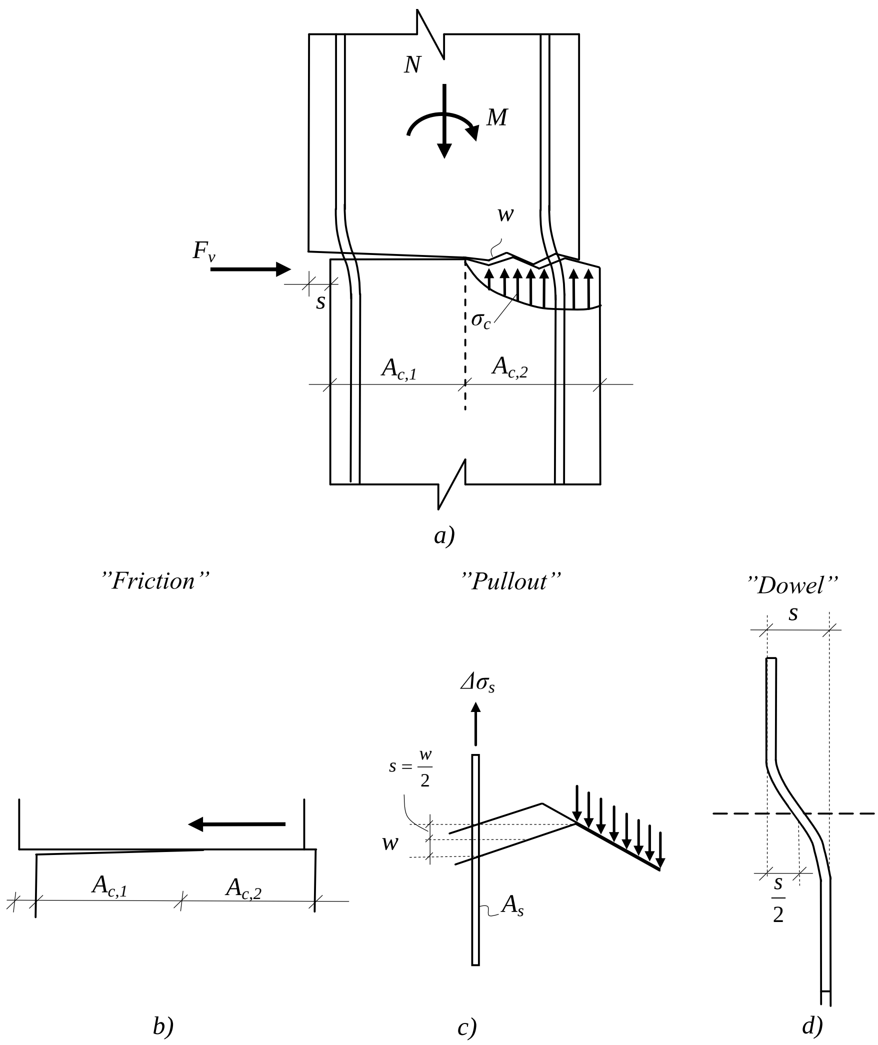


Figure 8.2 Imposed shear slip, s, mobilises dowel action and shear friction. The transverse bars are strained due to both dowel action (bending) and bar pullout (tension) that results from the joint separation , w, a) overview, b) friction between the joint faces, c) pullout resistance, d) dowel action. The figure is based on fib (2008) who adopted it from Tsoukantas and Tassios (1989).

8.2 Shear at the interface between concrete cast at different times

8.2.1 Requirements in Eurocode 2

In Eurocode 2 shear at the interface between concrete cast at different times is described in Section EC2 6.2.5. In this case the load is resisted mainly by shear friction. Joints of this type are for example a joint between a prefabricated and a cast-in situ part of a composite beam, see Figure 8.3, where the influence of friction, cohesion, normal stresses and reinforcement is of importance. The rules and recommendations in Section EC2 6.2.5 are in addition to those described for ordinary shear resistance in Section EC2 6.2.1-6.2.4, see Chapter 5.

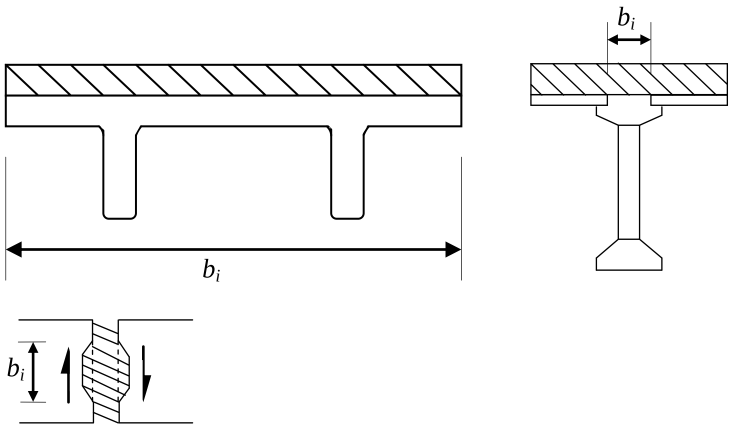


Figure 8.3 *Examples of interfaces between concrete cast at different times. The figure is based on SIS (2008).*

The design shear resistance should according to Eurocode 2, Paragraph EC2 6.2.5(1), be larger than the design value of the shear stress at the interface, i.e. the following condition should be fulfilled

$$v_{Edi} \leq v_{Rdi} \tag{8.1}$$

The design value of the shear stress at the interface is calculated as

$$v_{Edi} = \frac{\beta V_{Ed}}{z b_i} \tag{8.2}$$

- β ratio of the longitudinal force in the new concrete area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered
- V_{Ed} sectional shear force
- z lever arm of composite section
- b_i width of the interface

The design shear resistance is, according to Expression EC2 (6.25), determined as

$$v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} \left(\mu \sin \alpha + \cos \alpha \right) \leq 0.5 \nu f_{cd} \tag{8.3}$$

- c, μ factors which depend on the roughness of the interface
- f_{ctd} design tensile strength of concrete
- σ_n normal stress per unit area caused by the minimum external force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0.6\ f_{cd}$, and negative for tension. When σ_n is tensile $c f_{ctd}$ should be taken as 0.
- ρ ratio between the area of the reinforcement crossing the joint and the area of the joint itself.
- f_{cd} design compressive strength of concrete

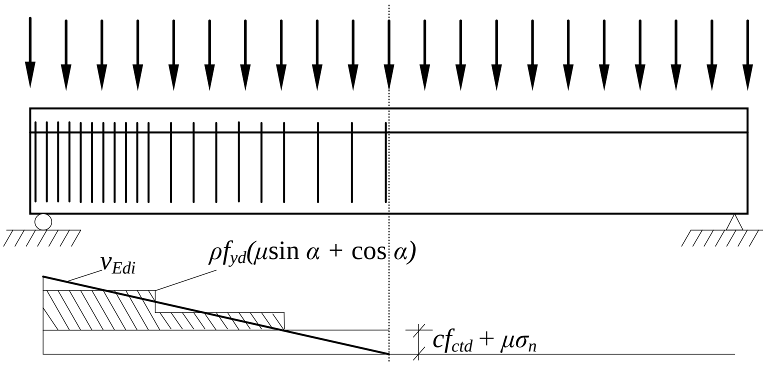
v strength reduction factor for concrete cracked in shear

The roughness of the joint face corresponds to a certain value of the frictional coefficient, μ. Different types of faces are classified in Paragraph EC2 6.2.5(2) and are reproduced in Table 8.1, SIS (2008).

Table 8.1 Classification of joint faces.

Name	***c***	***μ***	**Description**
Very smooth surface	0.25	0.5	Very smooth surface: a surface cast against steel, plastic or specially prepared wooden formwork
Smooth surface	0.35	0.6	Smooth: a slip formed or extrude surface, or a free surface left without further treatment after vibrating
Rough surface	0.45	0.7	Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregated or other methods giving an equivalent behaviour
Intended surface	0.50	0.9	Intended: a surface with indentations complying with

According to EC2 6.2.5(5) the reinforcement across the joint can be provided in the transverse direction or with an inclination and should only be designed for the shear stress that is not resisted by cohesion and friction of the external normal stress, see Figure 8.4.



$\rho f_{yd}(\mu \sin \alpha + \cos \alpha)$	*shear stress resisted by friction generated by reinforcement crossing the joint*
$cf_{ctd} + \mu\sigma_n$	*shear stress resisted by cohesion and friction of external normal stress*

Figure 8.4 Shear diagram where the required interface reinforcement is shown. The figure is based on SIS (2008).

8.2.2 Explanation and derivation

Equation (8.2), which describes the design value of the shear stress, is derived below for a concrete joint subjected to shear due to the compressive force in the flexural compressive zone caused by a bending moment. This can be described by Figure 8.5. The derivation is based on Johansson (2012a).

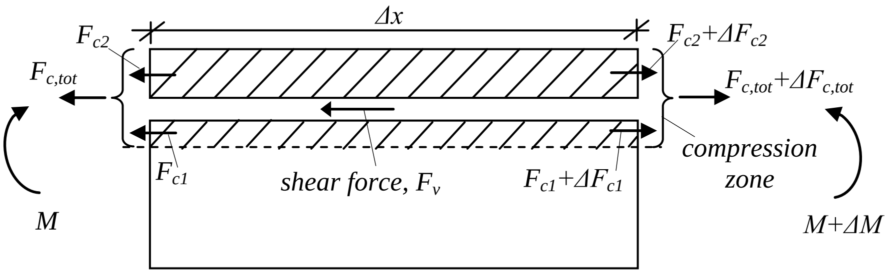


Figure 8.5 Schematic figure of the forces in a joint between concrete elements cast at different times.

The ratio between the longitudinal force in the new concrete area and the total longitudinal force acting in the compressive zone can be defined as

$$\beta = \frac{F_v}{\Delta F_{c,tot}} \tag{8.4}$$

where

$$F_v = \Delta F_{c2}$$

The increase of the compressive force, $\Delta F_{c,tot}$, over a certain length, Δx, of the structural member, can be calculated as the increase of the bending moment divided by the internal lever arm, z

$$\Delta F_{c,tot} = \frac{\Delta M_{Ed}}{z} \tag{8.5}$$

The shear stress at the interface is the same as the shear force, F_v, divided by the area over which it is acting. The shear stress, v_{Ed}, at the interface of the joint can then be determined as

$$v_{Ed} = \frac{F_v}{\Delta x b_i} = \frac{\Delta M_{Ed}}{\Delta x} \frac{1}{z b_i} \tag{8.6}$$

where

$$F_v = \beta \Delta F_{c,tot}$$

The shear force, V_{Ed}, can be defined as the derivative of the moment

$$V_{Ed} = \frac{\Delta M_{Ed}}{\Delta x} \qquad \text{when } \Delta x \to 0 \tag{8.7}$$

Hence, by inserting Equation (8.7) into Equation (8.6) an expression for the shear stress can be derived

$$v_{Edi} = \frac{F_v}{\Delta x b_i} = \frac{\beta \Delta F_{c,tot}}{\Delta x b_i} = \frac{\beta \Delta M_{Ed}}{\Delta x b_i z} = \frac{\beta V_{Ed}}{z b_i} \tag{8.8}$$

Figure 8.6a illustrates a slab with existing concrete and in Figure 8.6b new concrete is cast above the existing concrete. At this stage the new concrete is not hardened and it is important to notice that it is only the existing concrete that is able to carry the total self-weight. Consequently, the actions that affect the joint are those loads that will be added on the slab after the joint has hardened, see Figure 8.6c. It is important to notice that V_{Ed} in Equation (8.8) does not consider any contributions from the self-weights. It is the shear force caused by the load, q, see Equation (8.9). The same goes for the design moment, M_{Ed}.

$$V_{Ed} = V_{Ed}(q) \tag{8.9}$$

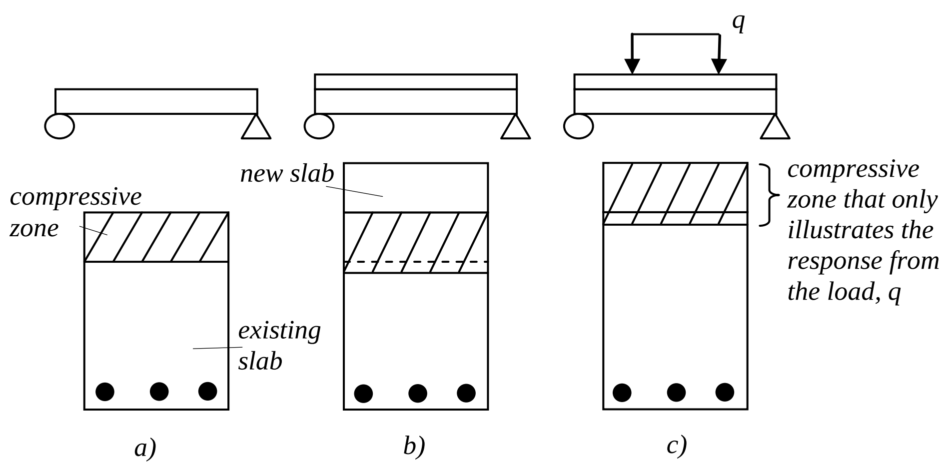


Figure 8.6 Schematical illustration of a joint between existing concrete and new concrete, a) existing concrete, b) existing concrete with new not hardened concrete, c) existing concrete with hardened new concrete.

The definitions of β in Equation (8.4) and β' in Equation (7.3), concerning shear between web and flanges, are similar, i.e. the ratio of how much of the shear force that is transferred in the chosen part that is to be designed for. It can therefore be argued that Eurocode 2 is not consistent, since β' is not included in Expression EC2 (6.20).

The background to Equation (8.3) is given in Section 3.2.2. There it is described that the shear resistance of a joint interface is based on a frictional model, see Figure 3.14. It is the roughness of the joint interface, the amount of transversal reinforcement across the joint and if the reinforcement is plain or ribbed that influence the shear capacity. In order to provide more information about what the different parts in Equation (8.3) takes into account these are explained below:

- cf_{ctd} the shear stress that can be resisted without any reinforcement or external compressive force acting over the joint, Boverket (2004)
- $\mu\sigma_n$ friction due to external normal stress, Johansson (2012a)

$\mu\rho f_{yd}$ friction due to normal stress generated by pullout resistance of transverse reinforcement

$0,5\upsilon f_{cd}$ upper limit with regard to crushing of small inclined struts (is derived in Section 8.3)

8.2.3 Discussion

When calculating the shear resistance at the interface between concrete cast at different times, the capacities of cohesion and friction are combined and added together in Equation (8.3), SIS (2008). Figure 3.18 shows how friction is generated by the pullout resistance of the transversal reinforcement crossing the joint. Tensile stresses in the transverse reinforcement result in compressive stresses at the joint interface. It can be noted that BBK 04 states the opposite, i.e. the addition principle is not used for calculation of the shear resistance at the interface between concrete cast at different times and the capacities of the different contributions are not added together, Betongföreningen (2010a).

It should be noticed that for high strength concrete the shear resistance at the joint interface will decrease due to fracture of the aggregates at crack formation, meaning that the effect of aggregate interlock will decrease, *fib* (2008).

There is risk to get confused when the correct value of the cohesion factor, c, should be chosen when looking at Paragraphs EC2 6.2.5(1) and EC2 6.2.5(5), see Section 8.2.1. The former paragraph discusses the joints shown in Figure 8.3, where the contribution to the shear strength, cf_{ctd}, should be taken as zero for $\sigma_n < 0$. This is because the transverse tensile stress, σ_n, will cause cracking which will result in loss of the aggregate interlock effects. Paragraph EC2 6.2.5(5) treats a special case where the joint is loaded by fatigue or dynamic loading where it is stated that the factor c should be halved. It can therefore be discussed what the value of the factor c should be if the joint at first has been subjected tension and thereafter becomes compressed. It can be argued that Paragraph EC2 6.2.5(5) gives some guidance to this question, i.e. some aggregate interlock effect can be regained when the crack is compressed and the factor c does not need to be taken as zero.

It can be added that according to Engström (2013) it is incorrect to say that the cohesion is completely gone after cracking of the joint. It is more correct to say that the cohesion due to the compressive stress acting on the joint can still be utilised since the joint interface regains some of the aggregate interlock effects.

8.3 Maximum transversal reinforcement

8.3.1 Requirements in Eurocode 2

In Eurocode 2 it is stated that the design shear resistance at the interface of a concrete joint should be limited according to Equation (8.10). This is stated in order to prevent crushing of small inclined compressive struts as illustrated in Figure 8.1. The derivation of the limitation of the design shear strength will also result in an upper limit of the reinforcement amount, A_s / s.

$$v_{Rd,i} = 0.5 v f_{cd} \tag{8.10}$$

v_{Rdi} is the design shear resistance, see also Equation (8.3)

In the derivation of the upper limit of the design shear strength the frictional coefficient, μ, will be included. The coefficient is explained in Section 8.2.1.

8.3.2 Explanation and derivation

The derivation of the maximum design shear resistance in Equation (8.10) is based on *fib* (2008). The combination of shear force along the joint interface and the tensile force in the transversal ties, i.e. the reinforcement crossing the joint, will result in an inclined compressive force that acts through the joint interface, with an angle, θ, see Figure 8.1. The compressive force is resisted by a series of compressive struts that are balanced by tensile forces in the transversal reinforcement. Due to the biaxial stress-state created from these tensile forces, the compressive strength of the inclined struts will be reduced according to

$$\sigma_{Rd,\max} = v f_{cd} \tag{8.11}$$

where

$$v \leq 1 \tag{8.12}$$

Nielsen (1984) came up with an approach that is based on theory of plasticity, i.e. both materials are assumed to have plastic behaviour and is illustrated in Figure 8.1, *fib* (2008). By letting the force in the steel be equal to the vertical component of the inclined compressive force, vertical equilibrium gives

$$\left(v f_{cd} \cdot b_i \cdot s \cdot \sin\theta\right) \sin\theta = f_{yd} \cdot A_{sv} \tag{8.13}$$

f_{cd} concrete compressive design strength

f_{yd} design yield strength of the reinforcement

A_{sv} cross-sectional area of one reinforcement unit

s spacing of transverse bars

b_i width of the joint section

θ angle of the compressive strut

The expression within the parentheses in Equation (8.13) represents the inclined compressive force. The product $s \cdot \sin\theta$ is the influence length of one reinforcement unit in the transverse direction of the compressive strut, i.e. perpendicular to the inclined compressive force, see the length x in Figure 8.7. In order to create equilibrium between the compressive force and the tensile force in the steel, the compressive force needs to be multiplied with $\sin\theta$ one more time, see Equation (8.13).

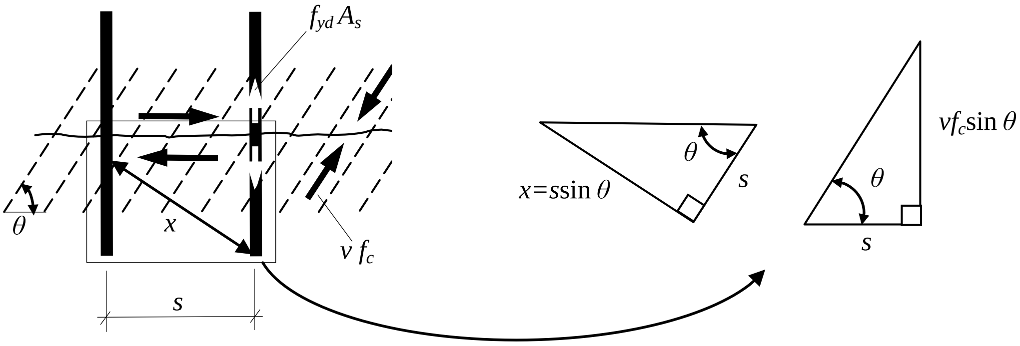


Figure 8.7 Illustration of the length, x, that is perpendicular to the compressive strut and dependent on the spacing of the transverse reinforcement units, s.

Equation (8.13) can be rewritten as

$$\sin^2\theta = \frac{f_{yd}}{vf_{cd}} \cdot \frac{A_{sv}}{b_i \cdot s} = \omega'_s \tag{8.14}$$

and

$$\cos^2\theta = 1 - \omega'_s \tag{8.15}$$

ω'_s mechanical reinforcement ratio

The desired response is a combined steel/concrete failure when the upper limit for the shear resistance, v_{Rdi}, is reached. In Equation (8.16) s·sin θ expresses the same length, x, that is shown in Figure 8.7. In order to create horizontal equilibrium between the inclined compressive force and the shear force acting along the joint interface, the expression within the parentheses in Equation (8.13) should instead be multiplied with cos θ, see Figure 8.8. Hence, the horizontal equilibrium for a part of the joint with length s becomes

$$v_{Rdi} \cdot b_i \cdot s = (vf_{cd} \cdot b_i \cdot s \cdot \sin\theta)\cos\theta \tag{8.16}$$

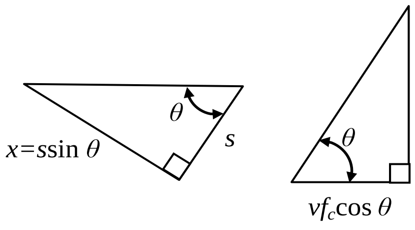


Figure 8.8 The horizontal component of the compressive stresses.

Inserting the expression of cos θ from Equation (8.15) into (8.16) and rearranging it results in

$$\frac{v_{Rdi}}{vf_{cd}} = \sqrt{\omega'(1-\omega')} \tag{8.17}$$

The frictional angle, ϕ, which was presented in Section 3.2.3, affects the angle of the compressive strut according to

$$\theta = 90° - \phi \tag{8.18}$$

This means that the compressive stresses will act perpendicular to the assumed saw-tooth model, where the frictional angle describes the angle of the geometry, compare to Figure 3.18. For rough or indented surfaces the frictional angle can according to *fib* (2008) be assumed to be within the interval 35º ≤ ϕ ≤ 54.5º (since the frictional coefficient is assumed to be 0.7 ≤ μ ≤ 1.4 according to the same reference). By inserting these values for the frictional angle into Equation (8.18) and using this expression in Equations (8.16) and (8.17) the upper limit for the design shear resistance at a joint interface can be derived

$$0.47 \leq \frac{v_{Rdi}}{vf_{cd}} \leq 0.50 \rightarrow v_{Rdi} \leq 0.50 \cdot vf_{cd} \tag{8.19}$$

From Equation (8.14) the maximum reinforcement amount can be determined according to Equation (8.20).

$$\frac{A_{sv}}{s} = \omega' \cdot \frac{b_i \cdot vf_{cd}}{f_{yd}} \tag{8.20}$$

8.3.3 Discussion

In Section 8.3.2 the upper limit in Eurocode 2 for the design shear strength at a joint interface is derived. The frictional coefficient used in the derivation is applicable for a rough surface, i.e. μ = 1.4. A rough surface will generate a large compressive force acting over the joint due to the pullout resistance of transverse reinforcement. Note that this will only be the case if the bond and anchorage of the reinforcement is sufficient. A rough surface, hence a large frictional coefficient, will cause large compressive forces acting at the joint interface which is unfavourable with regard to crushing of the concrete. It can be assumed that if a frictional coefficient of μ > 1.4 is used in design, the risk for crushing of the concrete is high. Probably, this will never be the case, since the magnitude of the frictional coefficient mentioned in Paragraph EC2 6.2.5(2) is much smaller than this, see Table 8.1.

8.4 Shear capacity due to dowel action

8.4.1 Requirements for dowel action

When the transverse reinforcement in a joint is not fully anchored in the concrete, dowel action will occur instead of a shear friction mechanism. Dowel action can also be the case if the surfaces of the joint interface are smooth and not cast against each other. This is because in such a case no joint separation, *w*, occurs. Dowel action is treated in BBK 04, but has unfortunately been left out from Eurocode 2, Boverket (2004). It is therefore of interest to highlight and describe this action.

In BBK 04, Equation (6.8.3a), the shear capacity of a dowel that has been arranged according to the requirements in Figure 8.9 can be determined according to Equation (8.21). A condition for Equation (8.21) is that splitting failure is prevented.

$$F_{vR} = \phi^2 \sqrt{f_{cd} f_{yd}} \tag{8.21}$$

F_{vR}	shear capacity of dowel
ϕ	diameter of the dowel
f_{yd}	design tensile strength of the dowel
f_{cd}	design concrete compressive strength
c	distance from the dowel to the free edge in the direction of the shear force

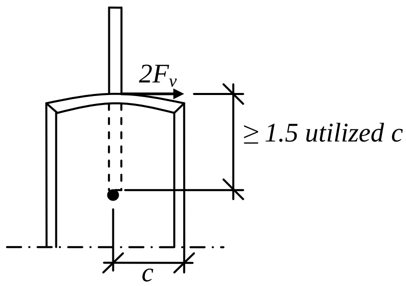


Figure 8.9 Geometrical requirements for a dowel subjected to a shear force F_v. The figure is based on Boverket (2004).

Engström and Nilsson (1975) suggest a minimum embedded length, l_a, of the dowel in order to be able to utilize the full capacity of the dowel action, see Equation (8.22).

$$l_a \geq 6 \cdot \phi \tag{8.22}$$

8.4.2 Explanation and derivation

Equation (8.21) is derived in the following text that is based on Højlund-Rasmussen (1963) in *fib* (2008). The derivation is at first performed for a dowel that is loaded in shear with an eccentricity, *e*. Thereafter an expression where *e* is equal to zero is derived. Theory of plasticity can be used when calculating the ultimate shear capacity in dowel action, because both the materials can be assumed to reach a plastic behaviour when the maximum shear force is approached, see Figure 8.10.

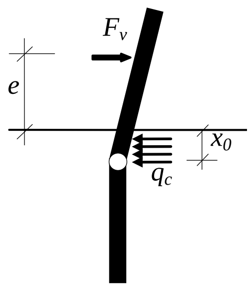


Figure 8.10 Model according to theory of plasticity for shear capacity of one-sided dowel pin embedded in concrete. When a one-sided dowel pin is subjected to shear force with a certain eccentricity e a plastic hinge will develop at a distance x_0 from the joint interface.

A tri-axial state of compressive stresses can be achieved for concrete if it is subjected to high stresses under a local bearing area, *fib* (2008). Sufficient concrete cover around the dowel pin is important in order to achieve this complex state of stress. It will increase the concrete compressive strength compared to the uniaxial compressive loading, CEB-FIP (1991). The increased compressive strength can be determined as

$$\sigma_{cc,\max} = kf_{cd} \tag{8.23}$$

where

k = 3.0 in BBK 04, Boverket (2004)

k = 4.0 in *fib* (2008)

When the failure mechanism has developed (ultimate state) the compressive stress in the concrete has reached its maximum value, which with regard to the tri-axial effects is expressed according to Equation (8.23). The concrete reaction, q_c, along the dowel pin per unit length, see Figure 8.10, is found as

$$q_c = kf_{cd} \cdot \phi \tag{8.24}$$

ϕ diameter of dowel pin

When the maximum shear force, F_{vR}, is reached, the section x_0, where the moment is at maximum, is found from where the shear force is zero.

$$x_0 = \frac{F_{vR}}{q_c} \tag{8.25}$$

To find the maximum load for when the dowel develops its failure mechanism the maximum moment, see Equation (8.26), is set equal to the plastic resistance moment, see Equation (8.27). By moment equilibrium at section x_0 in Figure 8.10 the maximum moment is found as

$$M_{\max} = F_{vR} \cdot e + F_{vR} \frac{F_{vR}}{q_c} - q_c \frac{1}{2} \cdot \left(\frac{F_{vR}}{q_c}\right)^2 = F_{vR} \cdot e + \frac{1}{2} \frac{F_{vR}{}^2}{q_c} \tag{8.26}$$

For a dowel pin with homogenous circular section, see Figure 8.11, the plastic resistance moment is found as

$$M_{yd} = f_{yd} \frac{\pi\phi^2}{8} \cdot \frac{4}{3} \frac{\phi}{\pi} = f_{yd} \frac{\phi^3}{6} \tag{8.27}$$

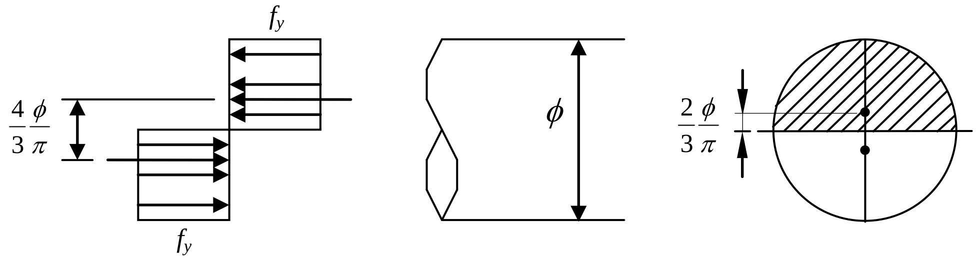


Figure 8.11 Plastic moment resistance of a dowel. The figure is based on fib (2008).

The shear resistance, F_{vR}, can be solved by letting the maximum moment be equal to the plastic moment. Then the shear resistance for a dowel loaded in shear with an eccentricity, e, can be expressed as

$$F_{vR} = c_0 \cdot c_e \cdot \phi^2 \sqrt{f_{cd} \cdot f_{yd}} \tag{8.28}$$

c_0 coefficient that considers the bearing strength of concrete

$$c_0 = \sqrt{\frac{k}{3}} \quad \text{(can be taken as } c_0 = 1{,}0 \text{ in design)}^3 \tag{8.29}$$

c_e coefficient that considers the eccentricity

$$c_e = \sqrt{1 + (\gamma \cdot c_0)^2} - \gamma \cdot c_0 \tag{8.30}$$

where

$$\gamma = 3\frac{e}{\phi}\sqrt{\frac{f_{cd}}{f_{yd}}} \tag{8.31}$$

If the eccentricity is set to zero in Equation (8.28) it can be shown that this is equal to Equation (8.21) that is the expression used in BBK 04 (6.8.3a), see Figure 8.12, Boverket (2004). For the full derivation of Equation (8.21) and Equation (8.28) see Appendix F.

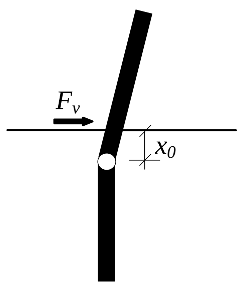


Figure 8.12 Model according to theory of plasticity for shear capacity of one-sided dowel pin embedded in concrete. No eccentricity of the shear force.

[3] In BBK 04 in Section 3.10.1 it is recommended to put the value of c_0=1.0; i.e., the value of k should not be higher than 3, Boverket (2004).

8.4.3 Discussion

It is unfortunate that shear resistance by dowel action has been left out in Eurocode 2. The structural engineers that are not familiar with using BBK 04 will most likely fail to identify situations when shear friction is insufficient and dowel action prevails. The shear capacity of the joint may in such a case be insufficient, if design is performed according to Eurocode 2, Section EC2 6.2.5, since the shear resistance due to dowel action is less than that of the shear friction. This is also why shear resistance due to shear friction is something the designer wants to obtain. However, when designing a joint between for instance a pre-fabricated wall and a slab, it may be difficult to obtain sufficient bond of the dowel pin and dowel action will be dominating the shear resistance of the joint. A factor that also influences the shear resistance is the interaction between the surfaces at the joint interface. Since the surfaces are smooth and not cast against each other, no or small joint separation, *w*, will be the case when shear slip occurs, and hence the shear resistance of the joint will be low.

It should be noticed that if a plain dowel is anchored by an end-anchor, a combined structural mode develops with a mix of both dowel action and shear friction, i.e. some contribution to the shear resistance is due to pullout resistance of the transverse bar.

According to Equation (8.21) the shear capacity will increase by increasing dimension of the dowel and strength of the concrete. In cases when the dowel is loaded with a shear force that acts with an eccentricity relative to the joint face, the shear capacity will be reduced, see Equation (8.28) and Figure 8.13. According to *fib* (2008) the reduction will about 40-60 %, if the eccentricity is of equal size as the bar diameter and such situations should thus be avoided.

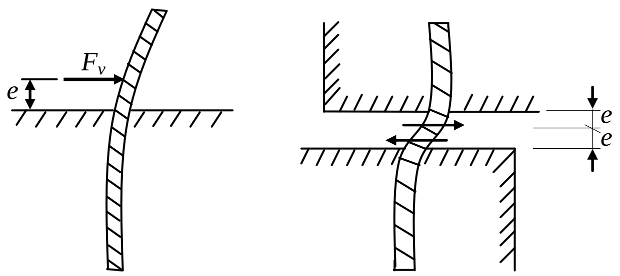


Figure 8.13 Shear transfer by dowel action in a bolt, pin or bar where a dowel pin with single or double fixation loaded with an eccentricity e is shown. The figure is based on fib (2008).

When the shear resistance is obtained by dowel action, no self-generated compressive force will be developed. Crushing of the concrete in the joint is in such a case not critical, as is the case for shear friction, why no upper limit of shear resistance needs to be taken into account. Instead the failure mechanism depends on the strengths of the different materials and the concrete cover, i.e. splitting failure must be avoided. If the concrete elements are small or if the concrete cover is insufficient, splitting of the concrete can occur. However, this can according to *fib* (2008) be avoided by providing sufficient concrete cover or splitting reinforcement. Since shear resistance by dowel action has been left out in Eurocode 2 the standard does not give any recommendations concerning this either, which is unfortunate.

9 Bond and anchorage

9.1 Structural response and modelling

In order to provide for a ductile structural behaviour the reinforcement in a part of a concrete member that is subjected to tensile stresses must be sufficiently anchored, Engström (2011a). If this is not the case, the type of failure that will occur, a so called anchorage failure, will be sudden and brittle. An anchorage failure is when the reinforcement is detached or released from the concrete, such as splitting or pullout failure, see Section 3.2.1. Sufficient anchorage means that the reinforcing bars are placed with an extension, beyond the considered section, that is long enough to build up a capacity equal to the applied force in this section. The force at the end of a bar must always be zero. The force increase in a reinforcement bar is achieved by bond stresses between the reinforcement steel and the surrounding concrete along a distance called transmission length. The bond mechanism has been described in Section 3.2.1.

At design of anchorage zones it is normally assumed that the bond stress, τ_b, at all sections along the anchorage length is equal to the bond strength, f_b. The bond strength represents the maximum force increase per surface unit area, S_b, of the reinforcing bar and is limited by the strength of concrete. It is also dependent of the surface characteristics of the reinforcement and the enclosing effect of surrounding concrete, SIS (2008).

If the provided anchorage length is too short, the maximum force increase will be exceeded and the reinforcing bars will be detached or released from the concrete, Engström (2011a). The maximum force increase per unit length is thus limited by the capacity of the mechanisms that enable force transfer between the reinforcement and the surrounding concrete as described in the introductory Section 3.2.1.

The length that is required in order to achieve the needed tensile capacity in a section is the required anchorage length, l_b, and is thus dependent on both the load effect and the maximum force increase per unit length. The maximum anchorage capacity corresponds to the length $l_{b,max}$ that correspond to the length required for the reinforcement to reach yielding without exceeding the maximum force increase per unit length, S_b. See the relation between maximum and required anchorage length in Figure 9.1.

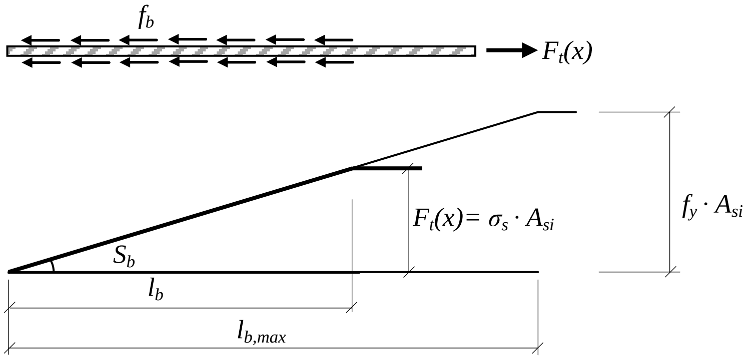


Figure 9.1 Relation between the required and maximum anchorage length.

A concrete member that is subjected to, for example, a uniformly distributed load will have a moment distribution that varies along the structure. The amount of longitudinal reinforcement needed in a member, subjected to bending moment, is calculated from this moment distribution and thus varies in the same way. The tensile force that should be resisted by the longitudinal reinforcement is equal to the moment, M_{Ed}, divided by the inner lever arm, z, in each section. Since the required amount of reinforcement varies, it is, due to economical and environmental reasons, advantageous to cut off the reinforcement such that the moment resistance follows the curve of the applied moment. This adaption is called curtailment.

When the reinforcement in a structure is curtailed it is important to also consider the need for anchorage. The reinforcement must be placed such that the force, which is intended to be resisted by the bar, can develop. This is often graphically illustrated as in Figure 9.2.

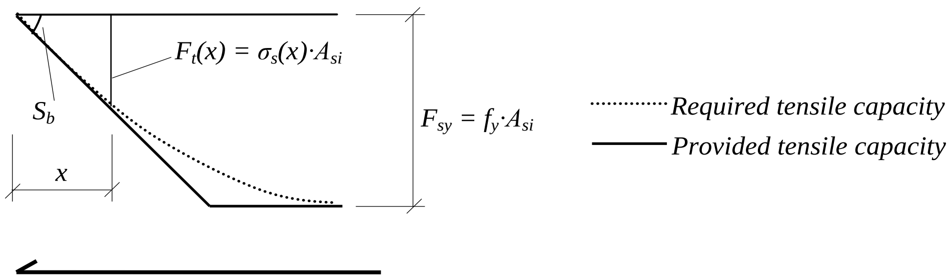


Figure 9.2 Illustration of how reinforcement must be placed in order to develop the needed tensile capacity in section x. The figure is based on Engström (2011a).

In Figure 9.2 it is shown that the provided tensile capacity is gradually increasing with an inclination corresponding to the maximum force increase per unit length, S_{bd}, in order to finally achieve the required tensile capacity. It should be noted that the maximum force increase per unit length of a group of bars is equal to the sum of all the bars individual contributions, see Figure 9.3.

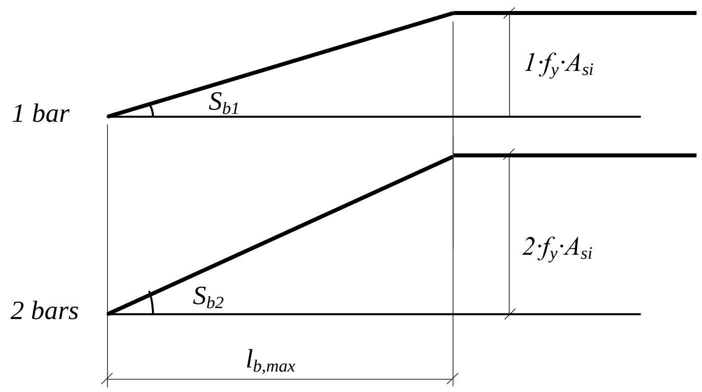


Figure 9.3 Both the total tensile force capacity and the maximum force increase per unit length increase in case of multiple bars. The figure is based on Engström (2011a).

The opposite of curtailment, where the reinforcement is cut off, is splicing, i.e. when reinforcing bars have to be spliced longitudinally in order to cover the whole length of the structure. There are different ways to enable splicing of bars where lapping is one of the most frequently used, Engström (2011a). The transmission of forces between bars in a lap joint is similar to the transmission of forces between reinforcement and surrounding concrete in case of anchorage. The bars in a lap splice do not have to be in close contact to each other. The load path is enabled through the concrete between the bars in a truss-like system, see Figure 9.4.

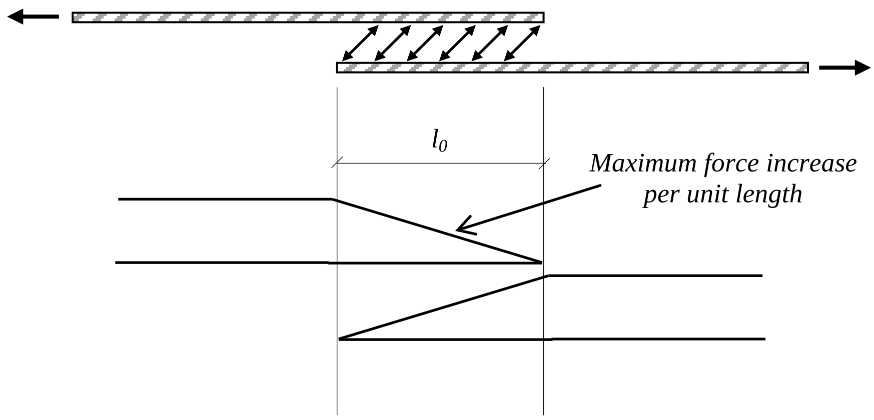


Figure 9.4 Transfer of forces between bars in a lap splice. The figure is based on Engström (2011a).

The contact forces are the same as described in Section 3.2.1 for bond between steel and concrete, inclined compressive forces with transverse and longitudinal components. Thus, the same types of failures, i.e. anchorage failures, must be considered in case of a lap splice. The lap length, l_0, must be long enough to build up a tensile capacity equal to the tensile force that needs to be transferred through the lap. This means that the lap length, l_0, is dependent of the basic required anchorage length, $l_{b,rqd}$.

9.2 Curtailment of reinforcement with respect to inclined cracks

9.2.1 Requirements in Eurocode 2

Curtailment of longitudinal reinforcement is treated in Eurocode 2, Section EC2 9.2.1.3 for beams. In this section it is stated that the reinforcement placed in a structure should be enough to carry the largest acting tensile force in each section, including the effect of inclined cracks in web and flanges. The additional tensile force due to inclined shear cracks, in a member with shear reinforcement, is in Eurocode 2 described as ΔF_{td}. The expression for ΔF_{td}, see Equation (9.1), can be found in Expression EC2 (6.18), Paragraph EC2 6.2.3(7). In order to better understand this expression see Section 5.5 where it is derived and explained more in detail.

$$\Delta F_{td} = 0.5 V_{Ed} \left(\cot\theta - \cot\alpha\right) \tag{9.1}$$

V_{Ed} shear force

θ angle of inclined struts, see Section 5.5

α inclination of shear reinforcement, see Section 5.5

As an alternative, it is according to Eurocode 2 allowed to estimate the influence of ΔF_{td} in each section by shifting the moment curve sideways, a distance a_l in the most unfavourable direction. The expression for a_l can be found in Expression EC2 9.2, see Equation (9.2).

$$a_l = \frac{z}{2}(\cot\theta - \cot\alpha) \tag{9.2}$$

z internal lever arm, see Figure 5.17, Section 5.5.2

Eurocode 2 also provides Figure EC2 9.2 that illustrates the effect of ΔF_{td} and a_l respectively, see Figure 9.5. It can be noted that l_{bd} is the design anchorage length which is explained more in Section 9.3.

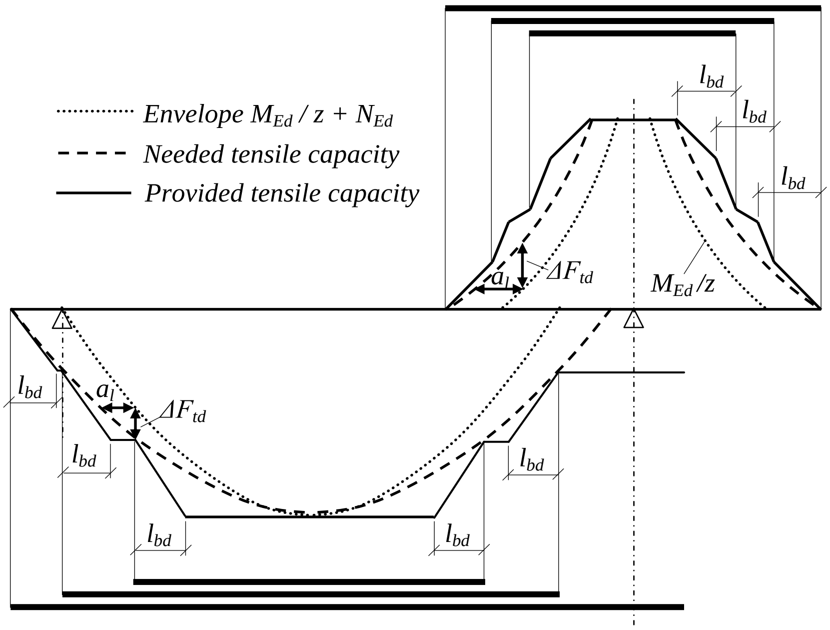


Figure 9.5 Curtailment of longitudinal reinforcement. The figure is based on SIS (2008).

It should be noted that for slabs all the rules in Paragraphs EC2 9.2.1.3 (1) to (3), concerning beams, are applicable. However, the shift a_l should for slabs be set equal to d, see Paragraph EC2 9.3.1.1(4).

9.2.2 Explanation and derivation

A structure subjected to shear force and bending moment will start to crack when the tensile strength of concrete is reached. The shear force, V_{Ed}, will cause inclined shear cracks, which is explained in Section 3.3 but also are presented more in detail in Chapter 5. In Section 5.5 it is also shown how the inclined shear cracks give rise to an additional tensile force, ΔF_{td}, which must be resisted by the longitudinal bending reinforcement. This additional tensile force is normally considered by the curtailment of reinforcement. In Figure 9.6 it is shown how this is achieved. The dotted line represents the force that must be resisted by the longitudinal reinforcement due to bending moment, i.e. $F_s = M_{Ed} / z$. When this curve is displaced vertically with various distances corresponding to $\Delta F_{td}(x)$ the required tensile capacity is obtained, see the dashed line.

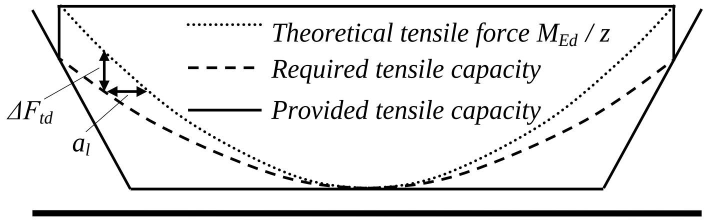


Figure 9.6 Illustration of how the longitudinal reinforcement is adapted to the required tensile force capacity.

In Paragraph EC2 9.2.1(3) it is stated that ΔF_{td} may be estimated by shifting the moment curve a distance a_l in the unfavourable direction. For a concrete member designed with shear reinforcement a_l is defined in Equation (9.2) in Section 9.2.1.

The relation between ΔF_{td} and a_l can from Equation (9.1) and (9.2) be identified as

$$a_l = \Delta F_{td} \frac{z}{V_{Ed}} \tag{9.3}$$

It should be noted that ΔF_{td} is the real increase of the tensile force, F_s, while a_l only is a mathematical consequence from the model. Hence, there is no physical explanation for a_l. However, a way to explain why a_l actually is a horizontal movement of the moment curve is by looking at the truss model of a beam subjected to a shear force, see Figure 9.7. The tensile force in each strut of the beam can be calculated by node equilibrium. Hence, the forces, F_{s1}, F_{s2} and F_{s3}, which are calculated, will include the whole contribution of the shear force.

If these forces are placed in relation to the envelope M_{Ed}/z, it is obvious that there will be sections that will have longitudinal forces higher than those from the moment distribution. This difference is the contribution to the longitudinal tensile force due to inclined cracks caused by shear force. However, this additional tensile force is divided in two, since it is distributed on the force couple that is resisting the bending moment, i.e. the longitudinal reinforcement and the compressive concrete zone. The distance a_l can thereby be illustrated as a horizontal movement, Johansson (2013).

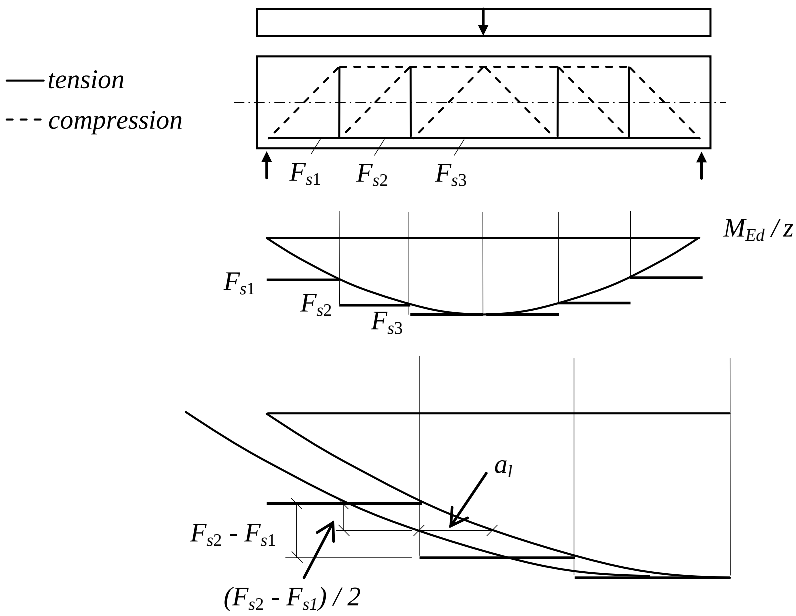


Figure 9.7 Illustration of a_l as a horizontal movement of the envelope of M_{Ed} / z.

It should be noted that the required tensile force in Figure EC2 9.2 never exceeds $M_{Ed,max} / z$, see Figure 9.5. This is because the tensile force is not affected by inclined cracks in these sections, Engström (2011a). This can be illustrated by a truss model as in Figure 9.8. As stated, the longitudinal forces calculated by means of a truss model will include also the contribution to the longitudinal tensile force from the shear force and the total tensile force will therefore not be larger than the force taken by the tie in the truss model.

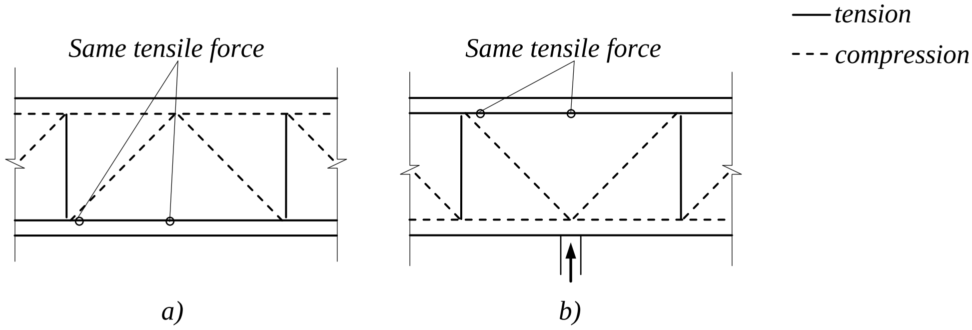


Figure 9.8 Truss model of beam subjected to bending moment, a) maximum moment section in span, b) maximum moment section at support. The figure is based on Engström (2011a).

9.2.3 Discussion

It is important to remember that the shear force and the shear reinforcement affect the amount of longitudinal tensile reinforcement needed in a structure. In Section 5.5 it is shown how the additional tensile force due to inclined shear cracks is dependent on the angle of the inclined struts. It is also shown that a smaller angle results in an increase of the additional tensile force. A small angle and an increased required longitudinal tensile capacity, F_{td}, will also increase the need for anchorage, see Figure 9.9.

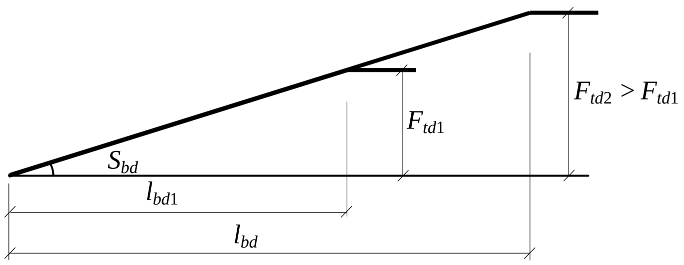


Figure 9.9 Larger required tensile capacity increases the need for anchorage.

From Paragraphs EC2 9.2.1.3(2) and EC2 9.3.1.1(4), see Section 9.2.1, it can be deduced that the horisontal shift, a_l, of the moment curve, in order to obtain the additional tensile force due to inclined shear cracks, is different for beams and for slabs. For slabs a_l should be equal to d. What this means can be shown by calculating the corresponding values of ΔF_{td} and θ.

Rearranging Equation (9.3) the relation between a_l and ΔF_{td} can be written as

$$\Delta F_{td} = a_l \frac{V_{Ed}}{z} \tag{9.4}$$

Inserting $a_l = d$ and assuming that z is equal to $0.9d$ gives

$$\Delta F_{td} = d \frac{V_{Ed}}{0.9d} = 1.11 V_{Ed} \tag{9.5}$$

If this is inserted in the definition of ΔF_{td}, see Equation (9.1), an expression for the angle θ can be obtained

$$\frac{1}{0.9} V_{Ed} = 0.5 V_{Ed} \left(\cot\theta - \cot\alpha \right) \tag{9.6}$$

For vertical shear reinforcement, i.e. $\alpha = 0$, the term cot α is equal to zero. Hence,

$$\cot\theta = \frac{1}{0.9 \cdot 0.5} \Leftrightarrow \tan\theta = 0.9 \cdot 0.5 \Leftrightarrow \theta = \tan^{-1}(0.45) = 24.2^{\circ} \tag{9.7}$$

This is a very small angle, compare to the lower limit of the inclination of the compressive strut that in Section 5.5.1 was stated as 22º in Equation (5.32). This implies that the rules stated in Eurocode 2 presume that no shear reinforcement is placed in slabs. This is because it is not possible to control the inclination of the compressive strut in members without shear reinforcement. Hence, in order to obtain a conservative solution a small angle is necessary, since it results in a large required tensile and anchorage capacity. It can be mentioned that for beams without vertical shear reinforcement the same value of $a_l = d$ applies, see EC2 9.2.1.3(2).

It should be noted that Paragraph EC2 9.3.1.1(4), concerning design of anchorage for slabs, states that the rules for beams in Paragraphs 9.2.1.3(1) to (3) apply also for slabs. However, according to Paragraph 9.2.1.3(2) the required anchorage capacity can be calculated by ΔF_{td} in Equation (9.1). This is contradictory, since a_l should be equal to d resulting in $\Delta F_{td} = 1.11 \cdot V_{Ed}$.

Paragraph EC2 6.2.1(4) states that it is not necessary to place shear reinforcement in slabs, see Section 5.4.1. However, it can be argued that it should be made clearer that the required tensile capacity calculated by $a_l = d$ presumes that no shear reinforcement is placed in the concrete member. It should also be made clearer that for cases when shear reinforcement is placed in slabs it is more favourable to calculate the required tensile capacity by Equations (9.1) or (9.2), since the strut inclination can be controlled and a smaller amount of longitudinal tensile reinforcement can be used. As an alternative to the above suggested improvements of Eurocode 2 it can be argued that the information should be changed so that all the rules presented in Paragraph EC2 9.3.1.1(4) applicable for beams also apply to slabs. Hence, the part of Paragraph EC2 9.3.1.1(4) that adds that the shift a_l should be set to d, should be removed, see Section 9.2.1.

Finally, it should be clarified that it is an approximate method to calculate the required tensile capacity of a reinforced concrete member by shifting the moment curve a distance a_l. In Figure 9.10 the required tensile capacity calculated by ΔF_{td} and a_l is compared. The contribution of ΔF_{td} has been directly added to the required force due to bending, M_{Ed} / z, in each section, x. The value of a_l has been considered by plotting the values of $M_{Ed}(x+a_l) / z$ at the position of x. The calculations have been made for a simply supported beam subjected to a uniformly distributed load of 1 kN/m. For more information about the calculation procedure, see Appendix G.

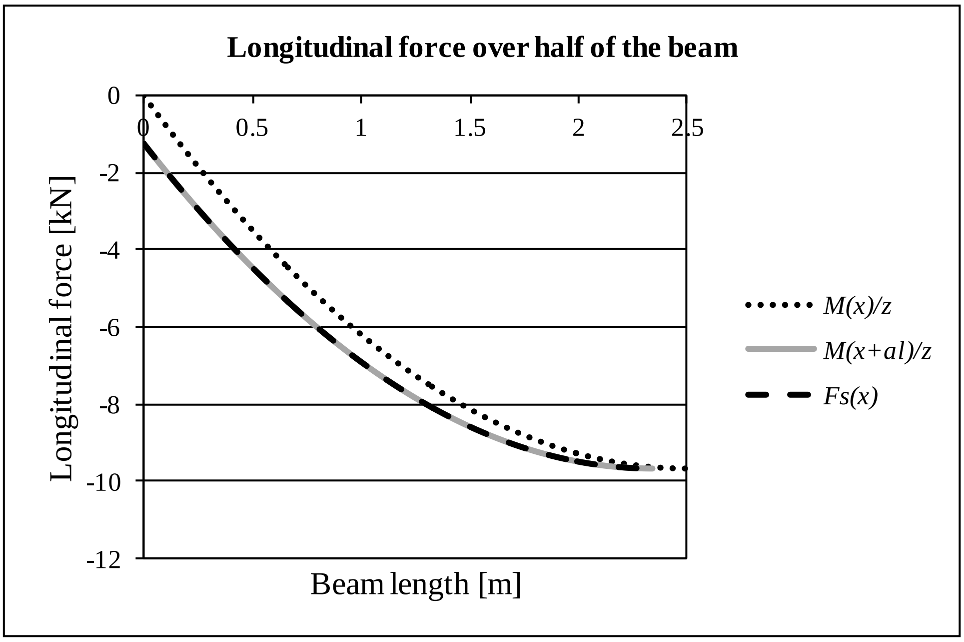


Figure 9.10 Comparison between required tensile capacities calculated by ΔF_{td}, see $F_s(x)$, or by using the shift method with a_l,, see $M(x+al) / z$.

It seems like a_l is a quite good approximation. It should be noted that for this load case the shift method with a_l resulted in slightly smaller values of the additional tensile force. However, the largest difference was only 3.2 %. The difference between the two methods, i.e. $F_s(x)$ and $M(x+a_l) / z$ was constant, see Appendix G.

9.3 Anchorage of bottom reinforcement at end supports

9.3.1 Requirements in Eurocode 2

Anchorage of bottom reinforcement at end supports is addressed specifically in Eurocode 2, see Section EC2 9.2.1.4. It is stated that at least 25 % of the amount of reinforcement required in the span section of a beam should be provided as bottom reinforcement at supports with little or no end fixity. For slabs the curtailment and anchorage may be carried out as for beams. However, for simply supported slabs the amount of reinforcement required in the bottom of support sections should be at least 50 % of the reinforcement in the span according to Paragraph EC2 9.3.1.2(1).

The design anchorage length, l_{bd}, at supports is according to Paragraph EC2 9.2.1.4(3) measured from the intersection point between beam and support, see Figure 9.11.

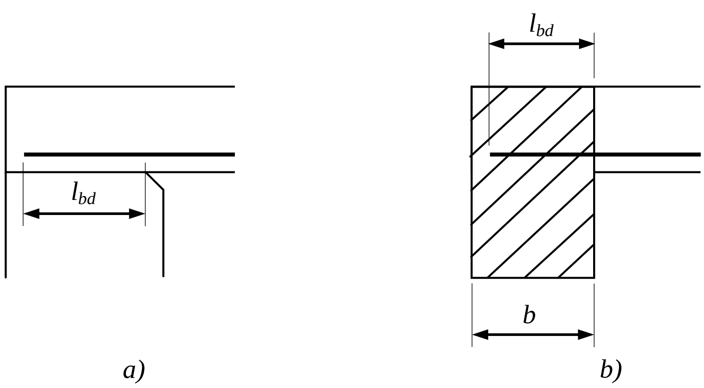


Figure 9.11 Anchorage length l_{bd} is measured from the line of contact between beam and support. The figure is based on SIS (2008).

The design anchorage length, l_{bd}, is in Eurocode 2 presented in Paragraph EC2 8.4.4(1), Expression EC2 (8.4) as a function of the basic required anchorage length $l_{b,rqd}$.

$$l_{bd} = \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd} \tag{9.8}$$

α_1- α_5 factors that consider different favourable effects within the anchorage zone, such as concrete cover, distance between bars and enclosing reinforcement

The basic required anchorage length, $l_{b,rqd}$, is according to Eurocode 2 calculated as

$$l_{b,rqd} = \left(\frac{\phi}{4}\right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}}\right) \tag{9.9}$$

σ_{sd} design stress of the bar at the position from where the anchorage is measured from

The design value of the bond strength, f_{bd}, can be found in Expression EC2 (8.2), see Equation (9.10).

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctd} \tag{9.10}$$

η_1 coefficient related to the quality of the bond condition and the position of the bar during concreting

η_2 coefficient related to diameter of the reinforcing bar

f_{ctd} design tensile strength of the concrete

The tensile force that needs to be anchored at support sections is affected by inclined shear cracks and can be determined according to Paragraph 6.2.3(7), see Equation (9.1) in Section 9.2.1, or by the shift rule according to Expression EC2 (9.3), see Equation (9.11)

$$F_E = |V_{Ed}| \cdot \frac{a_l}{z} + N_{Ed} \tag{9.11}$$

a_l see definition in Equation (9.2), Section 9.2.1.

N_{Ed} normal force that should be added or subtracted from the tensile force

In addition to the rules in Section EC2 9.2.1.4, Section EC2 6.5.4, concerning design of nodes according to the strut and tie method, provides information about anchorage that applies to support sections. In Paragraph EC2 6.5.4(7) it is stated that the anchorage length, when considered by a strut and tie model, should extend over the whole node region, and that the anchorage of reinforcement starts at the inner face of the supports, i.e. at the beginning of the node. In this case the support section is considered as a compression-tension node, see Figure 9.12.

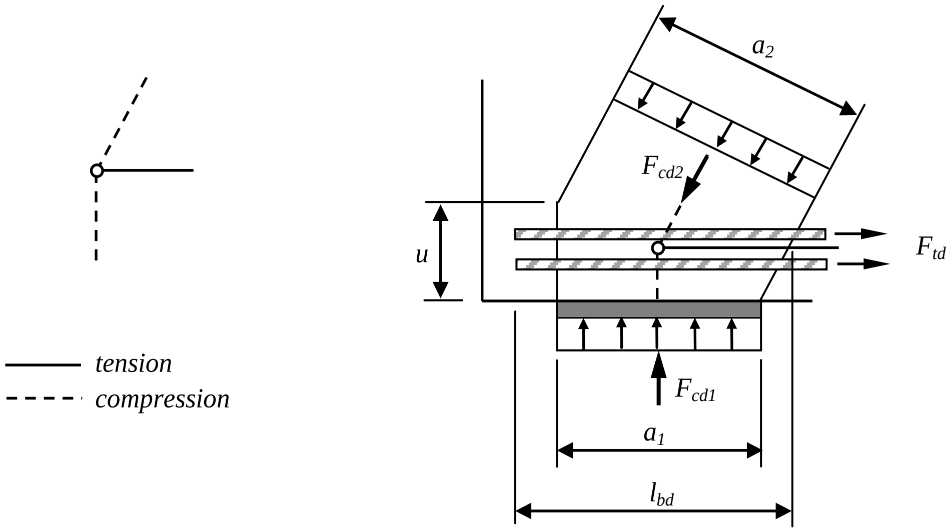


Figure 9.12 Definition of a compression-tension node and how this applies to anchorage in end support sections. The figure is based on SIS (2008)

9.3.2 Explanation and derivation

The basic required anchorage length is dependent on the design bond strength f_{bd}. As explained in Section 9.1, the bond strength can be seen as the maximum force increase per surface unit area of the reinforcing bar, Engström (2011a). This relation can be described as

$$f_{bd} = \frac{S_{bd}}{\pi\phi} \tag{9.12}$$

S_{bd} Maximum force increase per unit length

ϕ diameter of the reinforcing bar

The basic required anchorage length, $l_{b,rqd}$, is obtained by assuming an average bond stress, equal to the bond strength, which acts over the full perimeter of the bar and uniformly along the anchorage, Hendy and Smith (2010). The expression for $l_{b,rqd}$ given in Eurocode 2, see Equation (9.9), can be derived from Figure 9.1 in Section 9.1. According to Engström (2011a) the relation between maximum force increase per unit length, S_{bd}, and the required tensile force capacity, F_{td}, can be described as

$$l_{b,rqd} = \frac{F_{td}}{S_{bd}} \tag{9.13}$$

F_{td} Required tensile force capacity at the position from where the anchorage is measured

The required tensile force capacity can be expressed as

$$F_{td} = \sigma_{sd} \cdot A_{si} \tag{9.14}$$

σ_{sd} required stress in the bar at the position from where the anchorage is measured

A_{si} area of one reinforcement bar

By inserting Equations (9.12) and (9.14) into Equation (9.13) the expression for basic required anchorage length is derived as it is stated in Eurocode 2, see Equation (9.15).

$$l_{b,rqd} = \frac{\sigma_{sd} \cdot A_{si}}{f_{bd} \cdot \phi\pi} = \frac{\sigma_{sd} \cdot \phi^2 \pi}{f_{bd} \cdot 4 \cdot \phi\pi} = \frac{\sigma_{sd} \cdot \phi}{f_{bd} \cdot 4} \tag{9.15}$$

The maximum basic anchorage length when the design yield strength, f_{yd}, of reinforcement is utilised in design. This means that σ_{sd} is replaced by f_{yd} in Equation (9.15).

$$l_{b,\max} = \frac{f_{yd} \cdot \phi}{f_{bd} \cdot 4} \tag{9.16}$$

It is important to understand the difference between the basic required anchorage length, $l_{b,rqd}$, and design anchorage length, l_{bd}. As can be understood from the derivation $l_{bd,rqd}$ refers to a straight bar in a simple situation. The expression for l_{bd} on the other hand considers influences on the anchorage region, e.g. confinement or shape of the bars.

The expression in Equation (9.11) of the force that should be anchored at support sections is actually the same expression as the one considering the additional tensile force due to inclined shear cracks described before in Equation (9.1) , Section 9.2.1. The difference is that the force that will be anchored according to Equation (9.11) is that due to inclined cracks and, if any, the normal force, N_{Ed}, which acts on the structure. See comparison of the two expressions in Equation (9.17) and (9.18).

$$\Delta F_{td} = \frac{V_{Ed}}{2}(\cot\theta - \cot\alpha) \tag{9.17}$$

$$F_E = |V_{Ed}| \cdot \frac{a_l}{z} + N_{Ed} = \frac{|V_{Ed}|}{2}(\cot\theta - \cot\alpha) + N_{Ed} \tag{9.18}$$

θ inclination of the shear crack

α inclination of the shear reinforcement

However, the tensile force that should be anchored at end supports is not always the same as the additional tensile force at the intersection between beam and support. As an example anchorage of bottom reinforcement at support sections for a simply supported beam can be illustrated as in Figure 9.13. It is shown that also the bending moment in this section will affect the force that needs to be anchored.

Figure 9.13 also shows the reason why the anchorage length according to Eurocode 2 should be measured from the intersection between beam and support. This is because no shear cracks occur in the support section and no contribution from inclined cracks should therefore be considered behind the intersection point. Hence, the largest force and thus the one that must be anchored is the one at the intersection point.

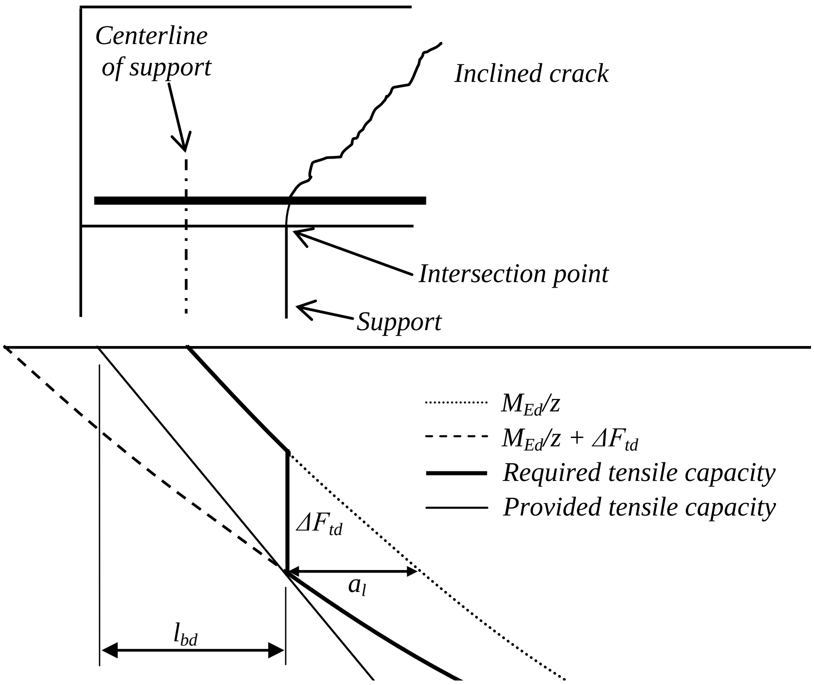
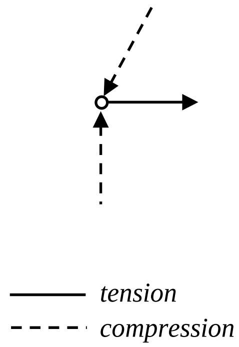


Figure 9.13 Anchorage of bottom reinforcement at end support.

As shown in Figure 9.12 the anchorage should extend over the whole node region. According to Betongföreningen (2010a) this is because each component of the reaction force at the support must be diverted by a transverse tensile force, i.e. the one provided by the longitudinal reinforcement. This is illustrated in Figure 9.14.



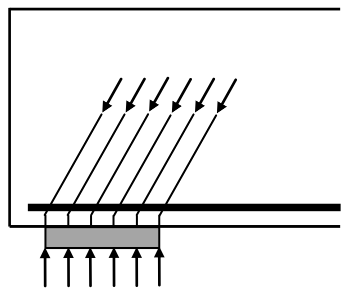


Figure 9.14 Each component of the compressive force needs a transverse tensile force to change direction. The figure is based on Betongföreningen (2010a).

According to Eurocode 2 it is always required to anchor a certain amount of reinforcement at the support sections. This is simply because of safety reasons. The general safety approach applied in Eurocode 2 is that all concrete structures should be reinforced in order to provide ductility and thereby reduce the risk for brittle failures.

9.3.3 Discussion

There are differing views on how to achieve sufficient anchorage of bottom reinforcement at supports, especially at end supports. Some argue that the anchorage capacity is sufficient as long as the design anchorage length is fulfilled beyond the intersection between beam and support. Other mean that the anchorage length at least must reach beyond the center of the support (the theoretical support) and some say anchorage must be provided over the entire node region.

Betongföreningen (2010a) provides a reasonable explanation why the entire node region should be anchored, presented in Section 9.2.1. However, it can be argued that this is not practically possible, since it in many cases there is too little space, if any, beyond the theoretical support section. Below, some anchorage situations are illustrated and discussed in order to provide some thoughts about this matter.

Figure 9.15 shows a situation where the required anchorage is fulfilled according to the explanation in Figure 9.13. However, anchorage is not provided over the entire node. This will result in that some of the reaction force must be diverted by tensile stresses in the concrete that will have an unfavorable effect on the load bearing capacity of the node, Betongföreningen (2010a). In order to reduce the stresses that act in the node region the reinforcement can be placed in several layers, since this increases the height, *u*, of the node region, see Figure 9.12.

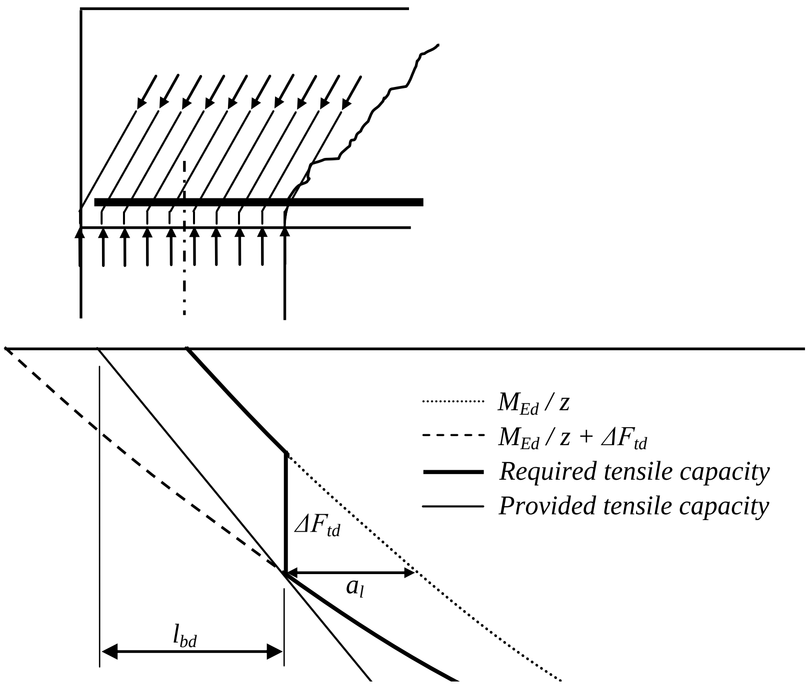


Figure 9.15 Anchorage of bottom reinforcement at end support. The entire compressive stress field is not balanced by the reinforcement.

It should be added that it is not always necessary to place the reinforcement over the full length of the support in order to anchor the entire node region. Such an example is shown in Figure 9.16. It should not be forgotten that theory of plasticity, in many situations, enables the designer to choose the way the forces are going to be resisted by the reinforced concrete member. Hence, the designer can thereby choose the effective width of the support, b_{ef}, which is a function of the concrete compressive strength and the compressive force acting in the support area.

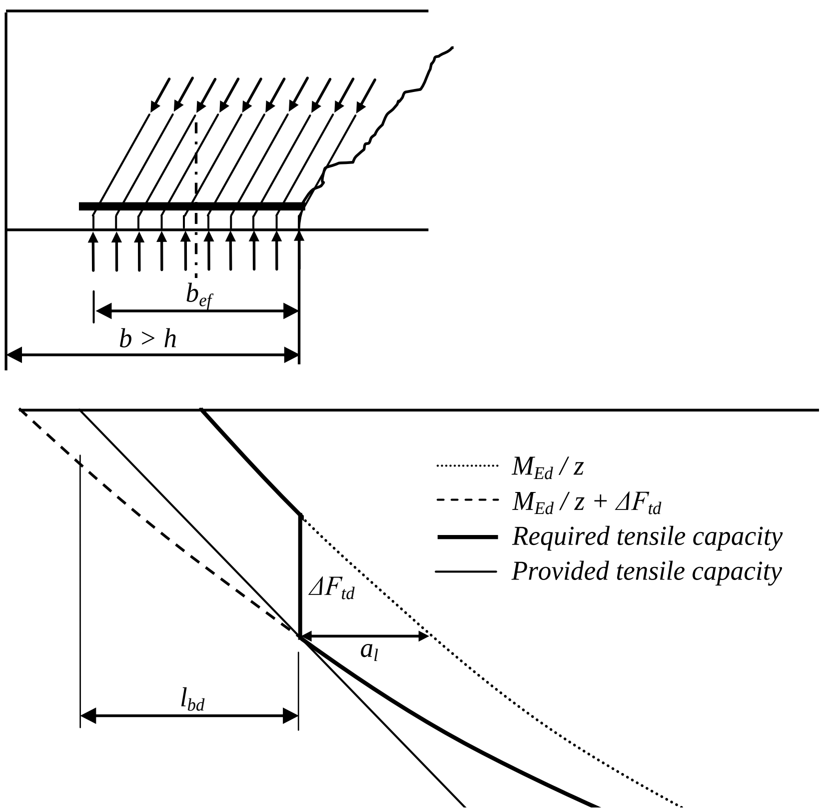


Figure 9.16 Anchorage of the entire node region at end support.

To alert users of Eurocode 2 that the support region can be considered as a node region, it would be good to have a reference between Sections EC2 9.2.1.4 and EC2 6.5.4, see Section 9.3.1. Such a reference might also increase the understanding of the behaviour of a support region and what it means for the anchorage of reinforcement.

Finally, it is important to also consider the contribution from the positive moment, i.e. M_{Ed} / z, when determining the required anchorage at support sections. It should be emphasized that by using Equation (9.11) there is a risk to forget about this contribution moment. However, if the detailing design is performed graphically as in Figure 9.13 this risk can be eliminated. The expression F_E, in Equation (9.11), can therefore be seen as a force that should be added to the required tensile force caused by bending moment but this must be made clearer in Eurocode 2. However, in order not to confuse the users of Eurocode 2 it is perhaps better to include the contribution from the bending moment into the expression for the tensile force that needs to be anchored at support sections, i.e. Equation (9.11) should instead be written as

$$F_E = |V_{Ed}| \cdot \frac{a_l}{z} + \frac{M_{Ed}}{z} + N_{Ed} \tag{9.19}$$

Alternatively, Equation (9.11) should be excluded from Chapter EC2 9 since the additional longitudinal tensile force due to inclined shear cracks, ΔF_{td}, already is stated in Paragraph EC2 6.2.3(7), see Section 5.5. The relation between a_l and ΔF_{td} should be explained in Section EC2 6.2.3, since it can be argued that Chapter EC2 9 should provide rules and recommendations on how to detail reinforcement and refer back to expressions for calculation of required capacities stated in previous chapters in Eurocode 2.

In addition to what have been stated about the amount of bottom reinforcement that should be placed at supports in Section 9.3.2 it should be noted that the required amount differs for beams and for slabs. The reason why more reinforcement is required in slabs is not known. However, a guess is that it might have something to do with enabling distribution of forces sideways, which is not as important in beams as it is in slabs.

9.4 Lapping of longitudinal reinforcement

9.4.1 Requirements in Eurocode 2

To be able to create reinforced concrete members longer than available reinforcement bars it is necessary to splice reinforcement such that forces can be transmitted between the bars. Splicing might also be necessary in some reinforcement configurations, such as torsional links or stirrups. According to Eurocode 2, Section EC2 8.7.1, splicing can be established by lapping, welding or mechanical devices. This section addresses lap splices, since these require careful reinforcement detailing.

In order to ensure transmission of forces between bars, prevent splitting failures of concrete and avoid large cracks, lap splices must be arranged with certain distances to each other, both longitudinally and transversally. The bars that are spliced together should also be placed within a certain distance to each other. Figure EC2 8.7 in Eurocode 2, Paragraph EC2 8.7.2(3), presents the rules that should be fulfilled for laps in tension, and is here shown in Figure 9.17.

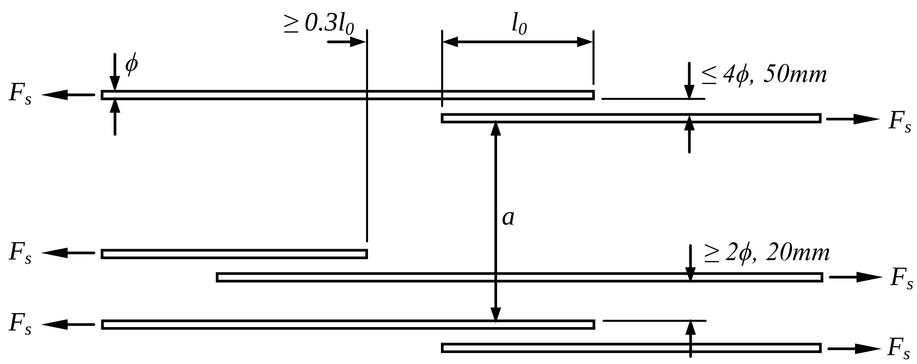


Figure 9.17 Detailing of adjacent lap splices in tension. The figure is based on SIS (2008).

The length, l_0, is the design lap length calculated according to Expression EC2 (8.10) in Eurocode 2 as

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 \; l_{b,rqd} \qquad l \geq l_{0,\min} \tag{9.20}$$

α_1, α_2, α_3 and α_5 are coefficients that consider different favourable effects influencing the lap length, the same as for anchorage.

$l_{b,rqd}$ basic required anchorage length, see Equation (9.9) in Section 9.3

α_6 coefficient dependent on the percentage of reinforcement, ρ_1, lapped within the section and is calculated as

$$1.0 \leq \left(\frac{\rho_1}{25}\right)^{0.5} \leq 1.5$$

see also Table 9.1 corresponding to Table EC2 8.3.

Table 9.1 Values of the coefficient α_6. The table is based on SIS (2008).

Percentage of lapped bars relative to the total cross-section area	< 25 %	33 %	50 %	> 50 %
α_6	1	1.15	1.4	1.5

In Paragraph EC2 8.7.2(4) it is added that when all the provisions in Figure 9.17 are fulfilled, it is allowed to lap 100 % of the bars in tension in one section, provided that they are all in one layer.

In addition to the requirements mentioned above it can be deduced from Section EC2 8.7.4 that transverse reinforcement should be placed in the lap zone. This is according to Paragraph EC2 8.7.4.1(1) in order to resist transverse tensile forces. It should be mentioned that it according to Eurocode 2 is allowed to take advantage of transverse reinforcement that have been placed in the structure for other reasons, if lapped bars have diameters smaller than 20 mm or if the percentage of lapped bars in any section is less than 25%.

9.4.2 Explanation and derivation

The expression for the design lap length, l_0, in Equation (9.20) resembles the expression for design anchorage length, l_{bd}, see Equation (9.8) in Section 9.3. In fact, the design lap length is almost the same as the design anchorage length multiplied with the factor α_6 that considers the percentage of bars that are lapped within the same section. It should be noted that the factor α_4, which considers the effect of welded transverse bars, has been left out of the expression for design lap length. The reason for this has not been investigated in this Master's thesis.

The factor α_6 varies between 1.0, when less than 25 % of all bars are spliced in the same section, and 1.5 when more than 50 % of the bars are spliced in the same section. It is therefore important to know the amount of bars that are spliced in one section. According to Eurocode 2 a section is determined as the region within the distance of $0.65l_0$ in each direction from the centre of the lap length considered, see Figure 9.18. All lapped splices with their centre lines within this region are considered to be in the same section as the considered splice. Splice B and C in Figure 9.18 are not considered to be in the same section as splice A. Further, two out of four bars are spliced within the considered section and the percentage ρ_1 is therefore 50 %.

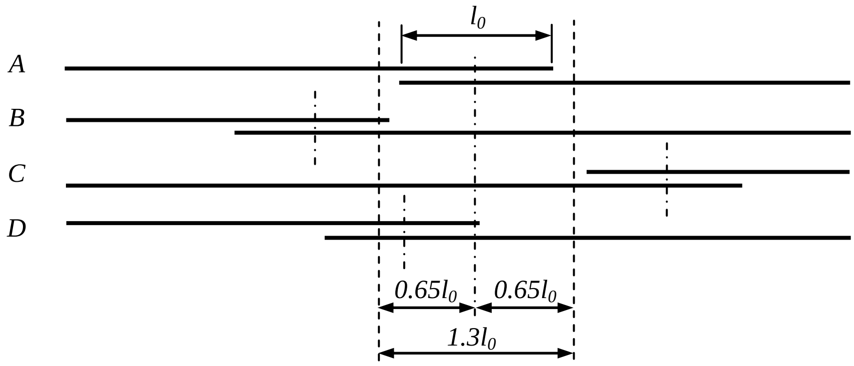


Figure 9.18 Definition of splice section for lap splices in tension according to Eurocode 2. The figure is based on SIS (2008).

There is one requirement provided by Eurocode 2 in Figure 9.17 that is worth some extra attention. It is the rule that implies that lap ends should be placed with a distance to each other of at least 0.3 times the design lap length, l_0. This requirement is new in Sweden, since BBK 04, Boverket (2004), doesn't provide any rules for the distance between lap ends.

Because of this difference between the current standard and the previous handbook it is interesting to investigate the background or purpose of the new requirement in Eurocode 2. In the background document Commentary to Eurocode 2, ECP(2008a), nothing about this is mentioned. Neither is it in Model Code 2010, *fib* (2012) and Model Code 1990, CEB-FIP (1991). However, in *fib* (2010) it is stated that staggered splices with a distance of $1.3l_0$ between the centrelines have no influence to each other. Moreover, stress concentration due to mutual influence of neighbouring splices can be limited or even eliminated by staggering of splices in the longitudinal direction.

This reason for the required distance between lap ends can also be found in Leonhardt (1974), which is written by the famous professor Fritz Leonhardt. According to Leonhardt (1974) the reason why the lap ends should be placed with a certain distance to each other is because of the transverse stress fields that are created for each bar in a lap due to bond stresses. In Section 3.2.1 it was shown that when a reinforcing bar is subjected to tension, the bond between reinforcement and concrete depends on inclined compressive stresses, see Figure 3.8 to Figure 3.11. Because of the inclined stress field there will be a transverse component that creates compressive stresses around the bar that must be equalised by tension in the concrete, Engström (2011). The bond stresses, τ_b, that hence are related to tensile stresses created in the concrete, increase from the end of the bar, see Figure 9.19.

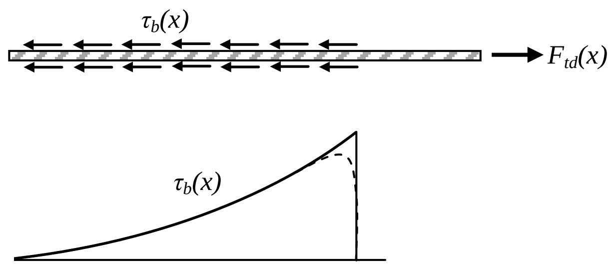


Figure 9.19 Distribution of bond stress along the transmission length. The figure is based on Engström (2011a).

This is also the case for bars in a lap splice. However, in this case the stresses of the two bars lapped together will be added to each other. The resulting distribution of transverse tensile strain over a lap splice, illustrated by Leonhardt (1974), can be seen in Figure 9.20. The stress σ_q here corresponds to the transversal component related to the bond stress illustrated in Figure 3.11, Section 3.2.1.

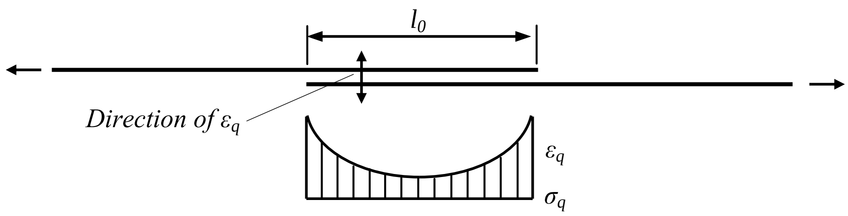


Figure 9.20 Qualitative distribution of transverse tensile strain over the transmission length of a lap joint. The figure is based on Leonhardt (1974).

In order to avoid high concentration of tensile stresses in the concrete around the bar that might cause splitting cracks in the concrete, see Figure 3.11 in Section 3.2.1, Leonhardt (1974) presents the following suggestions of appropriate and inappropriate solutions, see Figure 9.21. Note that the distance between lap ends in the appropriate configuration is the same as the requirement provided by Eurocode 2 in Figure 9.17.

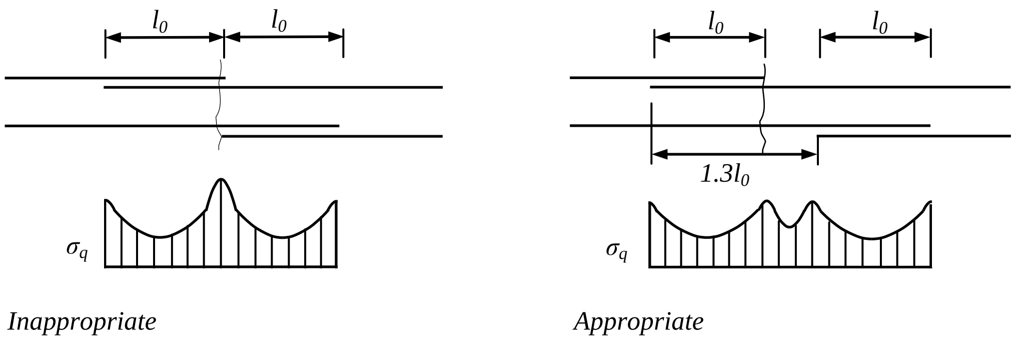


Figure 9.21 Illustration of appropriate and inappropriate arrangements of lap splices according to Leonhardt (1974).

It has not been verified that this is the reason for the requirements in Eurocode 2. However, it is known that Leonhardt (1974) have influenced the rules for reinforced concrete detailing previously used in Germany, which in turn are likely to have contributed to the development of the new European Standards. Therefore, it is believed that this is an important contribution to the rules for lapped splices in Eurocode 2.

According to Eurocode 2 it is, in addition to the rules concerning distances between bars and lap ends in case of lap splices, in most situations also required to place transverse reinforcement in the lap zone. According to Paragraph 8.7.4.1(1), transverse reinforcement is required in the lap zone in case of a lap splice subjected to tension. Because of the inclined stress field between the bars, see Figure 9.4, there will be a transverse component that tries to push the reinforcing bars away from each other, creating tensile forces which may result in splitting cracks in the concrete surrounding the splice zone. A so called splice failure can also occur due to spalling of concrete. More about this is explained in Section 3.2.1. However, the transverse forces can be balanced by transverse reinforcing bars that thus are of great importance for the capacity of the lap splice, Engström (2011). To know more about transverse reinforcement for a lap splice reference is made to Magnusson (2000). According to Magnusson (2000) several experiments performed by Eligehausen (1979) are the basis for what is stated about lap splices in CEB-FIP Model Code 1990 (1993).

9.4.3 Discussion

Laps between bars should according to Eurocode 2 normally be staggered and symmetrically placed. However, when all provisions given in Figure 9.17 are fulfilled and all the bars are in one layer, it is allowed to put all lapped bars in tension in one section. This can however seem a bit contradictory. If all splices are put in the same section the condition of a certain distance between lap ends in Figure 9.17 cannot be fulfilled, see Figure 9.22. It can be argued that the requirements in Eurocode 2 mean that when all the bars are placed with proper transversal distance to each other, all splices are allowed to be placed within the same section, Engström (2013).

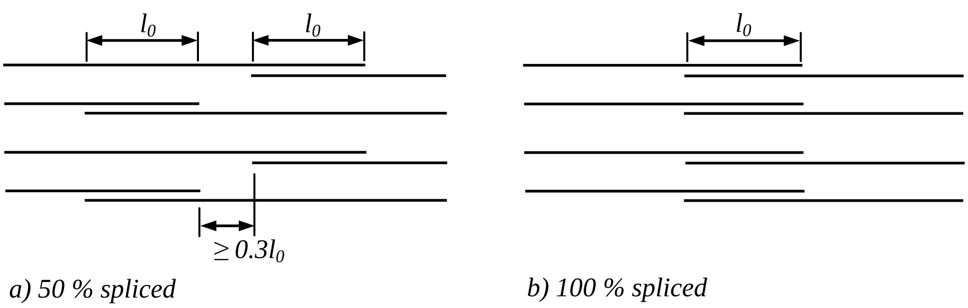


Figure 9.22 Illustration of different configurations of lap splices, a) 50 % of the bars are spliced in each section; required distance between lap ends is fulfilled, b) 100 % of the bars are spliced in one section; required distance between lap ends is not fulfilled.

According to BBK 04, Boverket (2004), it is allowed to splice all bars in one section, if sufficient concrete cover and spacing of bars is ensured and if transverse reinforcement in the lap zone has been arranged with special care, see also Svensk Byggtjänst (1990). This provides additional basis for Engström's argument in the preceding paragraph. It should also be noted that BBK 04 do not provide any requirements concerning minimum distance between lap ends, see Figure 9.23. According to BBK 04 the lap length, l_0, should be at least the same as the anchorage length l_{bd}, but should not exceed 80ϕ. It should be noted that a factor corresponding to α_6 is not included in BBK 04 meaning that the requirements in Eurocode 2 are more conservative due to an increased lap length in case of several splices in the same section.

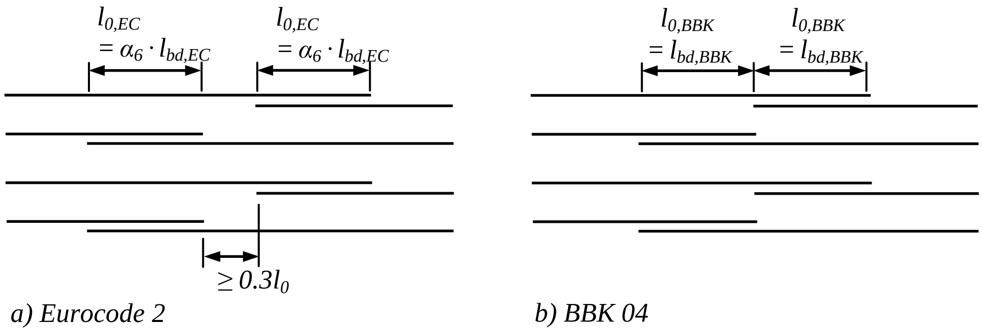


Figure 9.23 Comparison of lap splices arranged according to Eurocode 2 and BBK 04. The amount of spliced reinforcement in each section is in both examples 50 %.

No other reason for the required distance of $0.3l_0$ between lap ends in Eurocode 2, than the one expressed by Leonhardt (1974) presented in Section 9.4.2, has been found. In the Manual of Concrete Practice ACI 318-05, ACI (2007), there are however similarities. To allow splicing of all the bars in one section it is according to ACI (2007) required to increase the lap length from $1.0l_0$ to $1.3l_0$, see Figure 9.24. Otherwise only 50 % of all bars can be spliced in one section. A section is here defined as the region within the lap length. It should be noted that l_0 in Figure 9.24 corresponds to the design lap length. However, it has not been deduced from ACI (2007), if this also correspond to the design anchorage length.

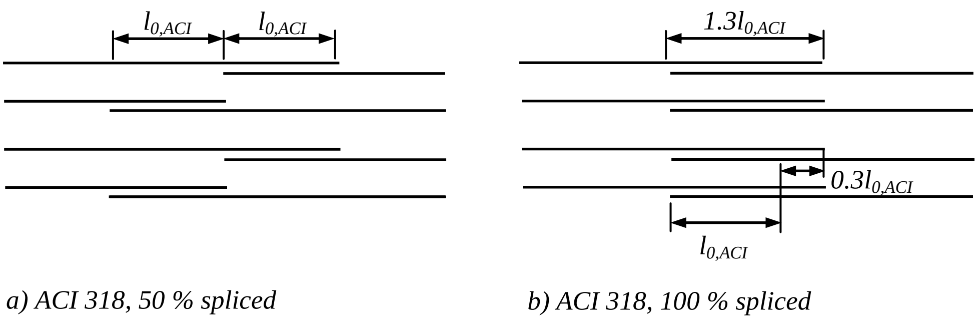


Figure 9.24 Lap splices according to ACI 318-05. The lap length should be increased from $1.0l_0$ to $1.3l_0$, if all laps are spliced in one section, ACI (2007).

Hendy and Smith (2010) provide another explanation of the rules set up in Eurocode 2. According to them, the percentage of lapped splices for determination of the factor α_6, for the arrangement in Figure 9.25 corresponds to 50 %. This is in agreement with the rules provided in Eurocode 2, see Figure 9.18. However, in contradiction to the definition presented in Figure 9.22, Hendy and Smith (2010) also states that the arrangement in Figure 9.25 shows an example of what is considered as 100 % splicing. This means that the solution in Figure 9.25 is only allowed in case of single layers, see the provisions in Paragraph EC2 8.7.2(4) stated in Section 9.4.1. However, the interpretation made by Hendy and Smith (2010) can be questioned.

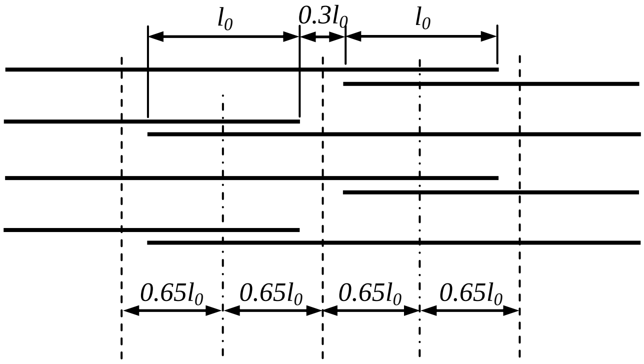


Figure 9.25 100 % lapping according to Hendy and Smith (2010). 50 % is lapped in each section for determination of α_6.

To be able to splice all bars in one section is favourable, since the same reinforcement units can be used repeatedly as in Figure 9.22b). If only 50 % of the bars can be spliced in each section, as in Figure 9.22a), the number of different reinforcement units is doubled. This means more work and planning at the construction site. However, it can be quite difficult to make room for all bars, if they are all spliced in the same section. It is also important to remember that the proportion of spliced bars in one section will influence the lap length. If 50 % of all bars in one section are spliced, then $\alpha_6 = 1.4$. However, if all bars are lapped within the same section, then $\alpha_6 = 1.5$ corresponds to a rather minor increase of only about 7 %. This increase will in many cases not be a reason to splice 50 % instead of 100 % of the bars in one section.

Another aspect to this is how splicing of shear and torsional reinforcement should be made. Not allowing to splice all bars in one section, results in a required staggering equal to at least $0.3l_0$ of lap splices. Due to geometric limitations it might be difficult to make room for staggered splices of this type of reinforcement. This was also discussed in Section 6.3.3.

Eurocode 2 urges on staggering laps between bars. The required distance between lap ends of $0.3l_0$ can, as in Hendy and Smith (2010), be interpreted as the definition of a staggered lap. There are however other requirements in Eurocode 2 as well as in other standards that provides motive and guidance in how to distribute laps along a structure. According to Eurocode 2 laps between bars should be avoided in areas subjected to large moments. The American code ACI 318-05, ACI (2007), as well as Model Code 2010, *fib* (2012a), provides somewhat more detailed requirements that can be used as a compliment or guidelines to Eurocode 2.

In ACI 318-05 the ratio between the provided and required reinforcement should be larger than 2 in order to avoid an extended lap length. This means that it is beneficial to splice bars in sections where this criterion is fulfilled. Note that this criterion is only valid if maximum 50 % of all bars are spliced in the intended section. If all bars are spliced in the same section, the lap length must be increased regardless of the utilisation rate of reinforcement. In Table 9.2 the guidelines in ACI 318-05, ACI (2007), are gathered.

Table 9.2 Guidelines for detailing of tension lap splices in ACI 318-05, ACI (2007).

$\dfrac{A_s^{provided}}{A_s^{required}}$	Maximum amount of A_s spliced within required lap length	
	50 %	100 %
≥ 2	$1.0l_0$	$1.3l_0$
< 2	$1.3l_0$	$1.3l_0$

Model Code 2010, *fib* (2012a), gives even more hands-on recommendations stating that lap splices, for bars with diameter larger than 12 mm, should be staggered so that not more than one third of the tensile force needs to be transferred by a lap.

It is important not to forget that lap splices, according to Eurocode 2, should be provided with transverse reinforcement within the lap zone, if the lapped bars have a diameter larger than 20 mm or if the percentage of lapped bars in any section is more than 25 %. This is according to Betongföreningen (2010a) more stringent demands in comparison to BBK 04 where it was only necessary to place transverse reinforcement when more than 50 % of the bars were spliced in the same section.

9.5 Concrete cover and distance between bars

9.5.1 Requirements in Eurocode 2

Eurocode 2 provides very limited information on how to place longitudinal reinforcement in order to provide a good detailing. The reason for this is that the codes should provide requirements with regard to safety and structural performance, but not govern the choice of solution. Hence, it should be up to the designer to develop a solution that fulfils the safety and serviceability demands in Eurocode. However, this can result in solutions that are difficult to execute at the construction site, i.e. solutions that the designer should avoid.

Paragraphs EC2 8.1(2) and EC2 4.4.1.2(1) state that requirements concerning minimum concrete cover should be satisfied in order to ensure safe transmission of bond forces, protection against steel corrosion and adequate fire resistance. The relation between concrete cover and bond of reinforcement is explained in Section 3.2.1. Minimum concrete cover, c_{min}, is determined by Equation (9.21) that can be found in Paragraph EC2 4.4.1.2(2).

$$c_{min} = \max\left(c_{min,b}; c_{\min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10 \text{ mm}\right) \tag{9.21}$$

$c_{min,b}$	minimum cover due to bond requirement
$c_{min,dur}$	minimum cover due to environmental conditions
$\Delta c_{dur,\gamma}$	additive safety element
$\Delta c_{dur,st}$	reduction of minimum cover for use of stainless steel
$\Delta c_{dur,add}$	reduction of minimum cover for use of additional protection

For separated bars placed in concrete with normal aggregate size the minimum concrete cover with regard to bond, $c_{min,b}$, should according to Table EC2 4.2 be equal to the diameter of the bar, ϕ.

A factor, Δc_{dev}, is also added in order to allow for some deviation in execution. The value that should be specified on drawings is therefore the nominal concrete cover

$$c_{nom} = c_{min} + \Delta c_{dev} \tag{9.22}$$

Eurocode 2 also provides information about how to place reinforcing bars in relation to each other by specifying the minimum clear distance, both horizontally and vertically, between them. This is according to Paragraph EC2 8.2(1) in order to enable satisfactory placing and compacting of concrete for the development of adequate bond. According to Paragraph EC2 8.2(2) the minimum clear distance between single bars, a, should therefore be determined as the larger of

$$a = \max\begin{cases} \phi & \text{bar diameter} \\ d_g + 5\,\text{mm} & d_g \text{ maximum aggregate size} \\ 20\,\text{mm} & \end{cases} \tag{9.23}$$

The definitions of the concrete cover, c, and clear distance between bars, a, are shown in Figure 9.26. It should be added that, if it is relevant, the concrete cover due to duration should be the distance to the reinforcement closest to the concrete surface, e.g. stirrups, links or surface reinforcement that for clarity has been left out from Figure 9.26. This can for example be when a certain concrete cover is needed in order to prevent corrosion of reinforcement. Also mounting reinforcement should be protected from corrosion by an adequate concrete cover, Engström (2010). It is also noteworthy that there are additional rules for clear distance between bars in case of lapping of reinforcement. More about this can be found in Section 9.4.1.

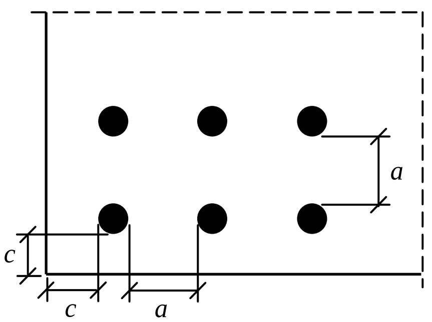


Figure 9.26 Definition of concrete cover, c, and clear distance between bars, a.

Further, Eurocode 2 states that reinforcement bars should be placed vertically above each other in case of separate horizontal layers. Eurocode 2 also states that in order to allow for good compaction of concrete access for vibrators should be provided between the bars.

9.5.2 Explanation and derivation

As explained in Section 3.2.1 sufficient concrete cover and clear distance between bars are required in order to prevent very brittle anchorage failure. If the concrete cover is sufficient, a brittle failure due to splitting can be avoided. However, this requires a concrete cover of about 3ϕ according to Engström (2011a). This is a very large concrete cover why it is common to use a smaller concrete cover and design for a for a pull-out failure in concrete with splitting cracks, see explanation in Section 3.2.1. The minimum concrete cover due to bond, $c_{min,b}$, for a single bar is ϕ according to Eurocode 2. The concrete cover, c, and clear distance between bars, a, will also influence the anchorage length by the factors α_1 and α_2, see Section 9.3.1. It should also be noted that the concrete cover might have to be increased to prevent material deterioration such as corrosion. This is considered by the factor $c_{min,dur}$. This will not be explained further, since it is out of the scope of this report.

The clear distance between bars is determined in order to provide for satisfactory placement and compacting of concrete and development of adequate bond. The minimum requirement of $d_g + 5$ mm will ensure that the largest aggregates in the concrete mix can travel around the bars so that there will be an even dispersion of aggregates within the structure. This will of course also affect the bond properties. The other requirement of a minimum distance not smaller than ϕ can be recognized to be the same as for concrete cover c. Hence, this distance is probably, in the same way as the concrete cover c, provided in order to achieve a certain expected failure mode and to reduce the risk for a very brittle anchorage failure.

9.5.3 Discussion

The reason why an extra factor, Δc_{dev}, is added to the minimum concrete cover is, as stated in previous section, to account for deviation in execution. Such a deviation of c depends on that it in reality is difficult to place the reinforcement with exact precision. Another reason for deviations can be the fact that the bars are not perfectly circular as assumed in design and therefore have a real section that differs slightly from the notional diameter. It is therefore quite interesting why no deviation factor is added to the clear distance, a. It is not easier to place reinforcement bars exactly in relation to each other, than it is to place them in relation to a concrete formwork. One have to presume that such allowance for deviation is included in the specified values in (9.23) for determining a. In case of lap splices there are additional rules for the clear distance between bars that suggest that this is the case.

As can be seen in Figure 9.17 in Section 9.4.1 the clear distance between adjacent bars of adjacent laps must be at least the larger of 2ϕ and 20 mm. The minimum distance is in this case dependent on twice the bar diameter. Motive for this increase can be to consider possible deviations at the construction site, such as reversing two bars that are spliced to each other, Johansson (2013). Another reason why the minimum clear distance between adjacent laps is increased compared to the requirement for single bars could be uncertainties related to the fact that nominal bar diameters are used on drawings. In reality the bars are not smooth and the diameter will deviate slightly from the nominal dimensions. When two bars are placed close together as in a lapped splice, the risk of deviations is increased.

Looking back at Figure 9.17 in Section 9.4.1 and comparing this requirement of clear distance between lapped bars to the requirement of clear distance for single bars, it is noticeable that Figure 9.17 can be interpreted as if the clear distance between adjacent laps does not need to fulfil the requirement with regard to maximum aggregate size, d_g + 5 mm. In order to clarify the requirements for minimum clear distance between bars provided in Eurocode 2, the criterion could be presented as in Figure 9.27.

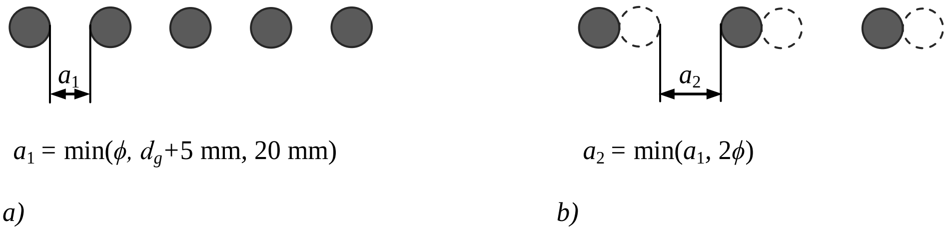


Figure 9.27 Suggestion of how to provide the rules for clear distance between bars, a) required clear distance between single bars, b) required clear distance between lapped bars. According to Johansson (2012a).

Eurocode 2 provides rules for clear distance, *a*, and concrete cover, *c*, in order to ensure adequate bond, fire resistance and protection against corrosion. In order to achieve these desired properties it is also required to enable proper casting of the concrete. This is to some extent considered in the rules concerning the clear distance between bars, see Equation (9.23). It is stated that good compaction should be enabled and access for vibrators should be made. However, no further or more detailed instructions are given. In practice, this means that free space for internal vibrators should be made.

In several previous codes and handbooks examples are given on how reinforcement more in detail can be arranged in order to provide enable satisfactory placing and compaction of concrete and free space for internal vibrators can be enabled. BBK 04, for instance, provides recommendation concerning this. It is stated that besides the minimum clear distance between bars, there should be gaps of minimum 100 mm width free from reinforcement. An example of such a configuration is shown in Figure 9.28.

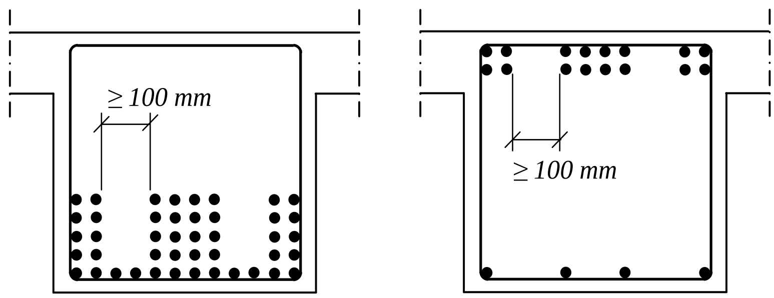


Figure 9.28 Arrangement of reinforcement with respect to satisfactory placing and compaction of concrete according to BBK 04, Boverket (2004).

In this matter BBK 04 also refers to Betonghandboken, Svensk Byggtjänst (1990), which in addition to BBK 04 states that free space for internal vibrators are recommended, if the reinforcement is placed in more than two layers, see Figure 9.29.

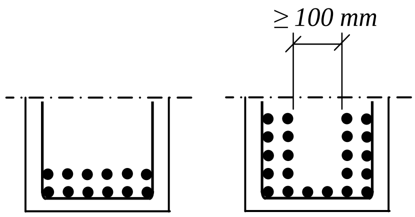


Figure 9.29 Arrangement of reinforcement with respect to satisfactory placing and compaction of concrete according to Betonghandboken, Svensk Byggtjänst (1990).

Another reference that provides quite extensive detailing recommendations is Reinforced Concrete Detailing, Barker (1967). It is based on the rules and guidelines for reinforced concrete design given in the Code of Practice (C.P.), BSI (1965), which was the previous code for reinforced concrete design in the United Kingdom. Barker (1967) states that when internal vibrators are intended to be used, a clear distance of about 75 mm between groups of bars should be left to enable the vibrator to be inserted. Figure 9.30 shows the recommendations given in Barker (1967).

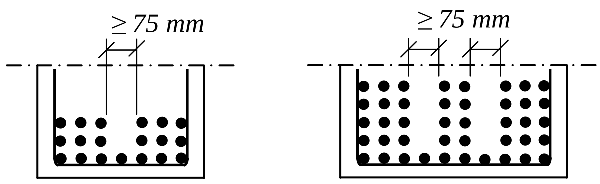


Figure 9.30 Arrangement of reinforcement with respect to satisfactory placing and compaction of concrete according to Barker (1967).

How frequently free space for internal vibrators should be provided is not stated. However, there is a table in Barker (1967) that indicates that a maximum amount of five bars in a row should be permitted before a new gap between the reinforcement bars is provided.

From the guidelines provided by Boverket (2004), Svensk Byggtjänst (1990) and Barker (1967) it seems reasonable to leave free space of about 75-100 mm in order to make room for vibrators. How closely spaced these gaps should be is still not clear and it would be interesting to know the opinion of a concrete worker about this issue.

9.6 Permissible mandrel diameters for bent bars

9.6.1 Requirements in Eurocode 2

In order to achieve sufficient anchorage or, as can be seen in Section 4.4, provide a proper detail in concrete frame corners it might be necessary to bend the reinforcing bar. However, the minimum diameter to which the bar is bent is limited according to Eurocode 2 Section EC2 8.3. It should be noted that there are many other cases where T-T-C nodes are solved by bends.

To avoid bending cracks in the reinforcing bar the permissible mandrel diameter is limited according to Table EC2 8.1N, see Table 9.3.

Table 9.3 Minimum mandrel diameter to avoid damage to reinforcement. The table is taken from SIS (2008).

Bar diameter	Minimum mandrel diameter for bends, hooks and loops
$\phi \leq 16$ mm	4ϕ
$\phi > 16$ mm	7ϕ

There are other requirements for welded bent reinforcement but they are not treated in this report.

As mentioned in Section 4.4 spalling of side concrete cover is a risk for frame corners subjected to bending moment. In Expression EC2 (8.1), see Equation (9.24), the splitting effect of the concrete is considered by limitation of the mandrel diameter, ϕ_m. According to Eurocode 2 this limitation will ensure that failure of concrete at the inside of the bar is avoided. Equation (9.24) usually refers to splitting failure but could be splitting failure if the bend is close to the free face. This case is further discussed in Section 9.6.

$$\phi_{m,\min} \geq \frac{F_{bt}\left(\left(1/a_b\right) + 1/(2\phi)\right)}{f_{cd}} \tag{9.24}$$

F_{bt} tensile force from ultimate loads in a bar, or group of bars in contact, at start of a bend

a_b for a given bar (or group of bars in contact), a_b is half of the centre-to-centre distance between bars (or groups of bars) perpendicular to the plane of the bend. For a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus ϕ / 2

ϕ bar diameter

f_{cd} design value of concrete compressive strength, should not be taken greater than that for concrete class C55/67

It should be noted that it is not necessary to check the mandrel diameter with respect to concrete failure if the bar is not positioned at an edge, the requirements in Table EC2 8.1N are fulfilled and the required anchorage is not longer than 5ϕ beyond the end of the bend.

9.6.2 Explanation and derivation

From Eurocode 2 it can be deduced that the minimum requirement for the mandrel diameter with respect to concrete failure provided in Equation (9.24) is stricter, i.e. provides a larger mandrel diameter, than the requirements with respect to bending cracks in the reinforcement. This means that even if the requirements in Table 9.3 are fulfilled it might be necessary to increase the mandrel diameter if the risk for splitting cracks is present, as for instance if the reinforcing bar is positioned close to an edge.

In the following text the requirement in Equation (9.24) will be treated. However, it may be noted that the bend of the bar affects the fatigue strength of the reinforcement, Johansson (2013), why it is important to avoid bending cracks, but this is out of the scope of this master's thesis.

High radial compressive stresses will be acting on the bar when it changes direction along the bend, see Figure 9.31. These stresses are associated with transverse tensile stresses that tend to split the concrete in the plane of the bend. The risk for splitting of the concrete depends on the magnitude of the splitting stresses, which in turn is influenced by the bar diameter, the steel yield strength of the reinforcement, the bending radius and the shape of the bar. As was mentioned in Section 9.6.1 the distance to the free face will influence the spalling of the concrete, see distance, *c*, in Figure 9.31b. A large side concrete cover will influence the resistance against spalling in a favourable way.

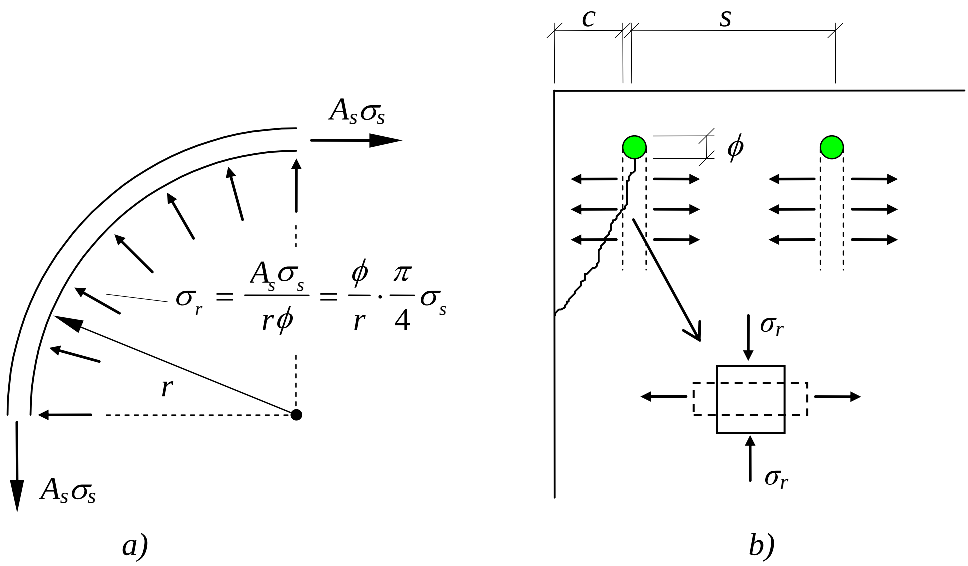


Figure 9.31 Schematic view of stresses acting on a bent reinforcing bar within a concrete member, a) radial compressive stresses, b) possible spalling cracks. The figure is based on Johansson (2000).

In Figure 9.31 it is shown how tensile stresses in the concrete occur because of radial compressive stresses, σ_r. The radial stresses decrease with an increased radius, *r*, which in turn also decreases the tensile stresses in the concrete. These tensile stresses can cause spalling if the bar is positioned close to an edge. The mandrel diameter, ϕ_m, equal to twice the radius, *r*, is for this reason limited in Eurocode 2 to a minimum value, $\phi_{m,min}$, according to Equation (9.24).

9.6.3 Discussion

It should be noted that the expression provided by Eurocode 2, regarding permissible mandrel diameter, does not take into account the shape or the angle of the bend of the bar, see Equation (9.24). The spalling effect is considered in BBK 04 by Equation (3.9.4.2a), see Equation (9.25), where also the angle of the bend of the bar is taken into account, Boverket (2004). Here $2r$ is the same as $\phi_{m.min}$ in the equation from Eurocode 2, see Equation (9.24).

$$\frac{r}{\phi} \geq 0.028 \frac{f_{yd}}{f_{ctd}} - 0.5 - \frac{1}{\sin(\beta/2)} \left(\frac{cc}{\phi} + 0.5 \right) \tag{9.25}$$

where

$$\frac{cc}{\phi} \leq 3.5$$

cc	concrete cover perpendicular to the plane of the bent bar. However, not bigger than half the centre to centre distance between parallel bent bars
ϕ	bar diameter
r	radius of the bent bar
β	angle of the bend of the bar
f_{ctd}	design tensile strength of the concrete
f_{yd}	design steel yield strength of the steel

In order to see the effect of the shape of the bend Equations (9.24) and (9.25) have been compared to each other in Figure 9.32 with the bar diameter as a variable. For further information about the calculations, see Appendix H. Equation (9.25) has been rearranged in order to be able to compare it to the minimum mandrel diameter, $\phi_{m,min}$, in Equation (9.24). This is shown in Equations (9.26) and (9.27).

$$\phi_{m,\min} = 2r \tag{9.26}$$

$$2r \geq 2\phi \left(0.028 \frac{f_y}{f_{ct}} - 0.5 - \frac{1}{\sin(\beta/2)} \left(\frac{cc}{\phi} + 0.5 \right) \right) \tag{9.27}$$

Note that the partial factors for steel, γ_s, and for concrete, γ_c, are affecting the whole expression from Eurocode 2, see Equations (9.28) and (9.29). According to Johansson (2013) this also leads to a more reasonable result, since the ratio r / ϕ thus becomes proportional to the reinforcement- and concrete strengths. In the expression provided by BBK 04 the partial factors only affect the first part of the expression, see Equation (9.29).

$$\phi_{m,\min} \geq \frac{f_{ky}/\gamma_s}{f_{ctk}/\gamma_c} \left(\left(1/a_b \right) + 1/(2\phi) \right) \qquad \text{Eurocode 2} \tag{9.28}$$

and

$$\frac{r}{\phi} \geq 0.028 \frac{f_{yk}/\gamma_s}{f_{ctk}/\gamma_c} - 0.5 - \frac{1}{\sin(\beta/2)} \left(\frac{cc}{\phi} + 0.5 \right) \qquad \text{BBK 04} \tag{9.29}$$

If the plots in Figure 9.32 are compared it can be seen that the requirement in BBK 04 result in more conservative values of the required mandrel diameter than the requirement in Eurocode 2. The difference between the results obtained from the two expressions is about 15-30 % for larger diameter of the bar. If the required minimum mandrel diameter is calculated according to Eurocode 2, with a bar diameter equal to $\phi 10$, it results in a value somewhere in between the values calculated according to the expression in BBK 04, with an angle of 90º and 180º of the bend bar.

In Figure 9.32 it is shown that the angle of the bend of the bar affects the required minimum mandrel diameter. A reinforcing loop of 180º requires a larger mandrel diameter than an L-shaped bend of 90º. This has also been discussed by Johansson (2000) who refers to experimental tests made by Stroband and Kolpa (1983) showing that if a 180º bend of the bar is used, in case of concrete frame corners, there is an increased risk of spalling, see also Appendix D. Johansson (2000) also obtained such results in his research program, which indicates that the angle of the bend of the bar should be taken into account also in Eurocode 2.

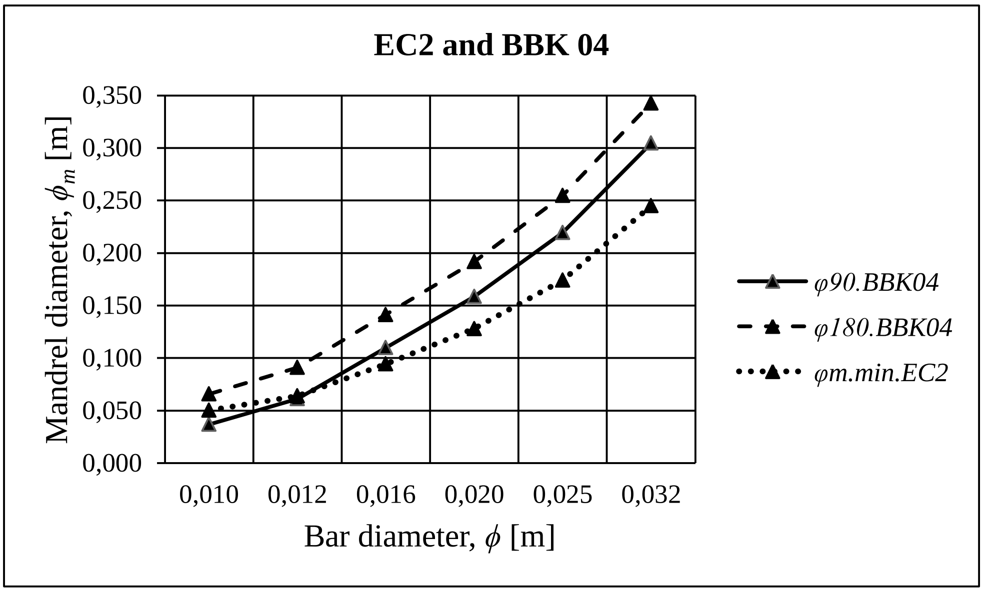


Figure 9.32 Equation (8.1) from Eurocode 2 and Equation (3.9.4.2a) from BBK 04 plotted against the bar diameter when using the design strength. The angle of the bend bar influences the minimum mandrel diameter in the expression from BBK 04.

10 Crack control

10.1 Structural response and modelling

Cracks will occur in reinforced concrete structures that are subjected to tensile stresses of the same magnitude as the tensile strength of concrete. These tensile stresses can be caused by external load with load effects such as bending moment, shear force or torsional moment. It is therefore appropriate to distinguish between bending cracks, shear cracks and torsional cracks. Cracks can also occur due to tensile stresses that arise from prevention of free movement or deformation of a structure, so called restraint cracking, Engström (2011d). For restraint cracking it is appropriate to distinguish between stress dependent strains and stress independent strains. Stress dependent strains are deformations that develop due to stress in a material that develop when the material is loaded. Creep is an example of stress dependent strain. Stress independent strains are in the other hand not caused by stresses. These types of strains are caused by deformations only that can occur due to for instance shrinkage or thermal expansion of concrete. Restraint cracking is a complicated phenomenon and such cracks are often difficult to predict.

The reinforcement placed in a concrete structure cannot prevent the concrete from cracking. However when the concrete has started to crack the reinforcement can be used to control the cracking; i.e. distribute the cracks and limit the widths of the cracks so that the structure can fulfil its purpose. One of the main reasons why crack widths should be limited is to prevent the structure from material deterioration. Large cracks might result in for example corrosion of the reinforcing steel. The corroding reinforcement will expand and may result in spalling of the concrete cover, thus resulting in a reduced capacity of the concrete structure. Another reason why cracks should be held as small as possible is for aesthetical reasons. Even if the structure is safe and strong enough to fulfil its purpose, it should also be perceived as such. Excessive deformation and cracking under normal load should therefore, if necessary, be prevented. It should be noted that crack control is related to design in the serviceability limit state.

The fundamental cracking process of reinforced concrete can be described by a prismatic concrete member with one centric reinforcing bar subjected to a tensile force N, see Figure 10.1. As have been explained in Section 3.2.1 it takes a certain distance, l_t, to transfer stresses between steel and concrete. In Figure 10.1 the relations between steel stress, concrete stress and bond stress are illustrated for such member before cracking.

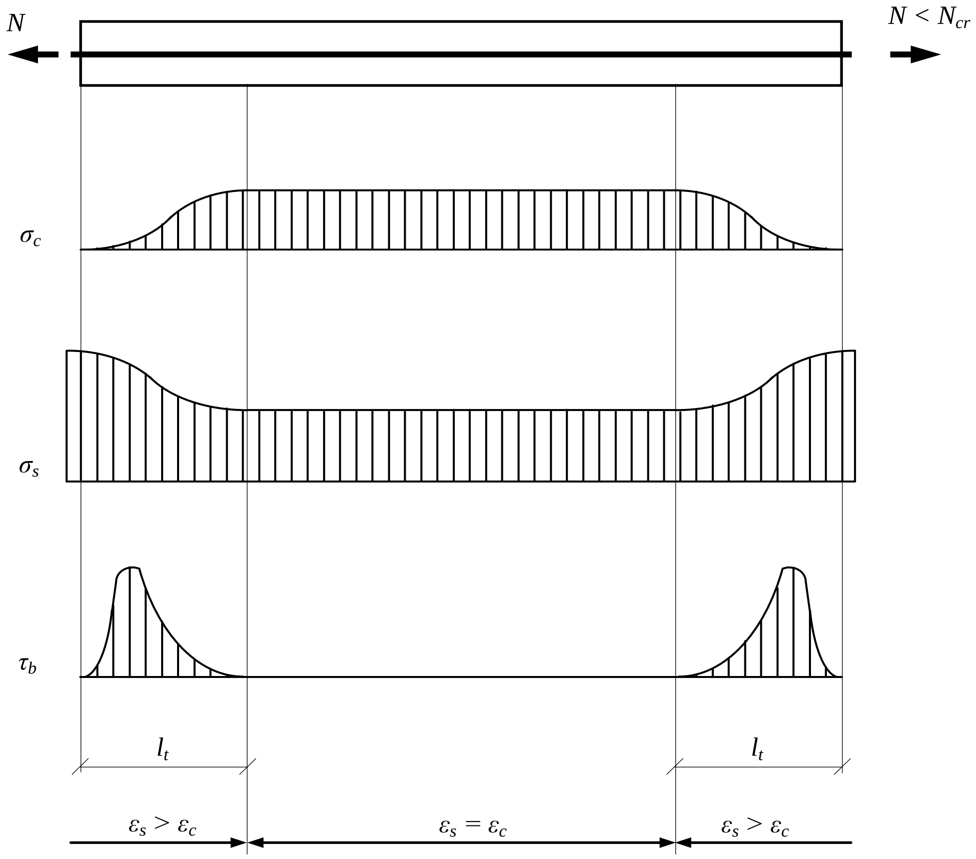


Figure 10.1 Distributions of steel stress, concrete stress and bond stress for a prismatic member loaded in tension. The figure is based on Engström (2011d).

In the mid-region of the concrete member there is compatibility between the deformations in the reinforcement and surrounding concrete, meaning that the concrete and steel strains are equal, Engström (2011d). When the load N is increased the tensile stress in the mid-region of the member increases until the tensile strength of concrete, f_{ct}, is reached. At this time the transmission length has increased to its maximum value $l_{t,max}$, which is the length necessary in order to build up tensile stresses that can cause a crack. When the tensile strength of concrete is reached a crack will therefore initiate anywhere in the mid-region of the member, see Figure 10.2b.

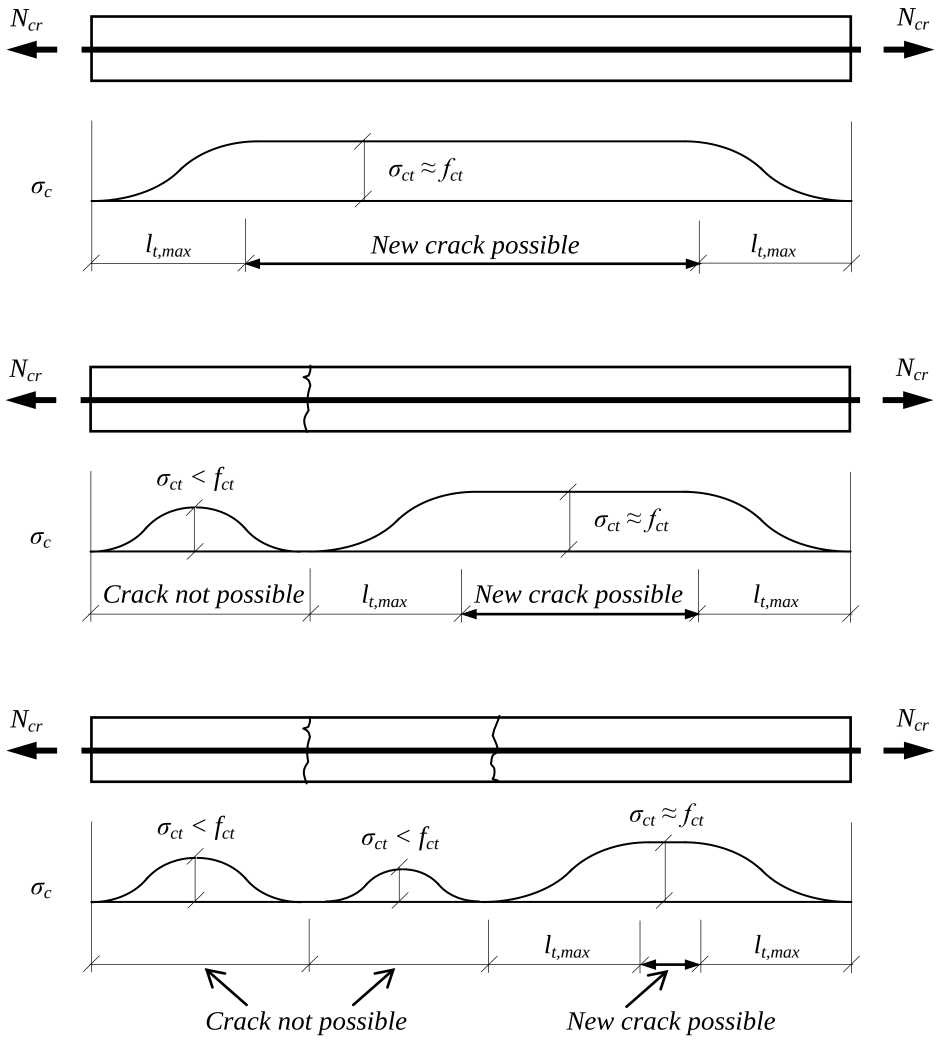


Figure 10.2 Fundamental cracking process of a prismatic member subjected to tension. The figure is based on Engström (2011d).

After the first crack has occurred new cracks continue to develop in other parts of the reinforced concrete member. Theoretically new cracks can occur without further load increase. However, in reality a small load increase is necessary to create new cracks. can occur , if the load is increased further. It should be noted that there is an upper limit for the number of cracks that can develop. This is limited by the length of the member. No crack can occur, if the remaining distance between the cracks is too short to develop tensile stresses high enough to initiate new cracks, see Figure 10.2c.

When no new cracks can develop, the crack formation phase is ended. If the load is increased further, the steel stress is also increased, resulting in larger crack widths of the existing cracks due to the difference in strain between steel and concrete. This phase is often referred to as stabilised cracking.

It should be noted that if the steel stress in a crack reaches the yield stress, the crack width increases without control. Hence, this is something that the designer should prevent in the service state.

10.2 Minimum reinforcement requirements for crack control

10.2.1 Requirements in Eurocode 2

According to Eurocode 2, Section EC2 7.3.2, minimum reinforcement must be placed in areas of a concrete member where tensile stresses are expected, if crack control is required. This is also clearly stated in Section EC2 9.2.1.1, concerning maximum and minimum reinforcement areas in beams, that refers back to Chapter EC2 7.3, where Expression EC2 (7.1) shows how a minimum reinforcement area for crack control can be calculated, see Equation (10.1).

$$A_{s,\min}\sigma_s = k_c k \; f_{ct,eff} A_{ct} \tag{10.1}$$

σ_s	absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack.
k_c	coefficient which takes account of the stress distribution within the section immediately prior to cracking and of the change of the lever arm
k	coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
$f_{ct,eff}$	mean value of the tensile strength of the concrete at the time when the cracks may first be expected to occur. This is equal to f_{ctm} or $f_{ctm}(t)$.
A_{ct}	area of concrete within the part of the section which is calculated to be in tension just before formation of the first crack

According to Paragraph EC2 7.3.2(1) the minimum amount may be estimated from equilibrium between the tensile force in the concrete just before cracking and the tensile force in reinforcement at yielding or at a lower level if necessary to limit the crack widths, SIS (2008). Hence, the value of σ_s may be taken as the yield strength of reinforcement or, if it necessary to satisfy certain crack widths, be chosen according to Table EC2 7.2N and EC2 7.3N in Section EC2 7.3.3. Table EC2 7.2N and EC2 7.3N are reproduced here in Table 10.1 and Table 10.2 respectively.

Table 10.1 Maximum bar diameters ϕ^*_s for crack control.

Steel stress [MPa]	Maximum bar size [mm]		
	w_k =0.4 mm	w_k =0.3 mm	w_k =0.2 mm
160	40	*32*	25
200	32	25	16
240	20	15	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

Table 10.2 Maximum bar spacing for crack control.

Steel stress [MPa]	Maximum bar size [mm]		
	w_k=0.4 mm	w_k=0.3 mm	w_k=0.2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

The calculated values in the two tables are based on rectangular cross-sections in pure bending with high bond bars and C30/37 concrete. The distance from the edge to the centroid of reinforcement, i.e. the Equation (*h-d)* was assumed to 0.1*h*. In order to be able to use Table EC2 7.2N for other geometries, loading situations and concrete types a correction must be calculated according to Expression EC2 7.6N and EC2 7.7N, see Equation (10.2) and (10.3). Then the maximum bar diameter ϕ_s should be modified as follows

$$\phi_s = \phi_s^* \left(f_{ct,eff}/2.9\right)\frac{k_c h_{cr}}{2(h-d)} \qquad \text{for bending} \qquad (10.2)$$

$$\phi_s = \phi_s^* \left(f_{ct,eff}/2.9\right)\frac{h_{cr}}{8(h-d)} \qquad \text{for uniform axial tension} \qquad (10.3)$$

ϕ^*_s maximum bar size given in Table EC2 7.2N

h_{cr} depth of tensile zone immediately prior to cracking

h-d is for pure tension the distance between the centroid of the reinforcement and the closest face of concrete

10.2.2 Explanation and derivation

The minimum reinforcement according to Equation (10.1) should be placed in a concrete member in order to enable a distribution of cracks along the structure. This distribution will make sure that new smaller cracks can develop, preventing large single cracks to form, Engström (2011d). This is also explained in Collins & Mitchell (1991) where reference is made to tests performed by Williams (1986) on large reinforced concrete elements subjected to pure tension, which showed that a minimum reinforcement amount is required if cracks are to be controlled. The effects of different reinforcement ratios, ρ, on crack distribution obtained from the tests are presented in Figure 10.3. It was also shown that if yielding of reinforcing steel occurred at the first cracking, adequate crack control was not obtained.

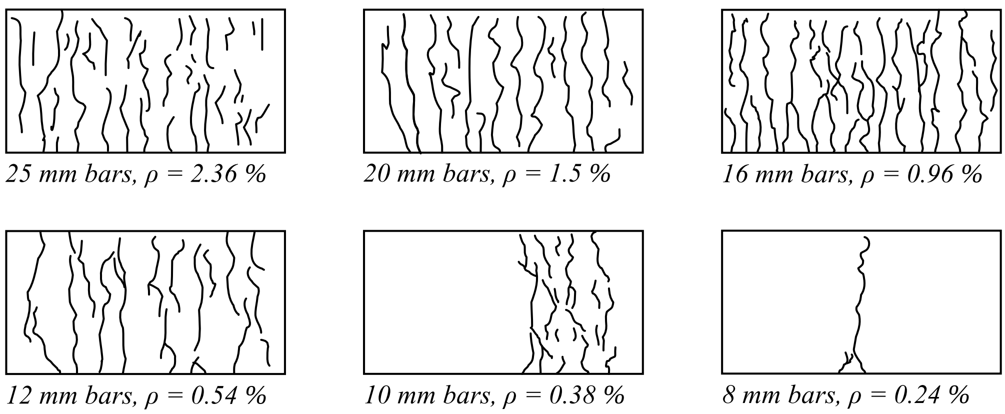


Figure 10.3 Distribution of cracks for specimens with different reinforcement amounts. obtained from tests made by Williams (1986).

As stated in EC2 7.3.2(1) the minimum amount can be estimated by force equilibrium between the tensile capacity of reinforcement and the tensile force in the concrete before cracking. When the concrete cracks the amount of reinforcement in the section should be sufficient to resist and transfer tensile forces to other areas of the member. At cracking the tensile force that was previously taken by the concrete is immediately transferred to the reinforcement. By ensuring that the stress in the reinforcement is below the yield strength, the tensile force can be transferred to other parts of the concrete member where new cracks can develop. In design of minimum reinforcement the value of σ_s is therefore set to a maximum value equal to the characteristic yield strength of the steel f_{yk}. The characteristic value is used, since crack control is related to the serviceability limit state and should be designed for accordingly.

It can be seen that the left hand side of Expression (10.1) corresponds to the force taken by the reinforcement, i.e.

$$F_s = A_{s,\min}\sigma_s \tag{10.4}$$

Hence, the right hand side, see Equation (10.5), corresponds to the force taken by the concrete in tension prior to cracking. In case of pure tension this force can be expressed as

$$F_{ct} = f_{ct,eff} A_{ct} \tag{10.5}$$

In order to account for other types of stress distributions than that of uniform tension the variables k_c and k have been introduced, Hendy and Smith (2010). The factor, k_c, takes into account the stress gradient over the cross-section. For pure tension the factor is equal to 1.0 and for bending with or without normal force it is smaller than 1.0. The factor k_c, has the effect of reducing the required amount of reinforcement in situations where the tensile stress decreases across the depth of the section. The factor k considers the influence from internal self-equilibrating stresses. These may arise where the strain varies non-linearly across the depth of the section, e.g. in case of non-uniform shrinkage. For high members or cross-sections with wide flanges, self-equilibrating stresses may increase the tension at the outer parts of the concrete section resulting in cracking at a lower tensile force than expected. The amount of reinforcement necessary to enable crack distribution can therefore be reduced.

In Eurocode 2 Commentary, ECP (2008a), the minimum reinforcement amount is partially derived. The derivation is based on a rectangular concrete section subjected to bending and an axial force N_{Ed}, see Figure 10.4.

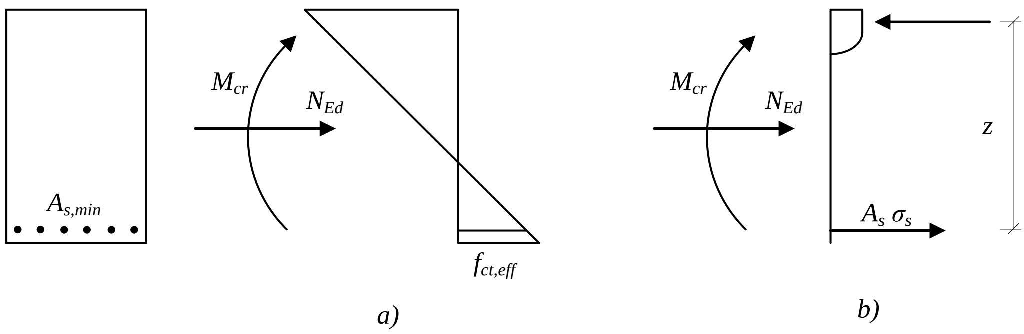


Figure 10.4 Model used for derivation of the minimum reinforcement amount according to ECP (2008a), a) uncracked cross-section b) cracked cross-section.

By equating the expression for the cracking moment derived from Figure 10.4a and the expression for the moment that can be taken by the reinforcement in Figure 10.4b an expression similar to the one used in Eurocode 2 is derived

$$f_{ct,eff} A_{ct} k_c = A_s \sigma_s \tag{10.6}$$

where the expression of k_c is derived as

$$k_c = 0.4\left[1 - \frac{\sigma_c}{f_{ct,eff}}\left(1 - 1.4\left(\frac{f_{ct,eff} - \sigma_c}{f_{ct,eff}}\right)\right)\right] \tag{10.7}$$

This expression can be compared to Expression EC2 (7.2) for bending of a rectangular cross-section in Eurocode 2

$$k_c = 0.4\left[1 - \frac{\sigma_c}{k_1\left(h/h^*\right)f_{ct,eff}}\right] \tag{10.8}$$

σ_c mean stress of the concrete acting on the part of the section under consideration: $\sigma_c = N_{Ed}/(bh)$

h^* should be taken as 1.0 m for cross-sections deeper than h otherwise use the cross-sectional height h.

k_1 coefficient considering the effects of axial forces on the stress distribution.

As stated in Eurocode 2 the expression for minimum reinforcement can also be used to some extent to limit crack widths by inserting values of σ_s taken from Table EC2 7.2N or EC2 7.3N. According to Hendy and Smith (2010) these tables were produced from parametric studies using the crack width calculation formula in Section EC2 7.3.4, see Section 10.3.1.

10.2.3 Discussion

Firstly, it should be noted that the minimum reinforcement requirement in Equation (10.1) is a prerequisite for the expressions for calculation of crack width and crack spacing in EC2 7.3.4, see Section 10.3.1. These expressions are based on the fact that so called stabilised cracking has been obtained, so that no further cracks can develop, see Section 10.1.

It should also be noted that the crack widths are not held within any certain limits just because the minimum reinforcement requirement is fulfilled. In order to obtain certain crack widths the stress in the steel must be limited according to Tables EC2 7.2N or EC2 7.3N. However, these tables should be used with caution, since they are based on certain concrete, concrete cover and bond properties of the reinforcement as well as certain stress distribution and load duration.

The basic principle in Eurocode 2 implies that crack control according to Section EC2 7.3 applies to cracks caused by direct loading, restraints or imposed deformations, see also Section (10.3.1). However, since Equation (10.1) is derived for a concrete cross-section subjected to bending, see Section 10.2.2, it can be discussed if the expression really is applicable also for distribution of cracks due to restraint. The derivation in ECP (2008a) is not explained in detail and the expression obtained for k_c is not exactly the same as the one used in Eurocode 2. This implies that there is more behind the expression than what is shown in that text. It is also noteworthy that the derivation is based on moment equilibrium between the cracking moment M_{cr} and the moment obtained by the force in the reinforcement, $F_s = A_s \cdot \sigma_s$, assuming a fully developed non-linear stress block, see Figure 10.4.

In Hendy and Smith (2010) it is stated that for cracks caused mainly by restraint, such as restrained shrinkage or thermal strain, table 7.2N can be used. This implies that the minimum reinforcement requirement actually has been developed to be used for many different design situations such as for example restraint situations. This argument is confirmed by Engström (2013), who believes that the expression for minimum reinforcement is applicable to both external loads and restraint situations.

Something that also supports this is that the expression for minimum reinforcement in Eurocode 2 in fact is very similar to the one used in BBK 04, Boverket (2004), see Equation (10.9), which is based on a restraint situation.

$$A_s \sigma_s \geq f_{cth} A_{ef,BBK} \tag{10.9}$$

f_{cth} = $1.5 f_{ctk,BBK}$. High value of the tensile strength of concrete: However, this value is close to f_{ctm} and will therefore not imply a large difference in relation to the expression in Eurocode 2.

$A_{ef,BBK}$ effective concrete area, i.e. that part of the tensile zone that have the same centroid as the reinforcement

This expression is derived from force equilibrium in the same way as the minimum amount of reinforcement can be estimated according to Paragraph EC2 7.3.2(1). The main difference between the expressions in Eurocode 2 and BBK 04 is the amount of concrete that is considered before cracking.

As stated just below Equation (10.1), the concrete area, A_{ct}, used for determining the minimum reinforcement is in Eurocode 2 defined as the concrete area in tension just before cracking. This can in deep structures, such as walls or deep beams, correspond to a very large concrete area and hence result in an unreasonably large amount of reinforcement, Johansson (2013). According to the Swedish handbook BBK 04, Boverket (2004), the concrete area in Equation (10.1) is instead defined as the effective concrete area, resulting in a much smaller amount of reinforcement, see Figure 10.5.

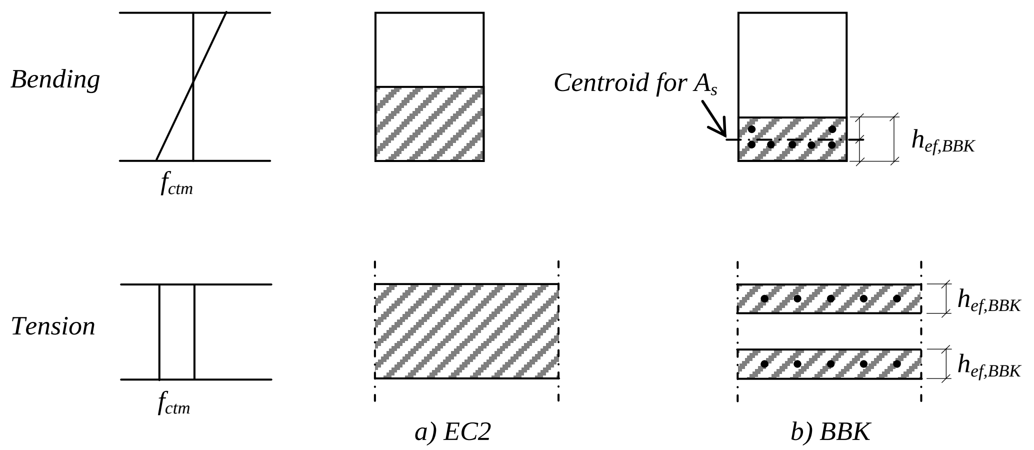


Figure 10.5 Different ways to consider the concrete area in tension for calculation of minimum reinforcement for crack control, a) concrete area in tension prior to cracking, A_{ct}, according to Eurocode 2, b) effective concrete area, A_{ef}, according to BBK04.

Consequently, the question about the applicability of the rule for minimum reinforcement amount in Eurocode 2 for a restraint situation once again arises. For a restraint situation, such as prevented shrinkage, the entire cross-section might be in tension before cracking, resulting in excessive amounts of reinforcement in order to fulfil the minimum reinforcement requirement, Johansson (2013). When cracking occurs due to restraint there will be a reduction of the restraint force immediately when the first crack opens, meaning that the force that must be resisted by the reinforcement is smaller than the one previously taken by the uncracked concrete. $A_{ef,BBK}$ can therefore be argued to result in a better approximation of the required reinforcement amount for a restraint situation.

It can be discussed which one of the expressions in Eurocode 2 and BBK 04 that is most correct to use. A Master's thesis project carried out at Chalmers University of Technology in 2008, Alfredsson & Spåls (2008), investigated this by performing FE-analyses on reinforced concrete prisms subjected to restrained deformations. The area of the concrete that is influenced by tensile stresses after formation of the first crack, i.e. the effective concrete area, obtained from the FE-analysis was compared to area A_{ct} according to Eurocode 2 and the effective concrete area in BBK 04, $A_{ef,BBK}$, see Figure 10.6. The effective concrete area, $A_{ef,EC2}$, used for calculation of crack widths in Eurocode 2, see Section 10.3.1, were also included in the comparison.

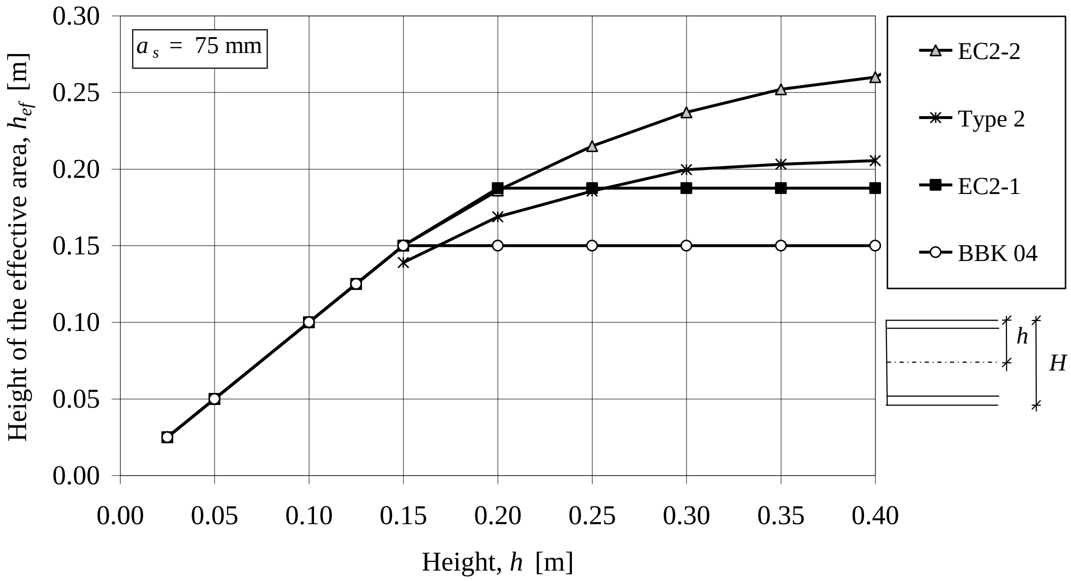


Figure 10.6 Comparison between FE-analysis, BBK 04 and Eurocode 2 of how the effective area varies with the height of the cross-section. (EC2-2 = A_{ct}, Type 2 = FE-analysis, EC2-1 = $A_{ef,EC2}$, BBK 04 = $A_{ef,BBK}$). The figure is taken from Alfredsson and Spåls (2008).

The conclusions that could be drawn from the comparison was that the amount of reinforcement is overestimated by using the concrete area A_{ct} as in Expression EC2 (7.1). However, using the effective area as defined in BBK04, $A_{ef,BBK}$, will underestimate the amount of reinforcement. The effective area according to the definitions in Eurocode 2, $A_{ef,EC2}$, will provide a reasonable amount of reinforcement for an edge distance $a_s = 75$ mm but for larger and smaller edge distances the reinforcement amount will be over- and underestimated.

It should be emphasised that crack control in thick members will only be achieved within the effective concrete area, Aef, around the reinforcement. In regions that are left without reinforcement there is a risk that single wide cracks can occur, see Figure 10.7.

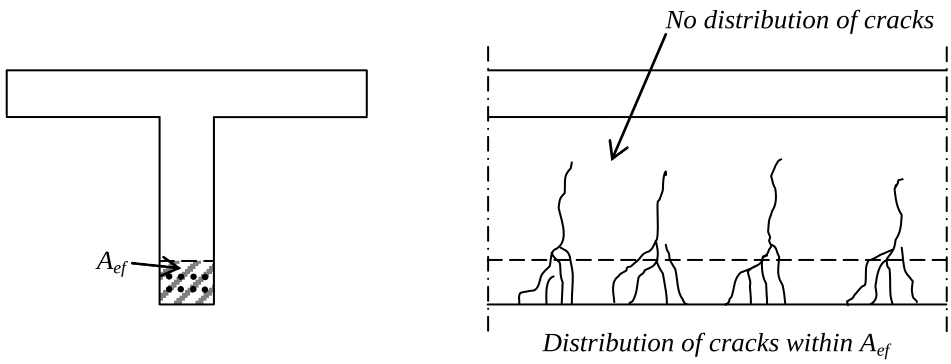


Figure 10.7 In areas where no reinforcement is located wide cracks can occur even if the total amount of reinforcement fulfils the minimum reinforcement requirement for crack control. The figure is based on Engström (2011d).

Eurocode 2 expresses not clearly that the minimum reinforcement only distributes the cracks within the effective concrete area. Hence, it can easily be misinterpreted that bending cracks will be distributed if the minimum reinforcement requirement is fulfilled by the bending reinforcement. For members that have large areas without reinforcement it is important to spread the reinforcement over the height of the cross-section.

However, there is another side to this. Reinforcement for crack control is often placed in order to prevent corrosion of reinforcement. In members with large areas without reinforcement it is perhaps not always important to limit crack widths in these area, since there will not be any reinforcement that can corrode there. Hence, it can be argued that the need for minimum reinforcement amounts to control cracking should mainly be applied in areas where reinforcement is located.

10.3 Limitation of crack widths for shear and torsion

10.3.1 Requirements in Eurocode 2

Crack control and limitation of crack widths are in Eurocode 2 treated in Chapter EC2 7.3. According to Paragraph EC2 7.3.1(2) cracks normally occur in reinforced concrete structures subjected to bending, shear force, torsion or tension caused by direct loading, restraints or imposed deformations. This implies that all these types of crack should be limited.

In Eurocode 2 there is, in Section EC2 7.3.4, a method for calculating characteristic crack widths, w_k, on the basis of the calculated steel stress, σ_s under the quasi-permanent load. The characteristic crack width, w_k, is calculated according to Expression EC2 (7.8) as a function of the maximum crack distance, $s_{r,max}$, and the difference between the average steel and concrete strains, $\varepsilon_{sm} - \varepsilon_{cm}$, see Equation (10.10).

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \tag{10.10}$$

The difference between the average steel and concrete strains is in Eurocode 2 defined as a function of the steel stress, σ_s, and the ratio, $\rho_{\rho,ef}$, between the reinforcement area and the effective concrete area, $A_{ef,EC2}$, surrounding the reinforcement.

According to Eurocode 2 the maximum crack spacing, $s_{r,max}$, is calculated by Expression (10.11) and is hence a function of the diameter of the reinforcing bar, ϕ, and the reinforcement ratio, $\rho_{\rho,ef}$. For a more detailed description of the notations see Eurocode 2, Paragraph EC2 7.3.4(3).

$$s_{r,\max} = k_3 c + k_1 k_2 k_4 \phi / \rho_{\rho,ef} \tag{10.11}$$

c	cover to the longitudinal reinforcement
k_1-k_3	factors that consider strain distributions and bond properties
k_4	constant

10.3.2 Explanation and derivation

The method for calculation of crack widths presented in Eurocode 2 Section EC2 7.3.4 is based on the basic case of a prismatic reinforced concrete bar as the one presented in Section 10.1, *fib* (2012b). The formulation given in Equation (10.10) for calculation of the crack width is according to Mosley *et al.* (2007) also applicable for flexural cracking in case of bending. However, for estimation of crack widths of inclined shear- or torsional cracks there is no established or generally accepted method. According to Betongföreningen (2010a) guidance for calculating stresses in the transversal shear- or torsion reinforcement are missing, which is a prerequisite in order to be able to calculate crack widths.

A reason why the standard method provided in Eurocode 2, Section EC2 7.3.4, for calculation of crack widths cannot be used for shear- or torsional cracks is found in Engström (2011d). It is described that the standard method in Eurocode 2 assumes that cracks occur with a certain distance to each other that corresponds to $s_{r,max}$, see Figure 10.8. The method is therefore not applicable where stabilised cracking is not reached, Engström (2011d). One single, or only a few, shear- or torsional cracks may occur over the height (or width) of the cross-section resulting in that a certain crack spacing cannot be determined.

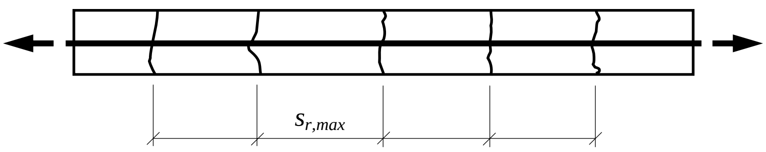


Figure 10.8 The method for limitation of crack widths provided in Eurocode 2 assumes that cracks develop at certain distances to each other so that a maximum crack distance $s_{r,max}$ can be defined.

It should be noted that even if only one or a few shear or torsional cracks have occurred over the height of the cross-section stabilised cracking may still has been reached. According to Johansson (2013) the problem with the method provided in Eurocode 2 is that it implies that the crack distance $s_{r,max}$ should be determined for the transversal torsional and shear reinforcement, see $s_{r,max,transv.}$ in Figure 10.9. However, the longitudinal bending reinforcement might also influence the width of shear and torsional cracks why it can be questioned if the crack distance in the longitudinal direction, $s_{r,max,long.}$ should be used also for calculation of shear and torsional crack widths. It can be noted that shear cracks often occurs as flexural shear cracks that per definition starts from a flexural crack..

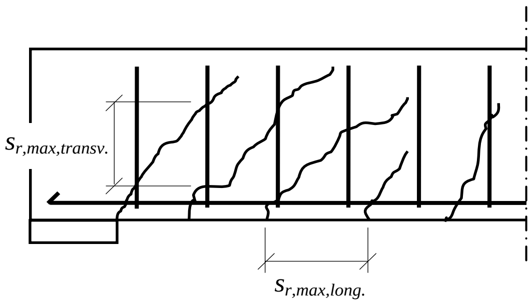


Figure 10.9 It can be discussed whether it is the longitudinal or the transversal reinforcement that determines the width of shear and torsion cracks.

Since there might be reason to believe that the method for calculation of crack widths provided in Eurocode 2 is not fully applicable for control of shear- and torsion cracks, three different proposals for estimating widths of shear cracks are presented and discussed in Section 10.3.3. Two are provided by Betongföreningen (2010a) and one is developed by Johansson (2012b).

10.3.3 Discussion

Betongföreningen (2010a) uses Expression (10.10) for calculation of shear crack widths. It is also stated that the two tables EC2 (7.2N) and EC2 (7.3N), see Section 10.2.1, are applicable for shear cracks as long as the stress in the transversal shear reinforcement can be found. Betongföreningen (2010a), Example X6, provides two different methods to calculate the steel stress, σ_s.

The first one, method a), calculates the reinforcement stresses based on linear elastic analysis of the web. It is assumed that the resulting normal and shear stresses should be resisted by compressive stresses in inclined struts and tensile stresses in the reinforcement in the cracked reinforced concrete. The steel stress can thereby, from the assumption of plane stress, be determined from equilibrium conditions, which can be written as in Equation (10.12) and (10.13), see Figure 10.10 for definitions

$$\rho_x \sigma_{sx} = \sigma_x + \cot\theta \cdot \tau_{xy} \tag{10.12}$$

$$\rho_y \sigma_{sy} = \sigma_y + \tan\theta \cdot \tau_{xy} \tag{10.13}$$

ρ_x reinforcement ratio in x-direction

ρ_y reinforcement ratio in y-direction

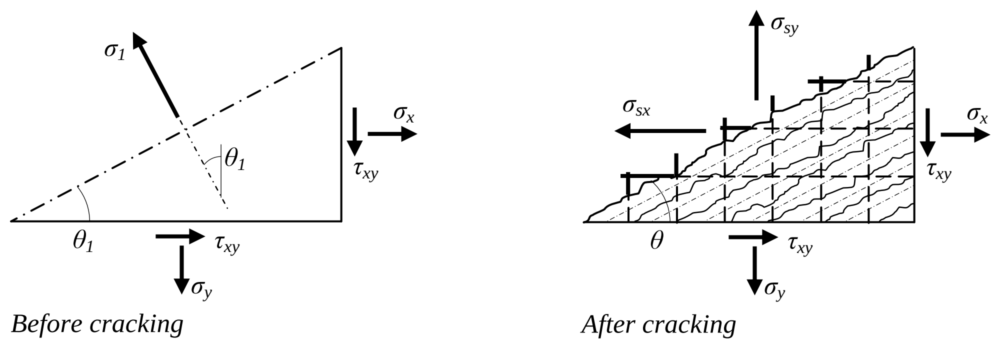


Figure 10.10 State of equilibrium, a) before cracking, b) after cracking. Figure is based on Betongföreningen (2010a).

The stresses σ_x, σ_y and τ_{xy} are solved by linear elastic analysis assuming uncracked concrete and ignoring reinforcement. It should be noted that the inclinations θ_1 and θ in Figure 10.10 are not necessary the same. In fact, θ after cracking is according to Betongföreningen (2010a) dependent on the shear reinforcement amount and should be chosen accordingly.

In order to find the strut inclination θ after cracking corresponding to the amount of reinforcement, the following simplified deformation conditions can be established, see derivation in Betongföreningen (2010a). ε_{sx} and ε_{sx} are strains in the longitudinal and transversal reinforcement respectively.

$$\varepsilon_{sx} = \varepsilon \sin\theta \tag{10.14}$$

$$\varepsilon_{sy} = \varepsilon \cos\theta \tag{10.15}$$

By combining the equilibrium conditions in Equation (10.12) and (10.13) with the deformation criteria in Equation (10.14) and (10.15), an expression for cot θ can be derived. The maximum crack spacing, $s_{r,max}$, can thereafter be calculated using the derived value of θ in Equation (10.16) that is provided in Eurocode 2. $s_{r,max,y}$ and $s_{r,max,x}$ are the maximum crack spacing obtained for the reinforcement in each direction, respectively, using Equation (10.11). It should be noted that the method thus is based on the assumption that stabilised cracking has occurred.

$$s_{r,\max} = \frac{1}{\dfrac{\cos\theta}{s_{r,\max,y}} + \dfrac{\sin\theta}{s_{r,\max,x}}} \tag{10.16}$$

In order to calculate the crack width w_k by Equation (10.10) the calculated crack spacing from Equation (10.16) is multiplied with the difference between the concrete strain and steel strain. The steel strain used in the calculation is according to Betongföreningen (2010a) determined from the maximum value of the steel stress in the longitudinal or transversal reinforcement which can be determined by rearrangement of Equations (10.12) and (10.13), see Equations (10.17) and (10.18).

$$\sigma_{sx} = \frac{\sigma_x + \cot\theta \cdot \tau_{xy}}{\rho_x} \tag{10.17}$$

$$\sigma_{sy} = \frac{\sigma_y + \tan\theta \cdot \tau_{xy}}{\rho_y} \tag{10.18}$$

The advantage with method a) is that it consider both longitudinal and transversal reinforcement when the crack spacing is calculated. However, it can be questioned that the strut inclination is calculated or chosen with regard to the amount of reinforcement. According to Engström (2013) and Johansson (2013) it should be the other way around. The amount of reinforcement should be chosen with regard to a reasonable strut inclination in the service state. It is more reasonable to choose an angle equal to or very close to 45°, since a smaller inclination than this requires plastic redistribution which should be avoided in the serviceability limit state.

The second method, method b), is also provided to obtain the steel stress but only in the transversal shear reinforcement after cracking of concrete. The method is based on a truss model similar to the one presented for design of shear reinforcement in Sections 5.1 and 5.2. The shear force, V, taken by the vertical shear reinforcement can be written as

$$V = A_{sw}\sigma_s \frac{z \cdot \cot\theta}{s} \tag{10.19}$$

V shear force under the quasi-permanent load

Hence, the steel stress σ_s that should be used to find the strain difference between steel and concrete, ε_{sm} - ε_{cm}, in Equation (10.10) can be derived as

$$\sigma_s = \frac{V \cdot s}{A_{sw} z \cdot \cot\theta} \tag{10.20}$$

The angle of the strut inclination, θ, can according to Betongföreningen (2010a) be chosen. In order to consider the fact that cracking occur in the service state Betongföreningen (2010a) suggests that the angle should be calculated according to Expression EC2 (6.65), where an angle suitable for fatigue loading is calculated from the angle chosen for design in the ultimate limit state.

Method b) is much easier to use than method a). However, the longitudinal reinforcement is not accounted for in method b), since the method is based on pure vertical equilibrium. Method a) is therefore more suitable for situations where both longitudinal and vertical reinforcement exist. An advantage with method b) is that it does provide a way to choose a strut inclination in such a way that it is more suitable for a design in the serviceability limit state. However, it can be questioned if it is a reasonable assumption to calculate the strut inclination in the service state from the inclination chosen in the ultimate limit state design. The calculated angle will still presume some plastic redistribution in the service state. It can be noted that both methods ignore effects of friction and aggregate interlock which are more important in the service state, Engström (2013). This can be argued to make the two approaches conservative.

The third method is a proposal developed by Johansson (2012b) after a reference group meeting with Vägverket and Banverket in 2007 where the method was presented and accepted. The method is not based on a certain crack distance. Johansson (2012b) instead uses a correlation between reinforcement stress, σ_s, and mean crack width, w_m, presented by Engström (2006), see Equation (10.21). This expression gives the relation between the mean crack width and the steel stress for a situation of a single crack.

$$w_m = 0.420\left(\frac{\phi \cdot \sigma_s^2}{0.22 f_{ctm} \cdot E_s\left(1 + \frac{E_s}{E_c} \cdot \frac{A_s}{A_{c,eff}}\right)}\right)^{0.826} + \frac{\sigma_s}{E_s} \cdot 4\phi \tag{10.21}$$

The relation between w_m and σ_s can also be illustrated by graphs for different concrete strength classes and bar dimensions, ϕ, see Engström (2006). It should be noted that in a more recent edition of Engström (2006), i.e. Engström (2011d), the expression have been altered so Equation (10.21) should be multiplied with a factor 1.24 in order to be applicable for long term loading. How this affects the method provided in Johansson (2012b) have not been investigated further.

The relation between characteristic crack width, w_k, and mean crack width, w_m, is according to Betongföreningen (2010a) the same in Eurocode 2 as in BBK 04, i.e.

$$w_k = 1.7 w_m \tag{10.22}$$

The allowable steel stress obtained from these calculations to keep a certain crack width is thereafter inserted into the expressions for transversal shear- and torsion reinforcement respectively in order to find the required reinforcement amounts with regard to crack widths. The strut inclination θ should according to Johansson (2013) be chosen to 45º.

$$\frac{A_{sw}}{s} \geq \frac{V_{Ed}}{\sigma_s z(\cot\theta + \cot\alpha)\sin\alpha} \qquad \text{Shear} \qquad (10.23)$$

$$\frac{A_{st}}{s} \geq \frac{T_{Ed}}{2A_k \sigma_s}\tan\theta \qquad \text{Torsion} \qquad (10.24)$$

For notations, see Chapters 5 and 6 for Shear and Torsion, respectively.

It can be argued that this method provides conservative values, since it calculates the crack width assuming one single crack, Johansson (2013). However, this method is perhaps a bit too conservative resulting in an unnecessarily large amount of reinforcement. This has not been investigated in this master's thesis. The way that the mean crack width w_m is defined in Johansson (2013) can also be questioned since it is defined as the horisontal distance over the crack. It might be more reasonable to believe that the crack width should be defined as the distance measured perpendicular to the direction of the crack.

It can be concluded that question marks remains if the method provided for calculation of crack width in Eurocode 2 should be applied to shear and torsional cracks. Especially what reinforcement that should be used to determine the maximum crack distance in order to obtain the expected crack widths. If it can be determined that the method is appropriate also for shear and torsional cracks it should clearly stated in Eurocode 2 what reinforcement that is intended to determine the maximum crack distance and especially if the bending reinforcement should be considered.

Since there is no well-established method for calculation of crack widths of torsion and shear cracks, it is also interesting to investigate how designers cope with the demands presented in Eurocode 2 on a regular basis.

Another thing that can be discussed is how the crack width in case of shear and torsional cracks should be defined. For a prismatic member, such as the one in Figure 10.8, the crack is perpendicular to the reinforcement and the crack width is defined as the distance along the reinforcement bar. A shear crack on the other hand crosses the shear reinforcement with an angle and it is therefore difficult to know if the crack distance should be taken as the distance perpendicular to the crack or in the direction of the shear reinforcement. However, this has not been investigated further in this report.

11 Investigation

11.1 Introduction

In Chapters 4-10 selected issues from Eurocode 2 have been presented and discussed based on information obtained from an extensive literature study. In order to get more detailed information about and further discuss the different subjects it was of interest to carry out interviews with engineers within the building industry who have a lot of experience and knowledge. Some of the chosen topics presented in the previous chapters have also resulted in questions concerning how structural engineers in general interpret the rules provided in Eurocode 2. A survey was therefore performed among engineers that work with structural engineering within the building industry. Section 11.2, which describe the interviews, and Section 11.3, that includes the survey, present the procedure and the result from each part, respectively.

11.2 Interviews

11.2.1 Overview

The engineers that were appointed for the interviews are presented in Table 11.1.

Table 11.1 People interviewed in the investigation.

Interviewed subject	**Company**	**Professional role**	**Type of interview**
Ebbe Rosell	Trafikverket	Responsible of new construction, client structural engineer	Telephone
Mikael Hallgren	Tyréns	Structural engineer, specialist within concrete structures	Telephone
Bo Westerberg	Bo Westerberg Konsult AB	Structural engineer, specialist within concrete structures	E-mail
Johan Söderberg	PEAB	Foreman, contractor	Personal meeting
David Eriksson	PEAB	Site manager, contractor, structural engineer	E-mail

The interview with Ebbe Rosell, who is manager of new construction at Trafikverket, was held in order to determine what questions that should be asked in the survey. Another interview with Mikael Hallgren, who is a structural engineer and specialist concerning concrete structures at Tyréns, was held after the survey in order to discuss questions that had come up in the survey and to further discuss the selected topics from Eurocode 2. The questions asked in the interviews were similar to those asked in the survey. However, since the survey was under development during the time when the interview with Ebbe Rosell was performed, some questions were not the same. After the interview with Mikael Hallgren one additional interview was held with Bo Westerberg, from Bo Westerberg Konsult AB, who also is an experienced structural engineer and specialist. This interview had the purpose to further discuss some of the background theories presented in Chapters 4-10. The interview also provided additional answers to the ambiguities that were not found during the literature study.

In order to capture to what extent structural engineers perform solution for reinforcement detailing solutions that facilitates the work at the construction site, one foreman and one site manager at PEAB were interviewed. The questions asked to the contractors are not entirely dependent on the ones asked at the other interviews or in the survey. However, these questions are related to what have been included in the report and the interviews also served the purpose of finding adequate questions to ask in the survey. All of the interviewed persons will be further presented in Section 11.2.2.

It should be noted that it is the personal opinions of the interviewed persons that are presented in Sections 11.2.3-11.2.7. If it is not the personal opinion that is provided in the answer, i.e. if it for instance is a company's or a committee's opinion, this is emphasised in the presentation of the result.

11.2.2 Interviewed persons

The persons that were selected for the interviews have different background and experience. Ebbe Rosell works at Trafikverket in Borlänge in the investment department as responsible for codes for new bridges. Ebbe Rosell will be referred to as Rosell further in the report. He has an experience of about 25 years within the building industry. Since he works at a client organisation, he and his colleges examine the documents that the structural engineers deliver, why it is of interest to see his point of view regarding detailing solutions in concrete structures. He has also been engaged in influencing the standards used in Sweden. The interview with Rosell was made by telephone, where Morgan Johansson from Reinertsen Sverige AB also participated.

Mikael Hallgren works at the consulting company Tyréns in Stockholm as a designer appointed to be a specialist within concrete structures. Mikael Hallgren will be referred to as Hallgren further in the report. He has a master's of science degree in civil engineering and a PhD in concrete structures. Hallgren has 23 years of experience within the building industry and has worked within the areas of housing and industrial buildings as well as bridges and tunnels. Hallgren is chairman in a committee called TK556 Concrete structures in the Swedish Standard Institute, SIS. This group is part of the European committee, CEN-TC250-SC2 where he is a Swedish delegate. Hallgren together with Bo Westerberg are responsible to respond on comments and questions concerning Eurocode 2 that are sent to SIS's helpdesk, see Section 2.1.1. He was also member of the group that wrote Svenska Betongföreningens handbok, Betongföreningen (2010). The interview with Hallgren was made by telephone.

Bo Westerberg works at the consulting company Bo Westerberg Konsult AB and is a very experienced structural engineer within concrete structures. Bo Westerberg will be further referred to as Westerberg. He has been working at the former J&W, today mostly known as WSP, for 14 years, Strängbetong for 6 years and at Tyréns for 15 years. Meanwhile, he has also been working on and off at KTH, including 10 years at part time as adjunct professor in structural concrete. He retired in 2009 but has continued his working career as a freelance consultant in his own company. Between the years 1990-2010 Westerberg was a member of European and Swedish committees and working groups for Eurocode 2. He has himself written about 15 % of the content in EN 1992-1-1 and has since 2008 held more than 20 courses concerning Eurocode 2.

Johan Söderberg is employed at the contractor PEAB as a foreman, where he has got a lot of experience during construction of reinforced concrete structures. Johan Söderberg will be referred to as Söderberg further in the report. He is holding great experience due to his participation in the construction of 15 bridges, working mainly as carpenter, but also as reinforcing steel worker. Since 2012 he has changed his professional role from being carpenter to being foreman. He has studied two years for a bachelor degree in civil engineering. The authors had the opportunity to meet Söderberg at his current construction site, the industry Perstorp, in Stenungsund.

David Eriksson is employed at the contractor PEAB as a site manager working with concrete bridges. David Eriksson will be referred to as Eriksson further in the report. He has studied 4.5 years for a master’s of science degree in structural engineering. After his education Eriksson has worked two years as a foreman and one year as a site manager. He, together with the client, has the largest influence during the construction of reinforced concrete structures. Eriksson has answered the same questions as Söderberg with the difference that his answers were delivered by e-mail.

11.2.3 Result from interview with Ebbe Rosell

11.2.3.1 Introduction

The interview with Rosell was carried out 5th of April in 2013 and is described in the following text. It should be noted that the interview with him was performed before the questions in the survey were finally formulated, where he contributed to highlight problem areas regarding design and detailing of reinforced concrete structures.

A client, such as Trafikverket, will have a large influence on the final documents delivered during the design process including, among other things, calculations and reinforcement configurations. Trafikverket examines and approves the documents and thereby has the possibility to come with opinions on the work of the structural engineer.

11.2.3.2 Reinforcement detailing of concrete frame corners

In Section 4.4 it was described that Eurocode 2 provides a larger number of recommended reinforcement configurations for concrete frame corners than the Swedish handbook BBK 04. For a concrete frame corner subjected to opening moment BBK 04 recommends to perform the reinforcement detailing as in Figure 4.10a, i.e. by reinforcement loops with an E-bar at the inside of the corner, that is also shown as a recommendation in Eurocode 2, see Section 4.4.1. Rosell was therefore asked about what reinforcement configurations that he believes are used in design of concrete frame corners. Rosell has not noticed any changes regarding reinforcement detailing provided by structural engineers on concrete frame corners subjected to opening moment due to the transition from the previous Swedish handbook BBK to the new standard Eurocode. However, Rosell has not seen so many detail solutions on concrete frame corners, why the statement above should be read with carefulness. For more information regarding this see Section 4.4 and question number 8 asked in the survey in Section 11.3.2.8.

11.2.3.3 Minimum shear reinforcement

The background to the requirement for minimum amount of shear reinforcement in Expression EC2 (9.5N), see Equation (5.27) in Section 5.4, has not been found. However, according to Rosell the requirement depends on some accidents that occurred in Germany. The failures were brittle in combination with settlements of the supports and unexpected thermal actions. For more information regarding minimum amount of shear reinforcement, see Section 5.4.

11.2.3.4 Configuration of shear reinforcement

Rosell has noticed that occasionally, when structural engineers design shear reinforcement, they do not enclose all of the longitudinal bending reinforcement as required according to Figure 5.22 in Section 5.6. Rosell believes that one reason for this is the stressful work of the structural engineers due to tight time schedules and economical pressure. Structural engineers are often short of time when calculations and reinforcement drawings need to be delivered to the client or the contractor. This can result in forgetting parts of the basic theory such as designing for the resulting tensile force that must be captured into the cross-section with shear reinforcement that encloses the longitudinal reinforcement. Another reason why designers might perform detailing of shear reinforcement wrongly can be because a node in a strut and tie-, or in this case a truss model is considered to be between the longitudinal reinforcement layers, i.e. in the centre of the tensile stress field, see Figure 11.1, and it can therefore be perceived as correct to locate the shear reinforcement to the node point and not around the tensile stress field. In Section 5.6.1 the problem regarding limited space in slabs has been discussed, for which a solution where shear reinforcement is not placed below the longitudinal reinforcement might seem attractive.

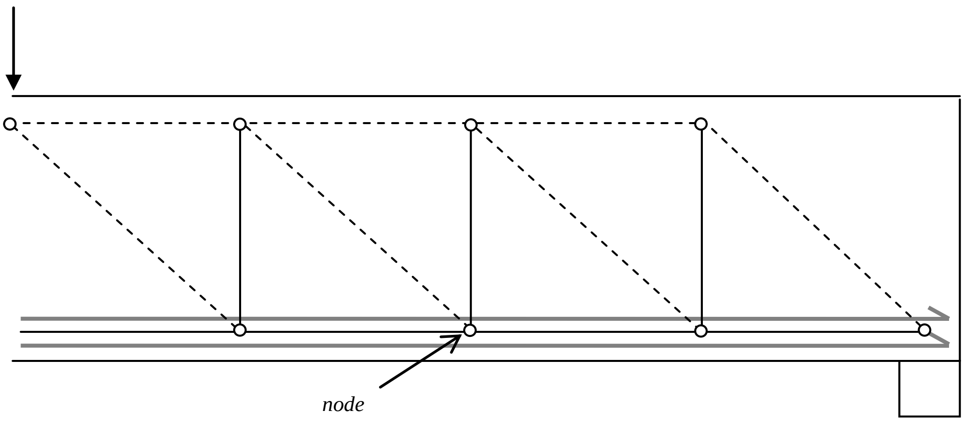


Figure 11.1 Truss model of a beam with the node point placed between the longitudinal reinforcement layers. The node does not indicate that the shear reinforcement must be extended to enclose all longitudinal reinforcement.

If a cross-section is designed with a minimum amount of bending reinforcement inside the shear reinforcement stirrups and a large amount of the longitudinal reinforcement outside the stirrup, the structural engineer relies on the tensile capacity of the concrete. This will lead to an incorrect detailing where the full capacity of the whole cross-section cannot be achieved. It should be noticed that when Trafikverket discovers such errors in detailing, the structural engineer of the consulting firm often opposes to the changes that have to be made and does not understand why the proposed solution will not work.

To get more information regarding shear reinforcement configurations, see Section 5.6. This subject was also included in the survey, see question number 6 in Section 11.3.2.6.

11.2.3.5 G-bars

According to the requirements in Paragraph EC2 9.2.2(4), see Section 5.6, at least 50 % of the shear reinforcement should consist of enclosing links. However, in Sweden this requirement does not apply to shear reinforcement in the form of bent up bars, why it is of interest to find out whether G-bars are considered as bent up bars or not.

Shear reinforcement designed as G-bars is used frequently according to Rosell. Trafikverket regards these in the same way as shear reinforcement in the form of bent up bars. Rosell believes that this is unclear in Eurocode 2, since it is not mentioned.

G-bars and bent up bars used as shear reinforcement will not enclose the outside of the longitudinal tensile reinforcement like stirrups or links do, see Figure 5.22a. However, “the enclosing effect” in this direction is according to Rosell replaced with the anchorage to the longitudinal reinforcement in the outer most layer. Within the building industry it is unclear whether the G-bars, when used as shear reinforcement, must be spliced with the longitudinal reinforcement or if it is enough to only anchor the bars in the outer most reinforcement layer. Anchorage of the bars instead of splicing will according to Rosell be more favourable with regard to limited space in the cross-section. Rosell also states that according to Trafikverket’s point of view it is allowed to anchor the G-bars in contact with the longitudinal reinforcement in the outer most layer.

Concerning the orientation of the G-bar, Rosell states that previously all G-bars were designed with an angle of 45º relative to the longitudinal axis. In such situations it is obvious in what direction the bars should be placed, i.e. crossing the cracks in the opposite direction. However, nowadays G-bars may also be designed with an angle of 90º, why it is not as clear in what direction they should be placed. Rosell believes that the recommended direction in Westerberg (1995), presented in Figure 5.28 in Section 5.6, is correct where the G-bars capture possible cracks within the bars.

To get more information regarding G-bars see Section 5.6. A question concerning this was included in the survey, see question number 5 in Section 11.3.2.5.

11.2.3.6 Suspension reinforcement

When Rosell was asked about how he perceives the knowledge about suspension reinforcement within the building industry, he answered that previously there were problems with reinforced concrete structures where additional reinforcement at indirect supports was left out. At that time the current requirements were not fully clear regarding this. Sometimes Trafikverket still has different opinions than structural engineers regarding additional suspension reinforcement at indirect supports.

Rosell described that in Bro94, Vägverket (1994), it is stated that the “normal” shear reinforcement could be accounted as suspension reinforcement. However, this is not correct. In the case of for instance an indirect support or a load that acts at the bottom of the cross-section, the load is coming into the main beam at the bottom and needs to be lifted by additional suspension reinforcement in order to take it into the inclined struts that carry the shear force. This is shown in Figure 5.42 and Figure 5.43. Rosell added that not until it was noticed that large cracks had occurred in structures that were not provided with additional suspension reinforcement, it was recognised that the requirements written in Bro 94 deviated from BBK.

Rosell also added that it previously, in Bro 94, was described that the shear reinforcement placed inside the grey area shown in Figure EC2 9.7, see Figure 5.39 in Section 5.8, could be accounted as suspension reinforcement, which is not correct. When this was realised the requirement was changed so that the grey marking corresponds to the area that should be provided with additional suspension reinforcement, i.e. the requirement that is valid in Eurocode 2 today.

For more information regarding suspension reinforcement see Section 5.8 and survey question number 7, see Section 11.3.2.7.

11.2.3.7 Configuration of transversal torsional reinforcement

Rosell believes that the requirement regarding splicing of transversal torsional reinforcement described in Paragraph EC2 9.2.2(3), placed in a section regarding shear reinforcement, is misplaced in Eurocode 2. The paragraph states that it is not allowed to have a lap joint near the surface of the web, if the link should enable a load path for torsion. It can, because of this unfortunate placing in the standard, be easy to miss this information and Rosell is unsure whether structural engineers in general are aware of this requirement. He does not know any reason why it is not allowed to place lap splices of torsional reinforcement in web sections.

Rosell commented that sometimes people, responsible for the Eurocodes, are very good concerning the scientific background of design requirements but not always have the same deep knowledge of contract language and contract situations. In this case a rule for splicing of torsional reinforcement is included in a chapter dealing with shear reinforcement. Hence, it can be argued that in a contract situation this requirement is not valid at all.

When the practical aspects of lapping transversal torsional reinforcement in webs were discussed with Rosell, he replied that it at the construction site usually is easier to perform lapping in webs than in flanges. However, according to the rules regarding detailing of transversal torsional reinforcement in Eurocode 2 it is not allowed to place a lap splice in the web.

During another discussion that was held during the interview it was said that Eurocode 2 can be assumed to primarily be written for reinforced concrete members in buildings and not for bridges and tunnel structures. In buildings bars with diameter of 8 mm are generally used for stirrups and links, which are possible to bend by hand.

However, in for instance bridges, bars with diameters of 16 mm are more commonly used for stirrups and links, which are more difficult to bend by hand. This means that certain detail solutions for stirrups or links works well for $\phi 8$ but not for $\phi 16$.

For more information regarding transversal torsional reinforcement see Section 6.3. A question concerning this was included in the survey, see question number 4 in Section 11.3.2.4.

11.2.3.8 Lapping of longitudinal reinforcement

According to Rosell, improvements of the requirements regarding lapping of longitudinal reinforcement in Section EC2 8.7 have been asked for, i.e. they need clarification. As implied in Section 9.4 it is not clear whether or not it is allowed to splice all the longitudinal reinforcement bars in the same section by means of lapping. However, Rosell believes that it is allowed to lap 100 % of the longitudinal bars in one section if transversal reinforcement is added. He understands that the rule concerning a distance of $0.3l_0$ between lap ends is disregarded in such case. In standards from other countries in Europe there are different rules concerning splicing of reinforcement, which unfortunately not have been captured in Eurocode 2.

The need for transversal reinforcement in the lap section was also discussed during the interview. In addition to what have been mentioned in Section 9.4 the requirements in Eurocode 2 state that the required transversal reinforcement needs to be anchored into the body of the cross-section, which according to Rosell is unpractical. Eurocode 2 also lacks information on how the amount of transversal reinforcement, which should be anchored into the body of the cross-section, can be calculated.

According to Eurocode 2 the transversal reinforcement needs to be added in the outer regions of the lap splice. This will not be a problem in slabs, but for a beam it will be more problematic to provide transversal steel. Rosell agreed on these thoughts and said that it is difficult to know how to interpret the requirements concerning lapping of reinforcement if applied to for instance shear reinforcement.

For more information regarding lapping of longitudinal reinforcement see Section 9.4. A question in the survey was asked regarding the amount of reinforcement lapped in the same section, see question number 3 in Section 11.3.2.3.

11.2.3.9 Minimum reinforcement requirement for crack control

How to interpret the minimum reinforcement requirement in Section EC2 7.3.2, see Section 10.2, is not obvious when designing a beam with relatively high cross-section, with for instance a cross-sectional height of 800 mm. If the requirement in Section EC2 7.3.2 is followed, it will result in too much reinforcement due to the use of the concrete area, A_{ct}, in the tensile zone just before cracking. In BBK 04 this is not the case, since the area should be taken as the effective concrete area, $A_{ef,BBK}$. Since this area is smaller than the one suggested in Eurocode 2, a problem with large reinforcement amounts in beams with relatively high cross-sections has occurred from the transition from BBK 04 to Eurocode.

It is unclear if crack reinforcement should be added at web faces or not, when the cross-section of a beam is relatively high. During the interview Rosell described that

Trafikverket has chosen to provide reinforcement everywhere in concrete structures. However, this requirement is very old. In the 80's it was required by the former Vägverket to provide surface reinforcement at least $\phi 10s300$ along the face of the whole structure. This requirement has later been increased with about 30 % to a minimum amount of at least 400 mm^2/m, which is still current for bridges. It does not matter if the intended section or zone is in compression or tension. The requirement has to do with stresses that can occur due to restraint, as for instance uneven shrinkage due to drying of the surfaces and alkali-silica-reactions in the concrete. Rosell means that a correct interpretation of Trafikverket's requirement is that there should be a reinforcement basket around the whole structure and said that the requirement is set also with regard to unforeseen events.

For more information regarding longitudinal crack reinforcement see Section 10.2. The corresponding survey question number 2 can be found in Section 11.3.2.2.

11.2.3.10 Limitation of crack widths for shear and torsion

There is no advised method in Eurocode 2 concerning check of shear or torsional crack widths, why a question concerning this was asked to Rosell. He described that a previous limitation of the steel stress was, according to Trafikverket set to 250 MPa in the ultimate limit state and this was used as a standard value. By this rule in the ultimate limit state it was assumed that cracks should be sufficiently small in the serviceability limit state without an explicit check. However, this requirement disappeared when Sweden changed from BBK 04 to Eurocode 2. The value of 250 MPa was only used as a fast solution at the time it was derived, but the development of the requirement unfortunately stopped. Another reason why the recommendation from Trafikverket was removed was the uncertainty of who became responsible, if a too large shear crack would occur. It was not clear if it should be Trafikverket who set the limitation or if it should be the one who performed the design who should have the legal liability.

For more information regarding check of shear and torsional cracks see Section 10.3. This subjected was also included in the survey, see question number 1 in Section 11.3.2.1.

11.2.3.11 Flexibility of reinforcement solutions

Due to late changes in drawings or at the construction site problems can occur, if reinforcement solutions are not flexible enough. The structural engineer might disregard problems with fitting the reinforcement within the formwork and still obtain appropriate concrete cover. Rosell has noticed that many reinforcement layouts provide solutions that are not as flexible as desired. One example, where changing of conditions at the construction site might occur, is if piles happen to be placed too close to the edge of a foundation slab resulting in that the size of the slab needs to be increased. If C-bars are used as reinforcement in the slab, the configuration will not be flexible, see Figure 11.2a. An alternative configuration, in order to avoid this kind of problem, is to use two B-bars instead, see Figure 11.2b. Rosell said that a configuration of two B-bars may, from a structural engineer's point of view, seem more expensive than a configuration of single C-bars, since the B-bars must be spliced with a lap length to each other.

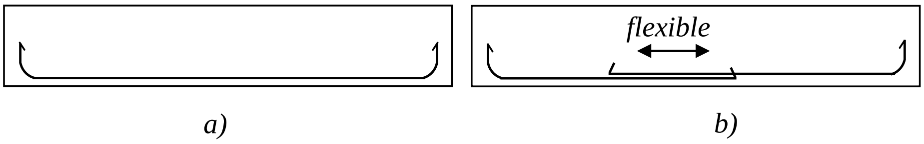


Figure 11.2 Two types of reinforcement solutions in a slab, a) C-bar, b) B-bars.

11.2.4 Result from interview with Mikael Hallgren

11.2.4.1 Introduction

The interview with Hallgren was performed by telephone at the 29th of May in 2013. It should be noted that the interview with Hallgren was performed after the survey was finished. Most of the questions asked to Hallgren are therefore related to the ones asked in the survey. Some questions that came up during the writing of the first ten chapters in the report were also asked.

In the beginning of the interview Hallgren described that the European committee, where he is a Swedish delegate, already has received about 400 comments on Eurocode 2, despite that it is only about six or seven out of 27 countries that have sent in their comments. This means that the committee has a lot of work to do before the new version of Eurocode 2 is finished. Hallgren says that the European committee has a goal to finish the new version in the year of 2019.

11.2.4.2 Minimum longitudinal reinforcement due to bending

Hallgren was asked about the background to the minimum reinforcement requirement for flexural reinforcement presented in Expression EC2 (9.1N), see Equation (4.4) in Section 4.2.1. In Section 4.2.2 it was shown that the requirement is derived from a rectangular concrete cross-section resulting in a value of 0.26 included in the equation. The discussion was about why a more general expression for the requirement is not used in Eurocode 2. Hallgren answered that the parameter 0.26 in Expression EC2 (9.1N) is a national selectable parameter. He also said that he was member of the reference group that brought up proposals to the national selectable parameters in Sweden to the first Eurocode together with Bo Westerberg who was also member of the group acting as chairman. According to Hallgren the whole group agreed on that the minimum reinforcement amount should be determined for a rectangular cross-section. If the minimum reinforcement amount should be calculated for other types of cross-sections, then a new parameters have to be derived. However, calculations performed for a rectangular cross-section are assumed to provide values on the safe side.

When the lower limit of the minimum reinforcement requirement was discussed Hallgren described that by assuming a value of the steel strength is assumed, as for instance $f_{yk} = 500$ MPa, the concrete strength corresponding to the lower limit can be determined from Expression EC2 (9.1N). This implies that the lower limit of Expression EC2 (9.1N) makes sure that also cross-sections with low concrete strengths will be provided with a minimum amount of longitudinal reinforcement. This is also explained in Section 4.2.3.

If comparing the expression provided in Eurocode 2 to the expression for minimum reinforcement in the American code, it can be seen that the two expressions are almost the same. Hallgren describes that this probably depends on that the countries give recommendations to each other when the codes are developed.

For further discussion regarding Expression EC2 (9.1N), see Section 4.2.

11.2.4.3 Reinforcement detailing of concrete frame corners

It can be argued that the recommendation in Annex EC2 J.2.3, regarding the limit of the reinforcement amount in concrete frame corners subjected to opening moment, is too high. The recommendation is set to 2 %. Hallgren had not noticed the high reinforcement amount before it was pointed out to him at the interview. However, when Hallgren briefly studied the recommendations in Annex EC2 J he agreed on that 2 % reinforcement amount seems to be very high. Hallgren said that he has not received any comments on this to SIS’s helpdesk. However, other comments might have been sent to Bo Westerberg.

Hallgren says that it should be noted that information in the informative appendices does not contain any rules, i.e. it is only recommendations. Hallgren describes that according to his opinion the most interesting in Annex EC2 J is the recommendations on how to establish a strut and tie model for a concrete frame corner so that the structural engineer can perform design calculations more in detail. It can be discussed how large a moderate opening moment is, since this appendix does not provide any recommendation concerning this.

For further information regarding reinforcement amount in concrete frame corner subjected to opening moment, see Section 4.4. This subjected was included in the survey, see question number 8 in Section 11.3.3.8.

11.2.4.4 Configuration of transversal torsional reinforcement

Hallgren states that it is unfortunate that the background to Chapter 9 of Eurocode 2 that concerns detailing of reinforcement is forgotten in ECP (2008a). At the interview a figure was shown to Hallgren including different configurations of reinforcement links, see Figure 11.3. It was discussed which of these that are allowed to be used as transversal torsional reinforcement. Hallgren thinks that only the links that are illustrated in Eurocode 2, see Figure 6.16, are acceptable solutions with regard to torsion. He added that it would be interesting to find the reason why it according to Eurocode 2 is not allowed to lap torsional links near the web surface. However, Hallgren has his own thoughts regarding this. If all laps are placed in the same section in the web and the anchorage length is fulfilled but not the lap length, then the stirrups will be sensitive when subjected to torsion. This depends on the fact that torsional cracks occur very locally, which is the reason why the torsional stirrup must be anchored in a mechanical way with bends. However, if the cross-sectional height is large and sufficient lap lengths can be achieved, then Hallgren does not see any physical reason why lapping in the web should not be allowed.

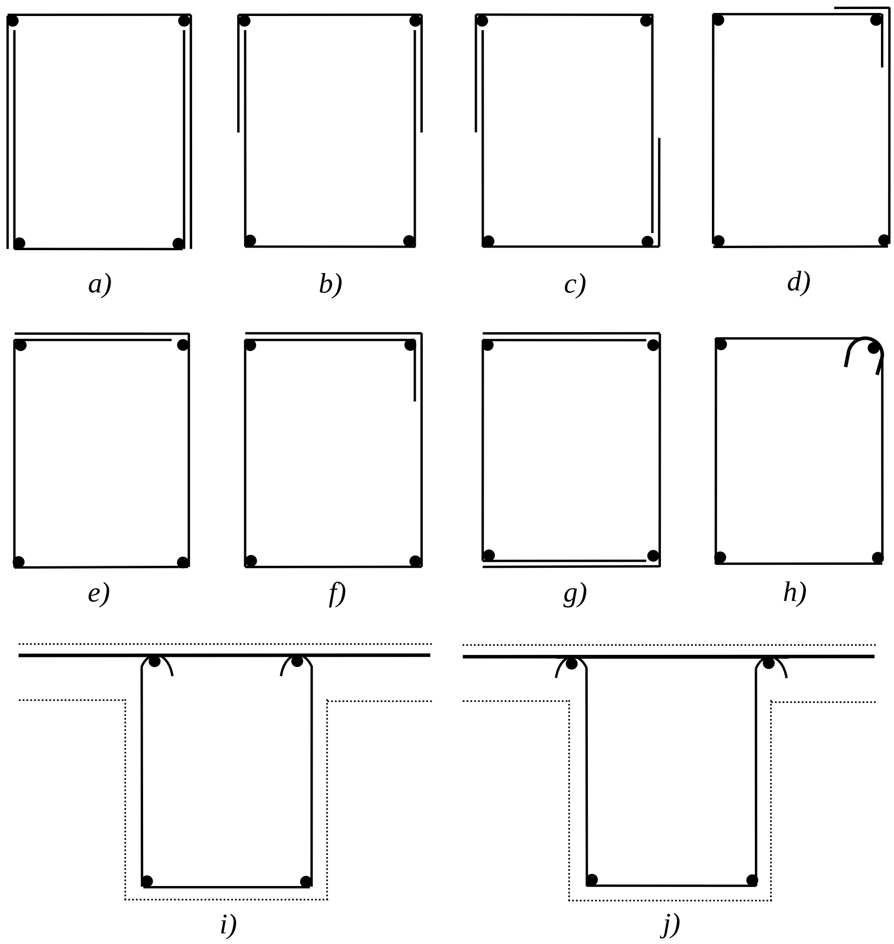


Figure 11.3 Different configurations of reinforcement links presented to Hallgren at the interview.

The comment from Hallgren to why Paragraph EC2 9.2.2(3) about shear reinforcement brings up information concerning torsional reinforcement is that when the superposition principle is used, i.e. when shear force and torsional moment are combined, the links becomes utilised in two ways. It is in such situations therefore important to know that the recommendations given for shear reinforcement are not sufficient. All of the recommended shear reinforcement configurations provided in Paragraph EC2 9.2.2(2), see Section 5.6.1, do not provide transversal reinforcement around the whole cross-section. Since the torsional moment is regarded as shear force when the superposition principle is used it is important to point out differences between torsional moment and shear force in Section EC2 9.2.2 concerning shear reinforcement. If it in Section EC2 9.2.2 is not mentioned anything about that it is not allowed to place lap splices in web sections, if the reinforcement is aimed to also resist torsion, it is possible that structural engineers believe that all the requirements concerning shear reinforcement are applicable also for situations when the superposition principle is used.

In situations when a concrete member should be designed for torsional moment only the recommended configurations in Section EC2 9.2.3, see Section 6.5.1, are the ones to be followed. Hallgren thinks that it is clear from Figure EC2 9.6, see Figure 6.16, that a lap in the vertical leg near the web surface is not allowed. Figure EC2 9.6 only provides recommend configurations where anchorage is achieved by bends and not by lapping, even if this is not written in text. The figure indicates that the links need to be anchored well, which was stated before.

For more information regarding transversal torsional reinforcement see Sections 6.3 and 6.5. In the survey a question regarding configurations of transversal torsional reinforcement was asked, see Section 11.3.3.4.

11.2.4.5 Combination of torsional moment and shear force

A question regarding design for combined shear and torsion was asked to Hallgren. The authors have interpreted the information in Table 6.1, which is taken from Betongföreningen (2010a), as that design with regard to superimposed shear force and torsional moment will result in that the required longitudinal reinforcement will not be spread around the whole cross-section. The longitudinal reinforcement will only be placed in the horisontal walls of the cross-section. This has been illustrated in Section 6.4.3, see Figure 6.15. Compare also Figure 5.6 and Figure 6.5 to each other. Hallgren is not completely sure about how to perform a combination of torsional moment and shear force, since he has not investigated this in detail. However, he recommended talking to Bo Westerberg in this matter, since he is the creator of the table in Betongföreningen (2010a). Hallgren mentioned that the documentation of the background information to Betongföreningen (2010a) was poor. However, most of the written material has been included in the handbook.

Hallgren said that the reason why the longitudinal torsion reinforcement must be spread evenly around the cross-section is because torsional cracks occur like a spiral around the whole cross-section. The cross-section needs to consist of links and longitudinal reinforcement that cross the torsional cracks in order to balance the compressive stresses in the inclined struts between the torsional cracks. He emphasised that this is why the longitudinal reinforcement should be spread in all the walls of the cross section.

The difference between torsional moment and shear force is according to Hallgren that the additional longitudinal reinforcement due to inclined shear cracks is calculated by using the bending moment. The required longitudinal tensile capacity due to bending calculated in one section is moved a certain distance to take into account that the crack activated by the bending moment is inclined due to shear force. This is in this report explained in Section 9.2.

Hallgren says that if shear force and torsional moment are combined in design, the latter will give a contribution to the shear force only in one of the vertical walls. This will in turn result in a contribution to the bending reinforcement due to the inclined shear cracks. In addition to this, extra longitudinal reinforcement calculated by Expression EC2 (6.28), see Section 6.2.1, should be spread around the cross-section to take into account the effect of torsional moment. However, after a while Hallgren realised that the extra longitudinal reinforcement only should be calculated for walls that have not been designed earlier, i.e. the horizontal walls. However, Hallgren considers that if Table 6.1 is interpreted as it is allowed to only provide longitudinal reinforcement in the horisontal walls, extra longitudinal reinforcement should be added in order to spread it evenly around the cross-section.

To get more information regarding combined shear and torsion, see Section 6.4.

11.2.4.6 Shear between web and flanges

Regarding Expression EC2 (6.20) that provides the design shear stress at the web-flange interface, see Equation (7.1) in Section 7.2.1, a question why it is allowed to use the length Δx instead of letting Δx go to zero in the expression was asked to Hallgren. If the length Δx is used there is risk that the member will not be designed with a sufficient amount of reinforcement to avoid utilising the tensile strength of concrete. If Δx instead goes to zero the design shear stress, v_{Ed}, will be determined for the actual shear force per unit length in each section, see Section 7.2.3. Hallgren's answer to this was that it is the increase ΔF_d of the normal force F_d that needs to be designed for. ΔF_d is the force increase that results in shear stress in the longitudinal section between the flange and the web. The maximum allowed length Δx is regulated in the last sentence of Paragraph EC2 6.2.4(3) and the increase ΔF_d over that length is approximated to be linear. The transverse shear force over the cross-section is resisted by calculating the required shear capacity of the cross section according to Expressions EC2 (6.2) and EC2 (6.5) or Expressions EC2 (6.8) and EC2 (6.9).

According to Paragraph EC2 6.2.4(5) it is written that the design for full shear stress and transversal bending moment added together is not needed. Hallgren commented that if the full contribution from shear stress and transversal bending moment would have been taken into account at the same time, the required reinforcement amount would have been unreasonably large.

To get more information regarding shear between web and flanges see Chapter 7.

11.2.4.7 Anchorage of bottom reinforcement at end support

When anchorage of bottom reinforcement at end supports was discussed two figures were shown to Hallgren. These can be seen in Figure 11.4. Figure 11.4a shows a situation where the anchorage is fulfilled at the face of the support. However, anchorage is not provided along the entire node region. Figure 11.4b shows the extension of the node region and a situation where the reinforcement has been extended through this region. According to Paragraph EC2 6.5.4(7) it is required to extend the reinforcement over the node region as in Figure 11.4b. However, in Section EC2 9.2.1.4 it is stated that the anchorage should be measured from the face of the support implying, that it is sufficient to provide anchorage as in Figure 11.4a.

Hallgren was asked if he thinks that Figure 11.4a is an adequate solution, since there is no reference between Section EC2 9.2.1.4 and EC2 6.5.4 in Eurocode 2. Hallgren described that anchorage should be provided along the entire node. He also added that from the first paragraph in the section in Eurocode 2 about nodes, i.e., Section EC2 6.5.4, it can be deduced that the rules provided for nodes also applies to regions which are not designed by the strut and tie method where concentrated forces are acting, such as for instance a support section. However, Hallgren said that he will probably use the recommendations in Section EC2 9.2.1.4, see Figure 11.1a, since Paragraph EC2 6.5.4(7), see Figure 11.4b, will result in too much reinforcement. This is despite that Paragraph EC2 6.5.4(1) is a principal rule.

For more information regarding anchorage of bottom reinforcement see Section 9.3.3

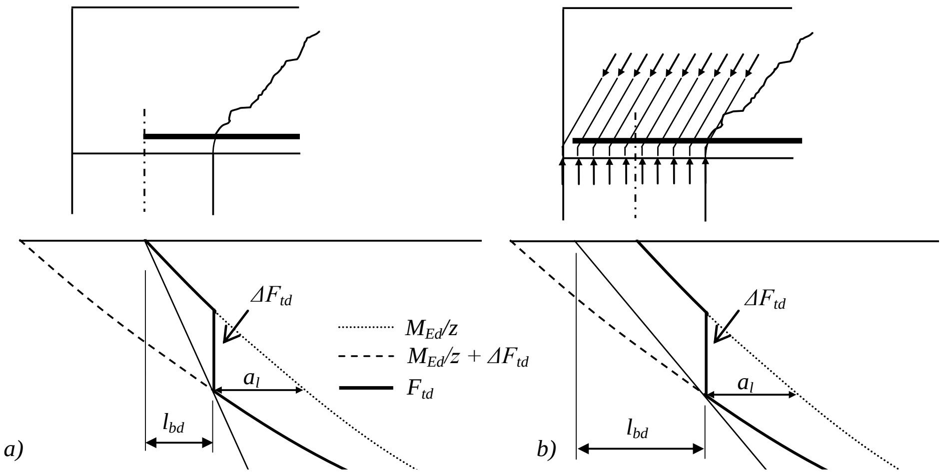


Figure 11.4 Different design approaches for anchorage of bottom reinforcement at end supports presented to Hallgren in the telephone conversation.

11.2.4.8 Lapping of longitudinal reinforcement

The same question that was discussed with Rosell was also asked to Hallgren, i.e. if he from Section EC2 8.7 in Eurocode 2 can determine whether or not it is allowed to lap all reinforcing bars in the same section. Hallgren agrees that there are contradictions between Sections EC2 8.7.2(3) and EC2 8.7.2(4) in Eurocode 2. These sections provide rules for lapping of reinforcement. He also described that this problem is discussed also at Tyréns in Stockholm. Despite this, he believes that it is allowed to lap all longitudinal reinforcement bars in one section provided that all bars are placed in only one layer. If the cross-section consists of more than one layer and all the bars in the first layer is lapped in the same section, the lap splices in the second layer need to be positioned at another place along the member. However, Hallgren knows that there are structural engineers who follow Paragraph EC2 8.7(3) without exception and therefore never place all lap splices in the same section.

In Eurocode 2 difference is made between lap length and anchorage length. According to Hallgren this difference is good and is an improvement compared to BBK 04 where no distinction between anchorage length and lap length is made. Hallgren thinks that it is clear from Eurocode 2 that if all bars are lapped in one section this will result in an increased amount of reinforcement in the member due to longer lap lengths. Hence, the structural engineer is rewarded, by a reduced amount of reinforcement due to decreased lap length, when taking time to design the member with only 50 % of all lap splices in the same section. In this case the structural engineer needs to spend time to work out different lengths in order to keep the provided distances between the laps. However, if the structural engineer wants to simplify the design and add to the cost by increasing the reinforcement, a configuration of 100 % splicing can be chosen, since the different lengths of the bars are more consistent. However, it should be noted that the necessary increase of reinforcement is very small.

For more information regarding lapping of longitudinal reinforcement see Section 9.4. A question regarding this was included in the survey, see question number 3 in Section 11.3.3.3.

11.2.4.9 Minimum reinforcement requirements for crack control

Expression EC2 (7.1), see Equation (10.1), is discussed in Section 10.2.3 where it is described that this expression, provided as a minimum reinforcement requirement for crack control, will generate a large reinforcement amount. This is because the whole area of the concrete section in tension before cracking is included in the expression. When Hallgren was asked about this he said that he agrees that the expression will give a very high reinforcement amount. According to Hallgren there are many people within the building industry that are of the same opinion as him. The expression will result in a much larger reinforcement amount than that the structural engineer received from the corresponding expression in BBK 04.

Hallgren added that nowadays concrete slabs with large thicknesses are often used, in situations when the structural member works as a composite foundation with piles. Hallgren described an example of such a foundation that was made in Skåne. The composite foundation was performed on clay with 600 mm thick slabs. This type of structural members will result in unreasonable amounts of reinforcement, if Expression EC2 (7.1) is used. If Hallgren remembers correctly, the composite foundation in Skåne required a reinforcement configuration of $\phi16s80$. This large amount of reinforcement has never been seen before, at least not within structural members in buildings.

A common opinion among structural engineers in Sweden and especially Hallgren and Westerberg (2013) is that they think that the calculation of minimum reinforcement amount for crack control can be performed with the corresponding expression from BBK 04 instead. This has been one of the comments sent to the developers of Eurocode 2. The part of the cross-section that can affect the cracking is the effective concrete area that surrounds the reinforcement. If top and bottom reinforcement is placed in a very thick slab this reinforcement cannot control cracking in the middle of the cross-section. The slab is only held together in those areas where the reinforcement is provided.

After it was discovered that Expression EC2 (7.1) can result in large reinforcement amounts compared to BBK 04 Hallgren wrote a report, Hallgren (2012), that was published at a workshop in Vienna in the year of 2012. The workshop brought up comments regarding the usage of Eurocode 2. It should be noted that the report only consists of recommendations and conclusions and did not result in the final suggestion of a new expression for minimum crack reinforcement that was sent to the European committee. It can be noted that Hallgren’s report also describes that the previous expression used in BBK 04 for calculation of minimum reinforcement will result in too little reinforcement. The condition, that a new crack must be able to form before the yield strength of reinforcement is reached after the first crack has occurred, will not always be fulfilled.

A master’s thesis that Mikael Hallgren has been supervising provides a more detailed investigation of the content in his report, see Björnberg and Johansson (2013). The investigation consists of numerical analyses where the authors as a result have come up with an expression that will decrease the effect of the concrete area. The project showed that the amount obtained from the requirement in Eurocode 2 was too large and that the requirement in BBK 04 provided too little reinforcement. The suggested modification of the expression results in a reinforcement area that is half of what had been calculated according to Eurocode 2.

It should be noted that Expression EC2 (7.1) does not provide a certain crack width if the steel stress in the expression is set equal to the characteristic yield strength. It only describes the required reinforcement amount in order to distribute cracks caused by restraint in order to achieve new cracks after the first one has occurred. This is why Hallgren considers this expression to be valid with regard to restraint, and thinks that it is most useful in this case. Hallgren has commented on this to his European colleagues in the European Committee. According to Hallgren the truth behind Expression EC2 (7.1) is somewhere in between restraint and bending.

Furthermore, Hallgren has noticed that Expression EC2 (7.1) unfortunately results in a different reinforcement amount compared to Expression EC2 (9.1N), see Equation (4.4), that provides a minimum reinforcement amount for flexural reinforcement in order to avoid brittle failure. According to Hallgren it is more logical that the minimum reinforcement requirement for distribution of cracks and the minimum reinforcement requirement in order to avoid brittle failure, in case of flexural bending, should result in the same reinforcement amount, since the motive in both situations is to avoid yielding of the reinforcing steel when the concrete cracks. Hence, Hallgren suggests that Expression EC2 (7.1) should be used when calculating the reinforcement amount due to restraint and Expression EC2 (9.1N) should be used when the beam is subjected to bending moment. After the minimum reinforcement amount is calculated the crack widths needs to be checked by other methods anyway. It should be noted that if Expression EC2 (7.1) is used in order to enable distribution of cracks characteristic values should be used since cracking occur in the service state.

To get more information concerning minimum reinforcement amount see Section 10.2.

11.2.4.10 Longitudinal crack reinforcement in beams with relatively high cross-sections

As a last comment in Section 10.2.3 it was discussed that it might not be necessary to place crack reinforcement in webs of relatively high beams, if there is no reinforcement that needs to be protected from corrosion. However, when this hypothesis was presented to Hallgren, he described that it according to Eurocode 2 always is required to place shear reinforcement in beams. This means that there always will be shear reinforcement in the web that needs to be protected from corrosion and that crack reinforcement therefore should be added also in that area. It is favourable to prevent cracking as much as possible by adding longitudinal reinforcement spread along the web. He also added that it actually has been shown that when cracking occurs, the cracks tend to follow the stirrups placed in the structure. This has for instance been seen at Slussen in Stockholm where rust has been created mainly in the areas close to the transversal shear reinforcement.

To get more information regarding longitudinal crack reinforcement, see Section 10.2. Question number 2 in Section 11.3.3.2, regarding if crack reinforcement is added or not in beams with relatively high cross-section, was included in the survey.

11.2.4.11 Limitation of crack widths for shear and torsion

In Section 10.3 it was stated that there is no generally accepted or established method for limitation of shear- and torsional crack widths. Hallgren was member of the group that wrote Betongföreningen (2010a), together with among others Bo Westerberg and Björn Engström. When limiting widths of cracks with regard to shear force and torsional moment Hallgren will use the method that is written in this handbook if he realises that there will be problem with large shear cracks, see Section 10.3.2. This can for instance be the case if the member has a large cross-sectional height and the web also is slender. The two suggested methods in Betongföreningen (2010a) concern plane stress. According to Hallgren the first method, see method a) in Section 10.3.2, describes equilibrium conditions through a diagonal cut over a quadratic element. The second method, method b) in Section 10.3.2, concerns a truss model and is inspired of the general method a). However, method b) is only useful when beam action is valid. In order to perform a more detailed analysis with regard to shear and torsional cracks Hallgren uses non-linear FE-analysis with crack models.

Limitation of crack widths for shear and torsion is discussed in Section 10.3.3 and this subject was also included in the survey, see question number 1 in Section 11.3.3.1.

11.2.5 Result from interview with Bo Westerberg

11.2.5.1 Introduction

The interview with Westerberg was held as an e-mail conversation between the 30th of May and 3rd of June in 2013. At first the interview was meant to act as a complement to the interview with Mikael Hallgren. In the cases when Hallgren could not provide adequate response to the questions asked to him, he urged on consulting Westerberg. However, a few questions that Hallgren already had answered were also asked to Westerberg in order to obtain further information and because he is a recognised structural engineer with extensive experience from the development of Eurocode 2.

11.2.5.2 Maximum longitudinal reinforcement

The thoughts presented in Section 4.3.3 concerning the misplacement of the ductility requirement $x/d < 0.45$ in Section EC2 5.6 under plastic analysis instead of Section EC2 5.4 concerning linear elastic analysis were shared with Westerberg in order to get his opinion about this. He agreed on that a linear elastic analysis based on an uncracked cross-section presumes some redistribution of moment, when the reinforced concrete member cracks. However, he thinks that this can be ignored.

Ductility requirements in Eurocode 2 have been presented in Section 4.3 in relation to the maximum reinforcement amount.

11.2.5.1 Reinforcement detailing of concrete frame corners

It was interesting to see if Westerberg also has perceived the limit of the reinforcement amount $\rho = 2\,\%$ for opening concrete frame corners, in Annex EC2 J.2.3, as a high value, see Section 4.4. Westerberg emphasised that 2% reinforcement amount is not a demand. It is only the limit for when to use one or the other reinforcement configuration according to Figures EC2 J.3 or EC2 J.4, see Figure 4.9 and Figure 4.10. This means that in most cases the simpler solutions in Figure 4.9 can be used. In BBK 04 a similar limitation is provided in order to be able to use the a configuration corresponding to the one presented in Figure 4.10a. This limitation is $5/f_{yk}$ which correspond to 1 % if f_{yk} = 500 MPa. Moreover, the configuration presented in BBK 04 always requires diagonal bars for concrete frame corners subjected to opening moments, which Eurocode 2 only requires for a reinforcement amount higher than 2%. Westerberg wonders if this could be the reason why the limit in Eurocode 2 might be perceived as liberal. He states that the requirements provided by BBK 04 are based on research that was performed in Nilsson (1973). Westerberg cannot comment on what is correct or not in this matter, without going to the sources. When he was asked about what he thinks about the reinforcement configuration in Figure 4.9b, he answers that it is probably good considering the theoretical relation between forces, but it might be a bit unpractical to perform at the construction site.

To get more information regarding concrete frame corners subjected to opening moment see Section 4.4. A question regarding this subject was asked in the survey, see question number 8 in Section 11.3.2.8.

11.2.5.2 Minimum shear reinforcement

Westerberg was asked about the background for the minimum reinforcement requirement for shear reinforcement stated in Expression EC2 (9.5N), see Equation (5.27) . Westerberg answered that he doesn’t have any good explanation for the expression itself, more than that it expresses some kind of general ductility requirement. The stronger the concrete is, the larger is the required amount of reinforcement and the stronger the reinforcement is, less is needed to match the concrete. In BBK 04 there is also a minimum shear reinforcement requirement, but unlike the one in Eurocode 2 this is only necessary to apply in regions where the shear capacity of concrete is not enough. According to Westerberg a minimum amount of shear reinforcement according to Expression EC2 (9.5N) should always be placed in beams, regardless if shear reinforcement is necessary or not.

To get more information regarding this see Section 5.4.

11.2.5.3 Configuration of shear reinforcement

According to Section EC2 9.2.2 it is recommended that a certain amount of the shear reinforcement should consist of links enclosing the longitudinal tension reinforcement, see Section 5.6. Westerberg was asked if he knows whether this may have something to do with the idea that links should enclose the longitudinal reinforcement in such a way that there will be a horisontal bar that can resist transverse tensile stresses that occur across the width of the cross-section, as described in Figure 5.23. Westerberg thinks that this is a good explanation to why a certain amount of the shear reinforcement should consist of enclosing links.

However, in Sweden it is allowed to use only bent up bars as shear reinforcement. It is, according to Westerberg, true that bent up bars cannot resist forces in the transverse direction across the width of the cross-section, but the bars can still form good nodes in a truss model. According to Westerberg G-bars can be used as bent up bars provided that they are placed in the correct direction, see Figure 5.28 and that they are spliced with a lap length to the longitudinal reinforcement.

To learn more about configuration of shear reinforcement see Section 5.6.

11.2.5.4 Load close to supports

When loads are applied close to supports there are rules in Eurocode 2 saying that the shear force contribution from those loads can be reduced if the loads are acting within a distance, a_v, between 0.5d and 2d from the edge of the support. When Westerberg was asked about why the lower limit of a_v is set to 0.5d, he answered that he agrees with the reasoning proposed in Section 5.7; that the entire load will be transferred directly to the support, if it is acting closer to the support than a distance d. Westerberg does not like that Eurocode 2 provides a lower limit of a_v. He thinks the rule is unnecessary and complicates things unnecessarily.

11.2.5.5 Suspension reinforcement

Westerberg was asked if he considers it to be a problem among structural engineers that they tend to forget to place suspension reinforcement in addition to the required shear reinforcement. He answered that he has no opinion about this, but he also stated that he always has taught about the different effects of loads that are applied on top or at the bottom part of a structural member and what that means for the shear reinforcement. As an example of where suspension reinforcement is required he mentioned beams that carry floor elements on bottom flanges.

Suspension reinforcement is discussed more in Section 5.8.

11.2.5.6 Longitudinal torsional reinforcement

In Eurocode 2 it is stated that the required amount of longitudinal torsional reinforcement may be reduced in compressive chords, see Section 6.2.3. A question concerning how to determine this reduction of longitudinal torsional reinforcement was asked to Westerberg. He stated that longitudinal torsional reinforcement is supposed to be able to resist a certain force. In a compression zone there is a force equal to M_{Ed} / z that can be subtracted from the force that otherwise would have been taken by the longitudinal torsional reinforcement in the compressive zone.

11.2.5.7 Configuration of transversal torsional reinforcement

According to Section EC2 9.2.2 it is not allowed to splice a link by lapping near the surface of the web if the link should be used in a load path for torsional moment. The question asked to Westerbeg was if there is any specific reason for this requirement. Westerberg replied that a torsional link, even if it is lapped in the web, probably will fulfil its purpose. However, the rule is an expression of caution since the knowledge about this was considered to be insufficient when the rules were determined. In Sweden this has been investigated in Nilsson (1973) on the behalf of former Vägverket. If he is not mistaken, Westerberg recalls that the result of this investigation was to allow lapping of shear reinforcement in webs. Westerberg added that he cannot see any difference between shear and torsion reinforcement in this respect. Configuration of transversal torsional reinforcement is discussed more in Section 6.4

11.2.5.8 Combination of torsional moment and shear force

According to Table 6.1 taken from Betongföreningen (2010a) the required longitudinal reinforcement due to inclined cracks can be calculated from a combined effect of shear force and torsional moment by using the expression for ΔF_{td}, see Equation (6.33). According to the authors this can be perceived as it is no longer necessary to evenly distribute the longitudinal torsional reinforcement along the sides of the cross-section. According to Paragraphs EC2 6.3.2(3) and EC2 9.2.3(4), see Section 6.2.1, longitudinal torsional reinforcement should always be distributed with a minimum distance between the bars. Table 6.1 therefore raises questions about how to handle the combined shear force and torsional moment. This question was specifically asked to Westerberg. According to Westerberg the intention is that the longitudinal torsional reinforcement can be designed by the additional tensile force, ΔF_{td}, calculated for a combined shear force equal to $V_{Ed,V+T} = V_V + V_T$, see Section 6.4.2. He also explained that the part of the longitudinal reinforcement that is related to shear force is always an increase of the bending reinforcement and the part of the longitudinal reinforcement that is related to torsional moment should in general be distributed in accordance to Paragraph EC2 6.3.2(3). However, this might be missed if only a truss model for shear force, i.e. Equation (6.33) , is applied in design.

To get more information regarding combined torsional moment and shear force see Section 6.4.

11.2.5.9 Shear at the interface between concrete cast at different times

According to Section EC2 6.2.5, see Section 8.2.1, it is stated that the cohesion factor, c, should be taken as zero if the stress across the joint interface, σ_n, is negative, i.e. tension. The question asked to Westerberg was if it is allowed to take the cohesion into account for a situation where the joint has cracked and thereafter been pressed together again. Westerberg thinks that the cohesion can be considered also after cracking. However, he does not consider the "c-term" as cohesion or bond, more like a point in a simple linear model adapted to test results, see Figure 3.14b in Section 3.2.2. In BBK 04 another adaption has been chosen, where c is set to zero but with a steeper inclination of the curve, i.e. a larger frictional coefficient μ. To get more information regarding shear at the interface between concrete cast at different times see Section 8.2.

11.2.5.10 Anchorage of bottom reinforcement at end supports

Anchorage of bottom reinforcement was also discussed with Westerberg. The same figures, see Figure 11.4, and the same question that was asked to Hallgren, i.e. if he thinks that Figure 11.4a provides sufficient anchorage of bottom reinforcement, were included in the e-mail conversation held with Westerberg. Westerberg replied that he does not think that Figure 11.4a is a good solution and that it should be taken into account that the node is extended along the support region. However, this might not be clear from Eurocode 2.

To get more information regarding anchorage of bottom reinforcement at end support see Section 9.3.

11.2.5.11 Lapping of longitudinal reinforcement

In Section EC2 8.7.2 instructions about how to perform lap splices are provided, see Section 9.4.1. In the situation presented to Westerberg one layer of longitudinal tensile reinforcement bars in a slab should be spliced. Westerberg claims that it is allowed to splice all bars in the same section under the conditions given in Paragraph EC2 8.7.2(4). However, the second indent in Paragraph 8.7.2(3), saying that there must be a distance of at least $0.3l_0$ between lap ends, should then be disregarded, since this requirement is irrelevant. Westerberg added that he thinks that this can be misunderstood from Eurocode 2.

To learn more about lapping of longitudinal reinforcement see Section 9.4.1. This subject was also included in the survey, see question number 3 in Section 11.3.2.3.

11.2.5.12 Clear distance between bars

The minimum clear distance between bars is, according to Paragraph EC2 8.2(2) determined to be the same as the diameter, ϕ, of the largest bar used in the configuration, provided that this is larger than 20 mm and the maximum aggregate size + 5 mm, see Section 9.5.1. Westerberg was asked about the background for the limit of 1ϕ. He was also asked why this distance is increased to 2ϕ in case of lapping, see Section 9.4.1, and why the criterion concerning maximum aggregate size have been left out from that requirement. According to Westerberg it is obvious that there should be a free space between bars. The limit of 1ϕ can be said to be compatible with rules for calculation of anchorage length and similar expressions. The reason why lap splices requires larger distances between bars has to do with increased risk of splitting failure. He also adds that the distance between bars in case of lapping also should fulfil the requirement of d_g+5 mm, regarding maximum aggregate size, even if this perhaps not is apparent from Eurocode 2.

To get more information regarding distance between bars see Section 9.5.

11.2.6 Result from interview with Johan Söderberg

11.2.6.1 Introduction

The interview with Johan Söderberg was performed at his current construction site Perstorp in Stenungsund at the 19^{th} of April in 2013. The questions asked to Söderberg were not related to the questions asked in the survey. However, his answers would together with the answers from the survey result in a wider perspective of where there might be problems within the building industry concerning reinforcement detailing of concrete structures.

11.2.6.2 Reinforcement detailing of concrete frame corners

Söderberg was shown the four recommended reinforcement configurations provided for concrete frame corners with opening moment in Annex EC2 J.2.3, see Figure 4.9 and Figure 4.10 in Section 4.4. When he was asked which of the four alternatives he has come across as a construction worker and which he prefers he answered that he has only seen the alternatives shown in Figure 4.9a and Figure 4.10a. However, he mentioned that in those configurations he have seen that the diagonal reinforcement bars are placed in a haunch at the inside of the corner.

To get more information regarding concrete frame corners see Section 4.4.

11.2.6.3 Configuration of shear reinforcement

A figure of different stirrup configurations were shown to Söderberg at the interview. The figure was derived from the configurations presented in Section 6.5.3 and therefore corresponds to the same figure presented to Hallgren in Section 11.2.4.4. He was also asked which of these configurations he prefers and if some of them are more difficult or not even possible to perform at the construction site. According to Söderberg the shape of the stirrups forming the shear reinforcement does not really affect the severity of the work at the construction site. Whenever there are problems with placement of the shear reinforcement, baskets are made by the stirrups and the longitudinal reinforcement that is placed inside the stirrups. The baskets are then lifted up by a crane to the bridge. However, this method will be more time consuming, but still favourable, since everything is gathered in one place on the ground instead of performing the work up on the bridge.

Söderberg said that almost all stirrups are bent at a factory, since there is no time to bend the bars by hand at the construction site. With regard to the rules that are set at the working place machines are always used when bars are bent today.

Söderberg was also asked which of the different configurations in Figure 11.3 he has seen as a construction worker. He described that he has never seen the shapes g), i) and j) at any construction site. However, there is no standard solution of the shape of the stirrups, since the different projects varies a lot from each other.

At the interview Söderberg said that he has seen the use of G-bars as shear reinforcement in many bridges. As an example he mentioned a bridge that was built in the year of 2012, where G-bars were provided close to the support at four or five rows. Söderberg explained that G-bars are a bit complicated to place in the formwork, since G-bars are almost always the last bars to be inserted. However, he added that it is not always complicated and that he thinks that it is difficult for the structural engineer to design the shear reinforcement in a way that to a great extent will improve and simplify the work at the construction site. Söderberg described that during his working time within the building industry he has never taken part in a project where the reinforcement has been impossible to place into the formwork. Söderberg adds that it is the order, when different reinforcement bars are placed into the structure, which is important. If the right order is identified, almost any type of reinforcement configuration is possible to place inside the formwork.

To get more information concerning configuration of stirrups see Section 5.6. A question concerning configuration of stirrups was included in the survey, see question number 4 in Section 11.3.2.4

11.2.6.4 Lapping of reinforcement

When configurations of lap splices of longitudinal reinforcement were discussed with Söderberg, he explained that his experience is that lap splices normally are not staggered, i.e. the requirements in BBK 04 are still used among structural engineers.

However, this probably depends on that the structures where he works at still have documents that are based on BBK 04 requirements since the projects started before 2011 when the transition to Eurocode took place. Söderberg described that 100 % lapping in one section occurs, but it is important to add extra transversal reinforcement for such situations. Reinforcement configurations of 50 % lapping in one section are usually used in order not to weaken the splice sections. In casting joints it is common to splice all the bars by lapping in the same section, since it is desired to end all the reinforcement in one section. In longer concrete members where 12 m bars are used the laps are placed at different sections evenly distributed along the member. If the laps are not evenly distributed along the member the reinforcement needs to be cut, which is not desired with regard to waste of the steel.

To get more information regarding lapping of longitudinal reinforcement see Section 9.4. A question regarding this was asked in the survey, see question number 3 in Section 11.3.2.3.

Lapping of transversal torsion reinforcement was also discussed with Söderberg. It is interesting to know, if it normally is possible to lap torsional links along the horizontal legs, since it in Eurocode 2 is prohibited to lap torsional links near the web surface. Söderberg said that if the splicing can be performed on the ground at the construction site it is possible to lap the links in the flange. Baskets of reinforcement are built where the longitudinal reinforcement is placed inside the links and are thereafter lifted up by cranes. However, if the construction site is by a river or at high heights this might not be possible, since the available ground to create the baskets on is insufficient.

To learn more about splicing of torsional links, see Section 6.3. This subject was also included in the survey, see question number 4 in Section 11.2.2.4.

11.2.6.5 Cooperation between contractor and structural engineers

It was of interest to ask what contractors think about the documents that structural engineers deliver to them. Söderberg does not feel that structural engineers in general perform poor detail solutions. However, he thinks that a greater cooperation would benefit both parts. This can for instance be meetings between the contractor and the structural engineer at an early stage, since errors that normally occur in the later stage of the design process can be prevented and avoided. As it is today errors in drawings are not detected before these are delivered to the construction site and this is a problem that needs to be highlighted. One reason for this is because the structural engineer is not present at the meetings with the contractor. Instead it is the client who is in contact with the structural engineer and the information from the designer is therefore forwarded to the contractor by the client. It is only at special occasions, when it is certain need for it, that the contractor and the structural engineer meet. Söderberg also described that about one time a year the drawings are not approved before casting of concrete takes place.

However, the poor communication between contractors and structural engineers is something that is known within the building industry, but the tighter time schedules and economical pressure prevent this desired increase in cooperation. Söderberg also thinks that the structural engineer should visit the construction site at least one time during each project. According to Söderberg the structural engineers are full of energy and want to learn more in the beginning of their careers, which result in that they are visiting the construction site more often. However, after some years when the structural engineers become involved in many projects at the same time, this will not be possible due to a more tight time schedule. It is not uncommon that changes are made at the construction site without changing the drawings and it can then be difficult for the structural engineer to completely understand what the changes imply. Söderberg suggests that when a structure is finished and built there should be a review of what have been changed from the original drawings so that the structural engineer, as well as others involved in the project, can learn from the mistakes made.

11.2.6.6 Changes of conditions

A question was asked to Söderberg about whether it is often the case that simplifications or alterations are needed due to changed conditions at the construction site, such as for instance that the formwork results in less space than intended or that late changes in drawings are made. Söderberg described that alterations often have to be made at the construction site because the structural engineer has not thought about the available space necessary to be able to put the reinforcement inside the formwork. Sometimes the construction workers have to remove one wall of the formwork in order to be able to place the reinforcement.

Whenever Söderberg or his colleges notice that a change at the construction site has to be made they have to communicate this to the structural engineer and get an approval. The time it takes to get an approval from the structural engineer varies between one hour and two weeks, depending on the complexity of the alteration. If the changes are difficult or complex, the structural engineer can give preliminary answers to the contractors as the times goes so that the work at the construction site can continue, instead of giving all the answers much later when all the recalculations have been made. However, it is mostly experienced structural engineers who are competent enough to do something like that. Söderberg described that he remembers one occasion at a bridge construction where C-bars had to be changed into B-bars, since the C-bars did not fit into the formwork due to changed conditions at the construction site.

Another time Söderberg received drawings of a wall that should be cast. However, according to the drawings the wall should be cast in two parts with a joint placed at about half the height of the wall, since a slab should be cast in between, see Figure 11.5a. Söderberg, who had high requirements on delivering a water proof structure detected this error and the first joint was removed and replaced by an indented joint instead, i.e. the wall was cast in one piece, see Figure 11.5b. This is one example of where the structural engineer had not included all the aspects, when the detail solution was determined. Söderberg also added that a structure that is cast in many steps will be much more time consuming to construct than if it is cast in one piece.

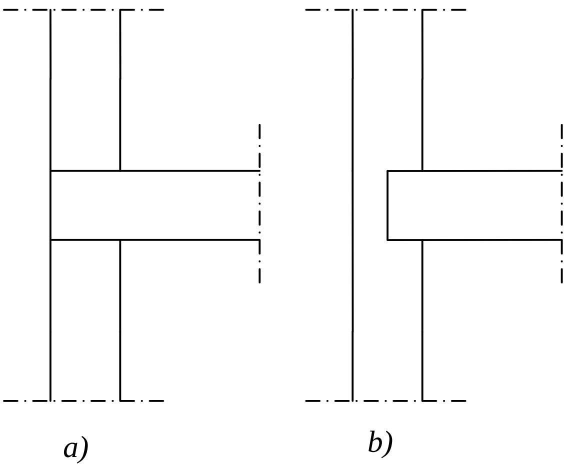


Figure 11.5 Two solutions on how a wall and a slab can be cast, a) the walls are cast in two steps, b) the wall are cast with an indented joint in one piece. Solution b) is a more effective solution and water proof.

As an example of where it can be difficult if late changes are made in a project, Söderberg told about a bridge in Värnamo that was to be built when he was working as a carpenter. He described that the supports of the bridge needed to be highly reinforced, see Figure 11.6. In this case the cross-section was full of steel and a very wet concrete had to be used in order to provide bond and anchorage. If changes were to be made here, it would not be possible to utilise the bond between the reinforcing steel and concrete.

Figure 11.6 Reinforcement at the end of a post-tension bridge in Värnamo. The pictures are taken by Söderberg (2012).

11.2.6.7 Providers of reinforcement

Söderberg experiences that experienced providers of reinforcement often can detect if something is wrong in the drawings. They can make changes of the specification of the reinforcement in order to facilitate for the steel workers who place the reinforcement. Söderberg added that he believes that 3D-modelling benefit both the structural engineers and the providers of reinforcement since it is easier to detect problems in a 3D-model. This can for instance be if two bars are going into each other or if it is too little available space to be able to place the reinforcement in the formwork.

As an additional question concerning changing conditions, see previous section, it was asked if Söderberg thinks that structural engineers provide too small margins in their reinforcement drawings. Söderberg answered that it is not often that he thinks that the margins are too small when the reinforcement should be inserted. However, when it is difficult to make room for the reinforcement into the formwork, it is often the provider of the reinforcement who has performed bends or splicing of the bars wrongly. The length of the reinforcement will then be a little too short or too long. When this type of problem occurs, it will probably be short of time until casting and new reinforcement cannot be ordered. The reinforcement then needs to be cut and spliced at the construction site.

11.2.6.8 Changing of professional role

Söderberg was asked if his relation to the structural engineer and if his priorities have changed from being a carpenter to being a foreman. He answered that when he worked as a carpenter he talked directly to the structural engineer when problems occurred, even if this is not usual for a carpenter. This is still the same after he changed his professional role. However, now he is one of the people at the construction site who handles the main contact with the structural engineer. The construction of the structure is what Söderberg prioritises the most. The cost for a building or structure made of reinforced concrete will decrease, if the production is efficient. Söderberg means that if the contractor does not produce anything, then no one will be satisfied. Another important priority is that the work at the construction site should be easy and flexible, if the conditions at the construction site change. In that way the working time can be decreased and the cost will be reduced.

11.2.6.9 Difference for the contractors between house- and bridge construction

During the interview with Söderberg the differences between design of bridges and houses were discussed. At the construction site where Söderberg works today the requirements from Eurocode 2 Part1-1 are applicable. He describes that it is easier to work with this part of the standard since it provides rules for structural members in buildings that are more easily handled than the rules for structural members in bridges or tunnels that are designed according to additional requirements from Trafikverket.

11.2.6.10 Bar dimension, spacing and length of reinforcement

When a bar diameter is presented on a drawing, it is the nominal bar dimension that is described, i.e. the average diameter between the solid inner area and the height of the ribs. According to Söderberg a ribbed bar with the dimension $\phi 16$ that is placed into the structure has in reality an outer diameter of about 18-19 mm.

According to Söderberg 6 and 12 meter long reinforcing bars are normally used in concrete structures. Reinforcement with bar dimensions of $\phi 6$, $\phi 8$, $\phi 10$ and $\phi 12$ are available in lengths of 500 m and are delivered on rolls, see Figure 11.7. In this case it is a machine that cuts the reinforcement to the desired lengths.

Figure 11.7 500 m reinforcement bar in a roll. The picture is taken by Söderberg (2012).

When a question was asked regarding what spacing of the reinforcement and what bar dimension and length that are preferable to work with, Söderberg answered that a bar dimension of $\phi 16$ normally can be carried without a crane and inserted to the structure by hand. Bars with dimensions of $\phi 20$ and $\phi 25$ are also sometimes handled by hand. However, 25 kg is the heaviest weight construction workers are allowed to carry. A consequence of this is that the standard weight of concrete bags has been changed from 50 to 25 kg. See also Appendix I for information regarding how long reinforcement bars, with different bar dimensions, can be in order to fulfil the requirement of 25 kg. Söderberg added that normally reinforcement is lifted with cranes. When buildings are constructed a lot of the concrete members today are prefabricated, which means that there will be a crane at the construction site anyway. The reinforcement in slabs in buildings often consists of fabrics that are rolled out to make the placing easier.

Söderberg described that at the construction of a bridge at Höga Kusten the construction workers handled 20 m long reinforcement bars with a bar dimension of $\phi 25$ and a weight of 49 kg each. There were two bending stations where the construction workers bent the reinforcing bars into C-shapes by hand. The total amount was about 1500 bars that were lifted up to the bridge one by one. The bars also had to be bent down by hand up on the bridge. It should be noticed that the delivery time is long for that type of large bars. However, the reinforcement will not be more expensive, since it is paid for each ton of steel. It is only the delivery cost that might increase.

11.2.7 Result from interview with David Eriksson

11.2.7.1 Introduction

The interview with David Eriksson was performed in writing where the answers were received at the 22nd of April in 2013. The questions asked to Eriksson were based on the questions asked to Söderberg.

11.2.7.2 Reinforcement detailing of concrete frame corners

The same figures of the four recommended reinforcement configurations of concrete frame corners in Eurocode 2 that were shown to Söderberg, see Figure 4.9 and Figure 4.10 in Section 4.4, were also shown to Eriksson. Eriksson described that he has not seen any of the solutions recommended in Appendix EC2 J.2.3. However, he replied that if he has to choose one of the solutions, he prefers the one in Figure 4.9a, since the other seem to be difficult to perform at the construction site. The general design for concrete frame corners is according to Eriksson to place a B-bar that is going to both the inside and outside of the corner at wall 2 from both the inside and outside of wall 1. The detail solution that Eriksson refers to can be compared to Type 1 in Figure 4.15.

To get more information regarding concrete frame corners see Section 4.4. See also survey question number 8 in Section 11.3.2.8.

11.2.7.3 Configuration of shear reinforcement

Eriksson was asked about what type of shear reinforcement that is preferred at the construction site. Eriksson described that the shear reinforcement used in general are in form of stirrups or links. In some cases it can be preferable to use G-bars in for instance thin walls. However, bent up bars are not to recommend, since it will cause unnecessary work for the persons that place the reinforcement.

Another question regarding configuration of stirrups was asked to Eriksson that was coupled with the question asked in the survey, see question number 4 in Section 11.3.2.4. The same stirrup configurations shown to Söderberg and Hallgren were also presented to Eriksson, see Figure 11.3. The shapes presented in a), b), c) and e) are according to Eriksson generally used as shear reinforcement.

To get more information concerning configuration of stirrups see Section 5.6.

11.2.7.4 Lapping of longitudinal reinforcement

Because of the transition to Eurocode 2 it is since 2011 required to place lap splices with a certain distance between the laps ends. Eriksson was asked if this is something that he has noticed at the construction site. According to Eriksson the structural engineer always delivers configurations of staggered lap splices or double lap lengths. This is further discussed in Section 9.4 and was also included in the survey, see question number 3 in Section 11.3.2.3.

11.2.7.5 Cooperation between contractor and structural engineer

During the interview the cooperation between the contractor and the structural engineer was brought up several times. Eriksson described that it is not often that structural engineers make errors in the drawings. The structural engineers usually deliver well performed detailing solutions to the contractors. It is helpful for the contractor when the structural engineers answer the questions from the contractors quickly. He also added that a good cooperation is based on that structural engineers are interested of the construction works and tries to contribute to construction that is as easy, good and cheap as possible.

11.2.7.6 Changes of conditions

Eriksson states that it is not often that modifications have to be made at the construction site due to changing conditions, such as errors in the drawings or that formwork differs slightly from the intended. However, if the structural engineer specifies the reinforcement wrongly, the contractor has to order new reinforcement or bend new bars by themselves. This is always time consuming and will cause problem and increased costs, especially if there is a tight time schedule. Sometimes simplifications that facilitate the work at the construction site can be identified by the construction workers themselves that result in changes of the design. It is for instance common to use the same length of all C-bars in a structure in order to avoid having to sort the bars at the construction site.

Another example that requires changes at the construction site is that the deliverer of the reinforcement does not cut and bend the reinforcement exactly according to the specification made by the structural engineer. In such situations the contractor can ask the structural engineer if it is allowed to make changes so that the delivered reinforcement will fit inside the structure. Otherwise new reinforcement has to be ordered. Eriksson also added that if the carpenter has performed the formwork wrong, the formwork has to be remade. This means that it will not affect the reinforcement design.

Eriksson described that situations sometimes occur where the space to place the reinforcement into the formwork is very limited. In such situations it can for instance be discussed with the structural engineer whether it is allowed to change one C-bar into two B-bars or to two B-bars plus one A-bar. However, if a bottom slab of a structure becomes larger than planned and the reinforcement already has been ordered, additional reinforcement is simply added in this area. Hence, this is not a large problem, why “flexible” reinforcement solutions are not that important.

11.2.7.7 Purpose of different reinforcement types

The reinforcing bars placed in a structure have different purposes and depending on what purpose the reinforcement has, different rules are required to be fulfilled in design and detailing. Eriksson was asked if construction workers in general are aware of what purpose the different reinforcing bars placed in a structure has and that different requirements apply to for instance shear- and transversal torsional reinforcement. Eriksson answered that the workers at the construction site normally not are aware of the purpose of the different reinforcement types. He wrote that the steel workers who place the reinforcement only follow the reinforcement drawings.

What purposes that are present for the reinforcement at hand are in fact irrelevant both for the persons that place the reinforcement and the work place management.

11.2.7.8 Difference between house- and bridge construction

Eriksson describes that there are differences between construction of houses and bridges. In the former situation the construction does not need to be performed with such carefulness as is required for bridges and tunnels. Eriksson states that this is why he thinks it is more satisfactory to work with construction of bridges instead of buildings.

11.2.7.9 Bar dimension, spacing and length of the reinforcement

The same question that was asked to Söderberg regarding what spacing of reinforcement and what bar diameter and length that is preferable to work with at the construction site was also asked to Eriksson. According to Eriksson a bar dimension of ϕ12 mm can be considered to be the maximum limit of what is possible to be bent by hand or with a manual bender. It might be possible to bend a bar with a diameter of ϕ16 mm with a manual bender, but it should be noted that it is difficult.

The spacing of reinforcing bars does not really matter. However, a spacing of 100 mm of the top reinforcement is a bit too narrow, since it will be difficult to tie the longitudinal reinforcement to the stirrups.

11.3 Survey

11.3.1 Procedure

The survey, which was the second part of the investigation, was sent to 37 structural engineers of whom a total of 19 persons chose to participate. 12 of the participants are working with design of bridges and tunnels and 7 of them are working with design of housing and industrial buildings. The companies that were involved in the survey are Reinertsen, Inhouse Tech, WSP, ELU, Chalmers, Skanska, Trafikverket, Sweco, COWI, Vattenfall and Structor. It should be noted that the survey was not intended to be statistically based, but in general to capture thoughts of experienced structural engineers. In Appendix J a copy of the Swedish original survey that was distributed to the participating structural engineers can be found.

In order to find adequate questions to ask in the survey the procedure started by an extensive literature search, see the result in Chapters 4-10. The goal was to find ambiguities in Eurocode 2 but also to find appropriate background information to the requirements in the code in order to be able to ask as convenient questions as possible. The questions included in the survey have been changed continuously during the process of determining the final questions. In order to formulate relevant and understandable questions the survey have been discussed with Morgan Johansson, structural engineer at Reinertsen AB, Björn Engström, Prof. at the Division of Structural Engineering, Chalmers University of Technology, Ebbe Rosell, structural engineer at Trafikverket and Vladimir Vantchantchin, structural engineer at Reinertsen AB. They are people within the industry with experience and great knowledge. Before distributing the survey it was also tested on a couple of people, working at Reinertsen AB, to make sure that the questions were understandable.

The survey was distributed to experienced structural engineers that in various ways are acquainted with Morgan Johansson. The questions consisted of multiple choice answers in order to facilitate the execution for the participants. A possibility to leave comments was also provided for each question in order to capture the thoughts of the designers. The participants were also encouraged to write comments if they experienced that they did not understand the question. In some of the questions it was also allowed to add additional alternatives if none of the provided were deemed suitable. It should be noted that the comments from the participants, which are shown in Section 11.2.2.1-11.2.2.9, are translated by the authors from Swedish to English.

It should be emphasised that a reinforcement detail should be designed in agreement with the codes but it is also important to consider the possibility to execute the design at the construction site. The risk for wrongly executed reinforcement configurations will increase if the designer delivers a detail solution that is correct according to Eurocode 2 but difficult to perform at the construction site. It is important that the designer remembers this and that the reinforcement sometimes might need to be adjusted due to this. It is difficult to capture this problem in a survey why this should be kept in mind when reading the results. This was also noted by the participants of the survey.

The result from the survey has been compiled in the following sections. However the full version of the result can be found in Appendix K. It was chosen to display most of the results in bar diagrams since the participants have been permitted to mark several answers. The total amount of answers in one question can therefore exceed 100 %. In the compilation of the result each answer was given one point and the total amount of answer for each alternative was thereafter divided with the total amount of participants of each question. In some exceptional cases an answer was counted as a half point. This was for instance if a participant did answer but made a comment that implied that he or she was unsure about something or if he or she considered the chosen alternative to be applicable only for certain situations. In those cases where a limiting value was asked for and the participants chose several alternatives the lowest value has been considered. The bars presented for each alternative answer in the bar diagrams have been divided in bridge- and tunnel design and housing- and industrial building design. In the cases when a person has experience from both areas it has been chosen to present these results in the group housing and industrial buildings, i.e. the results from these persons in Section 11.3.2 are shown in the light grey bars. In such cases it has been ensured that the intended persons have long experience within this area. The presentation of the result is divided into one section for each question. As an introduction to each question a description of why the specific question was deemed to be of interest and included in the survey is presented. After that the English translation of the question is given followed by answers and comments provided by the participating structural engineers.

11.3.2 Result from survey

11.3.2.1 Question 1

Eurocode 2 provides limitation of crack widths in Section EC2 7.3, see Section 10.2.1 and 10.3.1. However, there is no specific method for how to calculate crack widths of shear or torsional cracks. When Rosell and his colleges at Trafikverket examine documents delivered by structural engineers, they sometimes notice that check of shear cracks is left out. Previously Trafikverket recommended a limitation of the stress in the shear reinforcement to 250 MPa in the ultimate limit state. However, due to this recommendation Trafikverket became responsible if something would happen with the structure that involved shear cracks and the recommendation has therefore been withdrawn by Trafikverket. Despite this Rosell claims that structural engineers still use this recommendation with the intention to limit cracks widths caused by shear force. There is also a possibility that the requirements presented in Eurocode 2 Section EC2 7.3.3 or EC2 7.3.4 for limitation of crack widths are used also for shear cracks. Further, there are also methods in Betongföreningen (2010a), see Section 10.3.3. It is of interest to see what different methods structural engineers use in order to check shear cracks.

1) Eurocode 2 provides requirements regarding limitation of crack widths, where Section EC2 7.3 *Crack control* gives a method for calculation of crack widths and check of these requirements. However, there is no specific method for check of shear cracks.

1.1) How do you handle this in your daily work? Mark those alternatives that you think are consistent with how you perform the check

a) I do not check the widths of shear cracks

b) I use Trafikverket's previous rule of limiting the stress to 250 MPa in the ultimate limit state

c) I believe that the requirement of crack width due to shear cracks is fulfilled if minimum reinforcement, according to Expression EC2 (7.1), is provided

d) I believe that the required limitation of width of shear cracks is fulfilled if the requirements in Section EC2 7.3.3 are fulfilled

e) I believe that the required limitation of width of shear cracks is fulfilled if the requirements in Section EC2 7.3.4 are fulfilled

f) I use the method described in *Svenska Betongföreningens Handbok till Eurocode 2*, Betongföreningen (2010a), Section X6

g) I have my own method for this type of check (please, be kind to explain)

The answers to Question 1.1 are presented in Figure 11.8 and Table 11.2. A majority of the participants who work with bridges and tunnels chose alternative b) and a majority of the participants who are working with housing and industrial buildings chose alternative f).

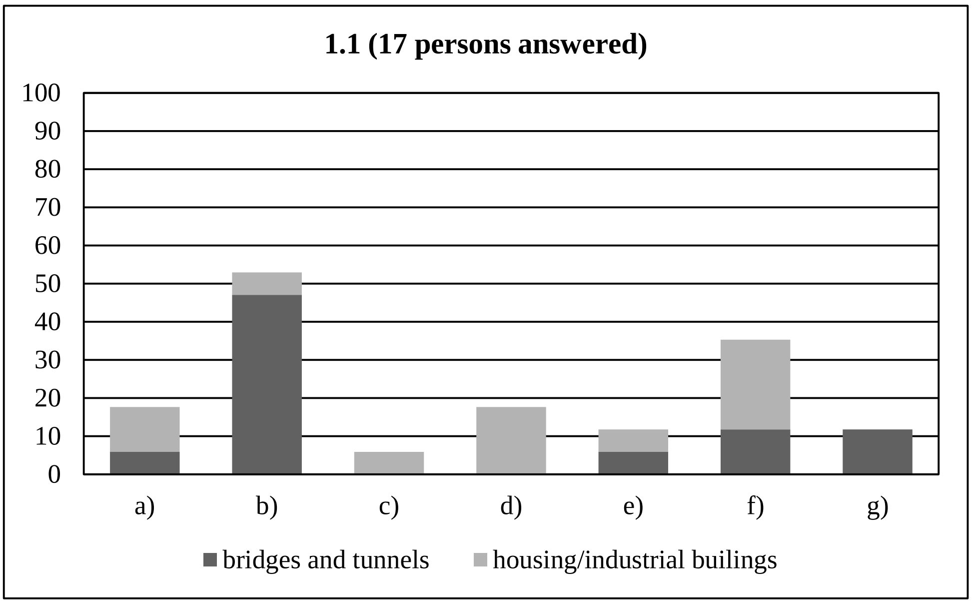


Figure 11.8 Result from Question 1.1.

Table 11.2 Comments from the participants on Question 1.1.

Answer	Comments
b)	With adjustment of 300 MPa.
	With adjustment due to exclusion of partial coefficient, 250·1.2=300 MPa.
b), g)	Use both of the above alternatives. Calculates the force in the reinforcement (transversal and longitudinal direction) with a strut and tie model for the quasi-permanent load by an assumed angle of the crack based on the direction of the principal stresses, generally 45º. Thereafter is the stress and final crack width calculated according to Eurocode 2. The check of crack width is performed on the reinforcement at the surface.
b), f)	Alternative f) is mostly used and alternatively b) is used in simplified cases with the steel stress taken for instance as 250 or 300 MPa.
e), g)	Why should there be any method in Eurocode 2? The requirement is stated, and then it is up to the structural engineer to fulfil it.

1.2) If you selected alternative c) in Question 1.1, which reinforcement stress do you use?

a) $\sigma_s = f_{yk}$
b) σ_s = reduced

If you selected alternative b) in Question 1.2, how do you estimate the stress in the reinforcement?

It was only one participant who answered on Question 1.2 and this person answered a). This is why no diagram is shown. However, the following comments were written in Question 1.2, see Table 11.3. Note that all the participants who commented on Question 1.2 work with design of bridges and tunnels. The left column shows how the participant answered on Question 1.1 and the right column presents the comments that were answered on Question 1.2.

Table 11.3 Comments from the participants on Question 1.2.

Answer in Question 1.1)	**Comments**
b)	Shear reinforcement is designed in ULS with f_{yd} = 250 MPa. Thus no further estimation is done.
	The shear reinforcement is designed for 250 MPa which also can be increased to 250·1.2 = 300 MPa, since the coefficient is acting on the load.
b), g)	As in ULS but with the yield stress taken as 250 MPa.
a), b)	When designing shear reinforcement the tensile stress in the stirrups is limited to 250 MPa (compare to 434 MPa).
f), g)	This is based on experience, which will of course sometimes give results on the unsafe side and safe side respectively, compare with for instance f).

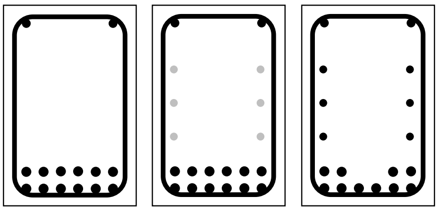
11.3.2.2 Question 2

When designing the longitudinal reinforcement in a beam with relatively high cross-section, it is sometimes unclear whether to add extra crack reinforcement along the side of the web or not and how this type of reinforcement in such case should be designed. The survey question was formulated to investigate how structural engineers consider the minimum reinforcement requirement for crack control in a beam with a relatively high cross-section. The question aimed at finding whether structural engineers think that there is sufficient crack control if the longitudinal reinforcement fulfils the amount required for crack control according to Eurocode 2 or if additional crack reinforcement must be added in the web. A third alternative was also added where some of the required longitudinal bending reinforcement corresponding to the amount needed for crack control is placed in the web.

The intension of this question was also to investigate if structural engineers think that it can be advantageous to avoid adding longitudinal reinforcement along the sides of the cross-section. The hypothesis was that it might be unnecessary to add crack reinforcement in areas where no other reinforcement, that needs to be protected from corrosion, is placed.

2) Configurations of longitudinal crack reinforcement in beams with relatively high cross-section.

2.1) How do you design this kind of crack reinforcement? The amount of main reinforcement in the beam is sufficient in order to fulfil requirements in the ultimate limit state, serviceability limit state and minimum requirements.



a) *b)* *c)*

a) No crack reinforcement
b) Main reinforcement plus <u>extra</u> crack reinforcement
c) Main reinforcement is distributed in the bottom and in the web acting as crack reinforcement

The answers to Question 2.1 are presented in Figure 11.9 and Table 11.4. It is shown that a majority of the participants place some kind of reinforcement along the sides of the web.

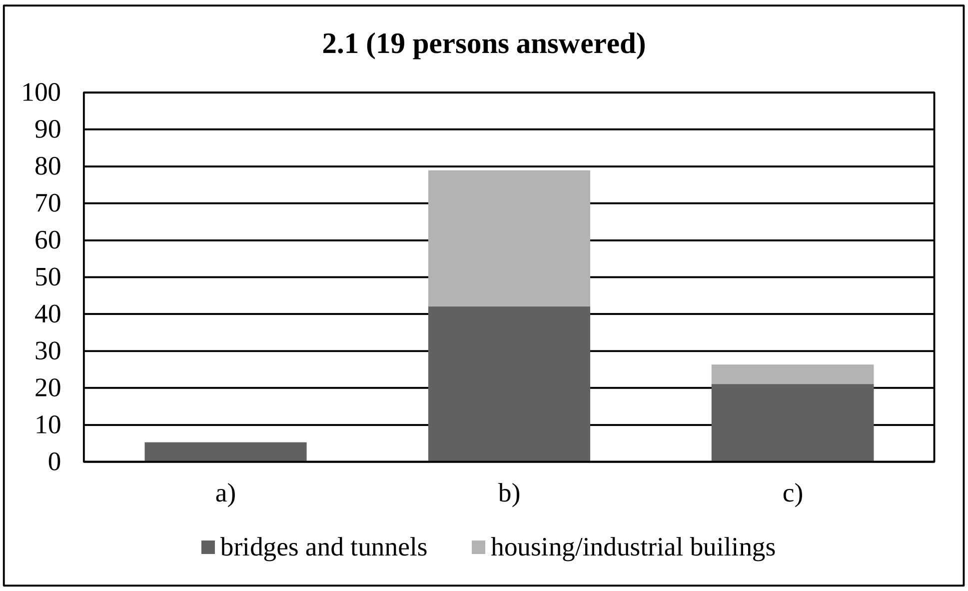
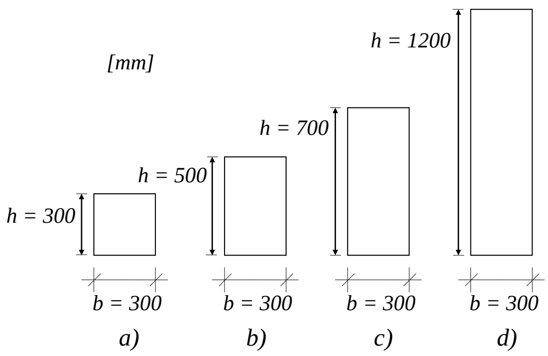


Figure 11.9 Result from Question 2.1.

Table 11.4 Comments from the participants on Question 2.1.

Answer	**Comments**
a)	These areas are often provided with longitudinal torsional reinforcement.
b)	Depends on the choice of bar dimension and spacing of the surface reinforcement. However, does not design with larger spacing than 300 mm.
	Interpret the term "crack reinforcement" as minimum reinforcement. In the areas where the main reinforcement is at least equal to the minimum reinforcement no extra reinforcement will be added.
	Most of the beams that I design are subjected to large torsional moment in ULS while it on the other hand is almost negligible in SLS. This leads to that the required longitudinal reinforcement due to torsion in ULS is also sufficient for control of crack widths. The check of reinforcement amount is performed according to TRVK Bro D1.4.1.1, where the reinforcement is designed in all surfaces.
b), c)	Alternative c) is also accepted. However, the actual placement of the bars needs to be taken into account.

2.2) If you selected alternative b) or c) in Question 2.1, how do you design the longitudinal crack reinforcement? For what approximate *h* do you think reinforcement at the surfaces of the web is required?



The answers to Question 2.2 are presented in Figure 11.10 and Table 11.5. A majority of the participants chose alternative b), which means that if the height of the cross-section is equal to or larger than 500 mm then crack reinforcement is added at the sides of the web.

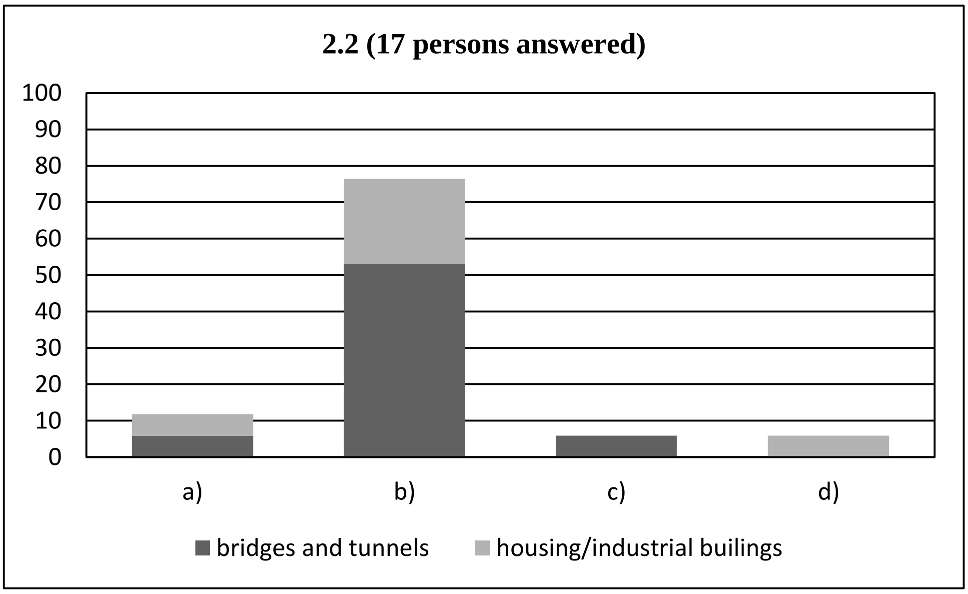


Figure 11.10 Result from Question 2.2

Table 11.5 Comments from the participants on Question 2.2.

Answer	Comments
a), b)	b) is a marginal case. Not more sparse than 300 mm. In some structures 200 mm spacing will be required.
b)	Depends on the design, otherwise h = 700 mm can become necessary.
	Minimum reinforcement according to Trafikverket's requirements in TRVK Bro 11, D.1.4.1.1 (all surfaces) and according to SS-EN 1992-1-1, 7.3.2. This concerns surfaces where tensile forces can occur.
	Depends on the bar dimension, ϕ, and spacing, s, of the surface reinforcement but never more sparse than 300 mm.
b), d)	Surface reinforcement is provided if the distance between the top and bottom reinforcement exceeds 200-250 mm.
	Trafikverket sets requirements concerning surface reinforcement which regulates spacing and reinforcement amount.
	$h \rightarrow$ min 350, max 500mm.
d)	1000 mm (note that the answer in question 2.3 is ϕ = 12 mm and s = 100-150 mm).
No answer	Is controlled by $A_{s,min}$ (TRVKBro11 D.1.4.1 is valid for bridges).
	Surface reinforcement s200 or s300 according to Trafikverket's requirements.
	500 mm and higher (note that the answer in Question 2.3 is s < 300 and A_s > 400).

2.3) Fill in those parameters that you believe are relevant for design of longitudinal crack reinforcement

bar dimension:	spacing:	reinforcement amount:
ϕ = ..……..mm	s = ..……..mm	A_s = ..……..mm^2/m

The answers to Question 2.3 are presented in Table 11.6. It can be concluded that some of the participants follow the requirement from Trafikverket regarding a reinforcement amount of at least 400 mm^2/m. Other answered with a bar diameter between ϕ10 and ϕ12 together with a spacing, s, between 150 to 250 mm. Two of the participants wrote that they did not understand the question.

Table 11.6 Type of bar diameter, spacing and reinforcement amount that is used for a beam with relatively high cross-section.

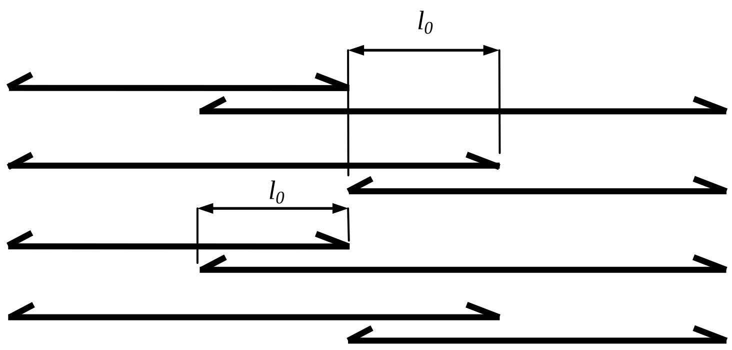
Bar diameter ϕ **[mm]**	**Spacing** s **[mm]**	**Reinforcement amount** A_s **[mm^2/m]**	**Comment**
X[1]	X	X	Consider all the parameters to be relevant for crack reinforcement.
12	265	427	If the conditions are that it is a bridge with a cross-sectional width of 300 mm and concrete C35/45, Trafikverket's requirements are governing.
X	X	X	
12	100 - 150		
		X	
X	X	X	
	< 300	> 400	
12	150		
10	200		Depends on the concrete strength and the restraint conditions. Weak bars with small spacing are preferred. At least ϕ10 s200.
12	250 alt. 200	450 alt. 560	Also consider the minimum reinforcement requirement corresponding to 0.05% and 0.08% of the cross sectional area, according to TRVK Bro D1.4.1.1, to be relevant for structures in outdoor environment.
	300		
10	200	785 / 2 = 392.5	
X	X	X	All the parameters are important.
X	X	X	All the parameters are equally important.
			Follows the requirements from Trafikverket.
			Minimum requirements according to the standard.
[1]The boxes marked with x means that the person believes the parameter is important when the longitudinal crack reinforcement should be determined.			

11.3.2.3 Question 3

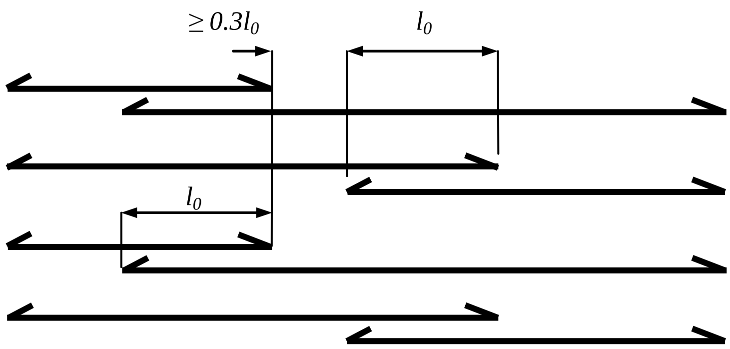
Recommendations concerning lap splicing of reinforcement are given in Section EC2 8.7.2. However, difficulties in the interpretation of Figure EC2 8.7, see Figure 9.17, in combination with Paragraph EC2 8.7.2(4) have resulted in dispersed opinions whether it is allowed to lap 100 % of the longitudinal reinforcement in one section or not, see Section 9.4.3. The survey included a question concerning whether different configurations of lap splices are allowed or not in order to see how the participants interpret the rules in Section EC2 8.7.

3) Recommendations concerning lap splicing of reinforcement are given in Section EC2 8.7.2. In this case it is one layer of main tensile reinforcement in a slab that is to be lap spliced with a bar diameter of $\phi 16$. The lap length, l_0, is considered to be sufficient.

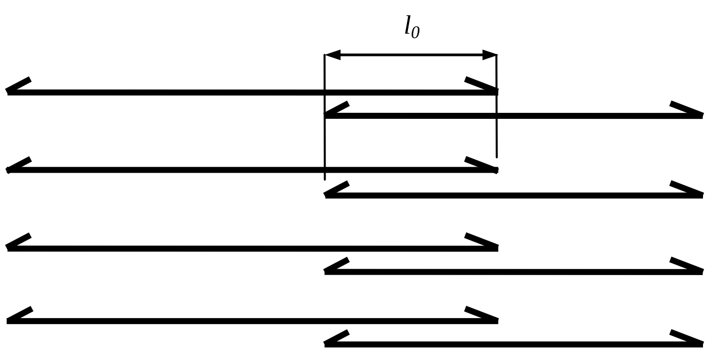
Which of the following alternatives do you use? It is allowed to choose more than one alternative.



a)



b)



c)

The answers to Question 3 are presented in Figure 11.11 and Table 11.7. As expected all of the participants do follow the requirement in Eurocode 2, see alternative b). However, some that are working with bridges and tunnels also chose alternative c).

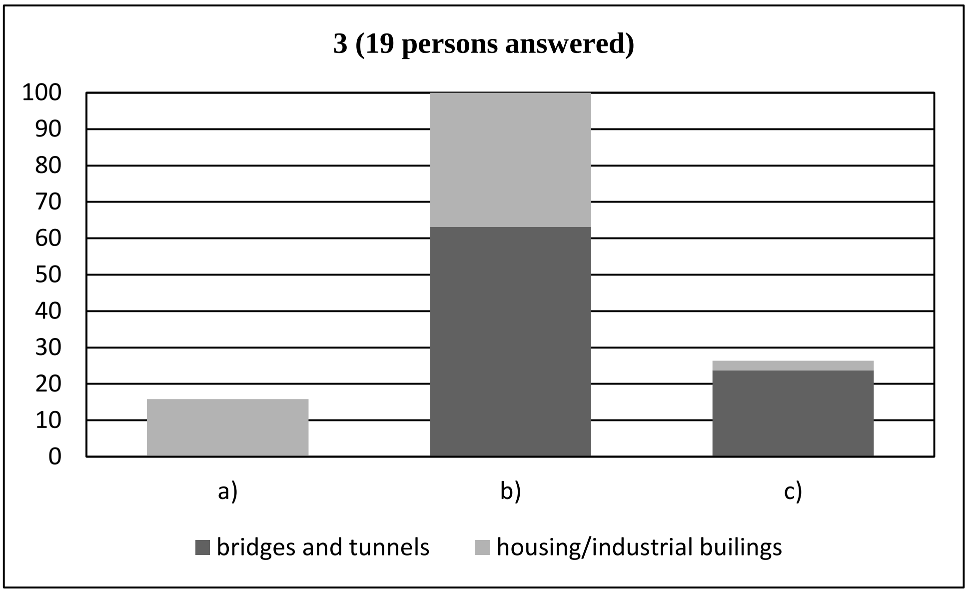


Figure 11.11 Result from Question 3.

Table 11.7 Comments from the participants on Question 3.

Answer	**Comments**
b)	Is used by tradition since it previously was not allowed to splice all the reinforcement in one section (according to former Vägverket/Banverket). Personally I believe that c) should be approved to be use under the condition that α_6 is set to 1.5 when calculating the lap length l_0 according to SS-EN 1992-1-1, 8.7.3. With the exception of longitudinal reinforcement in a bridge slab in a composite bridge, see TRVK Bro 11, D.1.4.1.5, where maximum half of the longitudinal reinforcement is allowed to be spliced in one section.
	Alternative c) is used when it is not possible to stagger the splices, $l_0 = 1.5l_{bd}$. In alternative b) it is 50 % spliced, which results in $l_0 = 1.4l_{bd.}$
	Eurocode requirement.
	Normally uses alternative b).
a), b)	Alternative b) should be used according to Eurocode 2. However, a) was allowed previously and is still occurring. Adaption is made for each situation.
	Sees alternative b) as a recommendation in Eurocode 2.
	When c) is used the utilisation needs to be checked.
b), c)	Alternative b) for bending reinforcement and alternative c) for shear, torsional and secondary reinforcement.
	Use both of the alternatives. As the standards is now alternative c) can be used for $\phi \leq 16$ mm if the percentage of splices in one section is 100 % and the reduced distance between bars are taken into account. $\phi \geq 20$ mm cannot be spliced 100 % in one section due to practical reasons (transversal reinforcement should consist of stirrups or links that are anchored in the cross-section). Does this concern every bar of the main reinforcement?
c)	c) if Section EC2 8.7.4.1 is fulfilled.

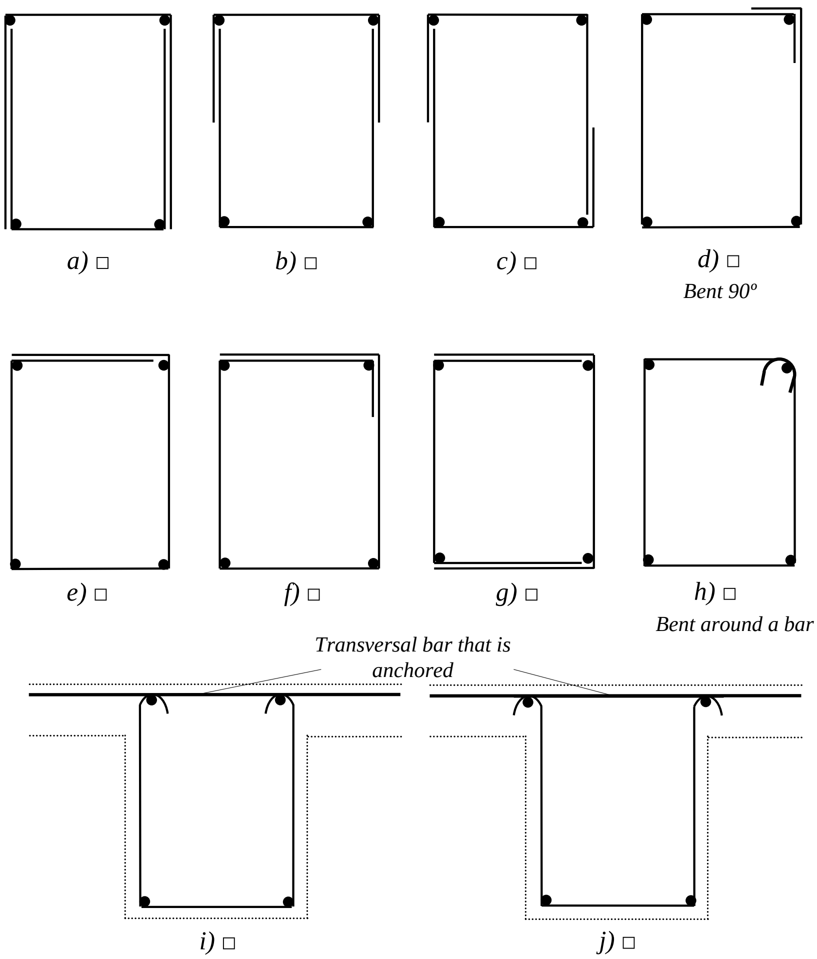
11.3.2.4 Question 4

The design of transversal torsional reinforcement is described in Section EC2 9.2.3 where recommended and not recommended shapes of torsional links are illustrated, see also Section 6.3.3. In the Swedish handbook, BBK 04, there is no distinction between configurations of shear and torsional reinforcement. However, according to Paragraph EC2 9.2.2(3), concerning detailing of shear reinforcement, it is stated that a lap joint near the surface of the web is permitted provided that the reinforcement link is not required to resist torsion. It is of interest to see if structural engineers have noticed this change in the new standard, since the requirement is stated in a paragraph not relevant for torsional reinforcement. It can be noted that shapes f), h) and i) are all recommended as torsional reinforcement in Eurocode 2 and shape d) is presented as "not recommended" in the standard.

4) Design of torsional reinforcement in a beam.

4.1) Which of the different shapes of stirrups do you consider to be allowed as torsional reinforcement in a beam? The stirrups are designed with sufficient anchorage length and a bar diameter between $\phi 8$ and $\phi 16$. It is allowed to choose more than one alternative.

From the alternatives that you have chosen, mark the one that you use mostly.



The answers to Question 4.1 are presented in Figure 11.12 and Table 11.8. The answers that were received from the participants were spread widely. It is interesting to see that even those alternatives that are not recommended in Eurocode 2 were chosen, i.e. alternatives a) to e), g) and j).

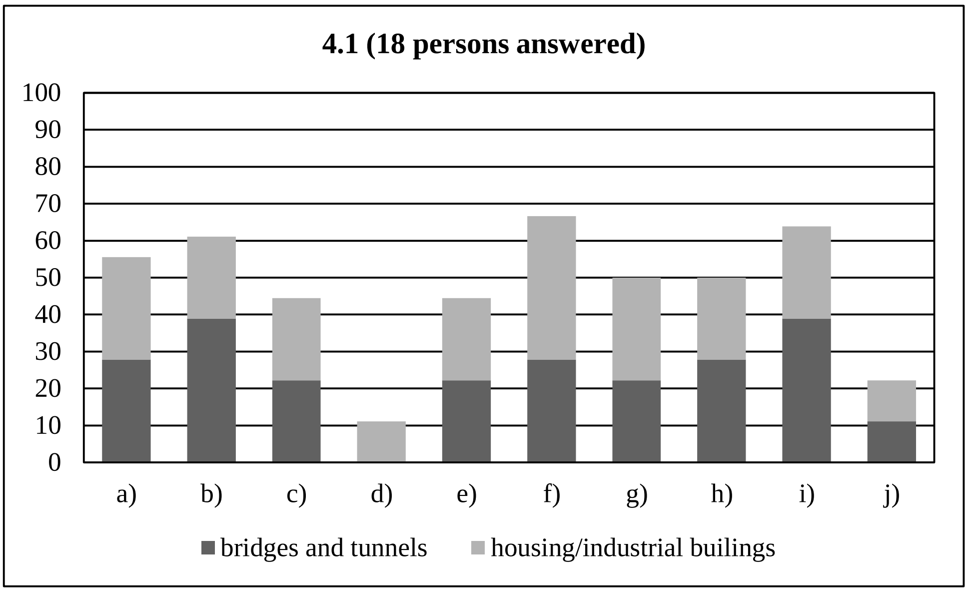
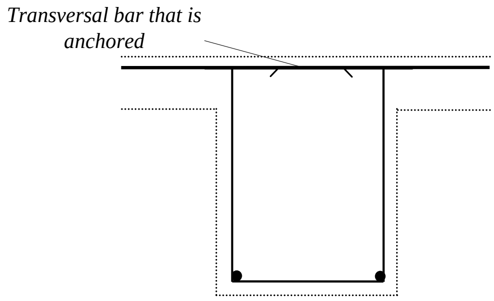
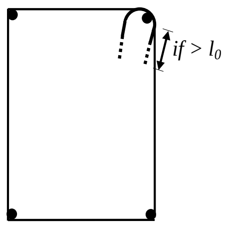


Figure 11.12 Result from Question 4.1

Table 11.8 Comments from the participants on Question 4.1.

Answer	Comments
a), b)	
a) - h)	The participant has added *"if > l_0"* at all the lap lengths in a) - g). This length is also added to the length of the bend in h), see below.  *h)* *Bent around a bar*
a), f), g), h)	In alternative h) the length of the bend has been increased so that it has the length l_0 , see figure above.
h), i)	Previously it was allowed to use for instance b) as both torsional and shear reinforcement. Why is this not allowed as torsional reinforcement any longer in Eurocode?
e) – j)	Use mostly i), even if the answer is e) - j), but wants to use a) - c).

In the question regarding what shape the designer uses most frequently the answers came out as follows, see Table 11.9. Note that only 12 persons answered this question. Alternative b) is not allowed as torsional reinforcement according to Eurocode 2 and it is unfortunate that this solution was chosen.

Table 11.9 Shape of stirrup that the designer uses most frequently.

Shape	Number of persons that chose the solution
a)	1
b)	8
f)	1
h)	1
i)	2

4.2) Does the bar dimension affect your choice of stirrup configuration?

a) Yes
b) No

The answers to Question 4.2 are presented in Figure 11.13. More who are working with design of houses and industrial buildings do consider the bar dimension when choosing stirrup configuration than the participants working with design of bridges and tunnels.

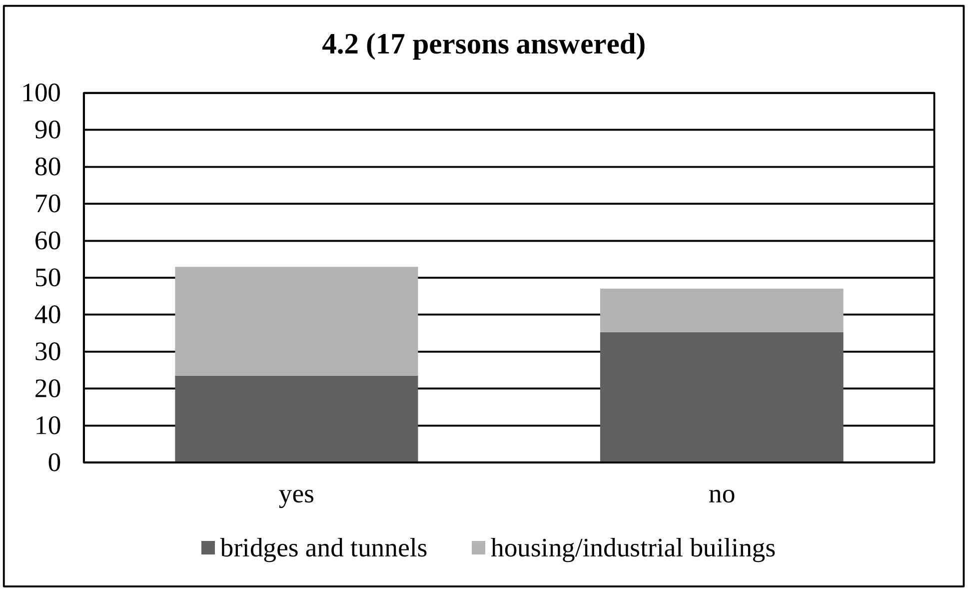


Figure 11.13 Result from Question 4.2

4.3) If you answered yes in Question 4.2), which of the chosen alternative in Question 4.1) do you use for a bar dimension of $\phi 16$?

The answers to Question 4.3 are presented in Figure 11.14. Alternative b) is the most commonly used configuration of torsional reinforcement with a bar dimension $\phi 16$ among participants working with bridges and tunnels. The answers from the participants who are working with design of houses and industrial buildings are spread between alternatives a) - c), e) and f). It is positive that alternative d) is not chosen in this case, since it is not allowed to be used according to Eurocode 2

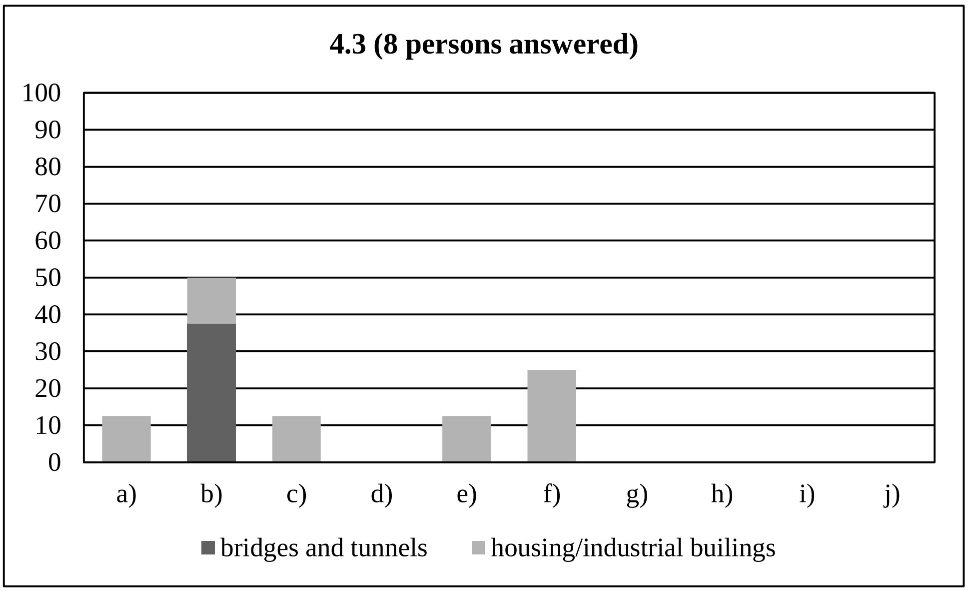


Figure 11.14 Result from Question 4.3.

The answers from Question 4.3 are compared to the answers from Question 4.1 in Table 11.10. Comments from the participants on Question 4.3 are shown in Table 11.10. It can be noted that solutions consisting of hooks, see alternative h)-j) in Question 4.1), are excluded when the bar dimension becomes larger. Alternative b), that is not allowed to be used according to Eurocode 2, is not excluded which is worrying.

Table 11.10 Shapes of stirrups used for a bar diameter of $\phi 16$.

Uses for a bar diameter of $\phi 16$[1]	**Answer in question 4.1**
b)	a), b), c), e), f), g), i)
b)	b), c)
b)	a), b), c), e), f), g), i)
f)	f), h), i), j)
b), e)	a), b) c), e), f), g), i)
f)	f), h), i)
a)	a), f), g), i)
c)	a), b), c), d), e), f), g)
[1]Each row correspond to one participant	

Table 11.11 Comments from the participants on Question 4.3.

Answer	**Comments**
b), e)	Use mostly b) and e). However, it depends on how the reinforcement practically can be placed into the structure at construction site

11.3.2.5 Question 5

G-bars can be used as shear reinforcement in beams and slabs. It is of interest to investigate if this type of configuration is used, since it is not described as shear reinforcement in Eurocode 2, see Section 5.6.1. Trafikverket has set up rules for how G-bars should be placed in a structure. It is therefore strange that it is considered as a risk that these are placed incorrectly. This is why a question regarding their direction has been included in the survey.

5) Z-shaped stirrups called G-bars are illustrated in Figure 11.15 and can be used as shear reinforcement.

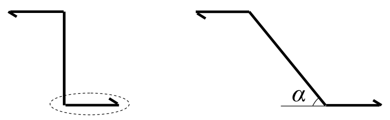


Figure 11.15 G-bars are used as shear reinforcement.

5.1) Are G-bars something that you use as shear reinforcement (not to mix up with bent-up longitudinal bars)?

The answers to question 5.1 are presented in Figure 11.16 and Table 11.12. The participants who are working with bridges and tunnels use G-bars as shear reinforcement.

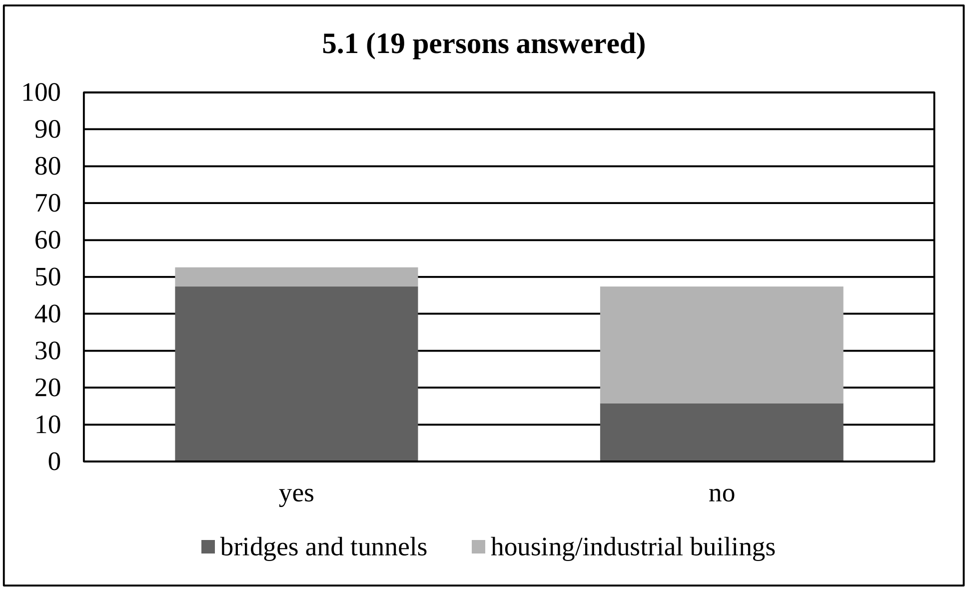


Figure 11.16 Result from Question 5.1.

Table 11.12 Comments from the participants on Question 5.1.

Answer	Comments
Yes	Yes for slabs but not for beams.
	Use them rarely.
No	Use them rarely.
	Only in very special cases.

5.2) Is it the anchorage length, l_{bd}, or the lap length, l_0, that determines the length of the horizontal leg in the G-bar (see the circle in the figure above)?

a) Anchorage length, l_{bd}
b) Lap length, l_0

The answers to Question 5.2 are presented in Figure 11.17. The participants who are working within the area of housing and industrial buildings chose alternative a), i.e. that the anchorage length determines the length of the horizontal leg. However, the persons who are working with bridges and tunnels mostly use the lap length, l_0.

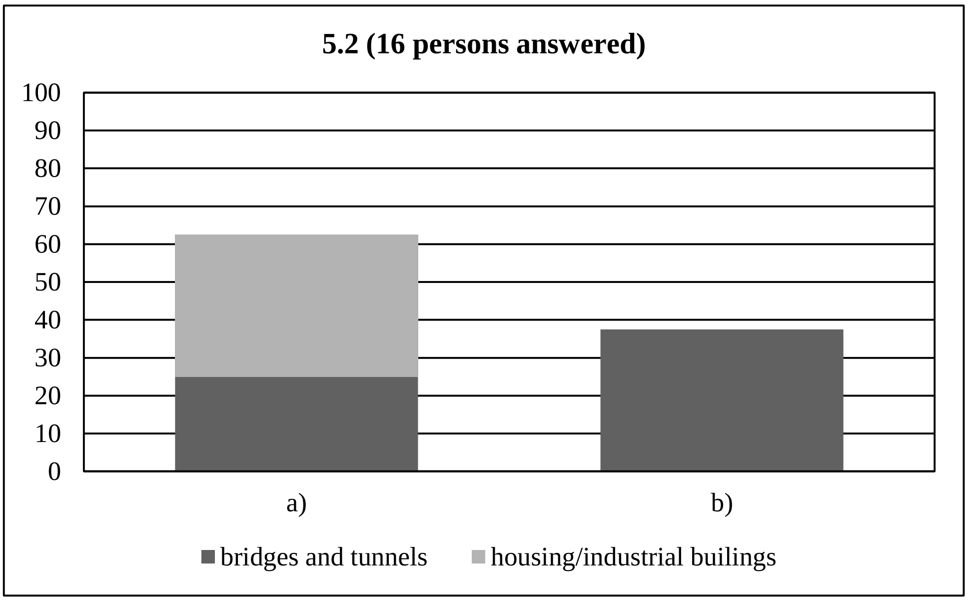


Figure 11.17   Result from Question 5.2.

5.3) Are G-bars used as shear reinforcement without additional enclosed stirrups?

a) Yes
b) No

The answers to Question 5.3 are presented in Figure 11.18 and Table 11.13. It should be noted that it is only the participants who answered yes to Question 5.1) who are presented in Figure 11.18. This is because on that it can be assumed that only they have sufficient experience to answer this question. The answer implies that it according to the participants is allowed to use G-bars as shear reinforcement without additional enclosing links.

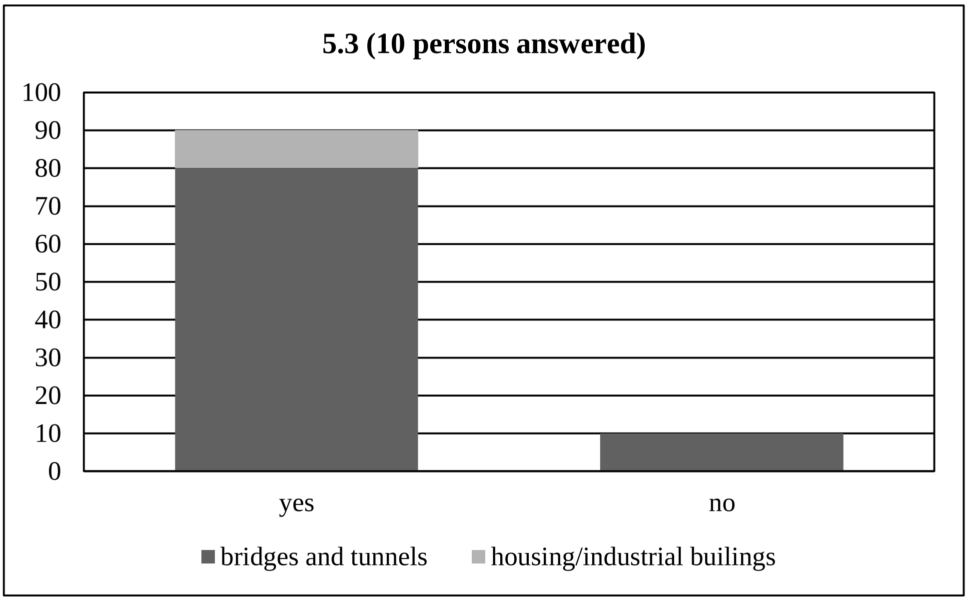
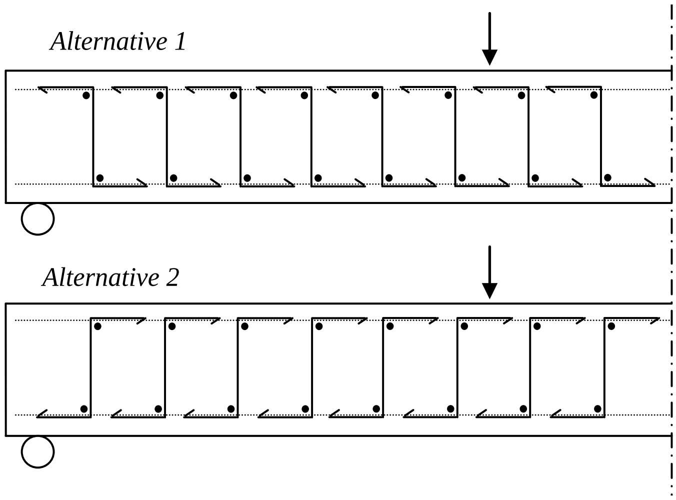


Figure 11.18   Result from Question 5.3.

Table 11.13 Comments from the participants on Question 5.3.

Answer	Comments
Yes	Yes for slabs but not for beams.
No	Yes for slabs but not for beams.

5.4) Which of the alternatives for positioning design of G-bars do you prefer?



a) Alternative 1
b) Alternative 2
c) It does not matter

The answers to Question 5.4 are presented in Figure 11.19 and Table 11.14. Several of the participants who are working with design of bridges and tunnels chose alternative a), i.e. configuration 1, which implies that they follow Trafikverket's recommendation.

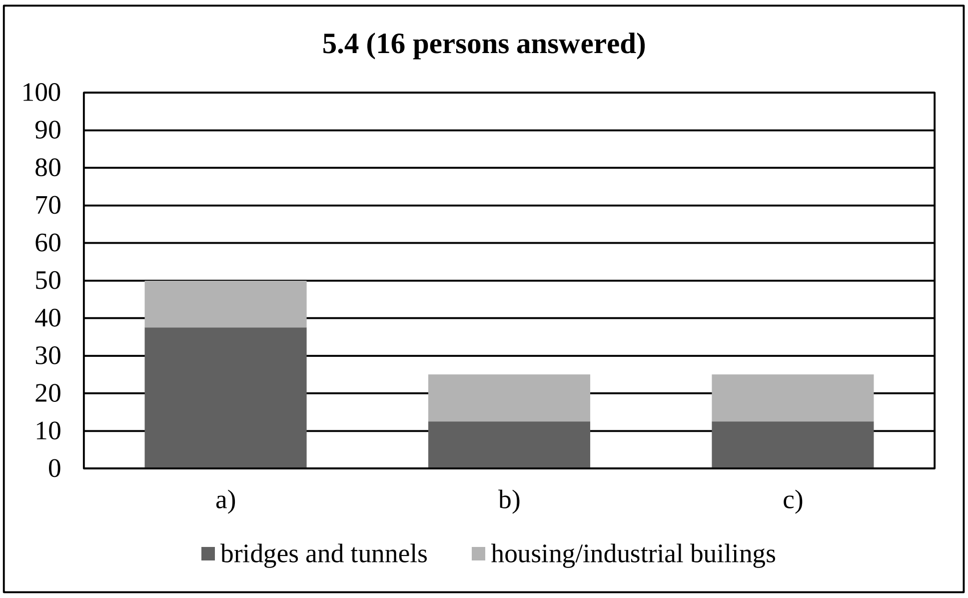
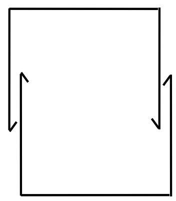
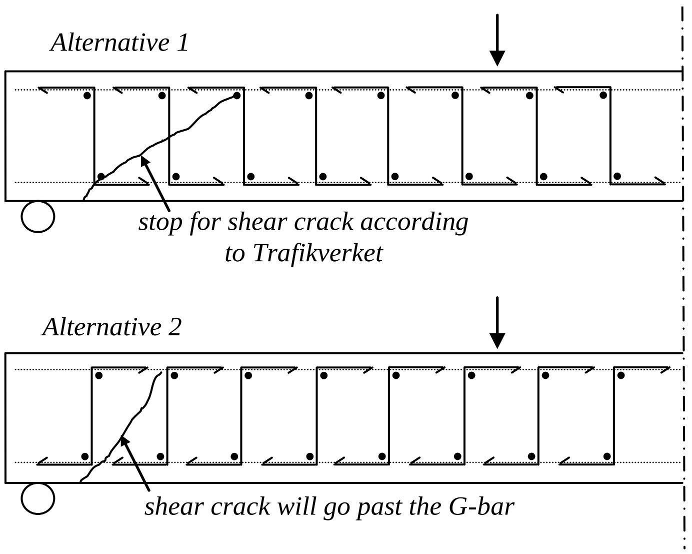


Figure 11.19 Result from Question 5.4.

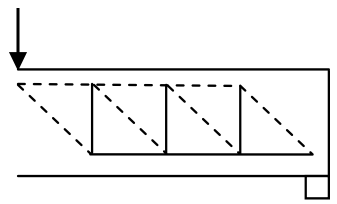
Table 11.14 Comments from the participants on Question 5.4.

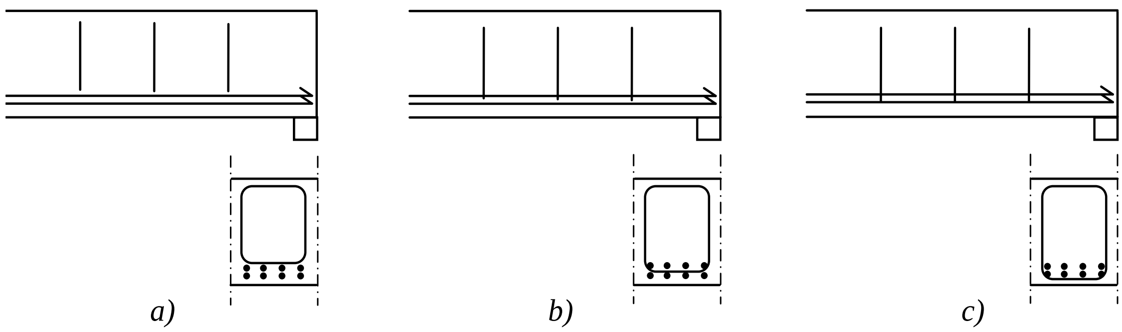
Answer	**Comments**
a)	Engineering judgement: The crack is captured within the bar.
	Otherwise, the node in the strut and tie model will not work adequately?
b)	Do not use G-bars. However, they may be useful if C-bars with sufficient lap length would not fit within the cross-section, see figure below. 
	Choose to place the bottom horizontal leg in the direction to where the smallest load is acting, i.e. towards the end of the beam or the slab.
c)	I have been in discussion with Trafikverket concerning this. They state that Alternative 1 is correct, since then the shear crack is captured within the bend of the bar, see figure below. In Alternative 2 the shear crack may miss the stirrup. However, as I consider it, it does not really matter if the comparison is made with "normal" stirrups, since then the crack will miss the bend anyway. It is the vertical reinforcement that should keep the shear cracks together and as long as the anchorage length is sufficient, calculated from full steel stress, then both alternatives should be ok. However, I always use Alternative 1 whenever it is relevant. 
No answer	It is often inappropriate to use G-bars in bridges where the cross-sectional height varies due to cambering of the bridge deck. The G-bars need to be adjusted along the member with the variation of the cross-sectional height.

11.3.2.6 Question 6

It is often limited space in cross-sections when the shear reinforcement in a beam or a slab is to be designed, i.e. it can be difficult to fit the reinforcement within the concrete cross-section. This is especially the case in slabs where the concrete cover and the main and transverse reinforcement do not leave much room for the shear reinforcement. It is important that the shear reinforcement encloses the main reinforcement in order to be able to lift the shear force and enable force transfer along the whole structure, see Section 5.6.1. It has been observed by for instance Ebbe Rosell at Trafikverket that many structural engineers tend to forget this basic theory when the time is short and available cross-sectional height is limited. According to Rosell it occurs that structural engineers for instance put shear reinforcement between two layers of main bending reinforcement instead of enclosing all of the layers. It was of interest to investigate if the participating structural engineers consider this basic theory in design in order to evaluate if the requirements in Eurocode 2 need to be further explained or specified.

6) Which of the following detail solutions of stirrups are suitable? It is allowed to choose more than one alternative.





The answers to Question 6 are presented in Figure 11.20 and Table 11.15. All of the participants perform their design by enclosing all of the longitudinal bending reinforcement with stirrups. However, three structural engineers have also chosen alternative b) but two of them have added a comment to their choice.

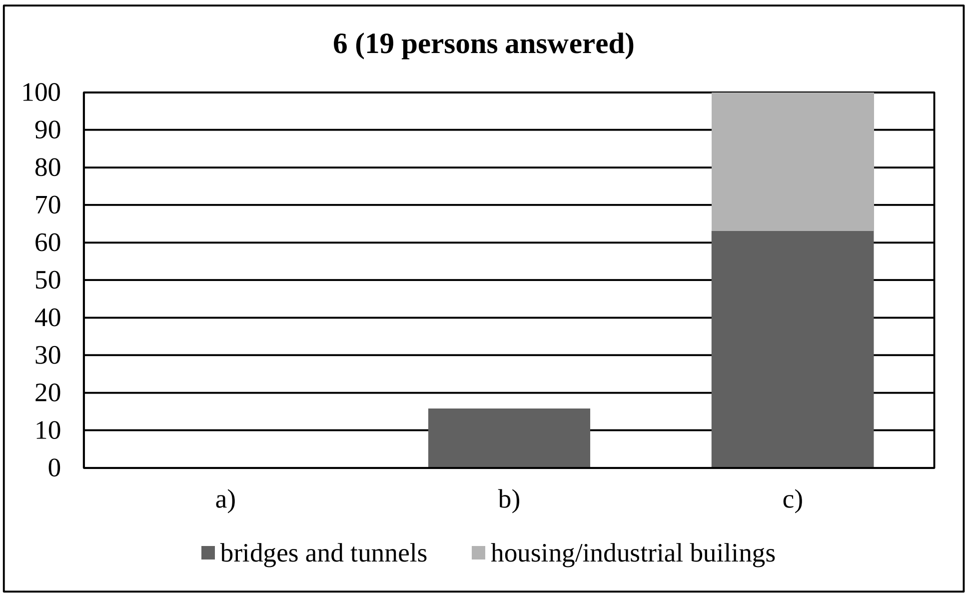


Figure 11.20 Result from Question 6.

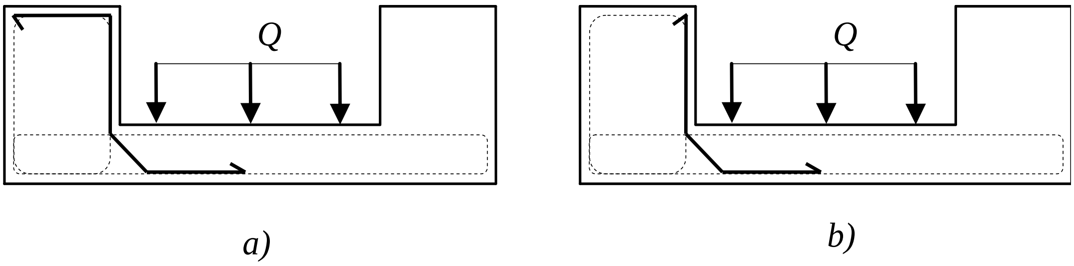
Table 11.15 Comments from the participants on Question 6.

Answer	**Comments**
b), c)	Alternative b) is only ok to use for design of transversal beams (secondary beams) in superstructures and over bridge support since it gives a more practical configuration of the reinforcement. However, in this case layer 1 is not used in the verification of the capacity of the section in ULS, but only in check of crack widths in SLS.
	Have never seen or applied alternative b) but a spontaneous reaction is that it should work.
c)	The stirrups will also act as surface reinforcement at the bottom edge.
	I would like to have reinforcement in all corners of the stirrup.
	The nodes in the strut and tie model will not be adequate in alternative a) and b) .

11.3.2.7 Question 7

At indirect supports it is important to provide additional reinforcement at the connection between the adjoining members in order to transfer the load caused by the supported member to the top of the supporting member, see Section 5.8.1. Before this rule was included in a clear way in the standards some previous damages and failures of structures depended on lack of appropriate suspension reinforcement, see Section 11.2.3.6. In these cases full utilisation of the cross-sections was not achieved. The reason why a question concerning suspension reinforcement was asked in the survey was to see if it from Eurocode 2 is clear how this type of reinforcement should be designed. It can be noticed that in alternative b) the full height of the cross section of the supporting main beam cannot be utilised, see Section 5.8.3.

7) Which of the following alternatives do you consider to be a possible reinforcement solution for a main beam in through bridge? It is the solid line that is to be considered. The dotted lines show the rest of the reinforcement that is considered to be sufficient.



The answers to Question 7 are presented in Figure 11.21 and Table 11.16. It was noticed by the authors that this question was difficult to interpret by the participants why it is problematic to comment on the result. However, a majority selected alternative a).

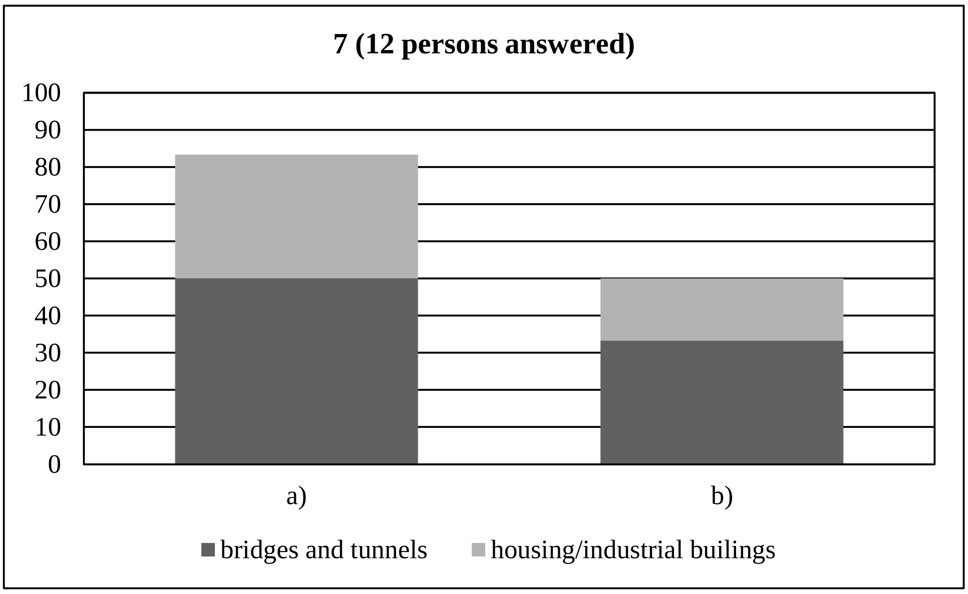
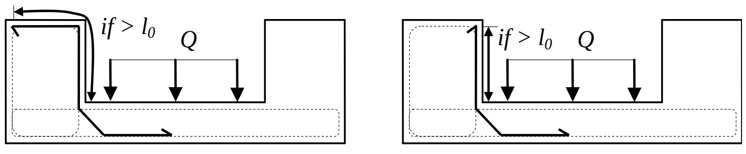
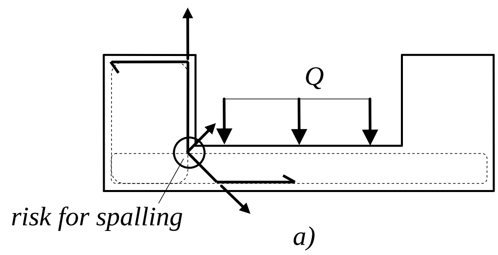
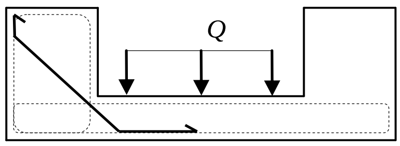
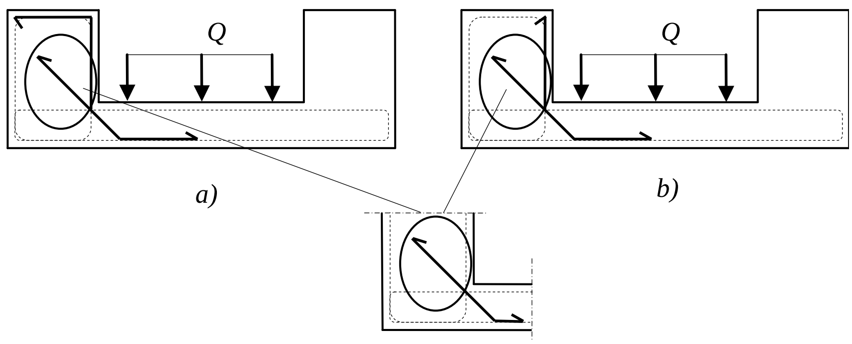


Figure 11.21 Result from Question 7.

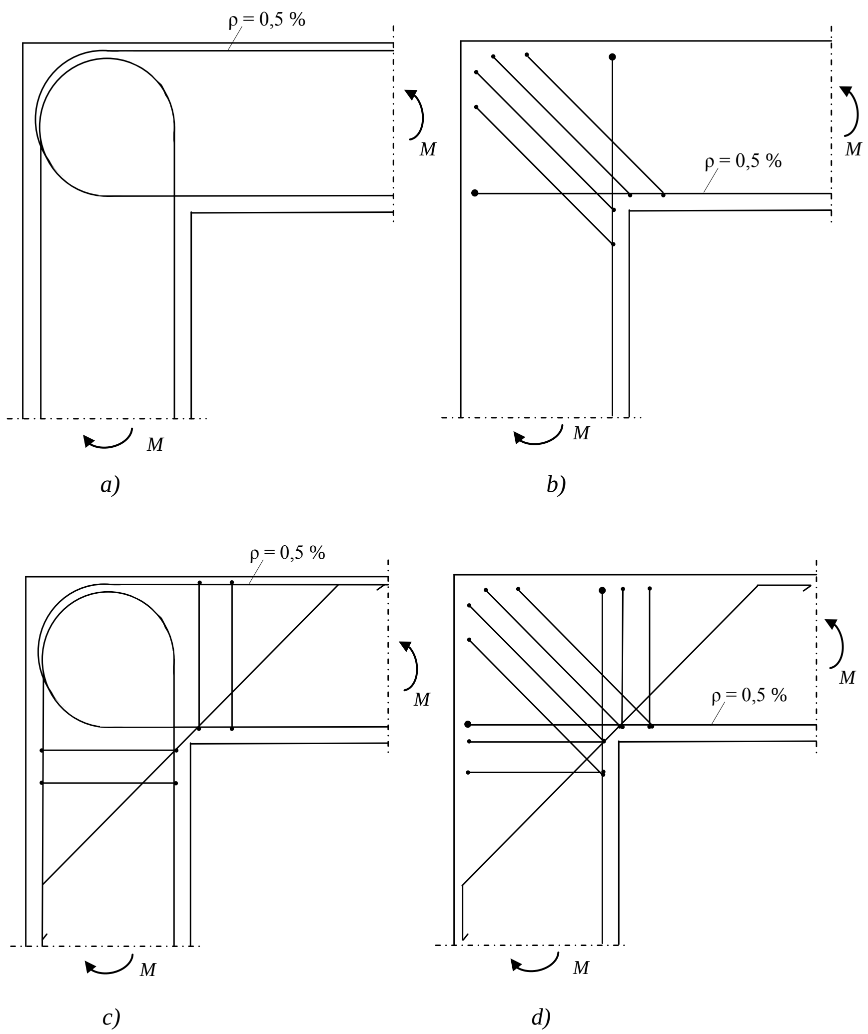
Table 11.16 Comments from the participants on Question 7.

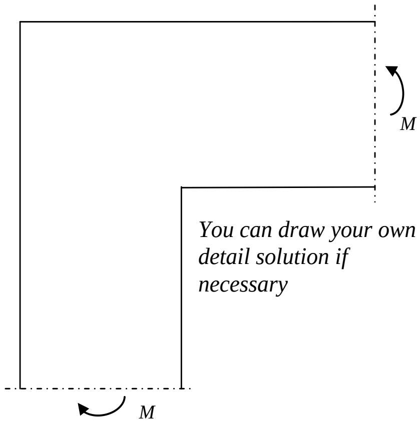
Answer	**Comments**
a), b)	If the vertical leg in b) is at least one lap length, l_0, then the solution should work.
	Does not matter.
	If the lengths shown in the figure below are larger than the anchorage length, l_{bd}.  a) b)
b)	Under the condition that the anchorage length is sufficient.
None of the solutions are correct	None of these.
	Am of the opinion that both solutions are insufficient since they will cause spalling of the concrete corner between the main beam and the transversal slab, see figure below.  a)
	None, do not design the bend at the first corner, see figure below.  b)
	None of the solutions seem to be adequate, diagonal reinforcement in opening corners are missing, see figure below. 
	One of the participant wrote that it was difficult to understand the question and one of the participant wrote that he/she did not know.

11.3.2.8 Question 8

Concrete frame corners subjected to opening moment are described in Section 4.4 where, among other things, recommendations in Eurocode 2 concerning the recommended reinforcement amount for different reinforcement configurations is discussed. There is reason to believe that the recommended limit of 2 % is too high and will in some cases result in a lower moment capacity than expected if the recommended configurations in Eurocode 2 are used, see Section 4.4. A question concerning this was included in the survey in order to see if the designers are of the same opinion and to see what configurations that are commonly used for a for a reinforcement amount of 0.5%. According to Eurocode 2 all configurations presented in the question can be used for this reinforcement amount but it is recommended to use alternative a) and b). It can also be noted that alternative c) also is recommended in the Swedish handbook BBK 04.

8) There is a need of 0.5 % bending reinforcement in the concrete frame corner below. It is subjected to an opening moment. Which detail solution do you prefer?





e)

The answers to Question 8 are presented in Figure 11.22 and Table 11.17. A majority of the participants selected alternative c). Alternative a) was only chosen by the persons working with design of bridges and tunnels. Alternative b) and d) are not that common.

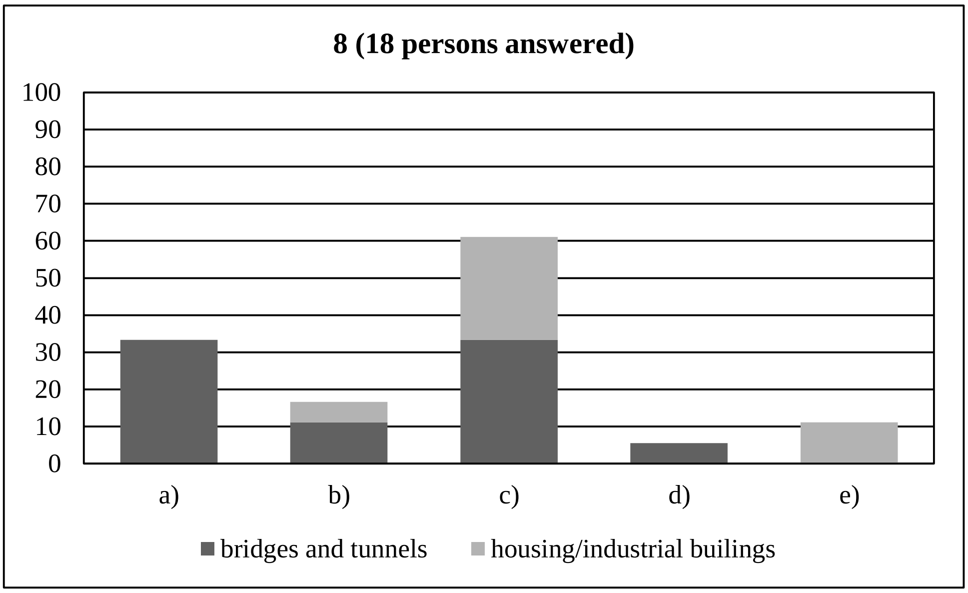
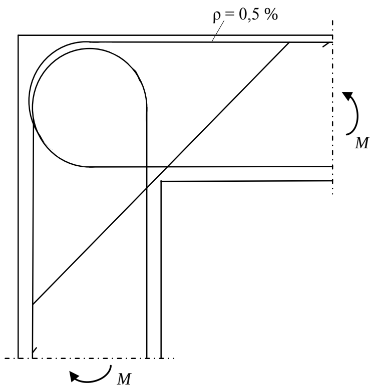
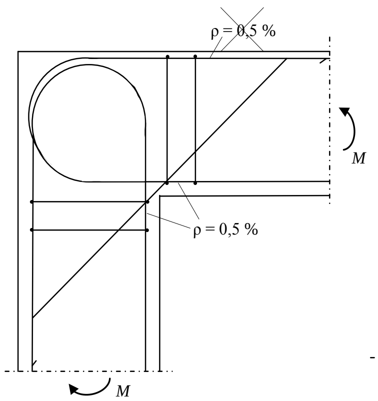


Figure 11.22 Result from Question 8.

Table 11.17 Comments from the participants on Question 8.

<table>
<tr><th>Answer</th><th>Comments</th></tr>
<tr><td>a)</td><td>Complements the solution with an E-bar, even if the standard does not require this</td></tr>
<tr><td rowspan="2">a), b)</td><td>A_s / bh < 2 %.</td></tr>
<tr><td>Alternative a) is combined with an E-bar, corresponding to alternative c). Alternative b) is OK as it is.</td></tr>
<tr><td rowspan="4">c)</td><td>Alternative c) if there is a need for shear reinforcement. If there is no need for shear reinforcement then alternative a) can be combined with an E-bar, see figure below.

</td></tr>
<tr><td>Alternative c), but without shear reinforcement, see figure above.</td></tr>
<tr><td>Would most likely use alternative c), otherwise alternative d).</td></tr>
<tr><td>In one of the answers in the survey the reinforcement amount has been moved as can be seen in Figure below.

</td></tr>
</table>

11.3.2.9 Other comments from the participants

A part of Eurocode 2 that is difficult to interpret is especially Section EC2 8.7, regarding lap splicing of longitudinal reinforcement. This is unfortunate since Ralejs Tepfers at Chalmers University of Technology came up with a reasonable model for anchorage failure.

12 Evaluation and analysis

12.1 General remarks

In this chapter the results from the survey and interviews presented in Chapter 11 are compared to the problem areas highlighted in Chapters 4-10 in order to reach some final conclusions. However, all topics that are presented in Chapters 4-10 are not followed up in Chapter 11 and are therefore not further discussed here. Comparisons between the different studied topics discussed in Chapters 4-10 are also made here.

The transition from BBK 04 to Eurocode 2 was made in 2011. However, Eurocode 2 was published in 2008 and has been used simultaneously as BBK 04 during a transition period. This means that structures that are constructed today might have been designed according to the previous Swedish handbook. Construction workers have probably therefore not yet seen all of the effects from the new standard, why the answers obtained from the interviews with Söderberg and Eriksson need to be analysed with this in mind.

The aim of the survey was to capture the opinions of structural engineers regarding chosen issues without the risk of asking leading questions and give answers in beforehand. As a result detailed problems were simplified to more general situations in the survey. This might have had the effect that some of the questions have been difficult to understand and the authors have interpreted the answers taking this into account.

When the material obtained from the interviews were compiled and presented it was noticed that the interviews made by telephone or by personal contact provided more comprehensive answers than the interviews where the answers were delivered by e-mail. However, when interviews were performed orally, the discussions often tended to diverge from the subject. The interviews have to a large extent been reproduced in full, except when issues that are not related to the content of the report were discussed.

The written answers were often more straightforward and were perceived as easier to retell, since an interpretation was not necessary in the same extent. However discussions could not be held in the same way as for the oral interviews, which can be seen as a drawback. It should also be added that all of the interviewed persons have been able to read and approve the text that has been written from their interview respectively.

12.2 Bending

12.2.1 Minimum longitudinal reinforcement

The Expression EC2 (9.1N) for minimum longitudinal reinforcement is derived in Section 4.2.2 on the basis of a rectangular cross-section of width *b*. In Section 4.2.3 it is discussed that the requirement more accurately can be expressed as the design moment capacity, M_{Rd}, in the ultimate limit state should be larger than the cracking moment, M_{cr}. Hallgren adds that the requirement is a national selectable parameter implying that the expression should be recalculated for other types of cross-sectional shapes, see Section 11.2.4. Even if the reference group that determined the national selectable parameters agreed on to keep the recommended expression, based on a rectangular cross-section and that Betongföreningen (2010a) states that this expression will be on the safe side for many types of cross-sectional shapes, the authors believe that it should be up to the designer to choose how to calculate the required capacity. A more general expression such as the one presented in Section 4.2.3 or at least an explanation to the to the requirement is therefore suggested as an improvement of Eurocode 2.

According to Hallgren the lower limit of Expression EC2 (9.1N) makes sure that cross-sections consisting of low strength concrete also will be reinforced. This supports the discussion in Section 4.2.3 concerning the background to the lower limit where the expression is compared to the corresponding requirement in ACI (2007). High strength concrete is more brittle than normal strength concrete, why the expression is dependent on the concrete strength. However, the authors believe that there is no need to further explain this in Eurocode 2, since when the lower limit is fulfilled the calculations are conservative.

12.2.2 Maximum longitudinal reinforcement

In Section 4.3.3 the reason why Sweden does not have any upper limit for the longitudinal reinforcement is discussed. According to Engström (2011b) the ductility of statically indeterminate structural members should always be verified at least according to the simplified requirement in Paragraph EC2 5.6.3(2), even if the member is designed according to linear elastic analysis. This is also implied in CEN (1991), CEB-FIP (1978) and CEB-FIP (1990), see Section 4.3.3. Westerberg does not agree on this and thinks that it is not necessary to have a ductility requirement also in Section EC2 5.4, regarding linear elastic analysis, see Section 11.2.5. However, the literature study speaks for that the ductility requirement $x / d < 0.45$ is misplaced in Eurocode 2 and should be moved to Section EC2 5.4. An evaluation of this is suggested.

According to Betongföreningen (2010a) the recommended expression for maximum reinforcement in Eurocode 2 is disregarded in Sweden, because other criteria are considered to better determine the maximum amount of reinforcement. However, if the ductility of a structural member designed according to linear elastic analysis does not have to be checked it can be argued whether or not there exists a requirement that provides a maximum reinforcement amount. According to Engström (2013) over reinforced sections are normally avoided in design and also the need for space in a concrete member will provide upper limits for the reinforcement amount.

12.2.3 Reinforcement detailing of concrete frame corners

Based on Johansson (2000) the limit of the reinforcement amount and detailing solutions provided for concrete frame corners subjected to opening moment in Annex EC2 J.2 can be questioned, see Section 4.2.4.3. Johansson (2013) argues that the reinforcement amount limiting the choice of different reinforcement configurations should be reduced and that the reinforcement configurations in Figure 4.9b and Figure 4.10b, both provided with radial stirrups, should be avoided. Hallgren agrees that the limit is very high and can be questioned. However, it should be noted that he has not considered the criterion before the interview. Westerberg has not reflected on if the limit value of reinforcement amount is high or not. However, he commented that the limit in BBK 04 was set to 1 % for characteristic yield strength of reinforcement equal to 500 MPa.

From the survey it can be seen that a majority of the participants have chosen alternative c), which according to Johansson (2000) is the preferable solution, see question number 8. The reinforcement amount ρ = 0.5 % is smaller than ρ = 2 %, which means that the recommended answer according to Eurocode 2 should be alternative a) or b). This might imply that the persons who answered c) do not consider the limit of the reinforcement amount in Eurocode 2 and instead follow the previous recommendations in BBK 04. Even if the reinforcement amount of 2 % is only a recommendation in Eurocode 2 and the persons from the survey do not seem to consider this recommendation, it is unfortunate that there is a possibility to choose the reinforcement configurations in Figure 4.9a and Figure 4.10a for that high amount of reinforcement. On the basis on Johansson (2000) an alteration of the recommended limitation of the reinforcement amount is desired.

From the literature study it has been shown that it is recommended to use the detail solution illustrated in Figure EC2 J.4a, see Figure 4.10a, for a upper reinforcement amount limit of 1.2 %. No recommendations for the reinforcement amount of the detail solution shown in Figure EC2 J.3a, see Figure 4.9a, have been found. However, in Section 4.2.2 it is shown that for a poorly designed concrete frame corner, see Type 1 in Figure 4.15, a upper limit of 0.2 % will provide quite sufficient corner efficiency. Hence, it can be argued that for the configuration in Figure 4.9a, that is a better configuration than Type 1 in Figure 4.15, a reinforcement amount of 0.2 % will result in a conservative detail solution.

12.3 Shear

12.3.1 Minimum shear reinforcement

The background to Expression EC2 (9.5N), see Equation (5.27) in Section 5.4.1, regarding the minimum requirement for shear reinforcement, has not been found. The only explanation is that this requirement will ensure a more ductile behaviour in the ultimate limit state, i.e. that the shear reinforcement, after cracking of concrete, is able to resist the force that previously was resisted by the uncracked concrete. This is also confirmed by Westerberg, who states that this can be understood from the fact that the expression is based on the principle that a high concrete strength requires a high reinforcement amount, see Section 11.2.5.4. Westerberg does not know how the expression has been derived. However, according to Johansson (2013) the expression in Eurocode 2 can be based on the requirement for minimum shear reinforcement provided in the Swedish handbook BBK, see Boverket (2004), but this has not been verified.

12.3.2 Configuration of shear reinforcement

12.3.2.1 Enclosing stirrups

In Section 5.6.3 it is discussed how shear reinforcement stirrups should be designed in order to utilise the full capacity of the cross-section. According to Rosell it is not uncommon that structural engineers miss to place shear reinforcement such that it encloses all of the longitudinal reinforcement. In the survey, see Section 11.3.2.6, it was shown that almost everyone that participated answered alternative c), i.e. the alternative in which all of the longitudinal reinforcement is enclosed by shear reinforcement stirrups. A few participants also answered alternative b), where only one reinforcement layer is enclosed by the shear reinforcement. However, from the comments from those who answered both alternative b) and c) it can be concluded that most of them are aware of that layer 1 cannot be utilised in the ultimate limit state but is there for other reasons.

It is difficult to draw any conclusions from the survey whether Rosell's concerns are motivated or not. However, it should be noted that it seems like structural engineers in general are aware of the importance of enclosing the longitudinal bending reinforcement. No need for improvement of Eurocode 2 can therefore be found.

12.3.2.2 Design of G-bars

In Section 5.6.3 it is discussed why it in Sweden is allowed to use only bent up bars as shear reinforcement instead of enclosing links. Westerberg's explanation to why bent up bars in Sweden can be used without additional enclosing links is that even if bent up bars cannot take forces in the direction across the width of the cross-section, they still can create good nodes in the truss model, see Section 11.2.5.5. It should be noted that Westerberg only consider the truss model from a 2D-perspective and does not reflect over the transversal tensile stresses that can occur across the cross-section as shown in Figure 5.23b. However, Westerberg's comment implies that he thinks that bent up bars and also G-bars can be used as shear reinforcement without additional enclosing links, just as the requirements in the Swedish Standard state. Rosell confirms that Trafikverket regards G-bars as bent up bars where no additional enclosing links need to be used, see Section 11.2.3.5.

The authors have not found any arguments why it in Sweden is allowed to use only bent-up bars as shear reinforcement, since the tensile strength of concrete is utilised in such a case in the same way as in Figure 5.23b. As stated in Section 5.6.3 it might be because previous standards in Sweden allow this type of utilization of concrete or that no cases of failure have occurred because of this type of shear reinforcement configuration. Motive for usage of only bent up bars might be to simplify the design by minimising the different types of reinforcement shapes and reduce the number of layers of reinforcement, which provides more available space in the structure. However, it should be noted that according to Eriksson the usage of bent up bars at the construction site will cause unnecessary work for the persons who place the reinforcement.

Since Eurocode 2 does not give any clear recommendations concerning G-bars, it was of interest to investigate if this is a common design of shear reinforcement, see question 4 in the survey described in Section 11.3.2.5. The survey showed that shear reinforcement in form of G-bars is a known configuration among the participants who are working with design of bridges and tunnels, see question number 5. However, some participants added that G-bars are only used as shear reinforcement in special cases and most commonly in slabs. The majority of the participants in the survey, especially those who are working with bridges and tunnels, thinks that G-bars can be used as shear reinforcement without additional enclosing stirrups, i.e. they can be regarded as bent up bars.

In Section 5.6.3 it was discussed whether to use the lap length or the anchorage length of the horizontal leg in order to be able to use G-bars as bent up bars. The opinions differ concerning this. Johansson (2013) believes that it is correct to use the lap length in design of G-bars. However, Rosell states that Trafikverket approves configurations where G-bars are used as shear reinforcement and placed in contact to the longitudinal reinforcement with an anchorage length. Westerberg states that G-bars can be used as bent up bars provided that they are placed with a lap length. It can be argued that it from economical aspects as well as geometrical reasons is more favourable to use an anchorage length, since this is shorter than a lap length.

The result from the survey question, asking whether the lap length or the anchorage length of the horizontal leg should be used when G-bars are used as shear reinforcement, shows that those who are working with design of bridges and tunnels mostly use the former and those who are working within the area of housing and industrial buildings think it is sufficient to use the latter. It is noteworthy that the opinions among the participants differ and that the anchorage length, that is shorter than the lap length, is more frequently used among designers working within the area housing and industrial buildings.

In Section 5.6.3 reference is made to Westerberg (1995) regarding the direction of the G-bars. This has been a debated question. Rosell agrees with Westerberg's recommendation where the crack should be captured within the bend of the bar, see alternative 1 in Figure 5.28. Most of the participants in the survey answered according to Alternative 1 in Figure 5.28. One of the comments on this question was that both directions are correct. The participant is aware of that the crack might not be captured within the bend if Alternative 2 is chosen. However, the participant argues that this is also the case when using "normal vertical stirrups" why the direction of the G-bars should not matter since the purpose of the shear reinforcement is to keep the shear crack together. It should be noted that the participant always uses Alternative 1, due to contact with Trafikverket. It is good that many structural engineers choose Alternative 1, which obviously might be a result from influence from Trafikverket.

As a final conclusion it can be noted that nothing about G-bars as shear reinforcement is mentioned in Eurocode 2, which is unfortunate since G-bars are used within the industry and that the opinions of how to perform a correct design differ.

12.3.3 Load close to supports

When shear reinforcement is designed in concrete members, it is allowed to reduce the effect of a load that is acting closer than a distance a_v from the support section. The distance a_v is in Eurocode 2 limited to be within the interval of $0.5d$ and $2d$. Westerberg agrees with the opinion presented in Section 5.7.3, where the lower limit of a_v equal to $0.5d$ is argued to be misleading. The lower limit of a_v could perhaps be changed into d instead, since the load acting within this area will go directly to the support due to the inclination of the first inclined strut, see Figure 5.37. It should be noted that it is not wrong to reduce effect of a load acting closer than a distance d to the support. What the authors opposes to is that the rule of av = 0.5 d implies that shear reinforcement should be placed within a distance closer to the support which is not required for uniformly distributed load. It might not be possible to say that it is not required to check the design shear force from a concentrated load at a distance less than *d* from the face of the support. However, it can be argued that loads placed closer to supports than a distance *d* should be designed for by strut and tie models rather than by truss analogy.

12.3.4 Suspension reinforcement

In Section 5.8.3 suspension reinforcement at indirect supports is discussed. It has been observed that this type of additional reinforcement sometimes is not provided in structures where it is needed. The question asked in the survey in order to see if structural engineers are aware of that suspension reinforcement should be placed, in for instance a through bridge and how this reinforcement should be placed in order to be able to utilise the full cross-sectional height in the load bearing capacity. However, the question turned out to be quite difficult to understand for the participants, see Section 11.2.2.7. In order not to ask a leading question the word suspension reinforcement was left out from the question. However, from the comments it was clear that this caused confusion among the participants and the focus of the answers did therefore differ from the one intended by the authors. The argument from Rosell presented in Section 11.1.3.6, that structural engineers still fail to provide additional suspension reinforcement in the structures, could therefore not be confirmed.

In order to see if the participating structural engineers add suspension reinforcement, a third alternative without any suspension reinforcement at all, would perhaps facilitate the interpretation of the question.

12.4 Torsion

12.4.1 Longitudinal torsional reinforcement

In Paragraph EC2 6.3(3) it is described that it is allowed to reduce the longitudinal torsional reinforcement in the compressive zone. However, no description on how to perform this reduction is presented. This is further discussed in Section 6.2.3. Westerberg, clarifies that the longitudinal reinforcement amount in the compressive zone can be reduced with a force equal to M_{Ed} / z, see Section 11.2.5.8. From this it is the authors' interpretation that this means that all longitudinal torsional reinforcement in the compressive zone may be reduced, also the reinforcement placed in the compressive zone in the vertical walls. It should be noted that this only applies if the force that should be resisted is larger than M_{Ed} / z. Westerberg's explanation partly contradicts the recommendations in Hendy and Smith (2010) where it is stated that the reduction by means of the compressive force should be made within a zone equal to twice the concrete cover to torsion links considered. Since there is different ways to reduce the longitudinal torsional reinforcement due to mutual bending an explanation or illustration on how this should be done in order to maintain a conservative solution is desired.

12.4.2 Transversal torsional reinforcement

Ambiguities concerning lapping of torsional links are discussed in Section 6.3.3, where Paragraph EC2 9.2.2(3) describes that laps in web sections are not allowed if the links should be able to lift torsional shear force.

Rosell describes that it can be easy to overlook the information in Paragraph EC2 9.2.2(3) when designing torsional stirrups, since it is stated in a section concerning shear reinforcement, see Section 11.2.3.7. However, as it is described in Section 11.2.4.4 Hallgren believes that this is not a problem, since Figure EC2 9.6 providing shapes of torsional links only shows links with anchorage in form of hooks and bends, see also Figure 6.16, Section 6.5.1. He claims that the figure in Eurocode 2 shows no configurations of torsional links that include laps placed in the web ended without a bend.

Hallgren indicates that it is not a good solution to lap splice torsional reinforcement, since it is difficult to obtain the required lap length over the cross-sectional height or width. This might be a reason why it in Eurocode 2 is emphasised that it is not allowed to do this. However, Hallgren does not see any physical reason why it should not be allowed to lap torsional reinforcement in the web, if the required lap length can be fulfilled. At the workshop held by Brosamverkan Väst, mentioned in Section 6.3.3, a question if the required staggering of $0.3l_0$ of lap splices also applies to torsional reinforcement were raised and sent to SIS. If a distance of $0.3l_0$ between lap ends is required also for torsional links it will be even more difficult to motivate lapping of shear or torsion reinforcement in webs. This is because the lap splices in such situations require larger cross-sectional heights. Johansson (2013) adds to the discussion that the recommended configurations illustrated in Section EC2 9.2.3 are most often only reasonable for small structures with beam heights lower than a full lap length.

According to Westerberg the rule in Eurocode 2, that states that it is not allowed to splice transversal torsional reinforcement in webs, is an expression of caution due to lack of knowledge or experience, see Section 11.2.5.9. He also adds that experimental investigations have shown that splicing by lapping is allowed for shear reinforcement and it is difficult to motivate a difference between transversal torsional reinforcement and shear reinforcement in this respect.

According to the opinions presented above it seems like there are many unresolved uncertainties related to splicing of torsional reinforcement. The authors have therefore identified a need for further research in order to see if the requirement regarding lap splices of torsional links in webs can be ignored or not.

Studying the results from question number 4 in the survey, see Section 11.3.2.4 where different configurations of reinforcing links were presented to the participants, it can be seen that there is a large distribution of the answers, with an exception concerning alternative d). This is probably because alternative d) corresponds to the configuration presented in Figure EC2 9.6 that is not recommended. It should be noted that alternative b) is the most popular configuration to use in design of transversal torsional reinforcement. However, this alternative is spliced by lapping in the web. The result shows that the recommended configurations in Figure EC2 9.6, i.e. alternative f), h) and i) are chosen by many participants. However, it is noteworthy that the structural engineers choose other configurations, especially the ones without hooks or bends, in contrary to what should be chosen according to what Hallgren believes.

The result from the survey indicates that Rosell is correct saying that the requirement regarding splicing of torsional links can be overlooked by structural engineers. A suggested improvement of Eurocode 2 is therefore to add references between Section EC2 9.3 concerning detailing of torsional reinforcement and Paragraph EC2 9.2.2(3).

The buildability aspect is considered in the next part of question number 4. This question, focusing on whether the bar diameter influences the choice of shape of torsional reinforcement, was written in the belief that reinforcing bars of smaller diameters may be bent by hand at the construction site. The hypothesis was that some of the configurations presented in question number 4 therefore are less suitable for larger bar diameters. From the interview with Söderberg this was found not to be true. He stated that almost all reinforcement is bent at the reinforcement factory and that all the suggested solutions therefore, with regard to that aspect, are acceptable solutions. The result from the survey is therefore from this point of view not very interesting. However, it can be mentioned that those who consider the bar diameter when choosing configuration, excludes alternatives h), i) and j) that all include hooks around another bar. Some participants have also chosen solution b) but avoided for instance solution a) for bar diameters equal to $\phi 16$. This might have something to do with that the required anchorage length increases with increased bar diameter.

12.4.3 Combination of torsional moment and shear force

In Section 6.4.3 questions were raised concerning design of longitudinal torsional reinforcement for combined shear force and torsional moment. The main question was if the longitudinal torsional reinforcement should be evenly distributed around the cross-section, when shear force and torsional moment are superimposed in order to calculate the additional longitudinal tensile force that needs to be resisted by longitudinal reinforcement, see Table 6.1. Westerberg points out that when combining torsional moment and shear force, i.e. using Expression EC2 (6.18) and calculating ΔF_{td}, see Equation (6.33), structural engineers might fail to follow Paragraph EC2 6.3.2(3), that states that longitudinal torsional reinforcement should be evenly distributed around the perimeter of the cross-section, see Section 11.2.5.10. Westerberg's comment implies that he believes that the longitudinal reinforcement still should be evenly distributed also for combined shear force and torsional moment. However, the authors think that if this is the case, Expression EC2 (6.18) should be used but without distributing the longitudinal component of the combined shear force $V_{Ed,V+T}$ $(\cot\theta - \cot\alpha)$ on the force couple that resists the bending moment. This means that the equation should be altered so that the combined shear force $V_{Ed,V+T}$ is not divided in two. However, this will result in a higher amount of reinforcement and that the shear reinforcement also will be spread around the cross-section.

From the interview held with Hallgren it was clear that he was not sure how to carry out superposition of torsional moment and shear force in a correct and appropriate way. However, both Hallgren and Westerberg seem to agree on that even if torsional moment and shear force are superimposed some longitudinal reinforcement should still be distributed around the whole cross-section to take into account that inclined cracks occur in all walls around the cross-section.

It can be questioned how superposition of torsional moment and shear force should be performed in a favourable way and at the same time fulfil the requirements provided for both shear and torsional reinforcement. More clarifications and explanations in Eurocode 2 are therefore requested by the authors.

12.5 Shear between web and flanges

In the discussion in Section 7.2.3 the focus was on Expression EC2 (6.20), see Equation (7.1), regarding the longitudinal shear stress at the web-flange intersection, in combination with Paragraphs EC2 6.2.4(5) and (6). It is discussed why it is allowed to ignore the highest shear force per unit length acting at the interface. There is a possibility that Δx is chosen such that the requirement which results in that no transversal reinforcement is needed, besides that for transversal bending, is fulfilled. If there is no need for transversal bending reinforcement or if this reinforcement is needed but is almost fully utilised it can be argued that the requirements result in a nonconservative design. Especially since the shear force, that is used to see if any transversal reinforcement is needed, is lower than the highest shear force acting in any section of the member.

Hallgren compares this to the paragraph in Eurocode 2 that states that no shear reinforcement is necessary in a section if the requirement in Expression EC2 (6.2) is fulfilled since friction and interlock effects can lift the shear force over the cracks. Hence, he thinks that it is nothing strange with Equation (7.1), see Section 11.2.4.6.

However, Johansson (2013) implies that Equation (7.1) does not say anything about how much of the shear force that can be resisted by the concrete. In general all tensile forces should be resisted by the reinforcement, but in this case some kind of superposition of the shear capacities due to both steel and concrete is used. According to Johansson (2013) a more reasonable method would be to include the effect of the concrete capacity by providing it a reasonable value.

Another solution to this problem can be to add a minimum requirement into the standard so that a certain reinforcement amount always will be provided into the structure, independent of the loading situation.

12.6 Shear at the interface between concrete cast at different times

The factor c in Expression EC2 (6.25), see Equation (8.3) and Section 8.2, can be interpreted as a cohesion factor which, according to Engström (2013), to some extent can be regained after cracking. Westerberg is of the same opinion, i.e. that this factor should be used also when compression again is acting on a joint that previously has been separated, see Section 11.1.5.11. However, he does not consider the factor c as cohesion, but more as a point in a model describing a behaviour derived from test results, see Figure 3.15c.

The authors think that the factor c is poorly described in Eurocode 2 and the question still remains, whether the loading history influences the cohesion or not. Two possible ways of determining the factor c can be distinguished. The first way is to assume that the loading history influences the value of c and the factor should therefore be taken as zero, if the stress σ_n has obtained a negative value at previous time. The second way is to assume that the loading history has no significant importance and that *c* should not be reduced when the joint is compressed. However, a suggestion is to assume that the cohesion to some extent can be regained after cracking as implied by Engström (2013). A proposal is therefore to halve the value of *c* and thereby follow Paragraph EC2 6.2.5(5) which is applicable for dynamic loads, see Section 8.2.3. It should be noted that this is a speculation and since it is difficult to know what value the *c*-factor should have in a situation when cracking of the joint has occurred, it is perhaps better to be conservative and set *c* to zero.

12.7 Bond and anchorage

12.7.1 Anchorage of bottom reinforcement at end support

In Section 9.3.3 it is discussed that there are differing views on how to provide sufficient anchorage of bottom reinforcement at end supports. Engström (2013), Westerberg and Hallgren are all in agreement that the reinforcement must extend over the entire node area. This is also stated in Betongföreningen (2013a). However, at the interview with Hallgren he implied that it can be difficult to make room for the reinforcement if it should extend over the entire node region. The authors think that this is one of the reasons why the opinions on how to provide anchorage at end supports differ among structural engineers. Another reason may also be that anchorage of bottom reinforcement at end supports is treated in two different sections in Eurocode 2, i.e. Section EC2 6.5.4 and EC2 9.2.1.4. In Section EC2 6.5.4, where anchorage of bottom reinforcement is illustrated by means of a strut and tie model, it is implied that the reinforcement should extend across the entire node region to provide sufficient anchorage. However, the section in Eurocode 2 that specifically is devoted for design of bottom reinforcement at end supports, i.e. Section EC2 9.2.1.4, only states that the design anchorage length, l_{bd}, should be measured from the intersection point between beam and support.

The authors think that it according to Eurocode 2 is quite clear that sufficient anchorage should be obtained at the face of the support and that the anchorage length should extend through the entire node area. However, this is under the condition that the requirements in both Section EC2 6.5.4 and EC2 9.2.1.4 are considered. In order to emphasise that the reinforcement should extend over the entire node region when anchorage of bottom reinforcement at end supports is designed, it is therefore desirable to add references between these two sections.

12.7.2 Lapping of longitudinal reinforcement

In Section 9.4.3 it is discussed whether it is allowed to lap all of the longitudinal reinforcing bars in the same section or not. Rosell, Hallgren and Westerberg all state that it is allowed to lap 100 % of the longitudinal reinforcement in one section, as in Figure 9.22b, see Section 9.4.3, provided that the recommendations in Paragraph EC2 8.7.2(4) are followed. It should be noted that this means that the second indent in Paragraph EC2 8.7.2(3), providing a certain distance between lap ends, is ignored. However, it is agreed that this can be difficult to understand from the text and figures provided in Section EC2 8.7.2.

From the interview with Hallgren one additional aspect concerning lapping of reinforcement was highlighted, since he emphasises on the difference between lap length and anchorage length, see Section 11.2.4.8. It can be argued that the bond stress that results in transversal tensile stresses around each bar is active along a certain distance corresponding to the anchorage length, l_{bd}. According to Eurocode 2 the required lap length is a function of the anchorage length times the factor α_6. Since the factor α_6 increases the lap length for those cases where several bars are lapped in the same section, it can be argued that this is a way to compensate for stress concentrations that might occur for several lap splices in the same section.

It should be noted that the distribution of bond stress assumed in Figure 9.19 and Figure 9.20 is based on a linear relation between the bond stress and bond slip. This relation is as explained in Section 3.2.1 true for moderate loads but for higher loads the relation becomes more and more non-linear. The model presented by Leonhardt (1974) does not consider that the bond stresses are evened out along the anchorage length in the ultimate state. However, lap splices that are not enclosed by large concrete covers or transversal reinforcement may result in a very brittle anchorage failure due to stress concentrations around the bars without the possibility of stress redistribution.

According to Engström (2013) an increased lap length is not always sufficient to increase the capacity of lap splices. A brittle anchorage failure may be triggered at a part of the anchorage length without redistribution of stresses along the increased length. The capacity is dependent on both lap length and the enclosement of the splice. Hence, it is of great importance to also ensure sufficient concrete cover and place transversal reinforcement in the splice zone. This is also reflected by the rules in Eurocode 2, Section EC2 8.7.4.1, providing stricter requirements on transversal reinforcement if more bars are spliced in the same section.

From the survey, see question number 3 in Section 11.3.2.3, it is clear that most of the participants choose to place lap splices with a distance between lap ends of $0.3l_0$, see Figure 9.22a. This is also in accordance with the required configuration in Figure EC2 8.7. It is noteworthy that so many of the structural engineers interpret Eurocode 2 that it is not allowed to place all lap splices in the same section. Even if the complete background to the requirements in Eurocode 2 has not been found, the result from the literature study and the interviews implies that it is allowed to lap all bars in one section. However it is clear from the survey that this is not the way the rules are being interpreted. It can be concluded that Figure EC2 8.7, see Figure 9.17, is misleading and it should be clarified that the required distance between lap ends of $0.3l_0$ can be disregarded if all other provisions provided in Section EC2 8.7.2 are fulfilled, see Section 9.4.1.

One comment from the survey was that alternative b) applies for bending reinforcement and alternative c) applies for shear, torsional and secondary reinforcement. This is an interesting point of view, since it in Eurocode 2 is stated that no laps are allowed to be placed in webs for transversal torsional reinforcement. A reason for this might be that it is difficult to ensure sufficient lap length if the lap ends have to be staggered with a distance equal to $0.3l_0$ to each other. However, if it can be determined that it is allowed to splice all bars in one section, then maybe it can also be argued that torsional reinforcement as well as shear reinforcement can be designed with lap joints in web sections, see also the discussion in Section 12.4.2.

12.7.3 Concrete cover and distance between bars

12.7.3.1 Clear distance between bars

In Section 9.5.1 the minimum requirements for clear distance between single bars were compared to the same requirements for bars in lap splices. Required clear distance between single bars is according to Eurocode 2 the larger of ϕ, $d_g + 5$ mm and 20 mm. For lapped splices the requirement is altered to the larger of 2ϕ and 20 mm. In Section 9.5.1 it was argued that the reason for the increased distance in case of lapped bars is to consider possible deviations at the construction site.

When Westerberg was asked about the reason why the requirement for single bars was set to one bar diameter he answered that the limit of 1ϕ can be said to be compatible with rules for calculation of anchorage length and similar expressions, see 11.2.5.14. The authors think that this reinforces the argument of why the distance between lapped bars is increased. If two bars in a lap splice by mistake are reversed at the construction site the minimum distance between the two bars closest to each other will still be at least 1ϕ, see Figure 12.1. However, it should be noted that this does not mean that it is allowed to place bars in a lap splice closer to each other than 2ϕ. If mistakes are found at the construction site they should be corrected.

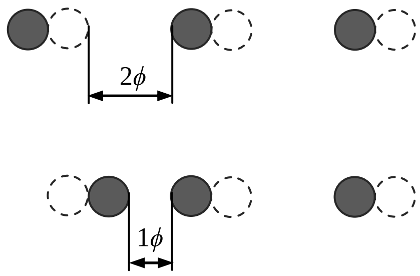


Figure 12.1 Required minimum distance between lapped bars might have been set to consider deviations such as by mistake reversing two bars at the construction site.

At the interview Westerberg also confirmed that the requirement of a clear distance between bars not smaller than $d_g + 5$ mm applies also for lapped bars. The recommended formulation of the requirements presented in Figure 9.27, where the requirement for single bars is seen as a basic requirement that should be fulfilled also for lapped bars, can therefore be considered as correct and can be presented as a suggested improvement of the requirements in the standard.

12.7.3.2 Free space for internal vibrators

No recommendations are presented in Eurocode 2 about how to enable sufficient free space between the reinforcement bars for internal vibrators in order to allow for good placing and compaction of concrete. This is discussed more comprehensively in Section 9.5.3. Recommendations found in BBK 04, Boverket (2004) and Barker (1967) provide information about suitable free space between groups of bars. However, no information about how close these gaps should be placed in relation to each other is presented. According to Söderberg one vibrator is assumed to affect the concrete within a distance of 350-400 mm around the gap, see Section 11.2.6. This implies that free space for internal vibrators should be enabled with a spacing of not more than 700-800 mm. In order to facilitate for structural engineers such information could be stated in Eurocode 2. However, there are also risks in providing too much information in the standard since it can be interpreted as a requirement that always must be fulfilled. If this type of information is provided in Eurocode 2 it should be clearly stated that it is a recommendation and not a principal rule.

12.7.3.3 Flexible reinforcement

It is important to provide sufficient concrete cover and clear distance between bars in order to ensure adequate bond between reinforcement steel and concrete. According to Rosell changed conditions at the construction site may result in problem to fulfil requested concrete covers, which can motivate the usage of more flexible reinforcement configurations. This can for instance be a combination of several bars spliced together, rather than one bar formed in the final shape, see Section 11.2.3.12.

Similar problems have also been noticed by Söderberg, who gives examples on occasions when flexible reinforcement configurations are convenient, see Section 11.2.6.7. In situations where there is limited space within the concrete formwork, the contractor often requests to change into more flexible reinforcement configurations in order to facilitate the placing of reinforcement. However, it can be concluded from both Söderberg and Eriksson that if the designers follow the requirements in Eurocode 2 in combination with common sense, sufficient margins for the reinforcement to fit within the structure is often provided.

There are some other things that structural engineers can consider in order to facilitate the work at the construction site. According to Eriksson it is difficult to bend a reinforcing bar with a dimension of $\phi 16$ with a manual bender. The structural engineer needs to be aware of this and perform detail solutions where these kinds of reinforcing bars can be bent at the factory and do not need to be bent by hand at site. The structural engineer also has to think about keeping sufficient clear spacing between the bars. For instance a clear spacing of 100 mm between longitudinal top reinforcement bars should be taken into account since it otherwise will be too little space to tie these bars to the stirrups. Another thing that also facilitates the work at the construction site is reinforcement composed of bars with the same lengths, dimensions and shapes in order to reduce the need for sorting.

It can be concluded that the knowledge about the contractors' needs is important for a structural engineer and that improvements within this area are asked for. It can also be concluded that flexible reinforcement configurations are desired already at the initial stage of design. Therefore, the authors would like to suggest that, for instance, in a further master's thesis investigate the needs and opinions of the contractors, in order to get a more practical point of view and to come up with actual examples of proper detail solutions.

12.8 Crack control

12.8.1 Minimum reinforcement requirements for crack control

In Section 10.2.3 it is discussed that Expression EC2 (7.1), see Equation (10.1), providing a minimum reinforcement amount for crack control, will result in a large reinforcement amount, especially in comparison to the corresponding expression in the Swedish handbook BBK 04. Hallgren agrees with this, particularly for slabs with large thicknesses, see Section 11.2.4.11.

A comment has been sent to the European committee regarding that Expression EC2 (7.1) perhaps can be replaced with the corresponding expression in BBK 04, where the effective concrete area, $A_{ef,BBK}$, is used instead of A_{ct} which is the concrete in tension just before cracking. This suggestion seems reasonable, since it is only the effective concrete area that is influenced by tensile stresses after formation of the first crack and the second crack will occur within the effective area. However, from investigations performed by both Alfredsson & Spåls (2008) and Björnberg and Johansson (2013) it is clear that neither the expression for the effective area provided in Eurocode 2 or the corresponding expression in BBK 04 will result in appropriate reinforcement amounts.

It can be concluded that using the minimum requirement for crack control provided in Eurocode 2 will provide reinforcement amounts large enough to provide a distribution of cracks. However, the amount of reinforcement can be argued to be unreasonably large, especially for large or thick members and members subjected to restraint forces. The minimum reinforcement requirement for crack control in Eurocode 2 therefore needs to be further developed in order to result in a more reasonable reinforcement amount. Perhaps the master's thesis mentioned in the interview with Hallgren, i.e. Björnberg and Johansson (2013), can be the foundation for such further research.

It is difficult to know for what situations the expression provided in Eurocode 2 is intended to be used. This is also implied by Rosell, see Section 11.2.3.11. The derivation of the expression in ECP (2008a) is based on a reinforced concrete member subjected to bending, see Section 10.2.2. According to Engström (2013) the expression can be used for both bending and restraint and Hallgren thinks that the truth behind the expression is somewhere between external loading and restraint situations. However, Hallgren states that it is more useful in restraint situations since it describes the required reinforcement amount in order to redistribute the deformation caused by restraint in order to achieve a new crack after the first one has occurred.

The authors think that the expression is poorly described in Eurocode 2. Even if the basic principle in Eurocode 2 implies that the requirements for crack control is applicable for both external load and restraint situations it should be stated for what situations the expression is derived and intended. It is also desirable to know what the consequences might be if the expression is used for different situations. Perhaps it might be necessary to derive different minimum reinforcement requirements for different situations in order to find appropriate reinforcement amounts and adequate crack control.

12.8.2 Crack reinforcement for relatively high cross-sections

The authors have noticed that it is unclear whether crack reinforcement should be added at web surfaces of beams with relatively high cross-section or not. It might be unnecessary to place longitudinal crack reinforcement in unreinforced areas, since the capacity of the member is not affected if large cracks occur in such areas. However, according to Hallgren such situations are very uncommon, since Eurocode 2 states that shear reinforcement always should be placed within structural members, why crack reinforcement is needed in webs in order to protect the shear reinforcement from corrosion, see Section 11.2.4.10. His comment can be confirmed for beams, see Section 5.4.3. Even if this is the case it is, according to Johansson (2013), not clear from Eurocode 2 how the crack width, w_k, and amount of crack reinforcement required in relatively large web sections should be calculated.

The participants in the survey were asked how they design crack reinforcement in beams with relatively high cross-sections, see question number 2 in Section 11.2.2.2. The relevance of the question was reduced due to the statement of Hallgren and since the presented cross-section in the survey contained shear reinforcement. However, it is still interesting to see whether structural engineers are aware of that crack reinforcement provides crack distribution only within the effective concrete area. A majority of the participants in the survey answered alternative b), which means that they place the crack reinforcement in the web in addition to the main reinforcement for bending resistance. The reason why most of the participants have chosen alternative b) instead of c) is not clear. It could be that they all think that crack reinforcement should be added to the main reinforcement or simply because alternative c) results in more complicated calculations, since the resulting tensile force moves.

The result from the survey indicates that the participants are aware of that the crack reinforcement only is active in the effective concrete area and that it is not enough to only place bending reinforcement in the bottom part of the cross-section and make sure that this amount fulfils the minimum requirement.

Since the minimum requirement in Eurocode 2 provides a very large amount of reinforcement for thick members, it was also investigated for what cross-sectional heights additional longitudinal crack reinforcement is considered to be required in webs. There is a risk that the participants have interpreted this question in another way than was intended since the word surface reinforcement unfortunately has been used in the formulation of the question. The intention was to ask about crack reinforcement, since it is a supplementary question to question number 2.1. However, most of the participants seem to have interpreted the question as intended. Many have answered that alternative b) is the limit for when crack reinforcement is added, which means that crack reinforcement should be placed with a spacing of maximum 300 mm. According to Engström (2013) this implies that an old rule given by the former Vägverket providing at least $\phi 10 s300$ is used, see also the interview with Rosell in Section 11.2.3.9. From the comments obtained in the survey it can also be noted that some of the structural engineers use the requirements provided by Trafikverket resulting in that all concrete surfaces should be reinforced.

From Table 11.6 in Section 11.3.2.2 it can be deduced that there are many different ways to arrange crack reinforcement, since the values of the parameters are spread widely. However, there is a tendency that the structural engineers use a spacing of about 150-300 mm. The bar diameter is in these cases chosen to 10 or 12 mm.

A conclusion that can be drawn is that structural engineers tend to apply rules or guidelines that are not stated in Eurocode 2 when they arrange crack reinforcement in beams with relatively high cross-sections. This implies that there is a need for or at least desirable to incorporate more guidelines concerning crack reinforcement in larger members in Eurocode 2.

12.8.3 Limitation of crack widths for shear and torsion

In Section 10.3 ambiguities concerning the use of the expressions for calculation of crack widths in Section EC2 7.3 also for check of torsion and shear cracks was discussed. Three additional methods to determine steel stresses in order to find the required amount of reinforcement were presented, where two are available in Betongföreningen (2010a). According to Rosell a previous recommendation from Trafikverket to limit the steel stress to 250 MPa in the ultimate limit state is still used by many structural engineers, which implies that there is lack of knowledge about how check crack widths in case of shear and torsion.

From the survey, see question number 1 in Section 11.3.2.1, it is clear that many participants working with design of bridges and tunnels follow the previous recommendation of 250 MPa from Trafikverket, see alternative b). As a comparison, the methods described in Betongföreningen (2010a) are the most frequently used by the participants who are working within the area of housing and industrial buildings. Since alternative b) is still used, even though Trafikverket does not recommend this anymore, it is clear that further investigation within this area needs to be carried out in order to improve the requirements in Eurocode 2. This is also asked for by Rosell.

12.9 Cooperation between structural engineers and contractors

It is unfortunate that both the creators of Eurocode 2 and structural engineers in general seem to not have sufficient knowledge about the contractors' needs. This is also implied by Rosell. By simply following the requirements stated in the code can lead to detail solutions of reinforcement that might be difficult to perform at the construction site. Johansson (2013) speculates that Eurocode 2 is written mainly for design of buildings and not design of bridges and tunnels, since many recommendations are not always applicable for large scale construction. Some examples of this that have been mentioned in this report are rules concerning crack reinforcement, lapping of transversal torsional reinforcement in web sections and rules for anchorage of longitudinal reinforcement in slabs that presumes no shear reinforcement. The authors' opinion is also that Eurocode 2 generally provides recommendations of reinforcement configurations that are more suitable for smaller bar dimensions. However, this could not be verified from the interviews performed with Söderberg and Eriksson. It would therefore have been interesting to get the opinion of construction workers who have experience more related to construction of houses to see if the dimension of the bars has larger influence of the work at the construction site for this type of buildings.

Söderberg and Eriksson are in agreement regarding that the documents delivered from structural engineers in general are well prepared, see Sections 11.2.6 and 11.2.7, respectively. However, Söderberg suggests that a more continuous cooperation between the contractor and the structural engineer is needed in order to improve the final result.

The usage of 3D-models has facilitated detection of errors in drawings and other design documents. Söderberg thinks that it is favourable that the providers of the reinforcement try to simplify the work at the construction site by changing and correcting the drawings if problem areas are noticed. However, it would be interesting to see if there are other ways to improve the cooperation between structural engineers and contractors. It would also be interesting to investigate how a continuous communication between the different actors can be achieved in a structured and efficient way.

12.10 Influence from professional background on the choice of detail solution

A general conclusion that can be drawn from the survey is that there are significant differences between the answers obtained from participants who are working with design of bridges and tunnels and the answers obtained from those who are working within the area of housing and industrial buildings. The differences often imply that the participants who are designing bridges and tunnels are more influenced by old rules given by Trafikverket and former Vägverket, see for instance question number 1 asking how limitation of shear and torsional cracks is checked, question number 3 about lap splicing of reinforcement, questions number 4.2 and 5.2 concerning design of shear reinforcement and question number 8 about placing of reinforcement in concrete frame corners.

13 Summary of results

13.1 Introduction

In this master's thesis project many different subjects have been treated. Some expressions in Eurocode 2 have been derived or explained. Other requirements have been discussed and unclear parts of the code have been highlighted. By means of interviews and a survey it has been investigated how experienced structural engineers and structural engineers in general interpret the rules and guidelines provided in Eurocode 2. During this project recommended improvements of Eurocode 2 have been suggested, and need for further research and clarification has been identified. In this chapter all these recommendations, thoughts, explanations and conclusions have been summarised. This will hopefully make it easier for the reader to find what he or she is looking for or identify new interesting topics.

Depending on the nature of the problems treated in this report and on the amount of background information that could be found, varying results have been obtained. Chapter 13 has therefore been divided in different sections, each describing the type of the presented results. Sections 13.2-13-6 consist of tables in which the different items are ordered in the same way as they appear in Eurocode 2. The first column of each table provides a reference number and the second column states the treated paragraph or equation in Eurocode 2. The third column provides a short summary of the conclusion or the obtained result. References to relevant chapters in this report are stated in the fourth column. The first reference describes the relevant section in the main chapters, i.e. Chapters 4-10, and is followed by a reference to the relevant section in Chapter 12 where the topic is evaluated and analysed based on information taken from the literature study, interviews and the survey. Some of the topics are also included in the survey why reference is made to the relevant question included in the survey presented in Section 11.3.2.

13.2 Recommended improvements of Eurocode 2

On the basis of the master's thesis project clarification and improvement of Eurocode 2 is suggested concerning the items that are stated in Table 13.1-Table 13.2. Table 13.1 presents recommended improvements where actual suggestions have been made. Table 13.2 are improvements that correspond to changes in the disposition of Eurocode 2.

Table 13.1 Topics where actual improvements have been suggested.

No	Items in Eurocode 2	Description	Sections in report
1	Paragraphs EC2 8.2(1), (3)	In order to allow access for concrete vibrators Eurocode 2 should include a recommendation to leave gaps of about 100mm with a spacing of not more than 700-800mm free from reinforcement.	9.5, 12.7.3.2
2	Paragraphs EC2 8.2(2), (4) Paragraph EC2 8.7.2(3)	It should be clarified that the rules for clear distance between single bars apply also to bars in lap splices.	9.5, 12.7.3.1
3	Section EC2 8.7.2	It should be better clarified in Eurocode 2 that the provision of a distance between lap ends of $0.3l_0$ provided in Figure EC2 8.7 can be disregarded when 100% of the bars in one layer is lap spliced in the same section.	9.4, 12.7.2, 11.3.2.3
4	Expression EC2 (9.1N)	The expression for minimum flexural reinforcement should be expressed more generally as $M_{Rd} \geq M_{cr}$.	4.2, 12.2.1
5	Paragraph EC2 9.3.1.1(4)	The rules referred to in Paragraph EC2 9.3.1.1(4), applicable for curtailment and anchorage of longitudinal reinforcement in beams, should also apply to slabs, without exceptions.	9.2

Table 13.2 Topics where Eurocode 2 can be improved by including references or by changes in the disposition.

No	Items in Eurocode 2	Description	Sections in report
1	Paragraph EC2 5.6.3(2)	The ductility requirement $x/d < 0.45$ is misplaced in Eurocode 2 and should be moved from Section EC2 5.6 to Section EC2 5.4.	4.3, 12.2.2
2	Section EC2 9.2.1.4	A reference between Section EC2 9.2.1.4 and Section EC2 6.5.4 is needed in order to clarify rules for anchorage of bottom reinforcement at supports.	9.3, 12.7.1
3	Paragraph EC2 9.2.2(2)	Rules and guidelines concerning shear reinforcement in the form of G-bars should be incorporated in Eurocode 2.	5.6, 12.3.2.2 11.3.2.5
4	Paragraph EC2 9.2.2(3)	There should be a reference in Section EC2 9.3, concerning detailing of torsional reinforcement, to Paragraph EC2 9.2.2(3).	6.5, 12.4.2 11.3.2.4
5	Section EC2 9.2.5	For design of suspension reinforcement references between Section EC2 9.2.5, Paragraph EC2 6.2.1(9) and Figure EC2 9.3 are needed.	5.8, 12.3.4 11.3.2.7

13.3 Needed improvements of Eurocode 2

In this master's thesis project it has been concluded that some parts of Eurocode 2 need to be improved. Table 13.3 presents different topics that have been treated in this report but where the reasons for the rules or how to apply the requirements in different situations still remain unknown. The items presented in Table 13.4 are examples of topics where a need for further research has been identified in order to be able to develop and improve the rules provided in Eurocode 2.

Table 13.3 Topics where the information in Eurocode 2 should be clarified or reviewed.

No	Items in Eurocode 2	Description	Sections in report
1	Expression EC2 (6.25)	It should be clarified whether the specified *c*-factor in Eurocode 2 is for the actual loading situation or if the loading history affects.	8.2, 12.6
2	Paragraph EC2 6.3.2(3)	Clarification of how reduction of the longitudinal torsional reinforcement amount in the compressive zone, due to mutual bending should be performed is needed.	6.2, 12.4.1
3	Expression EC2 (7.1)	The minimum reinforcement amount for crack control should not be based on the area A_{ct}, but on the effective concrete area A_{ef}.	10.2, 12.8.1
4	Section EC2 7.3.4	It should be made clear if also the bending reinforcement should be considered to determine $s_{r,max}$ for shear and torsional cracks in order to calculate crack widths.	10.3, 12.8.3
5	Paragraph EC2 9.2.2(6),(7) Paragraph EC2 9.3.2(4)	It should be clarified why the requirement for maximum longitudinal spacing of shear reinforcement units, e.g. bent up bars, differs between beams and slabs.	5.4
6	Annex EC2 J.3.2, Figures EC2 J.3 and J.4	The limitations of the reinforcement amount of concrete frame corners subjected to opening moment should be reviewed.	4.4, 12.2.3 11.3.2.8

Table 13.4 Topics where further research or investigation is needed.

No	Items in Eurocode 2	Description	Sections in report
1	Expression EC2 (6.20), Paragraph EC2 6.2.4(6)	The method of calculating the required transversal reinforcement amount in a section between web and flanges can be questioned why it needs to be further investigated.	7.3, 12.5
2	Paragraph EC2 6.3.2(2)	Identify the most favourable way to combine shear and torsional reinforcement by superposition.	6.4, 12.4.3
3	Expression EC2 (7.1)	Development of the minimum reinforcement requirement for crack control, based on previously performed master's thesis projects on the effective concrete area A_{ef} is requested.	10.2, 12.8.1
4	Section EC2 7.3.4	Find a reliable way and define how to determine widths of shear and torsional cracks in the serviceability limit state.	10.3, 12.8.3 11.3.2.1
5	Paragraph EC2 9.2.3(1) Paragraph EC2 9.2.2(3)	Investigate the effect of lap splicing of torsional reinforcement in webs on the load bearing capacity and performance of structural members.	6.3, 12.4.2 11.3.2.4

13.4 Derived and explained expressions

In this master's thesis project different expressions have been derived. In Table 13.5 the derived expressions are presented together with a description of what have been shown in this report.

Table 13.5 Topics where expressions have been derived or explained.

No	Items in Eurocode 2	Description	Sections in report
1	Expression EC2 (6.8) Expression EC2 (6.13) Expression EC2 (6.18)	Reinforcement requirements provided to resist shear force ensure that inclined compressive struts formed between shear cracks are balanced by transversal and longitudinal reinforcement.	5.2, 5.5
2	Expression EC2 (6.9) Expression EC2 (6.14)	The maximum shear reinforcement amount ensures that web shear compression failure is avoided and the expression is derived based on a cross-section with a rectangular web.	5.3
3	Paragraph EC2 6.2.3(8)	Design shear force from concentrated loads close to supports can be reduced since the shear force is resisted by both truss- and tied-arch action.	5.7
4	Expression EC2 (6.8), Expression EC2 (6.26) Expression EC2 (6.27)	Shear force and torsional moment are modelled in the same way in Eurocode 2 with the exception that torsion will also create a lever arm from each side wall of the cross-section.	5.2, 6.3
5	Expression EC2 (6.21)	The expression for the transversal reinforcement amount at a web-flange intersection is derived based on equilibrium between the inclined compressive struts and the transversal reinforcement.	7.3
6	Expression EC2 (6.25)	The upper limit of the requirement for design shear resistance at the interface of a concrete joint is stated in order to prevent crushing of the inclined compressive struts.	8.2
7	Expression EC2 (9.1N)	The expression for minimum flexural reinforcement ensures a ductile failure in the ultimate limit state and is derived based on a rectangular cross-section.	5.4
8	Paragraph EC2 9.2.2(6) Paragraph EC2 9.2.2(7)	The maximum longitudinal spacing of shear reinforcement units in beams ensures that each inclined shear crack will be crossed of at least one shear reinforcement unit.	5.4
9	Paragraph EC2 9.3.1.1(4)	The required longitudinal tensile capacity calculated by a horizontal shift $a_l = d$ of the moment curve is based on the assumption that no shear reinforcement is placed in slabs.	9.2
10	Not included in Eurocode 2	In this report derivations of expressions for shear capacity by dowel action are provided.	8.4

13.5 Clarified items

The reasons for the following requirements in Eurocode 2 have not been found during this master's thesis project. However, conclusions that have been drawn are stated in Table 13.6.

Table 13.6 Topics where the reasons for the requirement have not been found.

No	Items in Eurocode 2	Description	Sections in report
1	Expression EC2 (9.1N)	The lower limit of the expression implies that the minimum amount of reinforcement for concrete classes below C25/30 is independent of the concrete strength if the characteristic yield strength of reinforcement is 500 MPa.	4.2, 12.2.1
2	Paragraphs EC2 9.2.2 (2)-(4)	Shear reinforcement in the shape of stirrups or links must enclose all layers of bending reinforcement in order to enable a truss-like force pattern that can resist shear force and bending moment without utilising concrete in tension.	5.6, 12.3.2
3	Paragraph EC2 9.2.2(8)	The maximum transversal spacing of shear reinforcement enables forces to be evenly transferred across the width of the section and ensures that the transversal tensile component between the vertical bars, caused by the inclined stress field, can be resisted by transversal reinforcement.	5.4, 5.6, 12.3.2
4	Paragraph EC2 9.2.2(8)	In Sweden it is allowed to use only bent up bars as shear reinforcement. This will result in that the inclined stress-field will cause a transversal tensile component between the vertical bars which is not resisted by any reinforcement.	5.4, 5.6, 12.3.2,

13.6 Conclusions from survey and interviews

From the survey and the interviews general conclusions could be drawn which are presented in Table 13.7.

Table 13.7 Topics from the survey and interviews.

No	Description	Sections in report
1	Even highly experienced structural engineers are not always in agreement on how to interpret some of the rules and guidelines in Eurocode 2.	12.2.2, 12.4.3
2	A more continuous cooperation between contractors and structural engineers is desired according to contractors.	12.9
3	Significant differences between the answers obtained from structural engineers working with design of houses and industrial buildings and structural engineers working with design of bridges and tunnels have been identified.	12.10 11.3.2.1, 11.3.2.3-5, 11.3.2.8,

14 Conclusions

14.1 Concluding remarks

This master's thesis project has consisted of compiling, explaining and deriving expressions and paragraphs in Eurocode 2 that might be difficult to interpret and know the background to. The usage and interpretation of Eurocode 2 have also been evaluated in order to be able to identify need for and suggest improvements of the standard. Some of the identified problem areas in Eurocode 2, where need for improvements have been identified, are lapping of longitudinal reinforcement, concrete frame corners subjected to opening moment and limitations of crack widths for concrete members subjected to shear and torsion.

The overall general conclusions drawn from this master's thesis project are:

- Background information to the rules and guidelines in Eurocode 2 is very difficult to find.
- Rules and guidelines in Eurocode 2 are interpreted differently depending on the professional background of the structural engineer.
- Some parts of Eurocode 2 need to be clarified or explained more thoroughly to know how to apply the rules on other situations than standard cases.

Eurocode 2 is written in a very concise way why it sometimes can be difficult to know how to apply the rules and guidelines for cases other than standard cases. A lack of references between different sections in the code has also been identified. It should be noted that it is important that the standard provide rules in a short and concise way in order to make it manageable for structural engineers in their daily work. However, the background information in ECP (2008a) needs improvements, where for instance comments concerning Chapters EC2 8 and EC2 9 are totally missing today. It is also desirable to include additional information in the code explaining shortly the motive or the basic conditions for the provided expressions.

A survey directed to active structural engineers working with design of reinforced concrete structures has been carried out during this project. The result from the survey indicates that structural engineers follow the statements in Eurocode 2 thoroughly, even if some of the rules and presented solutions in the standard are only given in form of recommendations. The structural engineers are not always aware of the background to the expressions. From the survey it can be concluded that it sometimes is difficult to know how to perform the design in a correct way, if there are no clear recommendations in Eurocode 2. In the end this might lead to interpretations of what is an acceptable and not acceptable reinforcement solution that differs between structural engineers. Some expressions and paragraphs in Eurocode 2, that need improvement, have therefore been identified and are gathered in this report.

From the interviews many interesting thoughts about the background to the rules and requirements in Eurocode 2 were derived. However, some questions still remain unanswered. It is noteworthy that some parts of Eurocode 2 are not entirely clear even to highly experienced structural engineers and it can be questioned how inexperienced designers in such situations should be able to make correct interpretations and applications of the requirements written in the code.

14.2 Further investigation

This project has been focusing on the interpretation of the theoretical parts of Eurocode 2. Therefore, the authors would like to suggest that in a further master’s thesis, investigate the needs and opinions of the contractors, in order to get a more practical point of view. The authors’ belief is also that actual examples of proper detail solutions are desirable within the industry.

The following questions are proposed as a basis to a further master’s thesis project:

- Can detail solutions that are time consuming or often generates problem at the construction site be identified, and how can they be improved?
- How can the cooperation between the structural engineers and contractors be increased?

There are also many question marks that still remain from this master’s thesis that can be interesting to investigate even further by for example calculations, experimental tests or FE-modelling.

Examples of what the authors think is interesting to investigate even further are:

- Torsion reinforcement –What effect do lap splicing in webs have in case of torsion compared to shear, and does the configuration or anchorage of stirrups matter more if the reinforcement is used as torsional reinforcement?
- Superposition of shear and torsion –What is the most favourable way of combining longitudinal shear- and torsional reinforcement, and how should design and detailing be performed?
- Limitation of crack widths for shear and torsion –How can the methodologies in Eurocode 2 for calculation of crack widths be altered so that they are valid also for shear and torsion?
- Crack reinforcement in beams with relatively large cross-sections –Can the expression for minimum crack reinforcement be further developed so that it considers the effective area, A_{ef}, instead of the concrete area in tension before cracking, A_{ct}?

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Appendix A Compilation of ambiguities treated in the report

Section	Reference	Comment	Question
Concrete cover 4.4.1.1 (2)		c_{nom} Nominal concrete cover	See 4.4.1.3 (1)
Minimum concrete cover, c_{min} 4.4.1.2 (1), (2), (3)	Table 4.2	c_{mindur} concrete cover due to environmental conditions. (corrosion of reinforcement) $c_{min,b}$ concrete cover due to bond requirements	How has the minimum cover been determined?
Allowance in design for deviation 4.4.1.3 (1)		Δc_{dev}	Why is a deviation factor added to concrete thickness but not to distance between bars?
Plastic analysis for beams, frames and slabs 5.6.2(2)		Rotational capacity, required ductility $x/d < 0.25$	Derive requirement of ductility.
Rotation capacity 5.6.3(2)		Maximum reinforcement moment: $x/d < 0.45$. Calculate what this means for the reinforcement strain. Connect with ω_s. The ductility requirements should ensure that the reinforcement yield with sufficient margin.	Derive requirement of ductility.
Shear, general verification model 6.2.1(4), (5)	9.2.2 minimum reinforcement amount	May be neglected in plates, where transversal load distribution is possible. Minimum reinforcement may be left out in members of minor importance. Is added even if the concrete capacity is sufficient. Shear reinforcement is necessary where $V_{Ed} > V_{Rd,c}$ Additional requirement for bridges, see 9.2.2(5).	Explain why minimum reinforcement is important.
Shear, general verification model 6.2.1(7)	6.2.3(7)	The longitudinal tension reinforcement should be able to resist the additional tensile force caused by shear	Should also be a reference to design of flexural reinforcement
Shear, general verification model	6.2.2(6) 6.2.3(8)	The design shear force does not need to be checked at a distance less than d from the face of the	Why is that? Illustrate?

Section	Reference	Comment	Question
6.2.1(8)		support	
Shear, general verification model 6.2.1(9)		Where the load is applied near the bottom of a section sufficient shear reinforcement should be added to carry the load to the top of the section	Concerns suspension reinforcement, see 9.2.5
Members not requiring design shear reinforcement 6.2.2(5)	9.2.1.3(2)	Region cracked in flexure, moment curve should be shifted a distance a_l=d in the unfavorable direction.	What is the background to this? Linked to a_l=d for plates, see 9.3.1.1(4)?
Members not requiring design shear reinforcement 6.2.2(6)		Reduction of shear force with β at a certain distance to support	Explain why such reduction can be made. See 6.2.3(8)
Members requiring design shear reinforcement 6.2.3(2)		The angle θ should be limited. The angle of the shear crack can be chosen when shear reinforcement is added. The amount of shear reinforcement decreases and the shear force in the tensile zone increases when the angel decreases. Chosen angle for shear force control affects the shift of the needed shear force capacity, see 9.2.1.3.	What do different angles mean for the final design?
Members requiring design shear reinforcement 6.2.3(3), (4)		Vertical shear reinforcement Maximum shear reinforcement	Derive (6.8), (6.9), (6.12), (6.13), (6.14) and (6.15).
Members requiring design shear reinforcement 6.2.3(7)		Additional longitudinal tensile force due to inclined shear cracks, ΔF_{td}	Derive Equation (6.18)
Members requiring design shear reinforcement 6.2.3(8)		Shear reinforcement close to the support: reduced reinforcement amount within the distance 0.75a_v from the support for concentrated load. This requirement results in that the shear crack does not miss the stirrup. Wise to always think about the placement of stirrups	Explain the limitations of a_v and derive (6.19).

Section	Reference	Comment	Question
		close to the support. Ensures that all the stirrups contribute.	
Shear between web and flanges 6.2.4	9.3.1 minimum reinforcement	Minimum amount of longitudinal reinforcement. Transverse reinforcement per unit length. Avoid crushing of struts in flange.	Derive (6.20), (6.21) and (6.22).
Shear at the interface between concrete cast at different times 6.2.5(1)	In addition to the requirements in 6.2.1-6.2.4	Addition requirements for shear stress in joint. V_{Ed} in (6.24) is not the total shear force but the shear force that acts over the joint. Based on friction model. Conditions for Equation (6.25) where the shear capacity is the design value: - Rough surfaces will give a joint separation w when shear sliding occurs. Gives tension in transversal steel bar. - Good anchorage between steel and concrete gives large steel strains (locally) for a small joint separation. Small bar diameter is better. If the conditions above are not fulfilled another action will take place; dowel action that will give a lower shear capacity. Bending capacity is the design value. Note that this is not included in EC2.	Derive (6.24) and upper limit for (6.25) Derive the shear capacity for a dowel.
Shear at the interface between concrete cast at different times 6.2.5(3)		The reinforcement should only be designed for the shear stress that is not taken care by cohesion and friction of the external normal stress. A stepped distribution of the transverse reinforcement is allowed.	Derive the conditions. Cannot find how to design the reinforcement.
Shear at the interface between concrete cast at different times 6.2.5(4)	6.2.5(1) longitudinal shear resistance 10.9.3(12)	Shear resistance of grouted joints	How should the c-factor in 6.2.5(1) be determined if the crack is closed by a compressive force?
Torsion, Design			Derive (6.26), (6.27), (6.28)

Section	Reference	Comment	Question
procedure 6.3.2			and (6.30)
Torsion, Design procedure 6.3.2(2)		Superposition of shear and torsion	Explain how this is performed
Design procedure 6.3.2(3)		Longitudinal reinforcement torsion reinforcement	How is the longitudinal reinforcement reduced due to the available compressive force?
Design procedure 6.3.2(4)		Combination of torsion and shear	Derive (6.28), (6.29), (6.30)
Design with strut and tie models, ties 6.5.3(2)		Reinforcement should be adequately anchored in the nodes	Reference to how the anchorage is performed?
Nodes 6.5.4(7)	8.4-8.6	Anchorage length of bottom reinforcement at support. Anchorage starts at the beginning of the node.	Compare to 9.2.1.4(3).
Crack control Limitation of crack width for shear and torsional cracks 7.3.1(2)		Crack widths limitation also concerns shear and torsion reinforcement. Shear and torsion cracks may not be too large. Established calculation method for this is missing, Johansson (2013) has proposed a method. Requirement was considered to be fulfilled if f_{yd}=250MPa is used in ULS. Not unusual with $V_{IV:A}>>V_{V:B}$. Can result in unnecessary large reinforcement amount Method in BBK truss model.	It does not exist an established method. Compare the different methods.
Crack control, minimum reinforcement areas 7.3.2 (1), (2)	7.3.3(2)	Stress in reinforcement, σ_s. This requirement implies that there is a theoretical possibility to achieve a second crack at restraint loading. No guaranty to achieve a crack width that is small enough. A_{ct}, tensile concrete area for cracking. Important difference	Derive (7.1). When is it allowed to use a lower value for the steel stress?

Section	Reference	Comment	Question
		from BBK – harder requirements. For cross-section with varying width the controls are performed for different parts separately. K_c (7.2), (7.3) can give large differences in reinforcement amount at large compression forces. For instance T-shaped column and pre-stressing reinforcement in a bridge.	
Control of cracking without direct calculation 7.3.3 (2)	Rules in 9.3 solid slabs 7.3.2, table 7.2N, (7.1), table 7.3N	For minimum reinforcement according to 7.3.2 the crack widths are probably acceptable if the bar diameter is limited according to table 7.2N	Explain table 7.2N, 7.3N.
Calculation of crack width 7.3.4 (1), (2), (3)		Crack width, w_k. Bonded reinforcement $S_{r,max}$ (7.11). Not bonded reinforcement $S_{r,max}$ (7.14).	Discuss (7.8)-(7.10).
Spacing of bars 8.2		The distance between the bar layers should allow access for vibrators and good compaction of the concrete. Reinforcement in many layers should be located vertically above each other. Lapped bars may be allowed to touch one another within the lap length.	No explicit instructions concerning casting gaps?
Permissible mandrel diameters for bent bars 8.3(3)		It exist cases when these conditions does not need to be fulfilled. May be spalling of concrete if the conditions are not fulfilled. Will give larger radius than the reinforcement requirements. In Equation (8.1) the shape or the angle of the bend is not considered. Difference from BBK 04	Does it exist cases when the anchorage can be smaller than 5φ? Derive (8.1)? a_b, perpendicular to the plane of the bend?
Ultimate bond stress 8.4.2 (2)		Ultimate bond stress, f_{bd}	Derive or explain (8.2)
Basic anchorage length		Basic anchorage length, $l_{b,rqd}$	Derive or explain (8.3)

Section	Reference	Comment	Question
8.4.3 (2)			
Design anchorage length 8.4.4 (1)	8.6: for indirect supports	Design anchorage length, l_{bd}. Simplified alternative to 8.4.4(1), equivalent anchorage length.	Derive or explain (8.4). Tab 8.2, What is the intention with the reference to 8.6?
Laps and mechanical couplers, laps 8.7.2 (2), (3), (4)	Figure 8.8 lapping 50%	Interpretations of EC2 recommendations. Figure 8.7design of laps, important difference to BBK. All bars in compression and secondary (distribution) reinforcement may be lapped in one section. The given condition contradicts each other.	How should the requirements be interpreted? Is it allowed to lap 100 % of the bars in tension in one section?
Lap length 8.7.3 (1)		Lap length is the same as $\alpha_6 l_{bd}$.	Explain (8.10) and (8.11).
Minimum and maximum reinforcement areas 9.2.1.1(1)	7.3 Should also be reference to Appendix E	Minimum reinforcement. Minimum reinforcement for tension reinforcement ensures that the moment capacity in stadium I is smaller than that in stadium III. For structural members with less importance lower requirements are accepted. Cross-section with less reinforcement is considered to be unreinforced. The purpose is to prevent brittle failure. Suitable minimum amounts to use in slabs.	Derive (9.1N)
Minimum and maximum reinforcement areas 9.2.1.1(3)		Maximum reinforcement There is no limit for A_{smax} in Sweden (NA).	Why does A_{smax} exist?
Curtailment of longitudinal tension reinforcement 9.2.1.3(1),(2)	6.2.3(7)	Shear force reinforcement, additional tensile force, ΔF_{td}. Required tensile force: the effect of inclined cracks in web and flanges can be taken into account in two ways: - Horizontal shift of moment curve by a_l	Derive (9.2)

Section	Reference	Comment	Question
		- Vertical addition to moment curve by ΔF_{td}. Derivation by means of truss model.	
Anchorage of bottom reinforcement at end support 9.2.1.4(1)		Bottom reinforcement at end support, indirect support. 25% of maximum span reinforcement should be provided to end and mid support. Requirements of minimum anchorage length at mid support.	See 9.3.1.2
Anchorage of bottom reinforcement at an end support 9.2.1.4(2)	6.2.3(7) or (9.3)	Tensile force that needs to be anchored	Derive (9.3). What is the difference to 6.2.3(7)?
Anchorage of bottom reinforcement at an end support 9.2.1.4(3)	8.4.4	Anchorage at support, section where the beam meets the support is controlled. Figure 9.3. Critical shear crack from support edge.	Where should the anchorage be calculated from? Compare to 6.5.4(4) and figure 6.27?
Shear reinforcement 9.2.2(1)		Shear reinforcement should form an angle α between the interval 45-90º	What effect do different angles have on design solutions?
Shear reinforcement 9.2.2(2)		Shear reinforcement should consist of a combination of links enclosing the longitudinal tension reinforcement and the compression zone, Bent-up bars, cages, ladders etc. Show suitable direction of G-bars.	Are G-bars taken as bent-up bars? See 9.2.2(4)
Shear reinforcement 9.2.2(3)		A lap joint in the leg near the surface of the web is permitted provided that the links is not required to resist torsion. Will create problem, see 9.2.3(1). Difference between EC and BBK for lapping of stirrups.	Why is it not allowed to lap torsion reinforcement in the web? Must staggering according to 8.7.2 be followed?
Shear reinforcement 9.2.2(4)	Additional information in NA	At least β_3=50 % of the necessary shear reinforcement should be in the forms of links. In Sweden it is allowed to use 100 % bent up bars.	Why is it in Sweden allowed to use 100 % bent up bars? What is the motive for the recommendation?
Shear		Additional information for	Derive (9.4), (9.5N)

Section	Reference	Comment	Question
reinforcement 9.2.2(5)		bridges in NA (SIS). Additional information for wide webs in NA (VVFS). Minimum amount of shear reinforcement should be provided in sections even if the shear capacity is sufficient there, see 6.2.1(4).	
Shear reinforcement 9.2.2(6),(7)		Maximum distance in the longitudinal direction. (6), (7): $s_{l,max}$<0,75d is used to ensure that no cracks will appear in between the cracks Maximum distance, bent-up bars $s_{b,max}$<0.6d Additional information in NA, new value for $s_{b,max}$ <0,75d.	Derive (9.6N) and (9.7N) Why do requirements for links and bent up bars not differ in Sweden?
Shear reinforcement 9.2.2(8)		Distance in the transverse direction in between the shear reinforcements links should be larger than s_{tmax}. s_{tmax}<min(0,75d, 600mm). Will ensure an even distribution of forces in the transverse direction.	Derive (9.8N)
Torsion reinforcement 9.2.3	9.2.2(5), (6)	Requirements regarding splicing/anchorage. Configuration. Closed links, anchorage, lapping or end hook. Minimum requirements are the same as for shear force. Special requirements regarding spacing of torsion reinforcement in the longitudinal direction and distribution of it. Placement of longitudinal reinforcement, maximum distance of 350 mm. One bar in each corner. Evenly distributed inside the stirrups. Required tensile force in the longitudinal direction is over the whole cross-section.	What is the motive of the anchorage requirements? How should the recommendations be interpreted? Is it allowed to lap for instance two standing side links which is normally done in Sweden? Why do requirements differ from shear reinforcement? How should the longitudinal reinforcement be designed when shear and torsion are combined?
Indirect supports 9.2.5(1), (2)		Reference to figure 9.3b should be made. Suspension reinforcement should be added if the shear force needs	Are designers aware of the need for and how to place suspension reinforcement?

Section	Reference	Comment	Question
		to be lifted up. Suspension reinforcement: - Is required at indirect supports. - Is required if the load is not applied on top of the structure. - Will be in addition to other reinforcement. - The force needs to be lifted up to the top of the structure and will thereafter work as usual. - The self-weight below the shear crack can generally be ignored. Active area for suspension reinforcement is shown in figure 9.7. The stirrup should enclose the main reinforcement.	
Solid slabs, flexural reinforcement 9.3.1.1(4)	9.2.1.3(1)-(3), 9.2.1.4(1)-(3), 9.2.1.5(1)-(2)	Use a_l=d. Requirements for slabs with references to beams. The requirement regarding curtailments is the same as for beams. However, a_l=d is valid.	Illogical that the tensile force should be limited in this way. Why could the requirements not be the same as for beams?
Reinforcement in slabs near supports 9.3.1.2 (1)	9.2.1.3, 9.2.1.4, 9.2.1.5	Specific requirements for slabs that differs to those for beams. 9.2.1.4(1) contradicts 9.3.1.2(1) and thereby also 9.3.1.1(4). In 9.3.1.1(4) is the reference made to the method regarding beams. It is strange that it is not the same requirements for slabs and beams. According to 9.3.1.2(1) should 50 % of the field reinforcement go to the support. In the English version it is written “simply supported slab”. This should not be analogous with “enkelspända plattor” that is described in the Swedish version.	Contradiction, what requirement should be chosen? Why are there different requirements for beam and slabs?
Shear reinforcement 9.3.2 (2), (3)	9.2.2 6.2	Minimum reinforcement requirements for beams apply also for slabs. Other requirements apply if the some modifications are done If V_{Ed}< $1/3V_{Rd,\max}$ 100 % bent up	What are the motives for such modifications? Compare to 9.2.2(4)

Section	Reference	Comment	Question
		bars or shear reinforcement assemblies may be used	
Shear reinforcement 9.3.2 (4)		Maximum distance in the longitudinal direction, s_{max}.<0.75d for links s_{max}=d for bent up bars The requirements partly differ for the ones regarding beams	Unclear why the requirements for bent up bars deviates from the ones for beams? Is the angle of the crack considered differently in this case? Derive (9.9), (9.10)
Shear reinforcement 9.3.2(5)		Maximum distance in the transversal direction, 1.5d	Why is the maximum transverse distance longer for slabs than for beams?
Frame corners J.2		The specified recommendations are very high and wrong according to Johansson (2000). Opening moment is more dangerous than closing moment. Brittle failure will occur. The strut-and-tie model is not able to capture the real response. For design according to figure J.4 is ρ_{max}=1.2 % more reasonable instead of 2 %. The requirements give the impression that E-bar is not required for opening moment.	What configurations are used among designers?

Appendix B Compilation of ambiguities not treated in the report

Section	Reference	Comment	Question
Ductility characteristic 3.2.4 (2)	Annex C	Class A B and C	What are the classes influenced of? See also fig 3.8
Design assumptions 3.2.7 (2)		Two different types of working curves can be chosen.	When to choose which curve? See also fig 3.7
Exposure classes 4.2 (3)	Annex E : 7.3.2, 9.2.1.1 Tab 4.1: 7.3.2-7.3.4	Not the same reference in Annex E as in Table 4.1	
Minimum cover, 4.4.1.2 (5), (7), (8)	Annex NA	Table 4.3N, 4.4 are not used, c_{mindur} is determined according to NA Additional information in NA (VVFS, BFS)	
Minimum cover, 4.4.1.2 (11)		Increased concrete cover for rough surfaces	Why should the cover be increased and how should it be measured?
Concrete cover, allowance in design for deviation 4.4.1.3(4)	Annex NA	There are other values in NA (SIS, VVFS)	
Effective span of beams and slabs in buildings 5.3.2.2(3)		Beam or slab monolithic with its supports. The critical design moment should be taken as that of the face of the support.	What is the definition of monolithic?
Effective span of beams and slabs in buildings 5.3.2.2(4)		Where a beam or slab is continuous over a support the design support moment may be reduced	Explain the reason for the reduction
Linear elastic analysis with limited redistribution 5.5(4)		Redistribution of bending moments may be carried out without an explicit check of the rotation capacity	Derive why the simplification is ok.
Linear elastic analysis with		Redistribution should not be carried out in circumstances	Difficult to know when to

Section	Reference	Comment	Question
limited redistribution 5.5(5)		where the rotation capacity cannot be defined with confidence (e.g. in the corners of prestressed frames)	apply?
Linear elastic analysis with limited redistribution 5.5(6)		For the design of columns the elastic moments from frame actions should be used without any redistribution	Can this be illustrated?
Plastic analysis, general 5.6.1(2)		5.6.1 is relevant reading The ductility of the critical sections shall be sufficient.	How do the designer control this? Reference to 5.6.2(2)?
Analysis with strut and tie models 5.6.4(5)		Wrong translation from English to Swedish version! Load path method is translated as “stegvis pålastning”	
Method based on nominal stiffness 5.8.7.2(2),(3)	7.4.3	K_c and K_s are determined differently depending on reinforcement ratio ρ ρ > 0,002 or ρ > 0,01	What is the background to K_s and K_c for the two cases? What to do if the requirements are not fulfilled?
Shear, general verification model 6.2.1(4)	9.2.2 minimum reinforcement amount	May be neglected in plates where transversal load distribution is possible. Minimum reinforcement may be left out in members of minor importance. Is added even if the concrete capacity is sufficient. Additional requirements for bridges, see 9.2.2(5).	Explain why minimum reinforcement is important
Members not requiring design shear reinforcement 6.2.2(1), (5)		The design value for shear resistance $V_{Rd,c}$ The shear reinforcement may be calculated for the smallest value of V_{Ed} within a distance l	Derive expression? Derive expression for l
Torsion 6.3.1(2)	7.3 crack control 9.2 beam	Minimum reinforcement Torsion can be ignored in ULS if: - It arise from compatibility deformations - Minimum reinforcement is fulfilled according to 7.3 and 9.2.	Explain why this is the case
Design procedure	9.2.1.1 minimum	Approximately rectangular solid sections	Explain (6.31) Can it be rewritten for other

Section	Reference	Comment	Question
6.3.2(5)	reinforcement		cross-sections?
Design with strut and tie models, struts 6.5.2(3)	6.2.2, 6.2.3 alternative calculation methods	For struts between directly loaded areas such as corbels, short deep beams alternative calculation	What is the definition of a directly loaded area? What calculation methods are referred to?
Design with strut and tie models, ties 6.5.3(3)		Ties, tensile force, strut and tie model/load path method Figure 6.25: b_{ef} for partial and full discontinuity respectively.	Derive (6.58), (6.59)
Nodes 6.5.4 (4), (5), (6)	(3.24), (3.25)	Maximum stress in the node region Increase of design compressive stresses if certain conditions are fulfilled Tri-axial compression	Derive (6.60)-(6.62) Explain the conditions Explain why the expression can be used?
Anchorage and laps 6.6(2), (3)	8.4 to 8.8	Assumes constant bond stress	How does this differ from reality?
Stress limitations 7.2(2), (5)	Tab 4.1 exposure classes Annex NA	Longitudinal cracks. In order not to increase the concrete cover or confine the concrete by transverse reinforcement a limitation of the compressive stress can be done Tensile stresses in the reinforcement. Value on k_3 is found in NA (VVFS, BFS).	
Crack control, general considerations 7.3.1(5), (9)	Another table in Annex NA that should be used 7.3.4 or 7.3.3	Maximum crack width in Table 7.1N w_{max} is calculated according to NA (VVFS) Crack widths may be calculated according to the references	If BFS does not give any regulation, should VVFS be ignored? What are the preferences?
Control of cracking without direct calculation 7.3.3(1)	Rules in 9.3 solid slabs	Specific measures to control cracking is not necessary if h<200mm and if the provisions of 9.3 have been applied	
Control of cracking without direct calculation 7.3.3(3)	7.3.2(2), 7.3.4	Deep beams >1m should be provided with surface reinforcement	Insufficient recommendation, how should A_{ct} be determined for this surface

Section	Reference	Comment	Question
Control of cracking without direct calculation 7.3.3(4)	Chapter 8, 9 Requirements in these chapter are valid	Risk of large cracking occurring in sections where there are sudden changes of stress.	Illustrate the stress change?
Control of cracking without direct calculation 7.3.3(5)	9.2.2, 9.2.3, 9.3.2, 9.4.3	Control of cracking due to tangential action effects. Detailing rules	What is tangential action effect? Give examples
Calculation of crack width 7.3.4 (3)		Crack width, *w*, at concrete surface relative to distance from bar Another value for k_3 in NA.	Is the limit for reasonably close centers within the tension zone < 5(c+φ/2)? What is Figure 7.2 explaining?
Calculation of crack width 7.3.4 (4)		Reinforcement in two directions S_{rmax} (7.15) When does this case occur? How is the angle between the axis of principle stresses calculated?	Derive (7.15)?
Calculation of crack width 7.3.4 (5)	7.3.2	Early thermal contraction where the requirements in 7.3.2 is not fulfilled; $s_{r,max}$=1.3h	What is the background to this requirement?
Cases where calculations may be omitted 7.4.2(2)	7.4.1(4), 7.4.1(5), 5.3.2.2(1)	Deflection of reinforced concrete. The paragraph is difficult to read	Derive (7.16)
Checking deflections by calculation 7.4.3(3)		Describes the concept tension stiffening	Derive expression?
Detailing of reinforcement and prestressing tendons, general 8.1(2)	4.4.1.2 minimum concrete cover	Minimum concrete cover.	Where do the requirements come from?
Permissible mandrel diameters for bent bars 8.3(2)	6.8.4	The minimum diameter to which a bar is bent shall be such as to avoid bending cracks the bar, and to avoid failure of the concrete inside the bend of the bar. Additional information in NA. NA allows smaller bending radius	

Section	Reference	Comment	Question
		if certain conditions are fulfilled. The bending radius affects the fatigue strength of the reinforcement, see tab 6.3N.	
Anchorage of longitudinal reinforcement 8.4.1(1)		Transversal bars shall be provided if necessary. Reduces the risk of spalling of the concrete.	When is it necessary to provide transversal bars?
Anchorage of longitudinal reinforcement 8.4.1(2)	8.8(3) methods of anchorage	Bent bars anchored in concrete affects the anchorage length relatively little. Failure in concrete at the inside of the bent bar should be avoided.	Clarify the figures and l_b, l_{bd}, l_{beq}, l_{bdrqd}? What is the difference and how should the figures be interpreted?
Anchorage of links and shear reinforcement 8.5(1) (2)	8.6(2)	A bar should be provided inside a hook or bend. Detail of links: requirements differ from bent bars in concrete. Depends probably on that the link holds on to a bar. Smaller requirements than in BBK. Fig 8.5	The anchorage of a bent bar around a bar is probably more effective than anchorage only in the concrete?
Anchorage by welded bars 8.6(1)-(5)		The quality of the welded joints should be shown to be adequate The anchorage capacity of one welded transverse bar welded on the inside of the main bar Rules for welding of bars.	What does this mean for the structural engineer? Derive (8.8N) Derive (8.9).
Transverse reinforcement for bars in tension 8.7.4.1(3)		Transversal bars when lapping. For lapping >50% in one section stirrups might be anchored inwards in the cross-section New requirements compared to BBK.	How should ΣA_{st}, be calculated, should all the shear reinforcement be included?
Transverse reinforcement for bars in tension 8.7.4.2		Additional rules to the ones concerning tension reinforcement One bar of the transversal reinforcement should be placed at each side of the lap.	Why should the bar be placed like that?
Laps of the main reinforcement 8.7.5.1(5)	6.1, Tab 7.2 and tab 7.3	Increase the steel stress with 25 % for crack verification.	Difficult to understand, illustrate?
Laps of secondary or distribution		The lap of two secondary bars should be crossed by two main	Is this possible to achieve in reality?

Section	Reference	Comment	Question
reinforcement 8.7.5.2(1)		reinforcement bars	
Additional rules for large diameter bars 8.8(1)-(8)	8.4 & 8.7 9.2.4 or calculations of crack widths 7.3.4 9.2.4	Additional rules concerns also φ_n for bundled bars. Control of crack widths, surface reinforcement Dowel action is larger for large diameter bars Bundled bars should generally not be lapped Transversal reinforcement should be added to the shear reinforcement in anchorage regions Surface reinforcement	References might be needed Only place in EC2 where dowel action is mentioned Instead of lapping, how should it more preferably be done? Derive (8.12), (8.13) Background to the requirements for surface reinforcement?
Bundled bars 8.9.1 (1), (4)		The rules for individual bars also apply for bundles of bars. Equivalent diameter Where two touching bars is positioned one above the other such bars need not to be treated as bundle. The text in 8.9.2(2) and figure 8.12 should describe the same thing.	Derive (8.14) Difficult to achieve or make sure the correct placing in reality? Why is l_b used in the figure and $l_{b,rqd}$ used in 8.9.2(2)
Anchorage of bundles bars 8.9.2(1), (2)	Figure 8.12	Reference to how to l_{bd} is calculated The text in 8.9.2(2) and figure 8.12 should describe the same thing.	Difficult to interpret the section Why is l_b used in the figure and $l_{b,rqd}$ used in 8.9.2(2)
Lapping of bundles bars 8.9.3 (1), (3)	8.7.3	Lap length calculated using equivalent diameter Bundles consisting of more than three bars should not be lapped.	Reference in both directions might be necessary How should splicing be performed?
Other detailing arrangements 9.2.1.2(1)	9.2.1.1(1) 5.3.2	In monolithic construction, even when simple supports have been assumed in design, the sections at the face of the supports should be designed for a bending moment $M_s > 0{,}15M_f$. Minimum reinforcement according to 9.2.1.1(1) is valid.	What is the definition of monolithic construction? What structures are monolithic in reality?
Other detailing arrangements 9.2.1.2(2)	5.3.2 effective flange width	Certain concentrations in web width. Support moment in flange, the reinforcement is spread over the	Does more exact direction exist?

Section	Reference	Comment	Question
		effective flange width according to 5.3.2	
Other detailing arrangements 9.2.1.2(3)		Any compression longitudinal reinforcement which is included in the resistance calculation should be held by transverse reinforcement in order to be able to take advantage of the compressed reinforcement. Stirrups prevent buckling of the compressed reinforcement.	
Curtailment of longitudinal tension reinforcement 9.2.1.3(3)		The resistance of bars within their anchorage lengths may be taken into account. ”As a conservative simplification this contribution may be ignored”.	How does Figure 9.2 change due to this?
Curtailment of longitudinal tension reinforcement 9.2.1.3(4)		Bent-up bar, anchorage length. Anchorage requirements for bent-up bars that take shear force. Different requirements in tensile and compressive zone.	Why is the anchorage length altered for bent up bars?
Anchorage of bottom reinforcement at an intermediate support 9.2.1.5	9.2.1.4(1) 6.6 more refined analysis	The same as for end support. Some additional requirements Minimum anchorage length Reinforcement in order to take care of the positive moment.	Explain additional requirements
Solid slabs, flexural reinforcement 9.3.1.1(1)	9.2.1.1(1)	Reinforcement in the main direction. Minimum reinforcement in slabs: the reinforcement in the principal direction is designed in the same way as for beams, see 9.2.1.1(1), (3).	Do solid slabs mean all slabs that not are flat slabs?
Solid slabs, flexural reinforcement 9.3.1.1(2)		Secondary transverse reinforcement, >20% of the principal main reinforcement for one way slabs. Special requirements at support. Will ensure distribution of forces in the slab. Reasonable to use this also for two way slab.	Derive/explain requirements
Solid slabs, flexural		The distance between bars should not exceed $s_{max,slabs}$.	Strange that the maximum moment should control this.

Section	Reference	Comment	Question
reinforcement 9.3.1.1(3)		Different rules for spacing for main and secondary transverse reinforcement. s_{max}<250 mm for areas with concentrated loads and maximum moment.	Does EC2 means maximum moment due to point load?
Reinforcement in slabs near supports 9.3.1.2 (2)		Partly fixity slab, top reinforcement should be 25 % of the field reinforcement	Compare to 9.2.1.2 (1) Monolithic construction
Reinforcement at the free edges 9.3.1.4		At a free, unsupported edge of a slab, transversal reinforcement as well as closed stirrups should be placed. This reinforcement may consist of reinforcement that is placed for other reasons Similar requirements for a supported edge do not exist.	Difficult to know how much reinforcement that is required at edges.
Shear reinforcement 9.3.2 (1)		Requirements on thickness of the slab and spacing in the longitudinal and transversal direction. Minimum 200 mm thickness.	Why should shear reinforcement not be placed in thin slabs?
Flats slabs, slab at edge and corner columns 9.4.2(1)		Reinforcement required to transfer bending moment	Derive b_e in Fig 9.9
Punching shear reinforcement 9.4.3(1) (2)	6.4, Fig 6.22, 6.4.5(4)	Shear reinforcement with regard to punching shear At least two perimeters of link legs should be provided Required area of one link leg $A_{sw,min}$	Explain the requirement and clarify Fig 9.10? Derive (9.11)
Columns, longitudinal reinforcement 9.5.2(2) (3)	Additional information in NA	Minimum longitudinal reinforcement amount $A_{s,min}$ A_{smin}=0.002A_c Maximum reinforcement No requirement for maximum reinforcement in Sweden	Derive (9.12N) Why is there no upper limit in Sweden?
Columns, transverse reinforcement 9.5.3(3), (4)		The longitudinal distance between transversal bars in a column should not exceed $s_{clt,max}$ $s_{clt,max}$ should in certain cases be	Explain requirements

Section	Reference	Comment	Question
		reduced. Transversal links are used to prevent buckling of the longitudinal reinforcement. An enclosing effect is created resulting in a more ductile response	
Walls, vertical reinforcement 9.6.2(1), (2), (3)	Additional information in NA	Minimum and maximum reinforcement $A_{s,min} = 0.002A_c$ No maximum reinforcement requirement, $A_{sv,max}$, in Sweden If the minimum reinforcement requirement controls in design the reinforcement should be evenly distributed between the two surfaces. This implies that walls should be double reinforced Maximum distance between vertical bars: max(3t, 400 mm)	Can $A_{s,min}$ be derived? Why is there no upper limit in Sweden? Is this an accurate interpretation? Reason for maximum distance?
Walls, horizontal reinforcement 9.6.3(1), (2)		Minimum horizontal reinforcement $A_{sh,min}$ in all surfaces: 25 % of the vertical reinforcement and not less than $0.001A_c$ Maximum spacing between adjacent bars is 400 mm.	Can $A_{sh,min}$ be derived? Reason for maximum distance?
Walls, transversal reinforcement 9.6.4(1)	9.5.3 9.53(4)	Requirement of links for large amounts of vertical reinforcement in analogy with columns, see 9.5.3 Transversal links are used to prevent buckling of the longitudinal reinforcement	
Deep beams 9.7(1), (2)	5.3.1(3)	Minimum reinforcement requirement of an orthogonal reinforcement mesh in each surface: $A_{sdb,min}$ Maximum spacing between adjacent bars : min(2t, 300 mm)	Derive $A_{sdb,min}$ Reason for maximum distance?
Tying systems 9.10	Additional information in NA	Tying system is designed in order to enable alternative load paths in a structure. This is to avoid a progressive collapse	Exemplify such reinforcement configurations

Section	Reference	Comment	Question
Properties of reinforcement suitable for use with this Eurocode C.1		Reinforcement class A-C Class B is most commonly used in Sweden	
Tension reinforcement expressions for in-plane stress conditions F.1(2)-(4)		Plane stress, one of the principal stresses is equal to zero.	When is this Section applicable?
Surface reinforcement J.1	8.8	For main reinforcement consisting of large diameter bars of bundles of bars. Surface reinforcement is added to prevent splitting failure. Minimum reinforcement requirements both parallel and perpendicular to the main reinforcement, $A_{s,surf,min}$. Surface reinforcement may be taken into account as bending and shear reinforcement	Derive $A_{s,surf,min}$
Frame corners J2.3		The requirements give the impression that E-bar is not required for opening moment.	

Appendix C Reinforcement amount for different ductility requirements

C.1 Steel strain at failure for different ductility requirements

The ductility requirements provided in Eurocode 2, see Equations (C.1)-(C.4), can be derived by the relationship between concrete and steel strain in the ultimate limit state seen in Figure C.1.

$$\frac{x_u}{d} \leq 0.45 \qquad \text{for concrete strength class} \leq \text{C50/60} \tag{C.1}$$

$$\frac{x_u}{d} \leq 0.35 \qquad \text{for concrete strength class} \geq \text{C55/67} \tag{C.2}$$

$$\frac{x_u}{d} \leq 0.25 \qquad \text{for concrete strength class} \leq \text{C50/60} \tag{C.3}$$

$$\frac{x_u}{d} \leq 0.15 \qquad \text{for concrete strength class} \geq \text{C55/67} \tag{C.4}$$

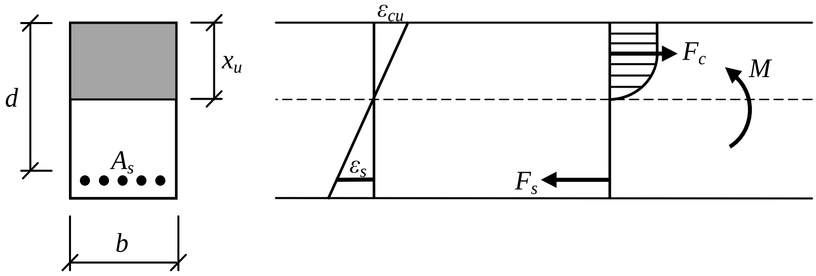


Figure C.1 Moment capacity of a cross-section in ultimate limit state.

It is of interest to find the steel strain, ε_s, when the concrete has reached its ultimate strain ε_{cu}. This can be calculated by the general expressions in Equations (C.5) and (C.6).

$$\varepsilon_s = \frac{d - x_u}{x_u}\varepsilon_{cu} \tag{C.5}$$

$$\frac{x_u}{d} = \alpha \tag{C.6}$$

$$\left.\begin{aligned} &\varepsilon_s = \frac{d - x_u}{x_u}\varepsilon_{cu} \\ &\frac{x_u}{d} = \alpha \end{aligned}\right\} \quad \varepsilon_s = \frac{1-\alpha}{\alpha}\varepsilon_{cu}$$

Hence, each criterion can be written as steel strain, ε_s, in relation to the ultimate concrete strain, ε_{cu}, see Table C.1

Table C.1 Ductility requirements written as steel strain, ε_s, in relation to ultimate concrete strain, ε_{cu}.

$\alpha = \frac{x_u}{d}$	0.45	0.35	0.25	0.15
ε_s [‰]	$1.22\varepsilon_{cu}$	$1.86\varepsilon_{cu}$	$3.00\varepsilon_{cu}$	$5.67\varepsilon_{cu}$

For concrete strength classes equal to or below C50/60 the ultimate concrete strain, ε_{cu}, is constant but for strength classes higher than C50/60 the ultimate concrete strain decreases, see Table C.2.

Table C.2 Ultimate concrete strain, ε_{cu}, for different concrete strength classes.

f_{ck} [MPa]	12-50	55	60	70	80	90
ε_{cu} [‰]	3.5	3.1	2.9	2.7	2.6	2.6

The steel strain, ε_s, for each ductility criterion must therefore be calculated for each concrete strength class respectively in order to relate it to the yield strain of reinforcement, ε_{sy}, see Table C.3 and Table C.4.

Table C.3 Steel strain corresponding to the ductility requirement $x_u / d = 0.45$ and $x_u / d = 0.35$.

f_{ck} [MPa]	12-50	55	60	70	80	90
$\frac{x_u}{d}$	0.45	0.35				
ε_s [‰]	4.3	5.8	5.4	5.0	4.8	4.8

Table C.4 Steel strain corresponding to the ductility requirement $x_u / d = 0.25$ and $x_u / d = 0.15$.

f_{ck} [MPa]	12-50	55	60	70	80	90
$\frac{x_u}{d}$	0.25	0.15				
ε_s [‰]	10.5	17.6	16.4	15.3	14.7	14.7

C.2 Reinforcement ratio, ρ, for x_u / d = 0.45

The moment capacity in the ultimate limit state of a rectangular reinforced concrete section, of width b and with tensile reinforcement at depth d, can be modelled as in Figure C.2.

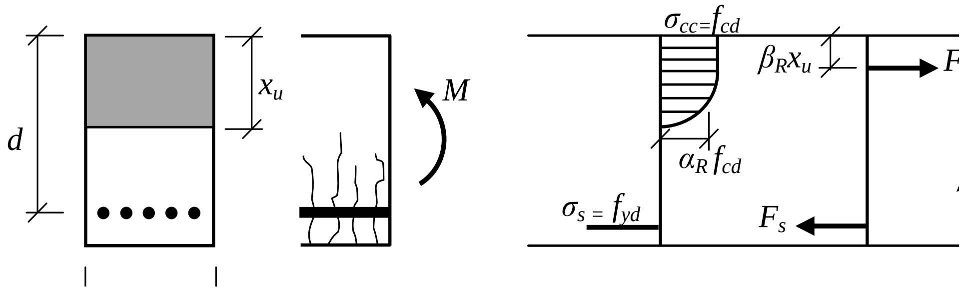


Figure C.2 Moment capacity of a cross-section in ultimate limit state.

An expression for the height of the compression zone, x_u, at failure can be derived from force equilibrium in the ultimate limit state, see Equation (C.7).

$$F_s = F_c \tag{C.7}$$

F_s force taken by the tensile reinforcement in ultimate limit state

F_c force taken by the compressed concrete in ultimate limit state

The force taken by the tensile reinforcement can be rewritten as in Equation (C.8). The characteristic reinforcement strength is assumed to be 500 MPa.

$$F_s = A_s f_{yd} \tag{C.8}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} \text{ MPa} \tag{C.9}$$

f_{yd} design yield strength of reinforcement

f_{yk} characteristic yield strength of reinforcement

γ_s partial factor for reinforcing steel at ultimate limit state

The compressive force taken by the concrete within the compression zone can be rewritten by the stress block factor, α_R, for a fully developed stress block.

$$F_c = \alpha_R f_{cd} b x_u \tag{C.10}$$

$$f_{cd} = \frac{f_{ck}}{\gamma_C} = \frac{f_{ck}}{1.5} \tag{C.11}$$

$$\alpha_R = 0.810 \qquad \text{for concrete strength class} \leq \text{C50/60} \tag{C.12}$$

f_{cd} design compressive cylinder strength of concrete

f_{ck} characteristic compressive cylinder strength of concrete

γ_C partial factor for concrete at ultimate limit state

α_R stress block factor for a fully developed stress block

The force equilibrium can now be rewritten as in Equation (C.13) and an expression for the compressive zone, x_u, can be derived, see Equation (C.14).

$$A_s f_{yd} = \alpha_R f_{cd} b x_u \tag{C.13}$$

$$x_u = \frac{A_s f_{yd}}{\alpha_R f_{cd} b} \tag{C.14}$$

The expression for the compressive zone, x_u, derived in Equation (C.14) is used to set up an expression for the ratio x_u / d

$$\frac{x_u}{d} = \frac{A_s f_{yd}}{\alpha_R f_{cd} b} \frac{1}{d} \tag{C.15}$$

The reinforcement ratio, ρ, is defined as

$$\rho = \frac{A_s}{bd} = \frac{A_s}{A_c} \tag{C.16}$$

By using the expression in Equation (C.14) the reinforcement ratio, ρ, can be rewritten as

$$\rho = \frac{x_u}{d} \alpha_R \frac{f_{cd}}{f_{yd}} \tag{C.17}$$

Transforming the design values of material strength into characteristic values by Equations (C.9) and (C.11) the reinforcement ratio, ρ, can be written as a function of the characteristic compressive strength of concrete, f_{ck}, see Equation (C.18). The characteristic yield strength of reinforcement, f_{yk}, is here assumed to be 500 MPa and α_R is set according to Equation (C.12).

$$\rho = \frac{x_u}{d} \alpha_R \frac{f_{ck}}{\gamma_C} \frac{\gamma_s}{f_{yk}} = \frac{x_u}{d} 0.816 \frac{f_{ck}}{1.5} \frac{1.15}{500} \tag{C.18}$$

f_{ck} in MPa

The ductility requirement for concrete strength class C50/60 and below can be seen in Equation (C.19), taken from Paragraph EC2 5.6.3(2).

$$\frac{x_u}{d} \leq 0.45 \qquad \text{for concrete class} \leq \text{C50/60} \tag{C.19}$$

The reinforcement ratio, ρ, can be determined by Expression (C.20).

$$\rho_{\leq C50/60} = 0.45 \frac{0.816 \cdot 1.15 \cdot f_{ck}}{500 \cdot 1.5} = 0.056304 f_{cd} \% \tag{C.20}$$

f_{ck} in MPa

The reinforcement ratio is dependent on the characteristic concrete compressive strength and must therefore be expressed for each class respectively, see Table C.5.

Table C.5 Reinforcement ratio corresponding to the ductility requirement $x_u / d = 0.45$ for different concrete types.

f_{ck} [MPa]	12	16	20	25	30	35	40	45	50
$\frac{x_u}{d}$	0.45								
$\rho = \frac{A_s}{A_c}$ [%]	0.7	0.9	1.1	1.4	1.7	2.0	2.3	2.5	2.8

Appendix D Concrete frame corners

D.1 Critical reinforcement amount, ρ

A simplified model concerning the critical reinforcement amount, ρ, is presented below, Johansson (2000). In this case all the data is determined from the geometry of the corner. Johansson (2000) derived an expression that is not dependent on tests and assumption. Here the stress field is assumed to have a triangular shape, instead of parabolic, to compensate for the large effective length.

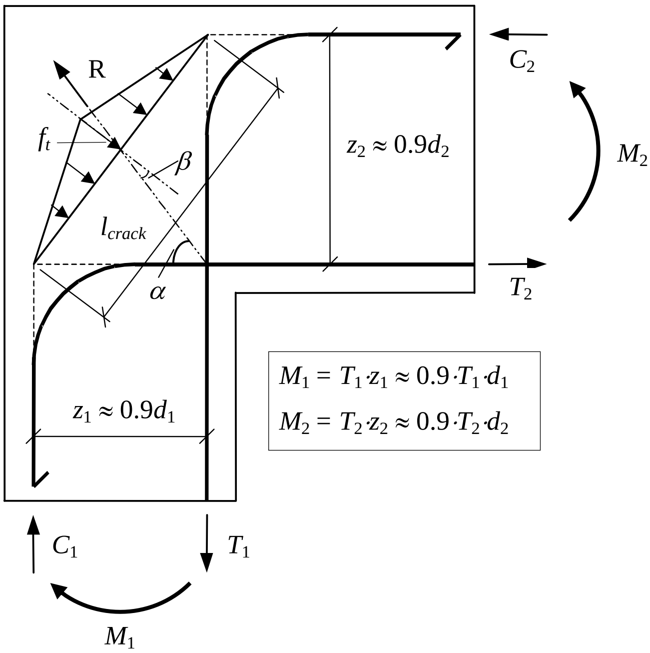


Figure D.1 The triangular stress distribution is assumed by Johansson (2000), which is used for the derivation of critical reinforcement amount. The figure is taken from Johansson (2000).

The effective length is assumed to be

$$l_{crack} = \sqrt{{z_1}^2 + {z_2}^2} \tag{D.1}$$

The diagonal force, R, acting in the corners is determined by Equation (D.2) when subjected to pure bending.

$$R = \sqrt{{T_1}^2 + {T_2}^2} = T_1\sqrt{1+\gamma^2} \tag{D.2}$$

T_1 tensile force in the smaller adjoining member

T_2 tensile force in the larger adjoining member

and the factor γ is obtained from the moment equilibrium $M_1 = M_2$, see Figure D.1

$$\gamma = \frac{z_1}{z_2} = \frac{T_2}{T_1} \tag{D.3}$$

A ductile behaviour will be reached if the reinforcement yield before the moment capacity is reached and if the corner is not over-reinforced. In this case the requirement of full efficiency is approximated to be fulfilled if the tensile reinforcement yields i.e.

$$R = T_1\sqrt{1+\gamma^2} = f_y A_{s,1}\sqrt{1+\gamma^2} \tag{D.4}$$

f_y yield strength of the reinforcement

$A_{s,1}$ cross-section area in member 1

The concrete resistance force, F_R, is, if assuming a triangular stress field, determined by

$$F_R = \frac{f_{ct}\cdot b\cdot l_{crack}}{2}\cos\beta \tag{D.5}$$

f_{ct} concrete tensile strength

b width of the corner

l_{crack} effective length

β angel between the direction of tensile force R and the normal to the assumed crack direction

By using the expression for l_{crack}, cos β can be expressed as

$$\cos\beta = 2\sin\alpha\cos\alpha = \frac{2z_1z_2}{z_1{}^2 z_2{}^2} \tag{D.6}$$

and setting

$$z = 0.9d_1 \tag{D.7}$$

d_1 effective height in the smallest adjacent member

The force F_R can now be rewritten as

$$F_R = \frac{0.9 f_{ct} b d_1}{\sqrt{1+\lambda^2}} \tag{D.8}$$

By setting $R = F_R$ the reinforcement amount, ρ, can be determined as

$$\rho = \frac{A_{s,1}}{bd_1} = \frac{0.9}{1+\lambda^2}\frac{f_{ct}}{f_y} \tag{D.9}$$

According to Equation (D.3) together with Equation (D.9) different size of the adjacent members will result in a higher resistance against cracking within the corner.

$$\gamma = \frac{d_1}{d_2} \le 1 \tag{D.10}$$

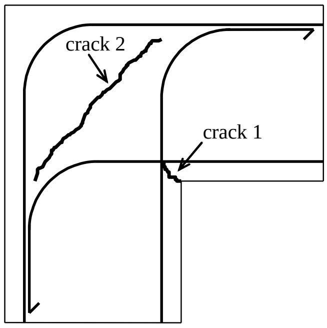
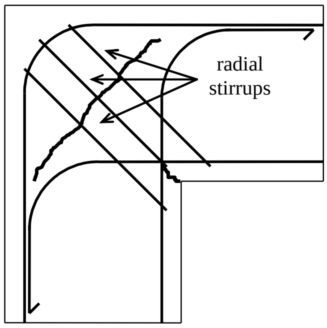
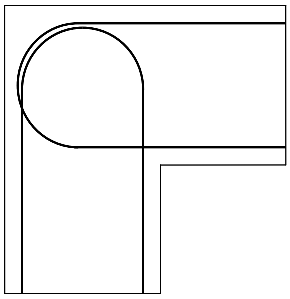
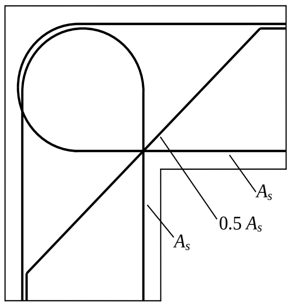
If the two adjacent members is of equal sizes the maximum reinforcement amount that is always allowed to use, while still reaching yielding in the reinforcement, is

$$\rho = \frac{0.45 f_{ct}}{f_y} \tag{D.11}$$

D.2 Summary of research performed by Johansson (2000)

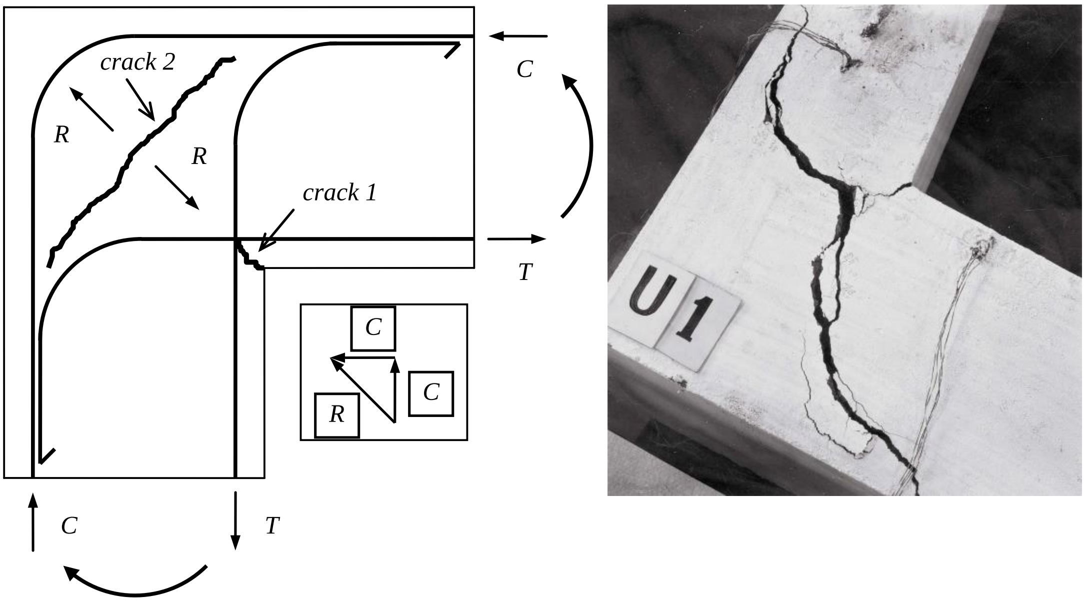
In Johansson (2000) a research on four different solutions of reinforcement arrangement was tested and investigated. In Table D.1 the solutions are summarized, to be further discussed in Section D.3 and D.4.

Table D.1 The investigated detail types used in Johansson (2000) research. The figures are taken from Johansson (2000).

<table>
<tr><th>Detail type</th><th>Comment</th><th></th></tr>
<tr><td rowspan="2">Type 1</td><td><u>Opening moment</u>
Poor detail – crack 2 will not be prevented</td><td rowspan="2">
</td></tr>
<tr><td><u>Closing moment</u>
Good detail - Less prone to fail by spalling of the side concrete cover</td></tr>
<tr><td>Type 2</td><td><u>Opening moment</u>
Poor detail – the radial stirrups is not fully anchored. Strut-and-tie model not valid.</td><td>
</td></tr>
<tr><td rowspan="2">Type 3</td><td><u>Opening moment</u>
Improved detail – the loops delay crack 2</td><td rowspan="2"></td></tr>
<tr><td><u>Closing moment</u>
Good detail - if there is sufficient side concrete cover to prevent spalling of the side concrete</td></tr>
<tr><td>Type 4</td><td><u>Opening moment</u>
Good detail – the diagonal steel bar delays crack 1 at the inner corner.</td><td>
</td></tr>
</table>

D.3 Structural response for the solutions of Type 1-4 subjected to opening moment

When performing the detailing of an opening frame corner it is not always that obvious to take the second inclined crack into account and Figure D.2 shows a detail where this has not been done. Unfortunately this detail was commonly used before. In this case it is no diagonal reinforcement that can resist the tensile force in the corner which will eventually push of the compressed concrete outside the tensile reinforcement. This results in a brittle failure since the compressive zone abruptly disappears. The concrete around the outside of the corner is in compression which means that the L-shaped reinforcement is also compressed and thereby is not used in a correct way, and hence may instead add to the "pushing off effect" of concrete in the outer corner. This means that without any diagonal reinforcement, the resistance of the corner depends on the concrete tensile strength to counteract the tensile force, *R*, in Figure D.2.



a) *b)*

Figure D.2 Detail Type 1, a) used in earlier practice which is not very successful, b) shows the typical crack pattern. The figure a) is taken from Johansson (2000). Photo is from Nilsson (1973).

By adding reinforcement perpendicular to the crack the tensile force, *R*, could be balanced and the failure of the corner is more prone to resist failure, see Figure D.3, Johansson (2000). This detailing depends, as for solution of Type 1, also on the concrete tensile strength, the reinforcement ratio and the yield strength. It has been shown that this detailing of reinforcement increases the efficiency of the corner and improves the structural behavior of the corner. To make sure that the reinforcement is able to take care of the whole tensile force, *R*, the needed amount can be determined according to Equation (D.4). Derivation of the tensile force is done in Section D.1. However, the radial stirrups are not anchored at the outside of the corner and equilibrium of the strut-and-tie model will not take place. In this case a critical crack may form outside the L-shaped reinforcement which will result in that the outer concrete is pushed off, see Figure D.3b.

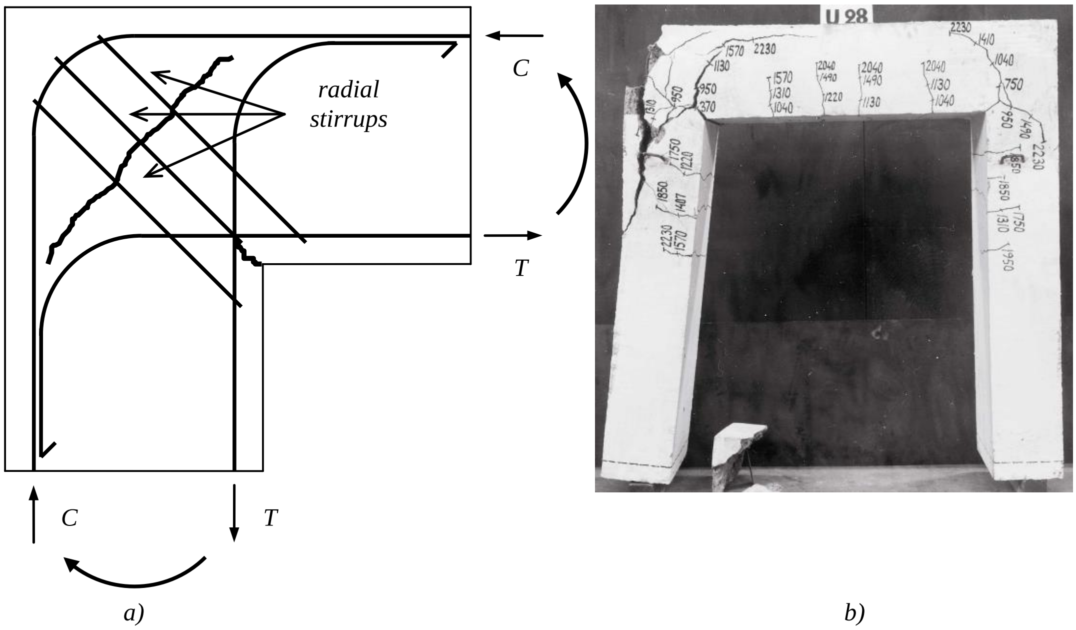


Figure D.3 Detail Type 2, a) where the stirrups are positioned perpendicular to the expected crack 2, b) the crack pattern at failure when using this solution. The figure is taken from Johansson (2000). Photo is from Nilsson (1973).

Then the solution of Type 3, in Figure D.4a, is a better solution, where the 180º bent bars provides some reinforcement perpendicular to crack 2, Johansson (2000). When the inner corner reinforcement is subjected to tension the loops prevents forming of the diagonal crack due to restraint. The reinforcement loops forces the tensile stresses to the outer corner, just as in the solution of Type 2, where the tensile stresses are less significant. However, instead of forming a diagonal crack within the corner the crack will grow along the reinforcement loops until reaching close to the compressive reinforcement and thereafter deviate and grow against the adjoining members, see Figure D.5. Also in this case the concrete corner outside the reinforcement will be pushed of and thereby result in decrease of capacity since the anchorage of the loops is reduced. However, despite this, a rather ductile behavior may still be obtained, see Johansson (2001) Paper I.

Many researches have been performed on the solutions of Type 2 and Type 3 which have resulted in comparable efficiency. However, the efficiency did not reach sufficient values for either detail solution. Due to this, improvement of detail solutions was wished for, and in Johansson (2000) it is referred to tests concerning this performed by Nilsson (1967, 1973). In Figure D.4b it is shown how the solution of Type 3 is strengthened by diagonal bars positioned at the inside of the corner. This additional reinforcement improves the structural behaviour in a manner where the first crack is delayed and hence also the propagations of the cracks outside the compressive zone. The experimentally tests was performed with different amount of inclined bars, in order to find an amount that will result in yielding of the bar, such that full capacity of the steel is utilized. This was fulfilled when 50 % of the main reinforcement was used, Karlsson (1999). It should be noticed that increased amount of main reinforcement would result in a similar increase of capacity. However, an advantage when using the diagonal bars may be to get cracking outside the compressive zone instead at the inside of the corner.

If there is a risk of spalling of the side concrete cover, when using one of the solutions in Figure D.4, Karlsson (1999) recommends adding radial stirrups, just as for the solution of Type 2, across the loop. Another favorable effect by doing this is that the stirrups will restrain the loops and hold them together when they are subjected to radial compressive stresses. The bar diameter also affects the spalling of concrete. It is preferable to use a small bar diameter since larger ones increase the risk of spalling.

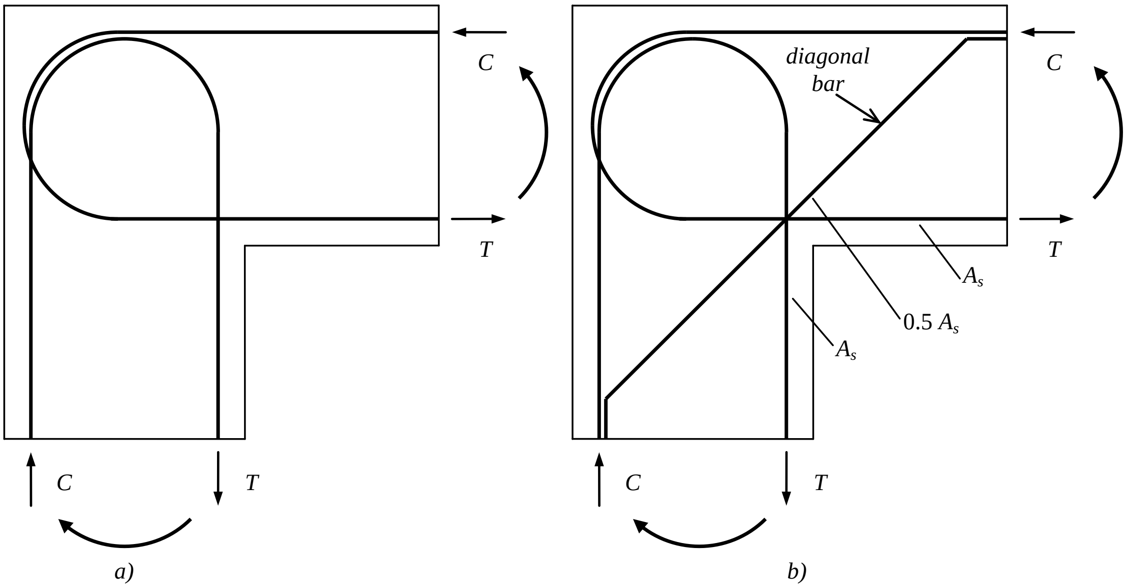
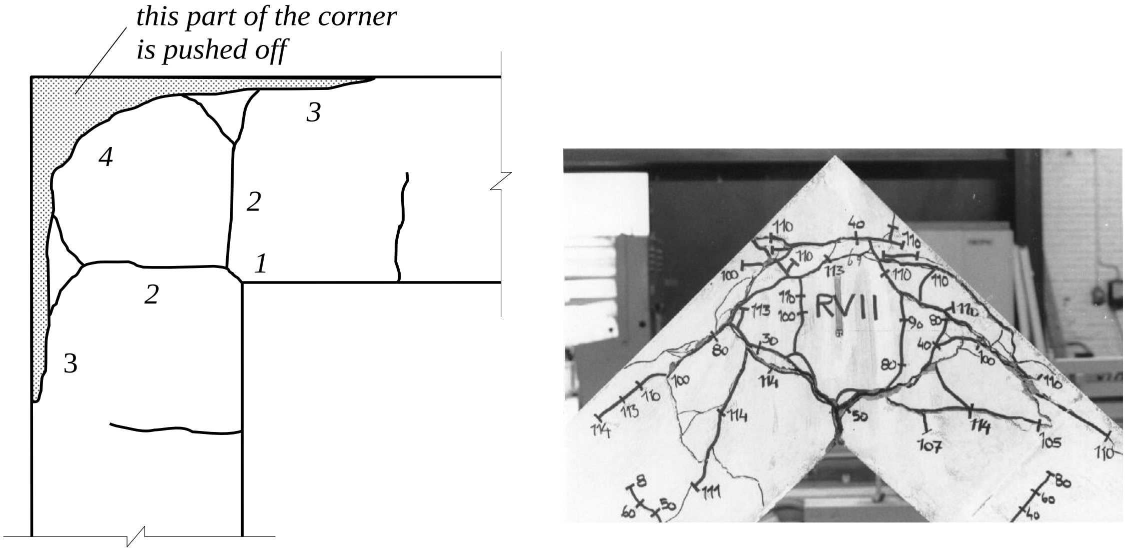


Figure D.4 *Arrangement of reinforcement for frame corners with opening moment, a) reinforcement loops are used in the solution of Type 3 to confine the concrete within the corner, b) the solution of Type 4 is further developed by adding diagonal bars used to strengthen the corner. The figure is taken from Johansson (2000).*



a) *b)*

Figure D.5 *How the crack pattern in frame corner develops when using the solutions of Type 3 and Type 4, a) the order of appearance of the crack propagation is shown by the numbers, b) the crack pattern when using these solutions. The figure is taken from Johansson (2000). Photo from Johansson and Karlsson (1997).*

D.4 Structural response for the solutions of Type 1 and 3 subjected to closing moment

Stroband and Kolpa (1983) have identified that for a corner subjected to closing moment there are problems with reaching full load capacity before failure due to that spalling of side concrete cover occurs. If the splitting stresses exceed the tensile strength of the concrete this might be the case, see Figure D.6. A large side concrete cover has a positive effect since it prevents the reinforcement, to some extent, to fail by spalling, Johansson (2000).

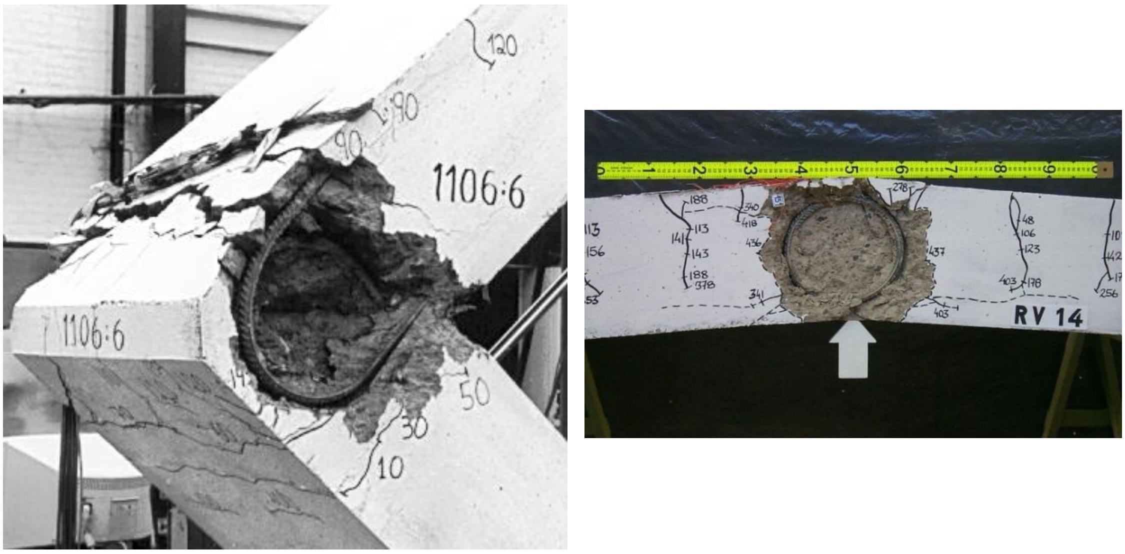


Figure D.6 Possible failure due to spalling of side concrete cover for a frame corner subjected to a closing moment. Photos from Johansson (1996) and Grassl (1999), respectively.

It is significantly easier to accomplish a reinforcement solution for a corner subjected to closing moment than opening moment. This is why there are several possible detail solutions where the forces still can be balanced even after cracking has occurred, for instance are the solutions of Type 1 and Type 3 both good options. The L-shaped detail in Type 1, see Figure D.2a, shows good performance regarding spalling of side concrete cover. However, experimental tests indicates that if a 180º bend of the bar is used, just as in the solution of Type 3, see Figure D.4a, there is an increased risk of this failure mode, see Stroband and Kolpa and Paper II. The reinforcement loop in the solution of Type 3 will cause higher radial compressive stresses which will result in splitting tensile stresses. An improvement of the detailing will occur if an increase of the side concrete cover is done. To restrain the tensile stresses perpendicular to the bend transverse reinforcement bars within the loops or radial stirrups around the loops may help.

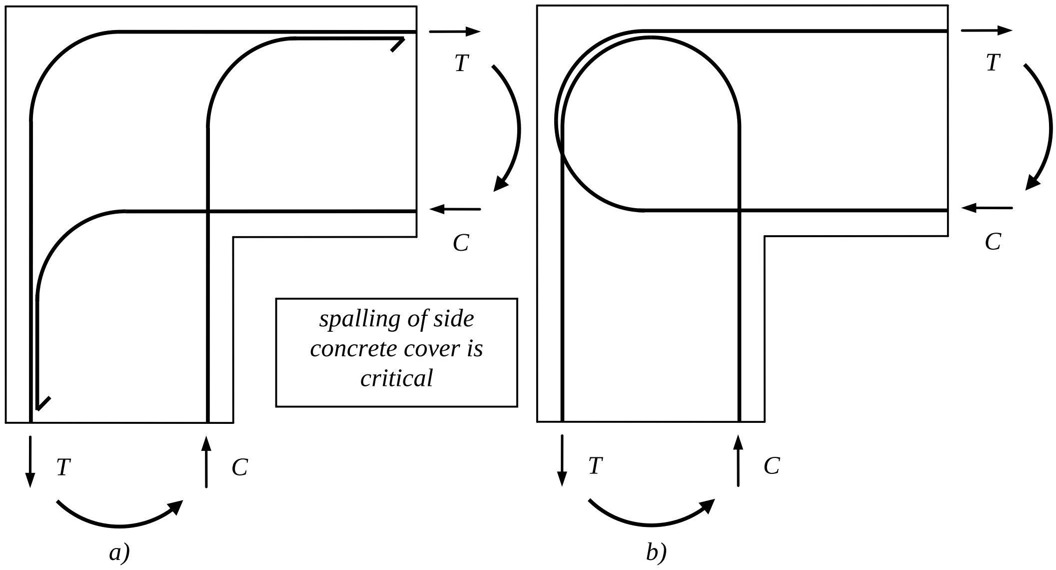


Figure D.7 Concrete frame corners subjected to closing moment where the solution of Type 1 in a) works well and the solution of Type 3 in b) creates high radial compressive stresses. The figure is taken from Johansson (2000).

According to Stroband and Kolpa in Johansson (2000) the load capacity will decrease suddenly, due to the loss of anchorage, if spalling of the side concrete cover will take place. This will happen when the splitting stresses reach the tensile strength of the concrete which will result in cracking in the plane of the loops and spalling of the side concrete. Johansson (2000) means that test has shown that this is not always the case. The interior reinforcement that still is anchorage in the concrete will delay the failure. Hence, this effect depends to a large degree on how much of the total reinforcement amount that is affected. If for instance, the concrete corner contains two bars with $\phi 6$ the failure will be brittle when the side concrete cover is spalled off. However, if only a limited number of bars are affected, the spalling effect will be little on the total load capacity.

Appendix E Action effect dependent on anchorage degree of transversal bars

E.1 Dowel action

In order to get the desired response of the dowel pin, where the failure mechanism is due to a combination of yielding of the steel and crushing of the surrounding concrete, a sufficient concrete cover should be designed using normal dimensions and strengths of the concrete and the steel.

The following cases can be distinguished regarding dowel action:

- Plain dowel pin with no end-anchors
- Plain dowel pin with end anchors (combination of dowel action/shear friction)
- Dowel pin anchored by bond (combination of dowel action/shear friction)

When a joint, which is designed with plain dowel pins with no end-anchors, is subjected to shear sliding the dowel pin is allowed to slide longitudinally inside the concrete, i.e. only flexural stresses are acting and the failure mechanism will be due to plastic hinges, see Figure E.1.

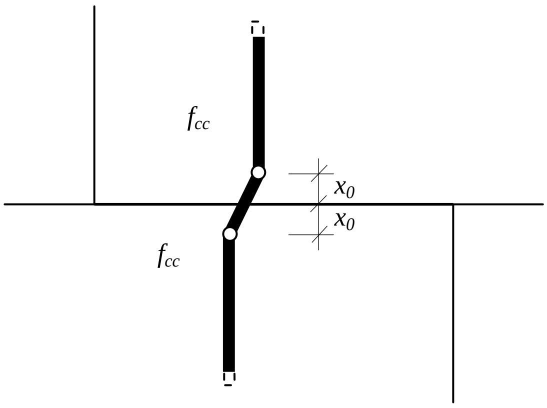


Figure E.1 Double-sided plain dowel pin across a joint interface (no joint gap) is equivalent to a one-sided dowel pin with no eccentricity. The figure is based in fib (2008).

If the plain dowel will be designed with end-anchors then there will be a combination of dowel action and shear friction. The dowel pin is not allowed to slide inside the concrete and will elongate uniformly along its length when it is anchored at each end. Axial restraint will develop as well as flexural stresses, see Figure E.2. The failure mechanism is due to plastic hinges but the dowel capacity will be reduced because of the axial restraint. However, the compressive force created over the joint will increase the friction between the interfaces and hence the overall shear capacity

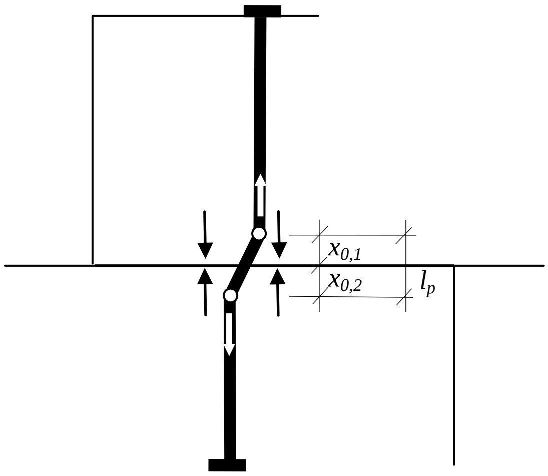


Figure E.2 Plain bar with end anchors. The shear displacement along the joint interface results in an elongation, n, of the dowel bar. The elongation can be assumed to be evenly distributed along the bar between the end anchors. The figure is based in fib (2008).

If the dowel pin is anchored by bond there will be a combination of dowel action and shear friction. This can be obtained with ribbed bars. The dowel pin will be loaded with an overall tensile force. However, in this case the stresses will be concentrated near the joint region instead of being evenly distributed along the dowel. This results in compressive stresses across the joint that will be high even for a small shear slip, which is favorable for the total shear capacity of the dowel pin.

E.2 Shear friction at a joint interface with transverse reinforcement

The size of the force that is developed in the reinforcement bar due to a joint separation depends on the anchorage of the bar. The failure due to shear friction will be by tension in the bar. In order to keep the amount of reinforcement as low as possible, without compromising the possibility to create a high self-generated compression, it is important to ensure high resistance of bar pullout.

The following cases can be distinguished regarding shear friction at joint interface with transverse reinforcement:

- Plain transverse smooth bar with no end-anchors
- Plain transverse bar with end-anchors
- Transverse bars embedded in the concrete
- External transverse bars not embedded in the concrete
- Interaction between pullout resistance and dowel action of transverse bars

For a plain smooth bar with no end-anchors it will, just as for the dowel pin, not create any significant axial stresses, i.e. no tensile force will develop that result in a compensating compressive force. The pullout resistance is therefore absent and the bar will fail by formation of plastic hinges due to bending.

For a plain bar with end-anchors it will, just as for the dowel pin, create tensile stresses that elongate the bar evenly along its length. The shear force will generate compressive stresses at the joint interface that clamps the members together.

For a transverse bar embedded in the concrete the bar will enable shear transfer by friction and the capacity is mainly dependent of bar pullout resistance, see Figure E.3. Tensile strain will localize near the region at the joint interface. The high tensile stresses close to the joint interface can obtain yielding for a small shear slip. This is something the designer wants to achieve. For the same shear slip and for the same steel area, the self-generated compressive force in case of a ribbed bar will be much higher compared to smooth bars. Ribbed bars provide higher resistance against shear loading. The shear force increases with increased shear slip until the bar reaches yielding or until the maximum joint separation, w_{max}, is reached. In this case it is generally not possible to take advantage of the dowel action since the bar will reach yielding because of the joint separation before any flexural deformation is developed. It should be noticed that a joint with smooth surfaces together with rough bars is not a preferable combination since it in such a situation not will be any joint separation, w, that results in pullout resistance.

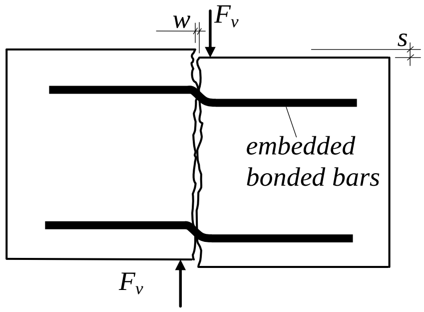


Figure E.3 The connection is tied together by embedded transverse reinforcement anchored by bond. The figure is based on fib (2008).

For an external transverse bar not embedded in the concrete the tensile rods will have an assumed uniform elongation, w, distributed between the end anchors, due to the shear slip, s. This means that the tensile stress will have the same magnitude across the whole joint. The tensile strain will therefore not be concentrated at the joint interface, see Figure E.4.

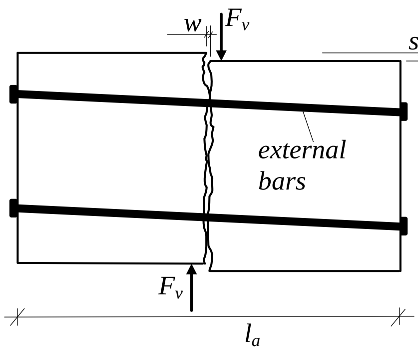


Figure E.4 The connection is tied together by external bars. The figure is based on fib (2008).

If the condition changes and the bar is instead plain and embedded in the concrete, still provided with end-anchors though, the behavior will be less stiff, see Figure E.5. The elongation can be assumed to be almost evenly distributed along the bar. In this case the significant contribution to shear resistance is due to dowel action. However, some shear friction due to wedging of the joint, also called aggregate interlocks effects, will still be achieved.

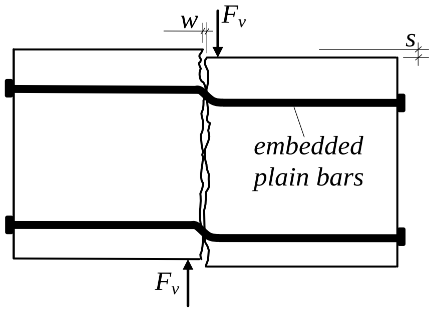


Figure E.5 *The connection is tied together by embedded transversal bars, anchored by end-anchorage. The figure is based on fib (2008).*

Appendix F Ultimate shear capacity due to dowel action

The ultimate shear capacity can be derived by equating the maximum moment, M_{max}, acting on the dowel and the plastic moment capacity, M_y, of the dowel pin. The expressions for the moments were derived in Section 8.4.2 and are repeated here in Equations (F.1) and (F.2).

$$M_{\max} = F_{vR} \cdot e + F_{vR} \frac{F_{vR}}{q_c} - q_c \frac{1}{2} \cdot (\frac{F_{vR}}{q_c})^2 = F_{vR} \cdot e + \frac{1}{2} \frac{{F_{vR}}^2}{q_c} \tag{F.1}$$

$$M_y = f_y \frac{\pi \phi^2}{8} \cdot \frac{4}{3} \frac{\phi}{\pi} = f_y \frac{\phi^3}{6} \tag{F.2}$$

Equating these two expressions gives

$$F_{vR} \cdot e + \frac{1}{2} \frac{{F_{vR}}^2}{q_c} = f_y \frac{\phi^3}{6} \tag{F.3}$$

$${F_{vR}}^2 + F_{vR} \cdot e \cdot 2 \cdot kf_{cc} \cdot \phi - f_y \frac{\phi^3}{6} \cdot 2 \cdot kf_{cc} \cdot \phi = 0 \tag{F.4}$$

$$F_{vR} = -e \cdot 2 \cdot kf_{cc} \cdot \phi \pm \sqrt{e^2 \cdot (kf_{cc})^2 \cdot \phi^2 + f_y \frac{\phi^4}{3} kf_{cc}} \tag{F.5}$$

$$F_{vR} = \sqrt{\frac{k}{3}} \cdot \phi^2 \sqrt{f_{cc} f_y} \left[\sqrt{1 + 3k \frac{f_{cc}}{f_y} \frac{e^2}{\phi^2}} - \sqrt{3k} \sqrt{\frac{f_{cc}}{f_y}} \frac{e}{\phi} \right] \tag{F.6}$$

Equation (F.6) can be simplified by defining an eccentricity factor γ as

$$\gamma = 3 \frac{e}{\phi} \sqrt{\frac{f_{cc}}{f_y}} \tag{F.7}$$

if this is inserted in the equation for shear capacity in case of eccentric loading the following expression is found

$$F_{vR} = c_0 \cdot c_e \cdot \phi^2 \sqrt{f_{cc} \cdot f_y} \tag{F.8}$$

c_0 coefficient that considers the bearing strength of concrete

$$c_0 = \sqrt{\frac{k}{3}} \qquad \text{(can be taken as } c_0 = 1{,}0 \text{ in design)} \tag{F.9}$$

c_e coefficient that considers the eccentricity

$$c_e = \sqrt{1+(\gamma \cdot c_0)^2} - \gamma \cdot c_0 \qquad \text{(F.10)}$$

If the eccentricity is set to zero in Equation (F.8), see Figure F.1, it can be shown that Equation (F.8) is equal to the expression for shear capacity presented in Section 8.4.1. See the derivation of Equation (F.14) below.

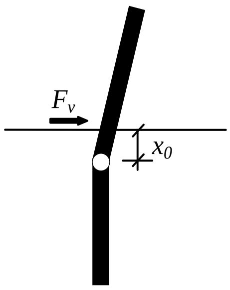


Figure F.1 Model for shear capacity of one-sided dowel pin embedded in concrete according to theory of plasticity. No eccentricity of the shear force.

$$M_{\max} = F_{vR} \frac{F_{vR}}{q_c} - q_c \frac{1}{2} \left(\frac{F_{vR}}{q_c}\right)^2 = \frac{1}{2} \frac{F_{vR}{}^2}{q_c} \qquad \text{(F.11)}$$

The ultimate shear capacity can as before be solved by setting the maximum moment equal to the plastic moment

$$\frac{1}{2} \frac{F_{vR}{}^2}{q_c} = f_y \frac{\phi^3}{6} \qquad \text{(F.12)}$$

$$F_{vR}{}^2 = f_y \frac{\phi^3}{6} \cdot 2 \cdot k f_{cc} \cdot \phi \qquad \text{(F.13)}$$

$$F_{vR} = c_0 \phi^2 \sqrt{f_{cc} f_y} \qquad \text{(F.14)}$$

$$c_0 = \sqrt{\frac{k}{3}} \qquad \text{(can be taken as } c_0 = 1{,}0 \text{ in design)} \qquad \text{(F.15)}$$

Appendix G Calculation of total longitudinal tensile force with regard to inclined cracks

G.1 Comparison between results obtained by ΔF_{td} or a_l

G.1.1 Notations, formulas and data used in the calculations

In Table G.1 the assumed values in the calculation of the longitudinal tensile force is presented. The calculations are performed in Excel and the formulas used are also presented in the table.

Table G.1 Data and formulas used in the calculation of the longitudinal tensile force.

Q	1	kN/m		Load	
L	5	M		Beam length	
R	2.5	kN		Support reaction	
H	0.4	M		Height of cross section	
D	0.36	M		Depth of reinforcement	=0.9h
Z	0.324	M		Inner lever arm	=0.9d
A	1.571	Rad		Angle of shear reinforcement	=90º
Θ	0.785	Rad		Angle of inclined struts	=45º
a_l	0.162	M			=$z\cdot(\cot\theta-\cot\alpha)/2$
$V(x) = R-qx$				Shear force	
$M(x) = Rx-qx^2/2$				Moment	
$F_s(x) = M(x)/z+\|V(x)\|/2\cdot(\cot\theta-\cot\alpha)$				Longitudinal tensile force	

G.1.2 Calculation procedure

In order to be able to compare longitudinal tensile force calculated by ΔF_{td} and a_l the force F_s is first calculated for different values of x, i.e. at different sections along the beam. The expression of F_s used in the calculations is

$$F_s(x) = \frac{M(x)}{z} + \frac{|V(x)|}{2}(\cot\theta - \cot\alpha) \tag{G.1}$$

where the expression for ΔF_{td} can be identified as

$$\Delta F_{td} = \frac{|V(x)|}{2}(\cot\theta - \cot\alpha) \tag{G.2}$$

The required tensile force in each section x is thereafter calculated in a second way by inserting x-a_l into the equation for $M(x)$ presented in Table G.2 and dividing with the internal lever arm z. This corresponds to a horizontal shift of the moment curve of a distance equal to a_l. If the obtained values are plotted or compared to the ones calculated by Equation (G.1) with the same values of x the difference between the two methods can be obtained. The calculation procedure can be understood from the formulas shown in Table G.2 where the data for half of the beam is presented.

Table G.2 Data and calculation procedure, only the data for half of the beam is presented.

x	*V(x)*	*M(x)*	*Fs(x)*	*x+al*	*V(x+al)*	*M(x+al)*	*M(x+al)/z*	*M(x)/z*	*|M(x+al)/z -Fs(x)|*	*(M(x+al)/z-Fs(x)) /Fs(x)*
0.0	2.5	0.000	-1.25	0.162	2.338	-0.392	-1.210	0.000	0.0405	3.240
0.1	2.4	-0.245	-1.956	0.262	2.238	-0.621	-1.916	-0.756	0.0405	2.070
0.2	2.3	-0.480	-2.631	0.362	2.138	-0.839	-2.591	-1.481	0.0405	1.539
0.3	2.2	-0.705	-3.276	0.462	2.038	-1.048	-3.235	-2.176	0.0405	1.236
0.4	2.1	-0.920	-3.890	0.562	1.938	-1.247	-3.849	-2.840	0.0405	1.041
0.5	2.0	-1.125	-4.472	0.662	1.838	-1.436	-4.432	-3.472	0.0405	0.906
0.6	1.9	-1.320	-5.024	0.762	1.738	-1.615	-4.984	-4.074	0.0405	0.806
0.7	1.8	-1.505	-5.545	0.862	1.638	-1.783	-5.505	-4.645	0.0405	0.730
0.8	1.7	-1.680	-6.035	0.962	1.538	-1.942	-5.995	-5.185	0.0405	0.671
0.9	1.6	-1.845	-6.494	1.062	1.438	-2.091	-6.454	-5.694	0.0405	0.624
1.0	1.5	-2.000	-6.923	1.162	1.338	-2.230	-6.882	-6.173	0.0405	0.585
1.1	1.4	-2.145	-7.320	1.262	1.238	-2.359	-7.280	-6.620	0.0405	0.553
1.2	1.3	-2.280	-7.687	1.362	1.138	-2.477	-7.647	-7.037	0.0405	0.527
1.3	1.2	-2.405	-8.023	1.462	1.038	-2.586	-7.982	-7.423	0.0405	0.505
1.4	1.1	-2.520	-8.328	1.562	0.938	-2.685	-8.287	-7.778	0.0405	0.486
1.5	1.0	-2.625	-8.602	1.662	0.838	-2.774	-8.561	-8.102	0.0405	0.471
1.6	0.9	-2.720	-8.845	1.762	0.738	-2.853	-8.805	-8.395	0.0405	0.458
1.7	0.8	-2.805	-9.057	1.862	0.638	-2.921	-9.017	-8.657	0.0405	0.447
1.8	0.7	-2.880	-9.239	1.962	0.538	-2.980	-9.198	-8.889	0.0405	0.438
1.9	0.6	-2.945	-9.390	2.062	0.438	-3.029	-9.349	-9.090	0.0405	0.431
2.0	0.5	-3.000	-9.509	2.162	0.338	-3.068	-9.469	-9.259	0.0405	0.426
2.1	0.4	-3.045	-9.598	2.262	0.238	-3.097	-9.558	-9.398	0.0405	0.422
2.2	0.3	-3.080	-9.656	2.362	0.138	-3.115	-9.616	-9.506	0.0405	0.419
2.3	0.2	-3.105	-9.683	2.462	0.038	-3.124	-9.643	-9.583	0.0405	0.418
2.4	0.1	-3.120	-9.680					-9.630		
2.5	0.0	-3.125	-9.645					-9.645		

It should be noted that in order to obtain as exact results as possible more values of x were used in the calculation procedure than the ones presented in Table G.3. The values of x went from 0 to 5 in steps of 0.001. This was done since the value of a_l was calculated to 0.162. See Table G.3 for the data obtained for the more accurate calculations over a small part of the beam.

Table G.3 Data for the first 2 cm of the beam obtained from the more accurate calculation.

x	*V(x)*	*M(x)*	*Fs(x)*	*x+al*	*V(x+al)*	*M(x+al)*	*M(x+al)/z*	*M(x)/z*	*|M(x+al)/z -Fs(x)|*	*(M(x+al)/z-Fs(x)) /Fs(x)*
0	2.500	0.000	-1.250	0.162	2.338	-0.392	-1.210	0.000	0.0405	3.2400
0.001	2.499	-0.002	-1.257	0.163	2.337	-0.394	-1.217	-0.008	0.0405	3.2214
0.002	2.498	-0.005	-1.264	0.164	2.336	-0.397	-1.224	-0.015	0.0405	3.2030
0.003	2.497	-0.007	-1.272	0.165	2.335	-0.399	-1.231	-0.023	0.0405	3.1849
0.004	2.496	-0.010	-1.279	0.166	2.334	-0.401	-1.238	-0.031	0.0405	3.1669
0.005	2.495	-0.012	-1.286	0.167	2.333	-0.404	-1.246	-0.039	0.0405	3.1492
0.006	2.494	-0.015	-1.293	0.168	2.332	-0.406	-1.253	-0.046	0.0405	3.1317
0.007	2.493	-0.017	-1.300	0.169	2.331	-0.408	-1.260	-0.054	0.0405	3.1143
0.008	2.492	-0.020	-1.308	0.170	2.330	-0.411	-1.267	-0.062	0.0405	3.0972
0.009	2.491	-0.022	-1.315	0.171	2.329	-0.413	-1.274	-0.069	0.0405	3.0803
0.01	2.490	-0.025	-1.322	0.172	2.328	-0.415	-1.282	-0.077	0.0405	3.0635
0.011	2.489	-0.027	-1.329	0.173	2.327	-0.418	-1.289	-0.085	0.0405	3.0470
0.012	2.488	-0.030	-1.336	0.174	2.326	-0.420	-1.296	-0.092	0.0405	3.0306
0.013	2.487	-0.032	-1.344	0.175	2.325	-0.422	-1.303	-0.100	0.0405	3.0144
0.014	2.486	-0.035	-1.351	0.176	2.324	-0.425	-1.310	-0.108	0.0405	2.9984
0.015	2.485	-0.037	-1.358	0.177	2.323	-0.427	-1.317	-0.115	0.0405	2.9826
0.016	2.484	-0.040	-1.365	0.178	2.322	-0.429	-1.325	-0.123	0.0405	2.9669
0.017	2.483	-0.042	-1.372	0.179	2.321	-0.431	-1.332	-0.131	0.0405	2.9514
0.018	2.482	-0.045	-1.379	0.180	2.320	-0.434	-1.339	-0.138	0.0405	2.9361
0.019	2.481	-0.047	-1.387	0.181	2.319	-0.436	-1.346	-0.146	0.0405	2.9209
0.02	2.480	-0.050	-1.394	0.182	2.318	-0.438	-1.353	-0.154	0.0405	2.9059

G.1.3 Graphs

The graphs obtained from the calculation explained in previous section are presented in Figures G.1 to G.7.

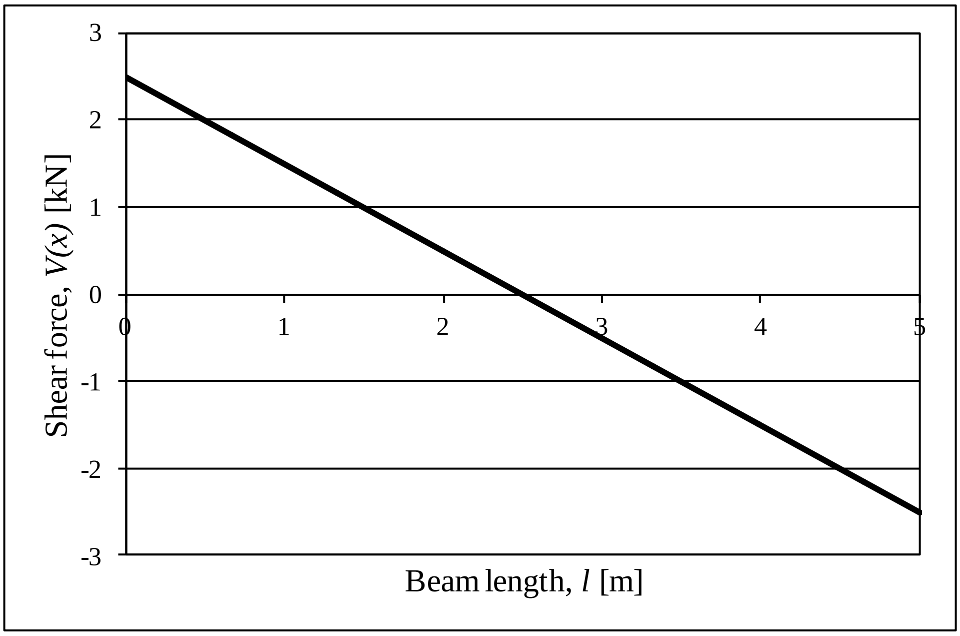


Figure G.1 Shear force distribution over the beam.

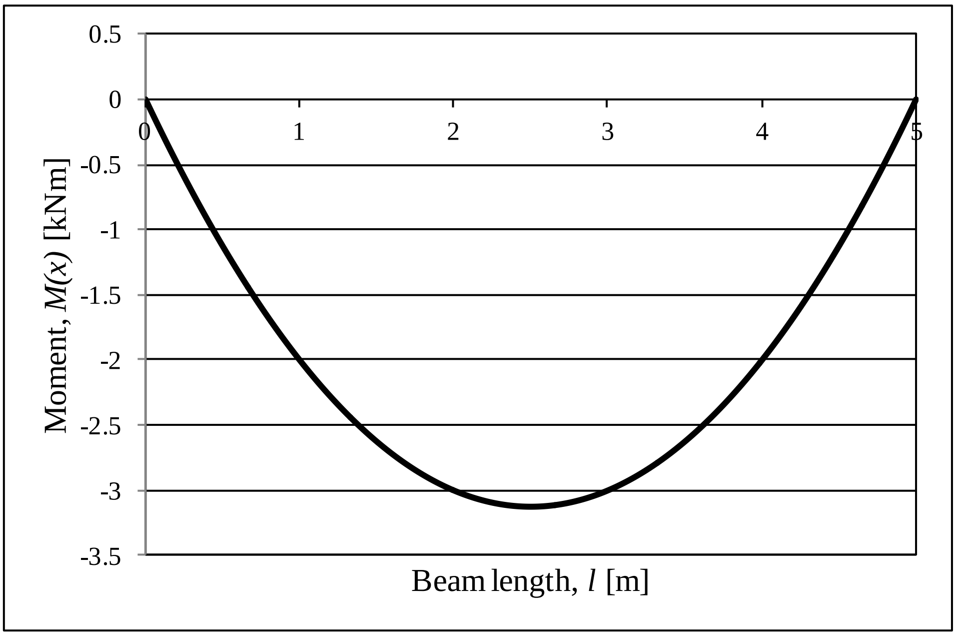


Figure G.2 *Moment distribution over the beam.*

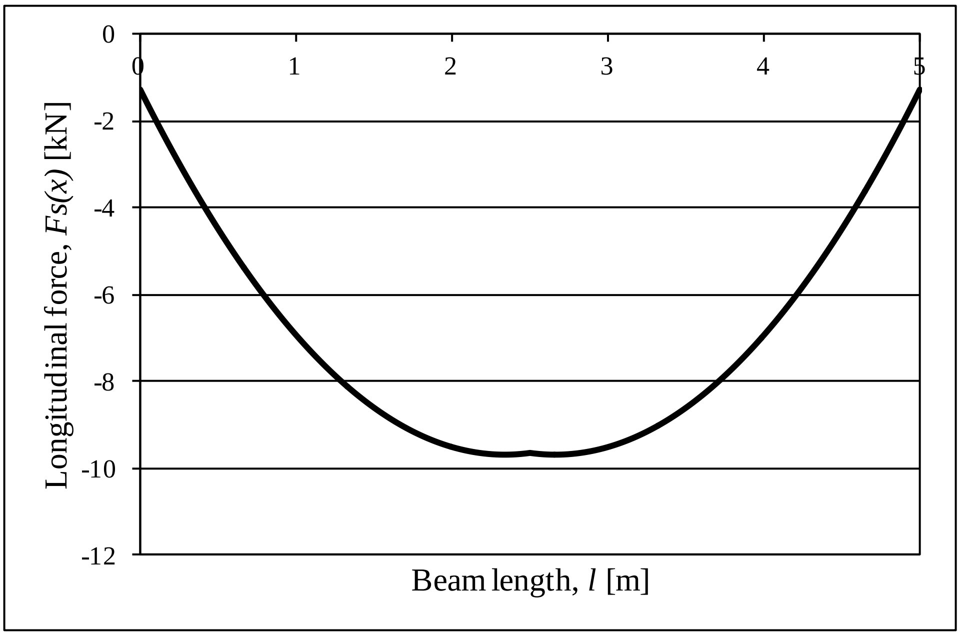


Figure G.3 *Distribution of longitudinal tensile force $F_s(x)$.*

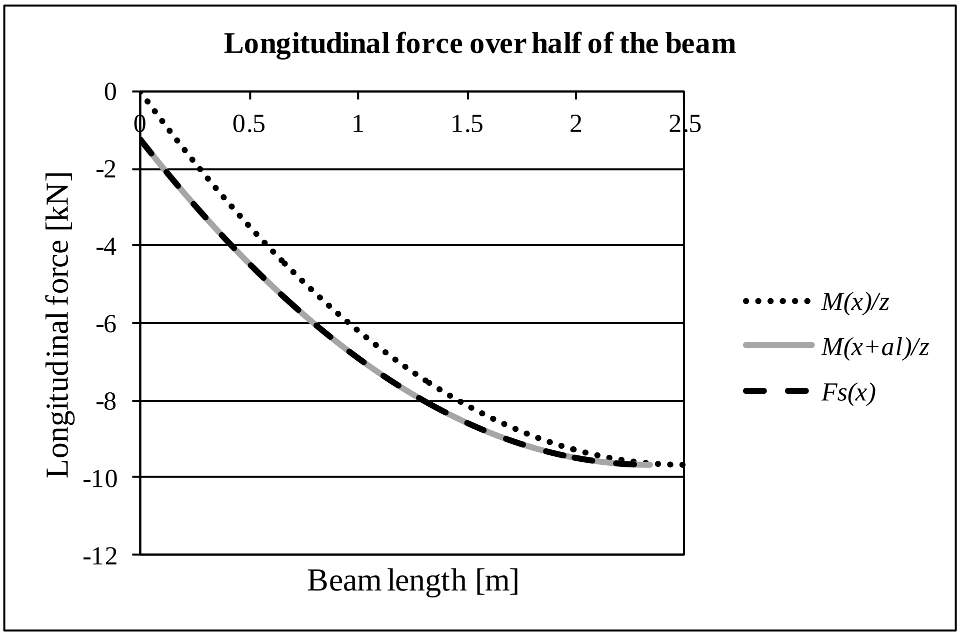


Figure G.4 *Distribution of longitudinal tensile force over half of the beam. The grey line represents the tensile force calculated by means of a_l and the dashed line is the tensile force calculated by means of ΔF_{td}.*

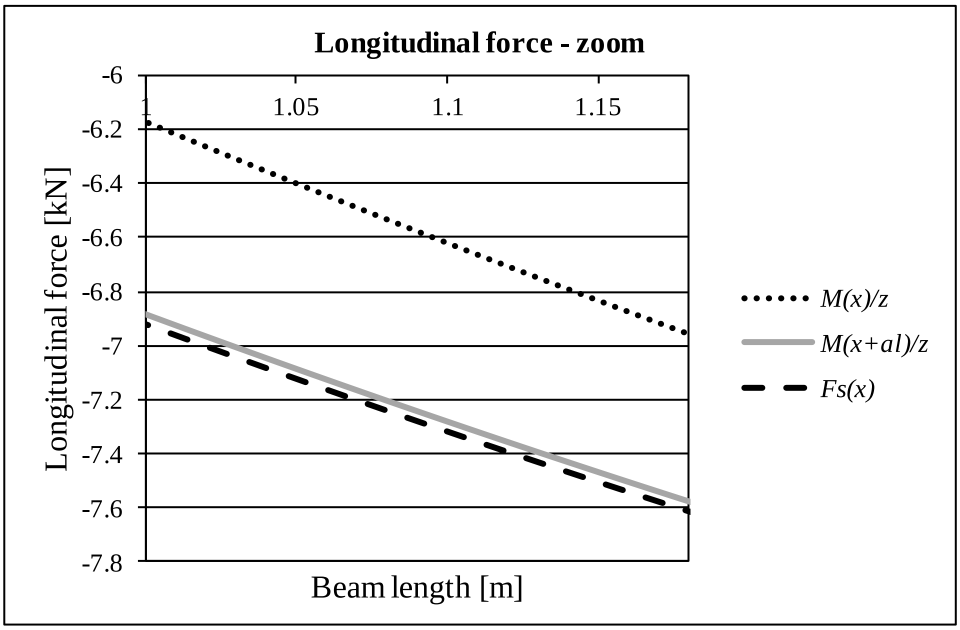


Figure G.5 *Zoom of Figure G.4.*

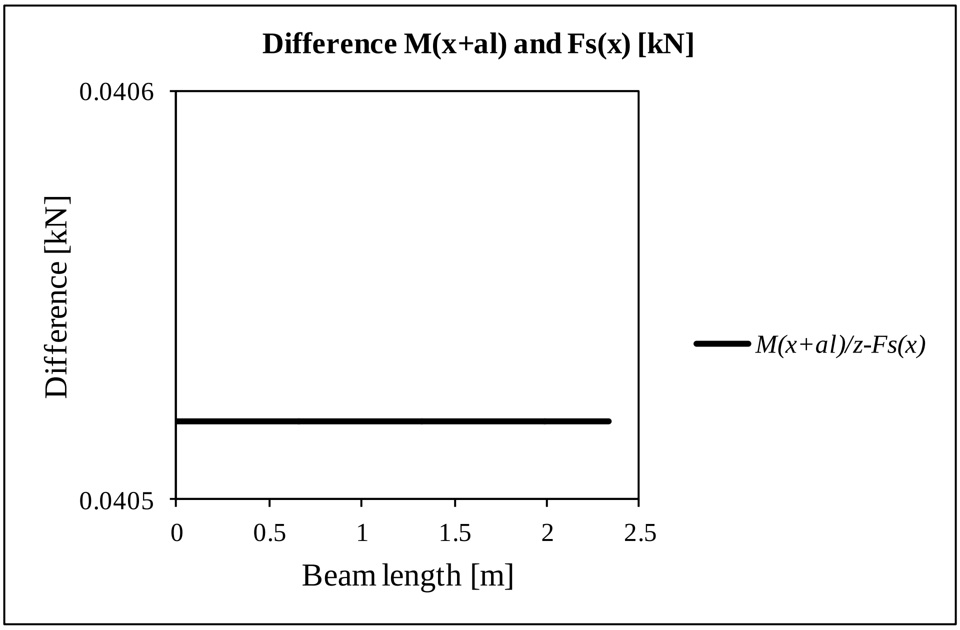


Figure G.6 *Difference between values of the longitudinal tensile force obtained by means of a_l or ΔF_{td}.*

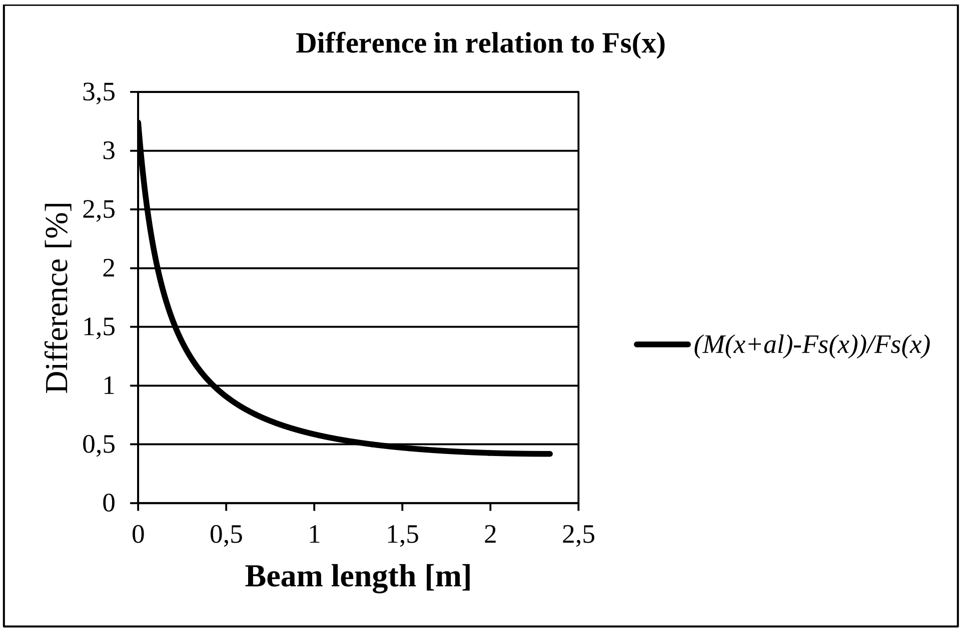


Figure G.7 *Difference in percent between values of the longitudinal tensile force obtained by means of a_l or ΔF_{td}.*

Appendix H Permissible mandrel diameter of bent bars

H.1 Comparison of minimum mandrel diameter according to Eurocode 2 and BBK 04

H.1.1 Data used in the calculations

Table H.1 shows areas for different bar diameters. All values in this section are, if nothing else is mentioned, provided in meters.

Table H.1 Bar diameter and area of one bar.

Bar diameter ϕ	Area of one bar $A_s = \pi\,\phi^2/4$
0.008	0.00005
0.01	0.00008
0.012	0.00011
0.016	0.00020
0.02	0.00031
0.025	0.00049
0.032	0.00080

The length, a_b, is included in the expression for minimum mandrel diameter presented in Eurocode 2, see Equation (H.1) in the following section. The length, a_b, depends on the concrete cover and the bar diameter, see Table H.2.

Table H.2 Calculation of the length, a_b, for a given bar, or groups of bars, perpendicular to the plane of the bend.

$a_b=c+\phi/2$
0.034
0.035
0.036
0.038
0.040
0.043
0.046
For a bar or group of bars adjacent to the face of the member, a_b, should be taken as the cover plus φ / 2.

The concrete cover and spacing of bars used in the calculations are presented in Table H.3.

Table H.3 Concrete cover and spacing of the bars.

Concrete cover	c =	0.03
Spacing between bars	s =	0.2
The concrete cover perpendicular to the plane of the bend bar.	cc=	0.03
cc should not be bigger than half the centrum distance when having many parallel bend bars		

In the expression for minimum mandrel diameter according to BBK 04, see Equation (H.2), the angle of the bend of the bar is included, see Table H.4.

Table H.4 Angel of the bend of the bar, [rad].

β_1 =90º=1.571 [rad]
β_2 =180º=3.142 [rad]

The partial factors for concrete and steel are presented in Table H.5.

Table H.5 Partial factors for concrete and steel, [-].

Concrete	γ_c =	1.5
Steel	γ_s =	1.15

H.1.2 Material properties

Table H.6 Characteristic and design strength of the concrete and the reinforcement and the tensile force acting in the reinforcement.

Characteristic strength [Pa]		
Steel B500B		
Yield strength	f_{yk} =	500000000
Concrete C40/50		
Compressive strength	f_{ck} =	40000000
Tensile strength	$f_{ct.0.05}$ =	2500000
Design strength [Pa]		
Steel B500B		
Yield strength	f_{yd} =	435000000
Concrete C40/50		
Compressive strength	f_{cd} =	26670000
Tensile strength	$f_{cd.0.05}$ =	1670000
Tensile force [N]		
ϕ = 0.008	$F_{bt.d} = f_{yd} \cdot A_s$ =	21854.4
0.01		34147.5
0.012		49172.4
0.016		87417.6
0.02		136590
0.025		213421.875
0.032		349670.4

H.1.3 Minimum mandrel diameter of the bar to in order to avoid spalling of the concrete in Expression EC2 (8.1)

In the expression for minimum mandrel diameter provided in Eurocode 2, see Equation (H.1), no account is taken for the angle of the bend of the bar.

$$\phi_{m,\min} \geq \frac{F_{bt}\left(\left(\frac{1}{a_b}\right)+\frac{1}{(2\phi)}\right)}{f_{cd}} \tag{H.1}$$

The permissible mandrel diameter obtained from the calculations, using the design strength values of the materials, is shown in Table.H.7.

Table H.7 Design values of the mandrel diameter.

ϕ	$\phi_{m.min.EC2}$
0.008	0.038
0.01	0.050
0.012	0.064
0.016	0.094
0.02	0.128
0.025	0.174
0.032	0.245

The values in Table H.7 are plotted in Figure 9.32 in Section H.1.1.4.

H.1.4 Minimum mandrel diameter in order to avoid spalling of the concrete in BBK 04 Equation (3.9.4.2a), Boverket (2004)

In the expression for minimum mandrel diameter provided in BBK 04, see Equation (H.2), account is taken for the angle of the bend of the bar. It should be noted that $2r$ corresponds to the mandrel diameter ϕ_m in Eurocode 2.

$$\frac{r}{\phi} \geq 0.028\frac{f_y}{f_{ct}} - 0.5 - \frac{1}{\sin(\beta/2)}\left(\frac{cc}{\phi}+0.5\right) \tag{H.2}$$

where

$$\frac{cc}{\phi} \leq 3.5$$

Equation (H.2) is rearranged in order to be able to compare it to the expression used in Eurocode in Equation (H.1).

$$\phi_{m,\min} = 2r \tag{H.3}$$

$$2r \geq 2\phi\left(0.028\frac{f_y}{f_{ct}} - 0.5 - \frac{1}{\sin(\beta/2)}\left(\frac{cc}{\phi}+0.5\right)\right) \tag{H.4}$$

Table H.8 Control of the condition $cc/\phi \leq 3.5$.

ϕ	Cc	cc/ϕ	
0.008	0.03	3.75	8 mm is not ok!
0.01	0.03	3	
0.012	0.03	2.5	
0.016	0.03	1.875	
0.02	0.03	1.5	
0.025	0.03	1.2	
0.032	0.03	0.938	

The permissible mandrel diameter obtained from the calculations using design strength values of reinforcing steel and concrete are shown in Table H.9 for a 90º bend of the bar and in Table H.10 for a 180º bend of the bar.

Table H.9 Design values of the mandrel diameter using 90º (L-shape) angle of the bent bar.

ϕ	$\phi_{90.BBK04}$
0.01	0.0369
0.012	0.0612
0.016	0.1099
0.02	0.1586
0.025	0.2195
0.032	0.3047

Table H.10 Design values of the mandrel diameter using 180º (loop) angle of the bent bar.

ϕ	$\phi_{180.BBK04}$
0.01	0.066
0.012	0.091
0.016	0.141
0.02	0.192
0.025	0.255
0.032	0.343

The values in Table H.9 and Table H.10 are plotted in Figure H.1 together with the values calculated from the expression in Eurocode 2, see Table H.7. The expressions are not plotted for bar diameter equal to $\phi 8$ since the expression from BBK 04 does not allow that.

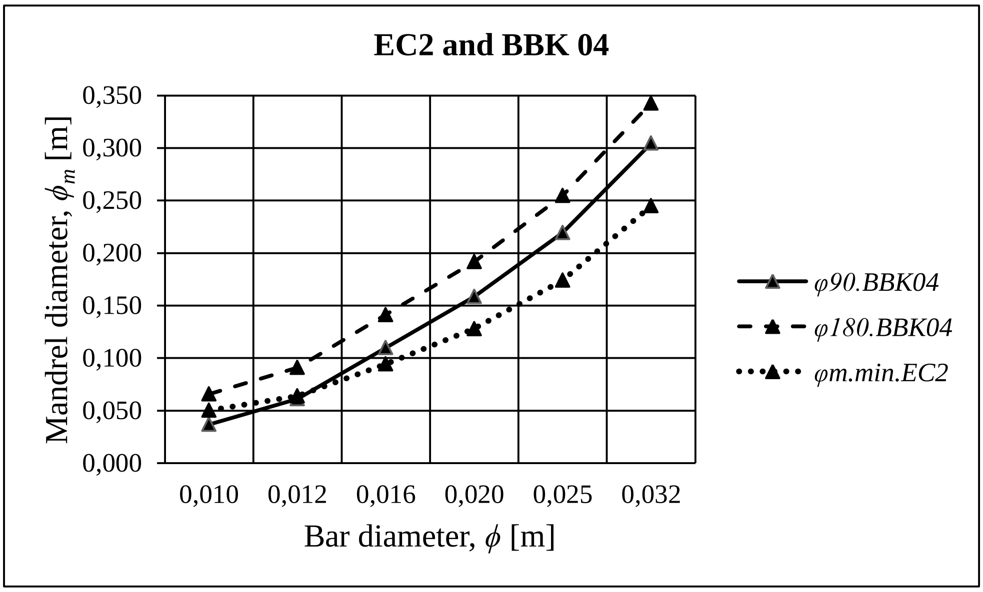


Figure H.1 Equation (8.1) from Eurocode 2 and Equation (3.9.4.2a) from BBK 04 plotted against the bar diameter when using the design strength. The angle of the bend bar influences the minimum mandrel diameter in the expression from BBK 04.

Appendix I Weight of different bar diameters and lengths

I.1 Standard length – 6 meter

In Sweden it is allowed to only carry 25 kg at the construction site. In Table I.1 it is calculated how heavy a reinforcement bar with the length 6 meter is with different bar diameter using Equation (I.1).

$$m = \frac{\phi^2 \pi}{4} l \cdot \rho [kg] \tag{I.1}$$

Table I.1 Weight of a reinforcement bar with a length of 6 m.

l [m]	*φ* [m]	*Π*		*m*^3	*ρ* [kg/m^3]	*m* [kg]
6	0.006	3.14	4	0.0002	7800	1.3
6	0.008	3.14	4	0.0003	7800	2.4
6	0.01	3.14	4	0.0005	7800	3.7
6	0.012	3.14	4	0.0007	7800	5.3
6	0.016	3.14	4	0.0012	7800	9.4
6	0.02	3.14	4	0.0019	7800	14.7
6	0.025	3.14	4	0.0029	7800	23.0
6	0.032	3.14	4	0.0048	7800	37.6

I.2 Standard length – 12 meter

In Table I.2 it is calculated how heavy a reinforcement bar with the length 12 meter is with different bar diameter.

Table I.2 Weight of a reinforcement bar with a length of 12 m.

l [m]	*φ* [m]	*π*		*m*^3	*ρ* [kg/m^3]	*m* [kg]
12	0.006	3.14	4	0.0003	7800	2.6
12	0.008	3.14	4	0.0006	7800	4.7
12	0.01	3.14	4	0.0009	7800	7.3
12	0.012	3.14	4	0.0014	7800	10.6
12	0.016	3.14	4	0.0024	7800	18.8
12	0.02	3.14	4	0.0038	7800	29.4
12	0.025	3.14	4	0.0059	7800	45.9
12	0.032	3.14	4	0.0096	7800	75.2

Appendix J Survey questions

Detaljutformning av armering i betongkonstruktioner

Enkäten utförs som en del av ett examensarbete på Väg- och vattenbyggnad, Chalmers, med inriktning konstruktion. Morgan Johansson från Reinertsen Sverige AB och Björn Engström från Chalmers tekniska högskola är handledare. Examensarbetet berör riktlinjer och regler för detaljutformning av armering i betongkonstruktioner. Det är av intresse att se hur branschen tolkar delar av Eurokod 2 gällande detta ämne. Enkäten bör utföras med Eurokod 2 tillhands.

Om du anser att det är svårt att veta hur någon av frågorna ska tolkas kommentera då gärna detta och försök förklara vilken tolkning Du har svarat utifrån. Frågorna har utformats så generallt som möjligt med principiella bilder vilket t.ex. innebär att frågeställningen syftar till både plattor och balkar då inget annat nämns. Anser du att det dock är av betydelse att veta konstruktionstypen skriv gärna detta som kommentar. Notera också att det är tillåtet att kryssa i flera svarsalternativ.

Vilken utbildning har du?

svar:……………………………………………………………

Kön?

a) Kvinna □
b) Man □

Vad jobbar du med? Om du kryssar i flera alternativ, ange hur många år du har jobbat inom varje bransch.

a) Hus □ antal år:…..
b) Industri □ antal år:…..
c) Anläggning (Trafikverket) □ antal år:…..
d) Annat (skriv gärna vad) □ antal år:…..

kommentar:…………………………………………………

Vilken yrkesroll har du?

svar:……………………………………………………………

Hur många år har du jobbat inom branschen?

svar:……………………………………………………………

1) I Eurokod 2 finns krav på sprickbreddsbegränsning där kapitel 7.3 *Begränsning av sprickbredd* ger metodik för beräkning och kontroll av dessa krav. Det finns dock ingen uttalad metodik för kontroll av skjuvsprickor.

1.1) Hur hanterar du detta i ditt arbete? Kryssa i de alternativ du tycker överensstämmer med hur du arbetar.

a) Jag kontrollerar inte sprickbredd för skjuvsprickor □

b) Jag använder mig av Trafikverkets tidigare förslag på en begränsning av spänningen till 250MPa □

c) Jag anser att sprickbreddskravet mht skjuvsprickor är uppfyllt om minimiarmering enligt ekvation 7.1 är inlagt □

d) Jag anser att sprickbreddskravet mht skjuvsprickor är uppfyllt om kraven i avsnitt 7.3.3 är uppfyllda □

e) Jag anser att sprickbreddskravet mht skjuvsprickor är uppfyllt om kraven i avsnitt 7.3.4 är uppfyllda. □

f) Jag använder mig av metoden i Svenska Betongföreningens Handbok till Eurokod 2, avsnitt X.6 □

g) Jag har en egen metodik för denna typ av kontroll (utveckla gärna) □

kommentar:…………………………………………………

1.2) Om du valt alternativ c), vilken armeringsspänning använder du dig av då?

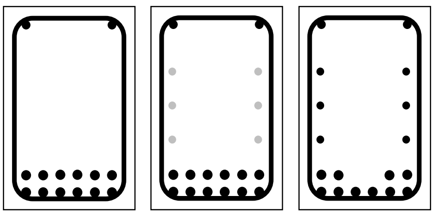
a) $\sigma_s = f_{yk}$ □
b) σ_s = reducerad □

Om du valt alternativ b), hur uppskattar du armeringsspänningen då?

svar/kommentar:……………………………………………

2) Anordning av längsgående sprickarmering i balkar med relativt högt tvärsnitt.

2.1) Hur anordnar du denna typ av sprickarmering? Huvudarmeringen i balken är den mängd som behövs för att uppfylla krav i brott- och bruksgränstillstånd samt minimikrav.



a) *b)* *c)*

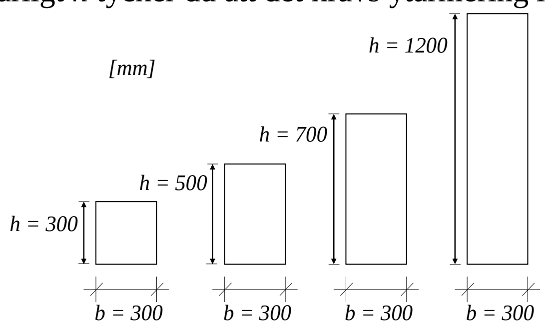
a) Ingen sprickarmering □

b) Huvudarmering plus <u>extra</u> sprickarmering □

c) Huvudarmeringen fördelas i botten och livet som sprickarmering □

kommentar:…………………………………………………

2.2) Om du svarat b) eller c) ovan, hur utformar du den längsgående sprickarmeringen i så fall? För vilket ungefärligt *h* tycker du att det krävs ytarmering längs livets ytor?



a) □ *b)* □ *c)* □ *d)* □

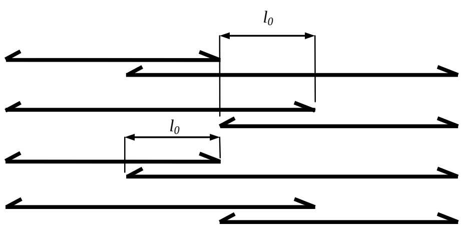
kommentar:………………………………………………

2.3) Fyll i den eller de parametrar du anser relevanta för placering av längsgående sprickarmering

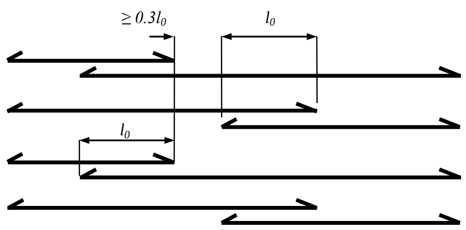
stångdiameter: ϕ = ..……..mm
centrumavstånd: s = ..……..mm
armeringsmängd: A_s = ..……..mm²/m
kommentar:…………………………………………………

3) I Eurokod 2, avsnitt 8.7.2 ges anvisningar om hur omlottskarvning ska utföras. I detta fall är det ett lager dragen huvudarmering i en platta som ska omlottskarvas med en stångdiameter ϕ16. Skarvlängden, l_0, anses vara tillräcklig.

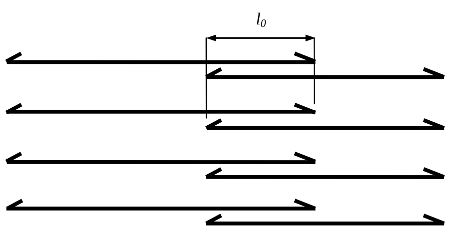
Vilken eller vilka av följande alternativ använder du dig av?



a) □



b) □



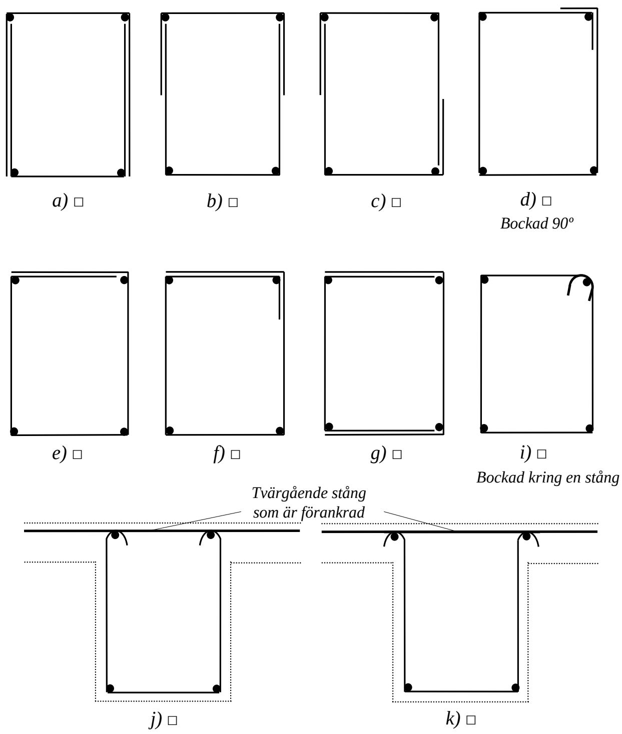
c) □

kommentar:…………………………………………………

4) Utformning av vridarmering i en balk.

4.1) Vilken eller vilka typer av byglar anser du är tillåtna som vridarmering i en balk? Byglarna är utformade med tillräcklig förankringslängd och en bygeldiameter mellan ϕ8-ϕ16.

Av de alternativen som du valt, ringa in den bygeln som du främst använder.



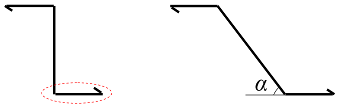
4.2) Påverkar bygeldiametern ditt val av utförandet av byglar?

a) Ja □
b) Nej □

4.3) Om du svarat ja på frågan ovan, vilket eller vilka av dina valda alternativ använder du för ϕ16?

svar:………………………………………………………………

5) Z-formade byglar (G-järn, se figur) kan användas som tvärkraftsarmering,



5.1) Är G-järn något som Du använder som tvärkraftsarmering (skilj från uppbockad längsarmering)?

a) Ja □
b) Nej □

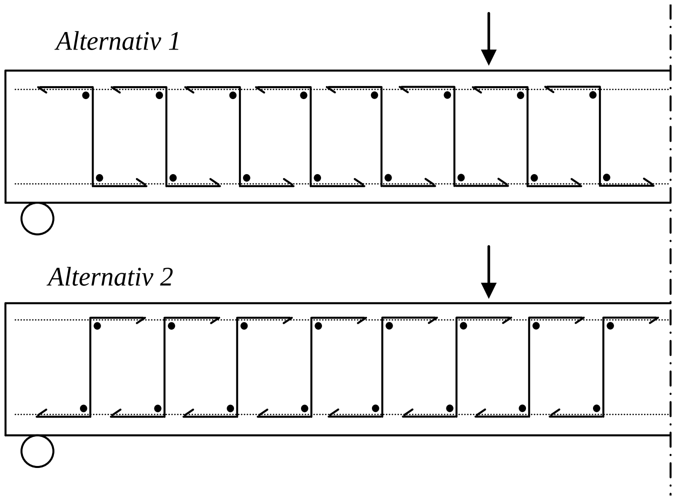
5.2) Används förankringslängden, l_{bd}, eller omlottskarvningslängden, l_0, som längd på den horisontella skänkeln i ett G-järn (inringad i figur ovan)?

a) Förankringslängd, l_{bd} □
b) Omlottskarvninslâng, l_0 □

5.3) Används G-järn som tvärkraftsarmering utan kompletterande omslutande byglar?

a) Ja □
b) Nej □

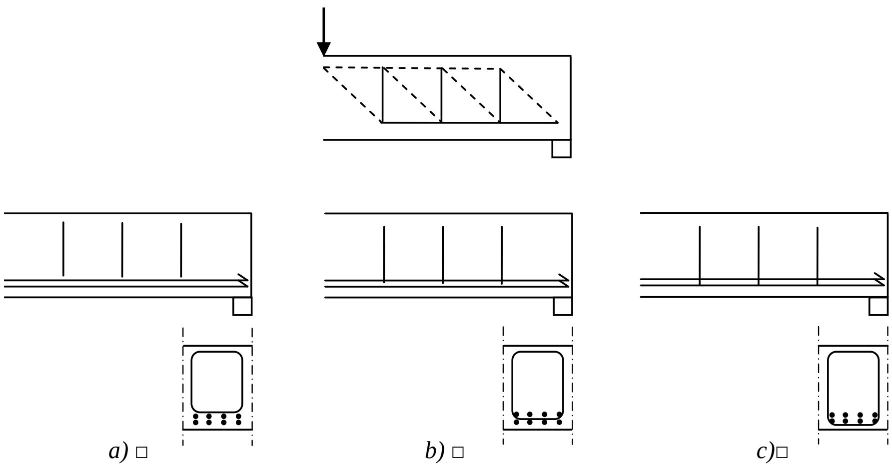
5.4) Vilket eller vilka av alternativen föredrar du som utformning av G-järn?



a) Alternativ 1 □
b) Alternativ 2 □
c) Spelar ingen roll □

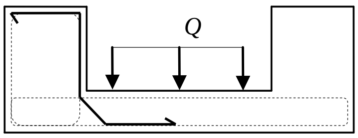
kommentar:………………………………………………………

6) Vilken eller vilka av följande detaljutformningar av bygelarmering är lämpliga?

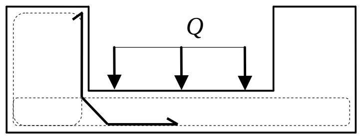


kommentar:………………………………………………………

7) Vilket alternativ nedan anser du är en möjlig armeringslösning för en huvudbalk i en trågbro? Det är den heldragna linjen som ska beaktas. De streckade linjerna visar övrig armering som anses vara tillräcklig.



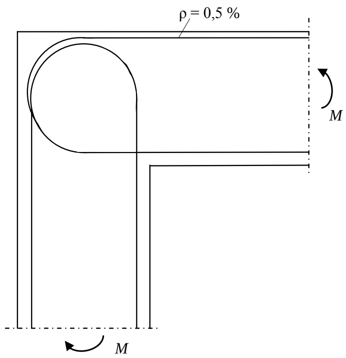
a) □



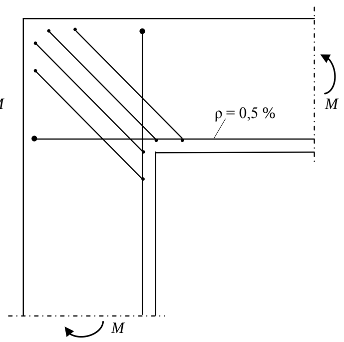
b) □

kommentar:………………………………………………………

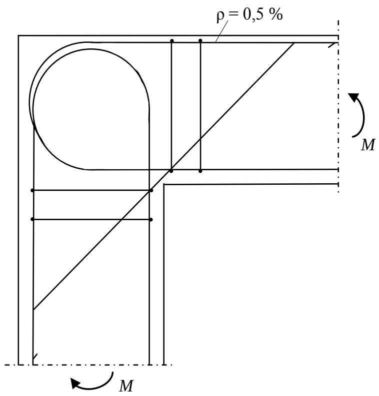
8) Det är ett behov av 0,5 % böjarmering i ramhörnet nedan. Ramhörnet är utsatt för ett öppnande moment. Vilken detaljutformning föredrar du?



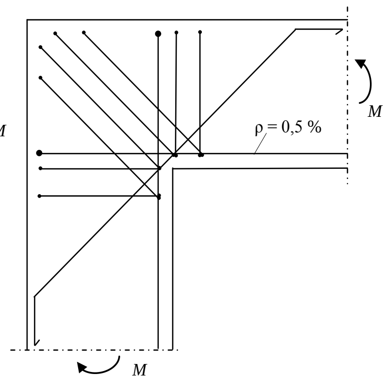
a) □



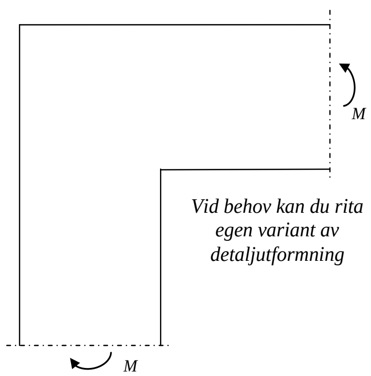
b) □



c) □



d) □



e) □

kommentar:………………………………………………………

Appendix K Result from the survey

K.1 Information about the participants

The result from the survey is stated in Section 11.2. In Appendix K the answers from the participants are shown and how the diagrams are produced. In Table K.1 shows the work experience in number of years for all of the participants in the survey.

- Number of men: 18
- Number of women: 1
- Total number of participants: 19

Table K.1 Additional information about the work experience from the participants.

Work area [year]																			
Housing				1									24		1	15	12	10	2
Industrial											2		24	6		15	12	10	9
Buildings																			
Bridges and	13	6	3	19	6	18	15	17	14	11	4	3						16	9
Tunnels																			
Other									6					1	4				

K.2 Survey question 1

17 people took part in answering on question number 1.1. The answers are presented in Table K.2. Table K.3 present the answers in percent.

Table K.2 Answers to question 1.1.

1.1																				**Summary:**
a)										1					1	1				3
b)	1		1	1	1	1		1	1		1				1					9
c)																1				1
d)														1		1		1		3
e)							1											1		2
f)								1		1			1				1	1	1	6
g)						1	1													2
																			Total=	26

Table K.3 Answers in percent to Question 1.1.

1.1	Housing/industrial buildings	[%]	Bridges and tunnels	[%]
a)	1	6	2	12
b)	8	47	1	6
c)	0	0	1	6
d)	0	0	3	18
e)	1	6	1	6
f)	2	12	4	24
g)	2	12	0	0

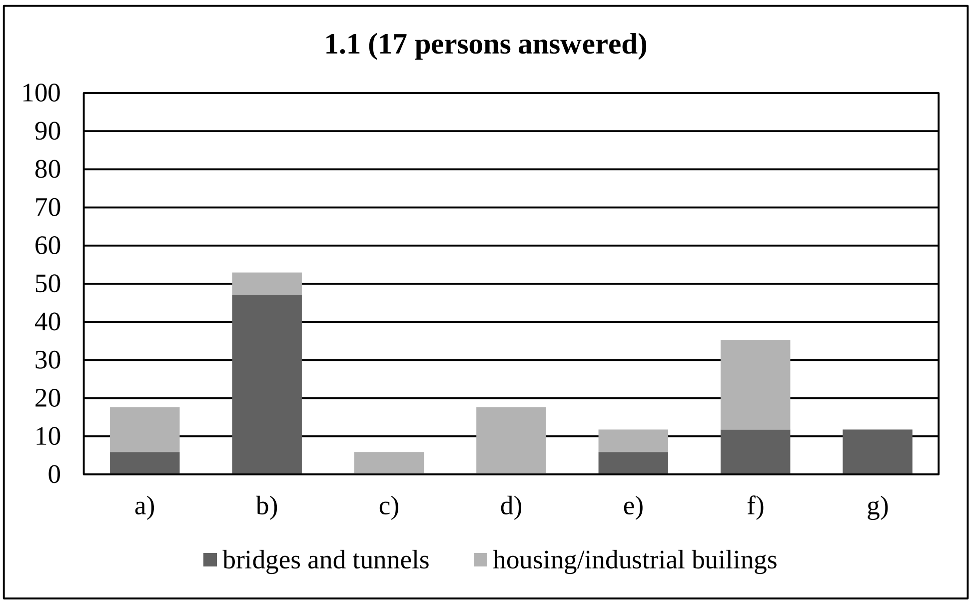


Figure K.1 Result from Question 1.1.

1 participant took part in answering on question number 1.2. The answer is presented in Table K.4.

Table K.4 Answers in percent to Question 1.2.

1.2																				**Summary:**
a)																1				1
b)																				
																			Total=	1

K.3 Survey question 2

19 people took part in answering on question number 2.1. The answers are presented in Table K.5. Table K.6 present the answers in percent.

Table K.5 Answers to Question 2.1.

2.1																				**Summary:**
a)											1									1
b)	1	1	1	1	1	1			1	1			1	1	1	1	1	1	1	15
c)						1	1	1				1		1						5
																			Total=	21

Table K.6 Answers in percent to Question 2.1.

2.1	Housing/industrial buildings	%	Bridges and tunnels	%
a)	1	5	0	0
b)	8	42	7	37
c)	4	21	1	5

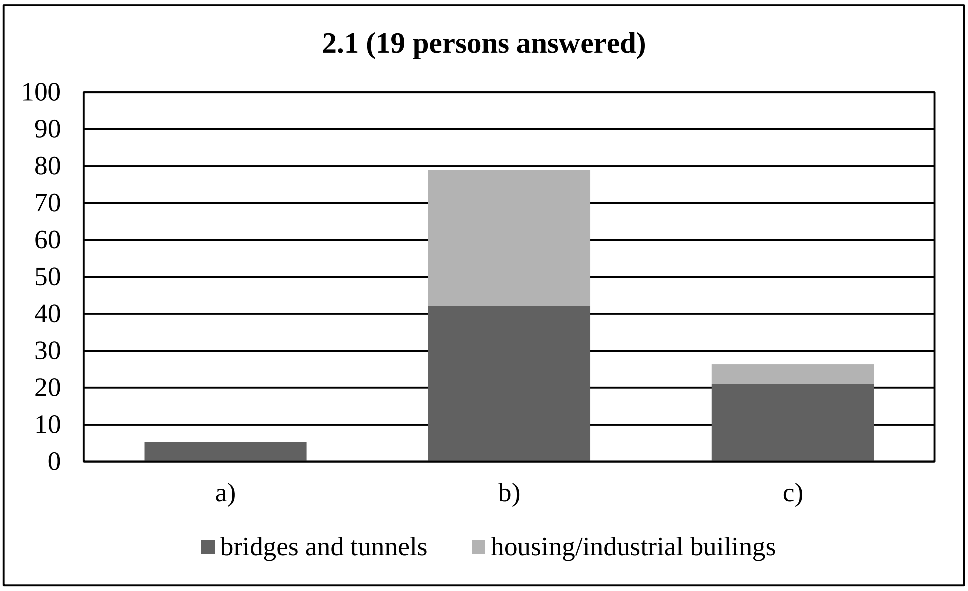


Figure K.2 Result from Question 2.1.

17 people took part in answering on question number 2.2. The answers are presented in Table K.7. Table K.8 presents the answers in percent.

Table K.7 Answers to Question 2.2.

2.2																				**Summary:**
a)							1											1		2
b)	1	1	1		1	1		1	1	1		1			1	1	1		1	13
c)											1									1
d)													1							1
																			Total=	17

Table K.8 Answers in percent to Question 2.2.

2.2	Housing/industrial buildings	%	Bridges and tunnels	%
a)	1	6	1	6
b)	9	53	4	24
c)	1	6	0	0
d)	0	0	1	6

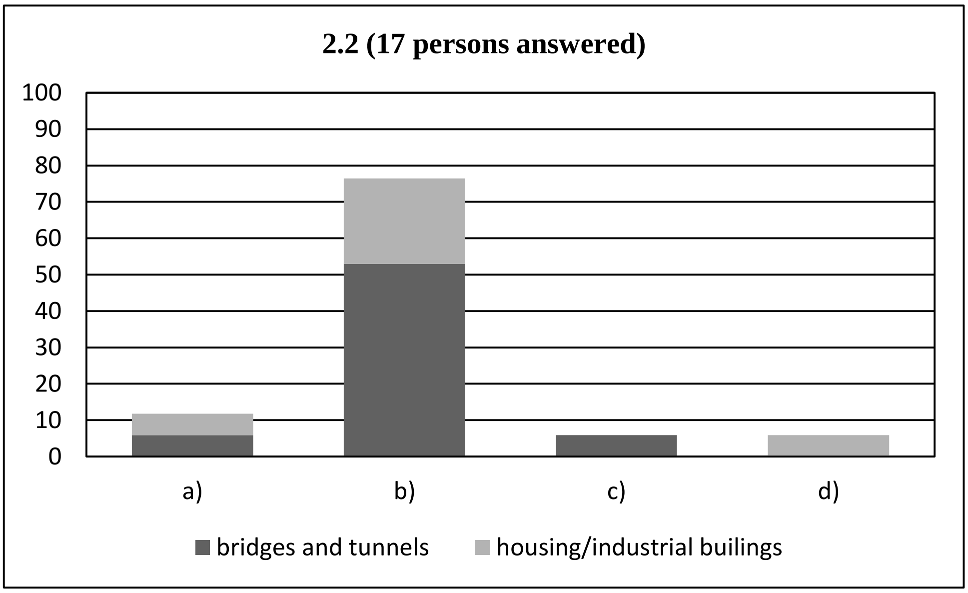


Figure K.3 Result from Question 2.2.

K.4 Survey question 3

19 people took part in answering on question number 3. The answers are presented in Table K.9. Table K.10 present the answers in percent.

Table K.9 Answers to Question 3.

3																				**Summary:**
a)																1	1		1	3
b)	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	19
c)	1					1		1		1		1				1				5
																			Total=	27

Table K.10 Answers in percent to Question 3.

3	Housing/industrial buildings	%	Bridges and tunnels	%
a)	0	0	3	16
b)	12	63	7	37
c)	5	24	1	3

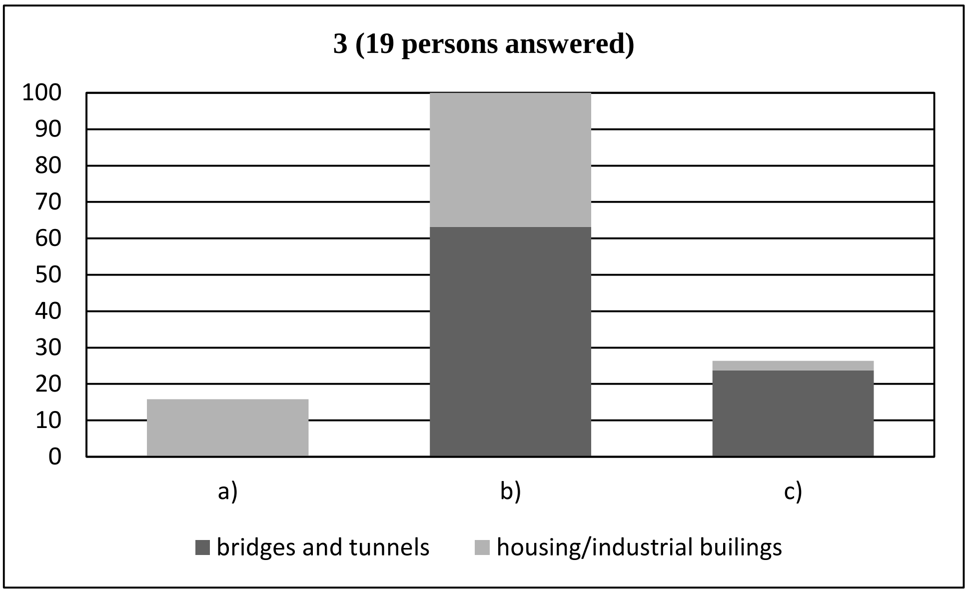


Figure K.4 Result from Question 3.

K.5 Survey question 4

18 people took part in answering on question number 4.1. The answers are presented in Table K.11. Table K.12 present the answers in percent.

Table K.11 Answers to Question 4.1.

4.1																				Summary:
a)	1				1	1	1		1					1	1	1	1		1	10
b)	1			1	1	1	1			1	1			1	1	1	1			11
c)	1			1	1	1								1	1	1	1			8
d)															1	1				2
e)	1				1		1					1		1	1	1	1			8
f)	1	1			1		1					1	1	1	1	1	1	1	1	12
g)	1				1		1					1		1	1	1	1		1	9
h)		1				1	1	1				1	1	1	1			1		9
i)	1	1			1	1	1	1				1	1	1			1	1	1	11.5
j)						1						1	1	1						4
Mostly				b)	b)	b)	i)			b)	b)	i)		b)	a)	b)	b)	f) or h)		
																			Total=	84.5

Table K.12 Answers in percent to Question 4.1.

4.1	Housing/industrial buildings	%	Bridges and tunnels	%
a)	5	28	5	28
b)	7	39	4	22
c)	4	22	4	22
d)	0	0	2	11
e)	4	22	4	22
f)	5	28	7	39
g)	4	22	5	28
h)	5	28	4	22
i)	7	39	5	25
j)	2	11	2	11

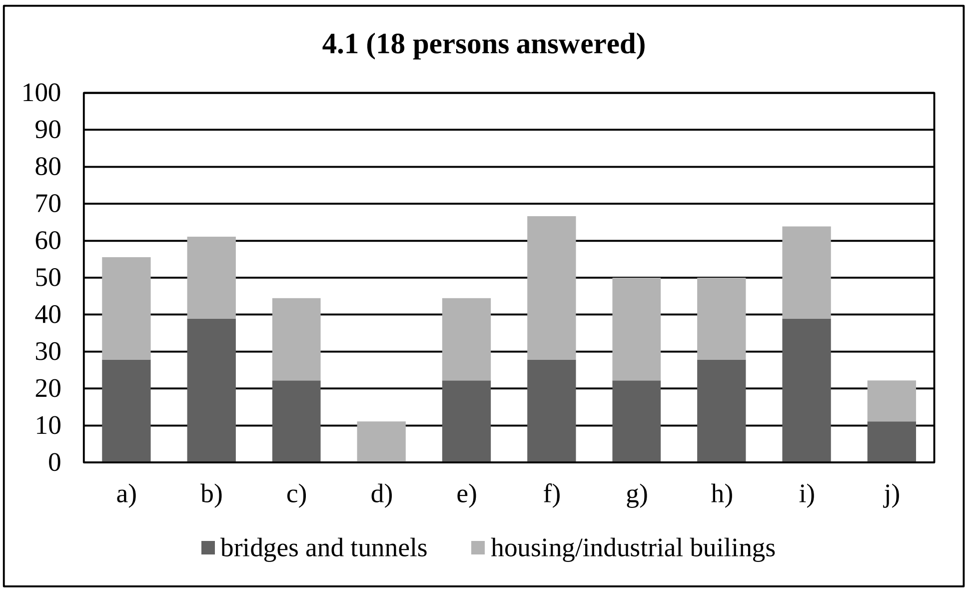


Figure K.5 Result from Question 4.1.

17 people took part in answering on question number 4.2. The answers are presented in Table K.13. Table K.14 present the answers in percent.

Table K.13 Answers to Question 4.2.

4.2																					**Summary:**
Yes	1			1	1	1							1			1	1	1	1		9
No		1					1	1	1	1	1			1	1						8
																				Total=	17

Table K.14 Answers in percent to Question 4.2.

4.2	Housing/industrial buildings	%	Bridges and tunnels	%
Yes	4	24	5	29
No	6	35	2	12

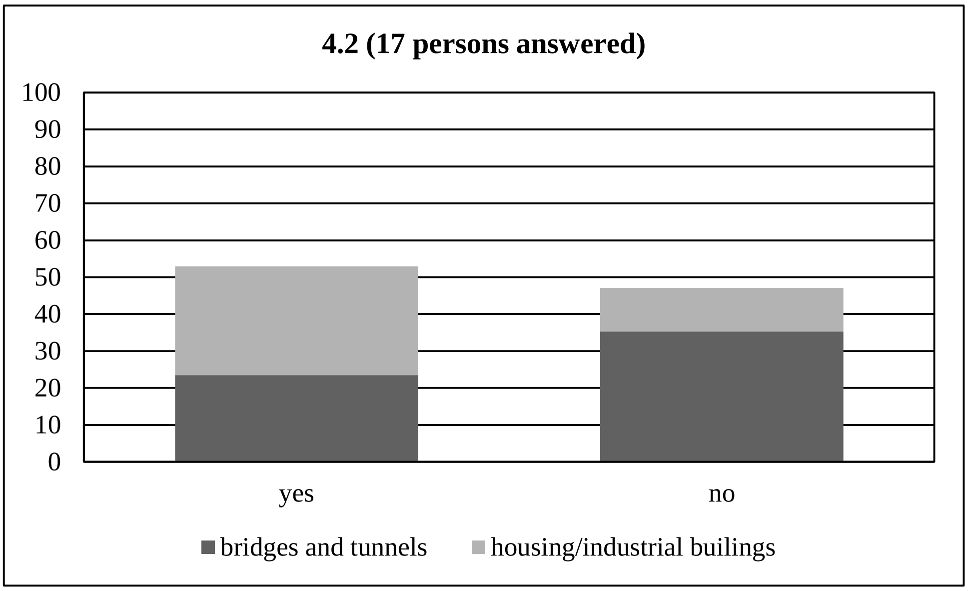


Figure K.6 Result from Question 4.2.

8 people took part in answering on question number 4.3. The answers are presented in Table K.15. Table K.16 present the answers in percent.

Table K.15 Answers to Question 4.3.

4.3																				**Summary:**
a)																			1	1
b)	1			1	1												1			4
c)																1				1
d)																				0
e)																	1			1
f)													1					1		2
g)																				0
h)																				0
i)																				0
j)																				0
																			Total=	9

Table K.16 Answers in percent to Question 4.3.

4.3	Housing/industrial buildings	%	Bridges and tunnels	%
a)	0	0	1	13
b)	3	38	1	13
c)	0	0	1	13
d)	0	0	0	0
e)	0	0	1	13
f)	0	0	2	25
g)	0	0	0	0
h)	0	0	0	0
i)	0	0	0	0
j)	0	0	0	0

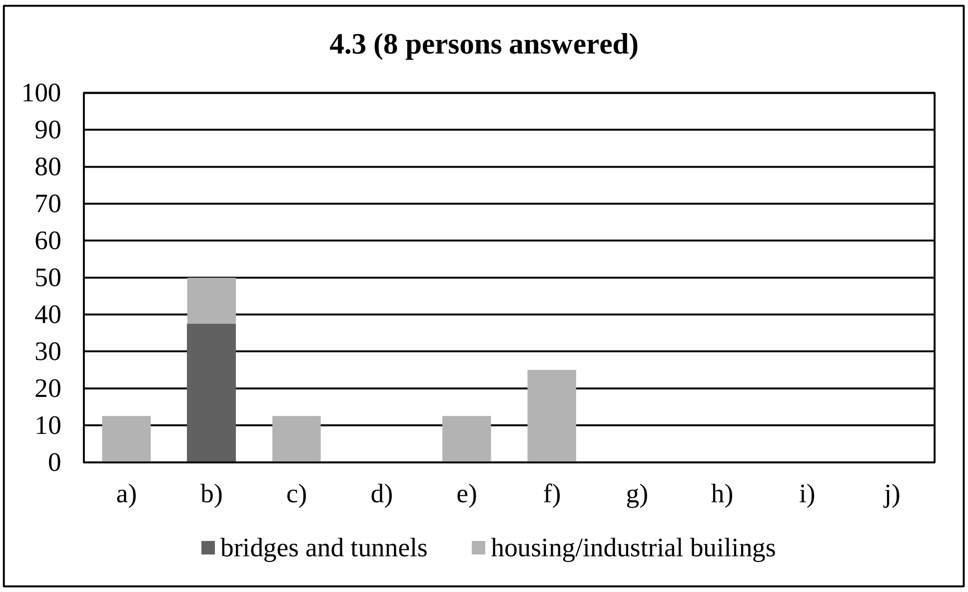


Figure K.7 Result from Question 4.3.

K.6 Survey question 5

19 people took part in answering on question number 5.1. The answers are presented in Table K.17. Table K.18 present the answers in percent.

Table K.17 Answers to Question 5.1.

5.1																				**Summary:**
Yes	1	1			1	1	1	1		1	1	1			1					10
No			1	1					1				1	1		1	1	1	1	9
																			Total=	19

Table K.18 Answers to in percent Question 5.1.

5.1	Housing/industrial buildings	%	Bridges and tunnels	%
Yes	9	47	1	5
No	3	16	6	32

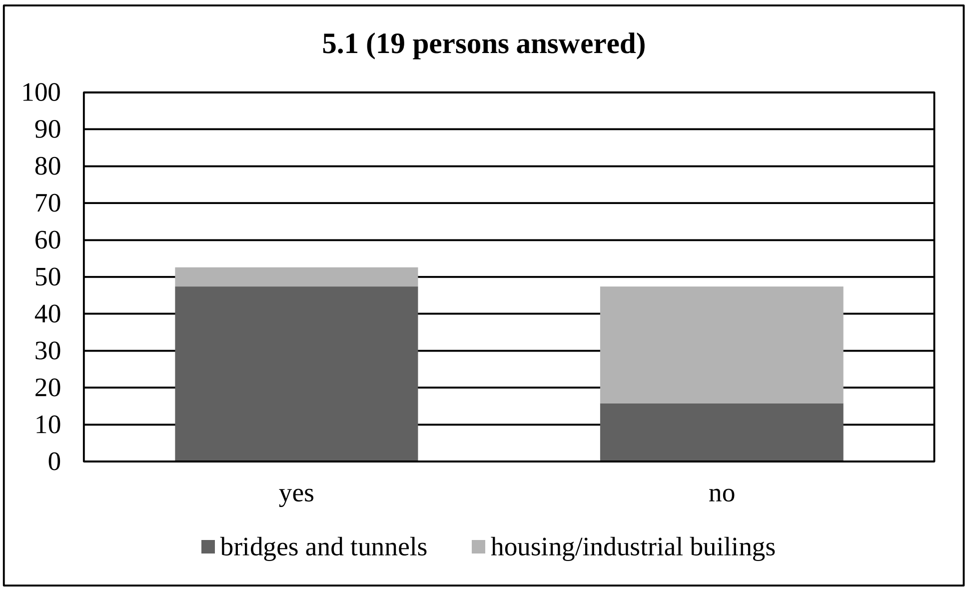


Figure K.8 Result from Question 5.1.

16 people took part in answering on question number 5.2. The answers are presented in Table K.19. Table K.20 present the answers in percent.

Table K.19 Answers to Question 5.2.

5.2																				**Summary:**
a)							1		1		1	1		1	1	1	1	1	1	10
b)	1	1			1	1		1		1										6
																			Total=	16

Table K.20 Answers in percent to Question 5.2.

5.2	Housing/industrial buildings	%	Bridges and tunnels	%
a)	4	25	6	38
b)	6	38	0	0

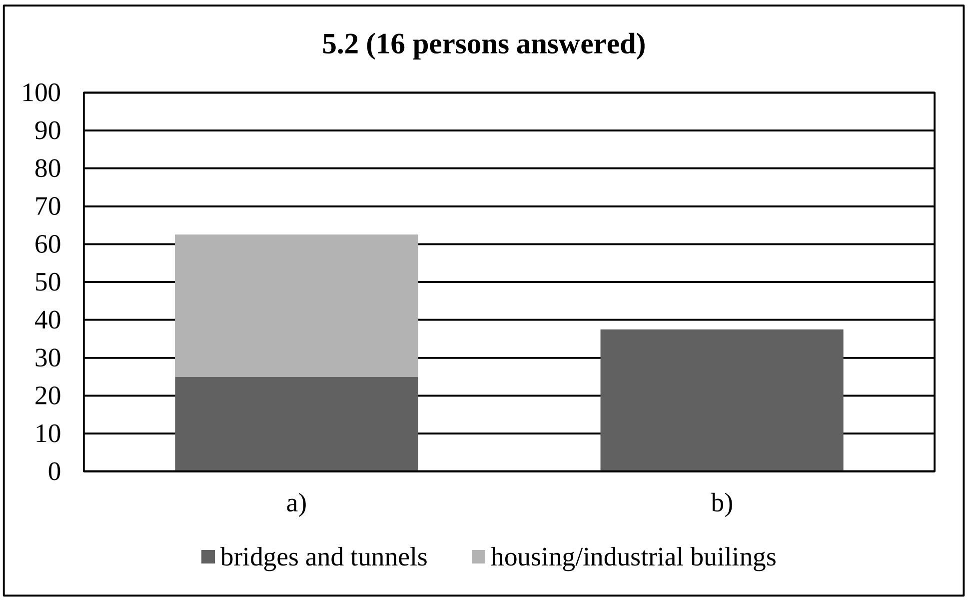


Figure K.9 Result from Question 5.2.

10 people took part in answering on question number 5.3. The answers are presented in Table K.21. Table K.22 present the answers in percent.

Table K.21 Answers to Question 5.3.

5.3																				**Summary:**
Yes	1	1			1	1	1	1		1		1			1					9
No											1									1
																			Total=	10

Table K.22 Answers in percent to Question 5.3.

5.3	Housing/industrial buildings	%	Bridges and tunnels	%
Yes	8	80	1	10
No	1	10	0	0

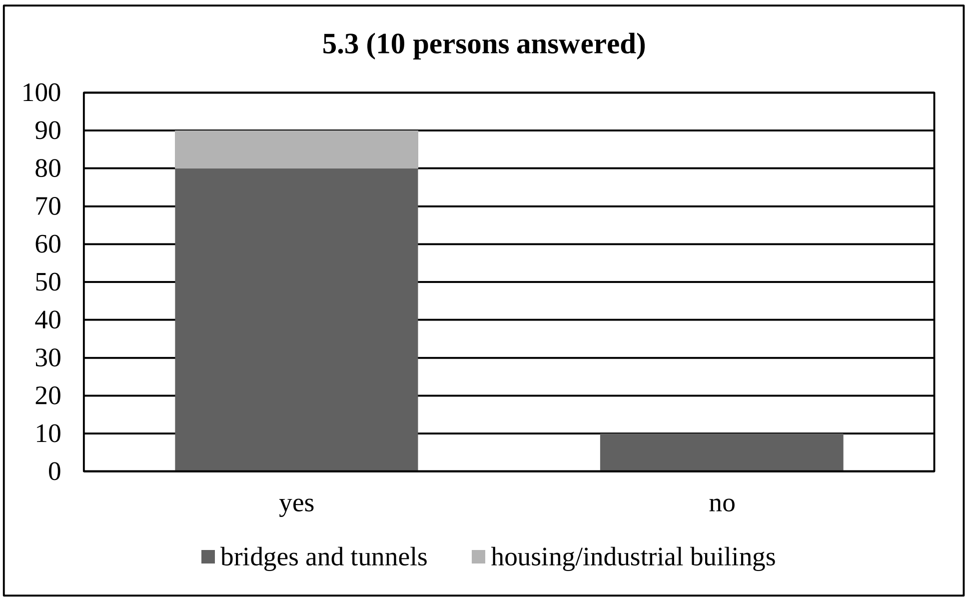


Figure K.10 Result from Question 5.3.

16 people took part in answering on question number 5.4. The answers are presented in Table K.23. Table K.24 present the answers in percent.

Table K.23 Answers to Question 5.4.

5.4																				Summary:
a)		1				1	1	1	1	1				1				1		8
b)				1							1						1		1	4
c)	1				1										1	1				4
																			Total=	16

Table K.24 Answers in percent to Question 5.4.

5.4	Housing/industrial buildings							%		Bridges and tunnels					%
a)				6				38					2		13
b)				2				13					2		13
c)				2				13					2		13

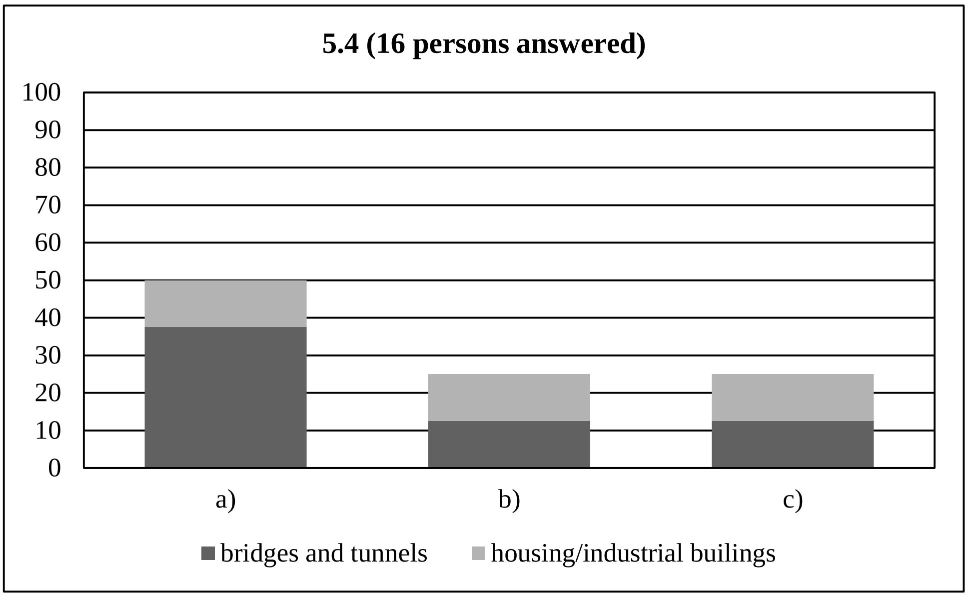


Figure K.11 Result from Question 5.4.

K.7 Survey question 6

19 people took part in answering on question number 6. The answers are presented in Table K.25. Table K.26 present the answers in percent.

Table K.25 Answers to Question 6.

6																					**Summary:**
a)																					0
b)			1						1	1											3
c)	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1		19
																				Total=	22

Table K.26 Answers in percent to Question 6.

6	Housing/industrial buildings	%	Bridges and tunnels	%
a)	0	0	0	0
b)	3	16	0	0
c)	12	63	7	37

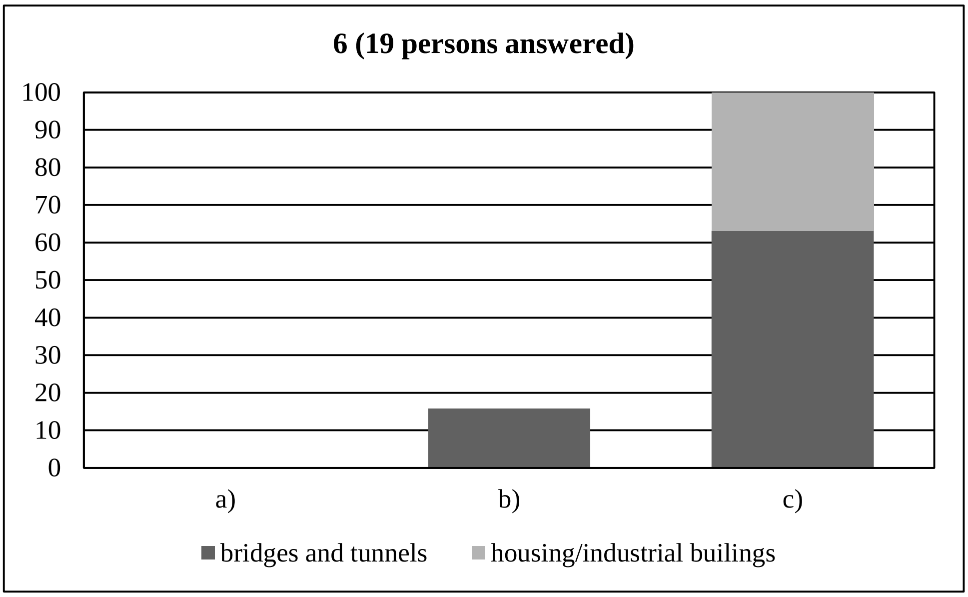


Figure K.12 Result from Question 6.

K.8 Survey question 7

12 people took part in answering on question number 7. The answers are presented in Table K.27. Table K.28 present the answers in percent.

Table K.27 Answers to Question 7.

7																				**Summary:**
a)	1	1	1		1	1			1						1	1		1	1	10
b)	1		1	1	1										1		1			6
																			Total=	16

Table K.28 Answers in percent to Question 7.

7	Housing/industrial buildings	%	Bridges and tunnels	%
a)	6	50	4	33
b)	4	33	2	17

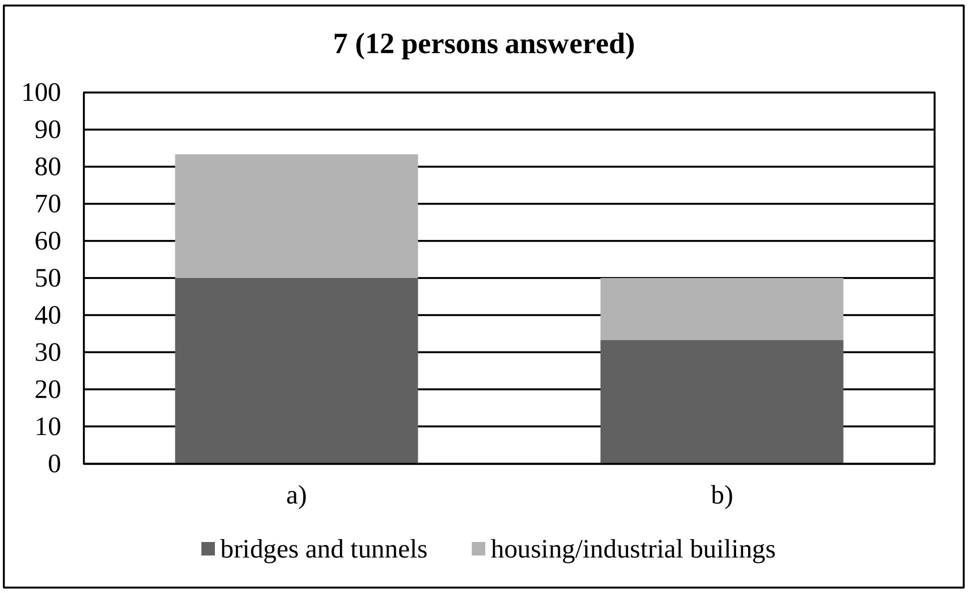


Figure K.13 Result from Question 7.

K.9 Survey question 8

18 people took part in answering on question number 8. The answers are presented in Table K.29. Table K.30 present the answers in percent.

Table K.29 Answers to question 8.

8																				**Summary:**
a)	1	1			1	1		1			1									6
b)	1				1													1		3
c)			1	1		1	1		1	1			1			1	1	1	1	11
d)									1											1
e)														1	1					2
																			Total=	23

Table K.30 Answers in percent to Question 8.

8	Housing/industrial buildings	%	Bridges and tunnels	%
a)	6	33	0	0
b)	2	11	1	6
c)	6	33	5	28
d)	1	6	0	0
e)	0	0	2	11

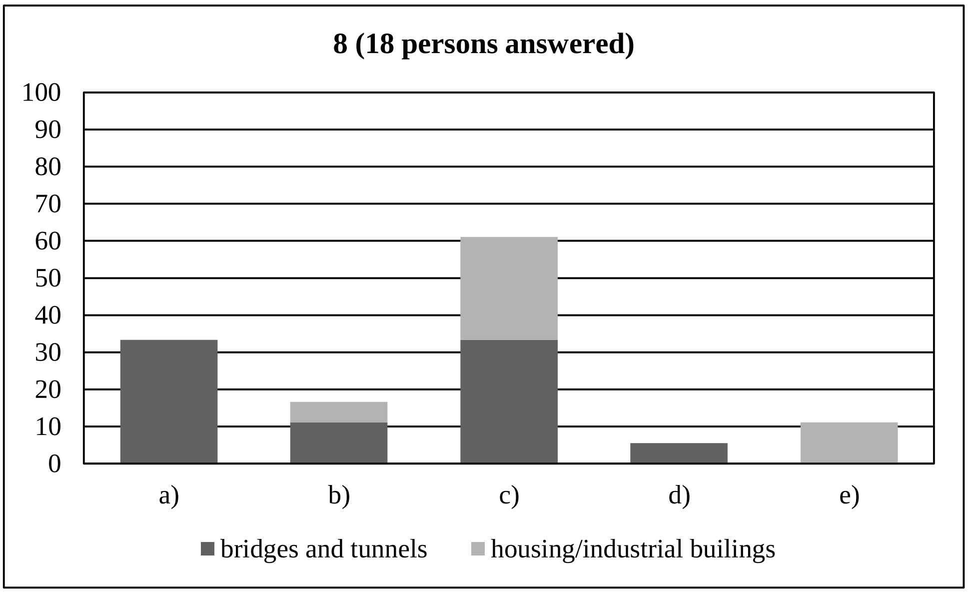


Figure K.14 Result from Question 8.