Master's thesis

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

Lillian Sangvik

Review of the proposed rules for Eurocode 5 - Part 1-1, focusing on the cross-section and member verifications

Master's thesis in Civil and Environmental Engineering
Supervisor: Haris Stamatopoulos
June 2022

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TITLE:

Review of the proposed rules for Eurocode 5 - Part 1-1, focusing on the cross-section and member verifications

Gjennomgang av de foreslåtte reglene for Eurokode 5 – Del 1-1

BY:

Lillian Sangvik

SUMMARY:
The objective of this thesis is to review "CEN/TC 250/SC 5 N 1489: Consolidated draft prEN 1995-1-1 with markups" (EN1995-1-1-Draft). EN1995-1-1-Draft will be compared to "Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings" (EN1995-1-1). This thesis focuses on the design rules for cross-sections and members. The different topics covered in this thesis are the basis of design, structural analysis, cross-section verifications, stability of members, members with special geometry, and serviceability limit state design. The two major topics connections and Cross Laminated Timber (CLT) are not covered in this thesis.

The EN1995-1-1-Draft contains more rules and guidance on the topics covered in EN1995-1-1. EN1995-1-1-Draft also covers topics not covered by EN1995-1-1. One of these new topics include holes in members. EN1995-1-1-Draft includes more design checks for combined stresses than EN1995-1-1 has. This includes the combined shear stress and tensile or compressive stresses perpendicular to the grain, the combined shear stresses from two axis bending, and the combined torsion and bending shear stresses. EN1995-1-1-Draft has included a new method to calculate the buckling, the κ-method. This method gives in some cases a utilisation below 0 for the member when buckling is considered. CEN/TC 250/SC 5 has proposed a new alternative method that fixes this problem.

RESPONSIBLE TEACHER: Haris Stamatopoulos

SUPERVISOR: Haris Stamatopoulos

CARRIED OUT AT: Department of Structural Engineering

Abstract

The objective of this thesis is to review "CEN/TC 250/SC 5 N 1489: Consolidated draft prEN 1995-1-1 with markups" (EN1995-1-1-Draft). EN1995-1-1-Draft will be compared to "Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings" (EN1995-1-1). This thesis focuses on the design rules for cross-sections and members. The different topics covered in this thesis are the basis of design, structural analysis, cross-section verifications, stability of members, members with special geometry, and serviceability limit state design. The two major topics connections and Cross Laminated Timber (CLT) are not covered in this thesis.

The EN1995-1-1-Draft contains more rules and guidance on the topics covered in EN1995-1-1. EN1995-1-1-Draft also covers topics not covered by EN1995-1-1. One of these new topics include holes in members. EN1995-1-1-Draft includes more design checks for combined stresses than EN1995- 1-1 has. This includes the combined shear stress and tensile or compressive stresses perpendicular to the grain, the combined shear stresses from two axis bending, and the combined torsion and bending shear stresses. EN1995-1-1-Draft has included a new method to calculate the buckling, the κ-method. This method gives in some cases a utilisation below 0 for the member when buckling is considered. CEN/TC 250/SC 5 has proposed a new alternative method that fixes this problem.

Sammendrag

Målet med denne oppgaven er å gjennomgå "CEN/TC 250/SC 5 N 1489: Consolidated draft prEN 1995-1-1 with markups" (EN1995-1-1-Draft). EN1995-1-1-Draft blir sammenlignet med "Eurokode 5: Prosjektering av trekonstruksjoner - Del 1-1: Allmenne regler og regler for bygninger" (EN1995-1-1). Denne oppgaven fokuserer på reglene for kontroll av elementene og tverrsnittene til dem. De forskjellige temaene denne oppgaven dekker er grunnlag for prosjektering, konstruksjonsanalyse, dimensjonering av tverrsnitt, knekking av elementer, elementer med varierende tverrsnitt og design for bruksgrensetilstanden. To store temaer som denne oppgaven ikke dekker er forbindelser og regler for massivtre.

EN1995-1-1-Draft inneholder flere regler og tilleggsinformasjon på temaene dom er dekket i EN1995-1-1. EN1995-1-1-Draft dekker også temaer som ikke inngår i EN1995-1-1. Et av disse temaene er elementer med hull. EN1995-1-1-Draft har lagt til flere kontroller for kombinasjoner av spenninger enn det EN1995-1-1 har. Disse nye kontrollene er for kombinasjon av skjærspenninger og spenninger vinkelrett på fiberne, kombinasjon av skjærspenninger for bøying om to akser, og for kombinasjon av skjærspenninger fra torsjon og bøying. EN1995-1-1-Draft har introdusert en my metode for å sjekke et element for knekking, κ metoden. Denne metoden kan i noen tilfeller gi en negativ utnyttelse av et element når det skal sjekkes for knekking. CEN/TC 250/SC 5 har kommet med et nytt forslag til regler for knekking som ikke har dette problemet.

Preface

This thesis was written as the final part of the two-year Master's Degree Programme "Civil and Environmental Engineering" at Norwegian University of Science and Technology (NTNU). The thesis was written in the course "TKT4950 - Structural Engineering, Master's Thesis" at the Department of Structural Engineering.

I am grateful to Associate Professor Haris Stamatopoulos and Postdoctoral Fellow Francesco Mirko Massaro for the opportunity to gain valuable insight into the forthcoming revised "Eurocode 5 - Part 1-1". It has been interesting to see how much new information and rules are presently included in the draft of "Eurocode 5 - Part 1-1" and to investigate how some of the changes affect the design of a member.

I would like to thank my supervisor, Associate Professor Haris Stamatopoulos, for valuable guidance throughout this semester and interesting conversations about the topic of this thesis. I would also like to thank Postdoctoral Fellow Francesco Mirko Massaro for valuable assistance on the topic of this thesis and help throughout the semester.

Trondheim, June 2022

Lillian Sangvik

Lillian Sangvik

Contents

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List of Abbreviations

CEN	European Committee for Standardization
CLT	Cross laminated timber
CRC	Corrosion resistance class
GLT	Glued laminated timber
LTB	Lateral torsional buckling
LVL	Laminated veneer lumber
MC	Moisture content
PLY	Plywood
PT	Project team
SC	Subcommittees (Applies in chapter 1)
SC	Service class (Applies from chapter 2)
SLS	Serviceability Limit State
ST	Structural timber
TC	Technical committee
ULS	Ultimate Limit State
WG	Working group

List of Symbols

Latin upper case letters

A	cross-section area
A_D	cross-section of the diagonals in the bracing
B	bending stiffness of a bracing system
B	floor width
B_d	bending stiffness of a bracing system
B_{ef}	effective width
C_E	atmospheric exposure category
$E_D A_D$	lever arm of diagonals
$(EI)_L$	bending stiffness along the floor span
$(EI)_{ST}$	bending stiffness of discrete bending member at mid-span transverse to floor span
$(EI)_T$	ending stiffness transverse to the floor span
E_0	modulus of elasticity parallel to grain
$E_{0,d}$	design modulus of elasticity parallel to grain
$E_{0,05}$	fifth percentile of the axial modulus of elasticity
F_d	design value of an action
F_E	critical Euler load
F_h	vertical force caused by the assumed weight of a walking person
$F_{r,d}$	load in a truss connection
$F_{t,90,Ed}$	design force in the reinforcement
$F_{t,90,M,Ed}$	design tensile force perpendicular to the grain from the transfer of bending stresses around the hole
$F_{t,90,V,Ed}$	design tensile force perpendicular to the grain from the transfer of shear stresses around the hole
$G_{k,1}$	characteristic value of permanent actions
G_0	fifth percentile of the shear modulus
G_{05}	fifth percentile value of the shear modulus
$G_{0,05}$	fifth percentile characteristic value of the shear modulus
I_m	mean modal impulse
I_{tor}	torsional moment of inertia
I_x	torsional moment of inertia
I_y	second moment of inertia about the y-axis
$I_{y,ef}$	second moment of inertia about the strong y-axis
I_z	second moment of inertia about the z-axis
Kd	stiffness of the fasteners
KR	stiffness of the restraints
$KR - B$	stiffness of connection restraint to bracing
$KS - R$	stiffness of connection of the restraint to the member to be stiffened
Ky,d	spring stiffness
L	length
L	span of the floor

M^*	modal mass
$M_{ap,d}$	design moment at the apex
M_d	design moment
$M_{res,d}$	design bending moment from the frame action around the hole
$M_{y,crit}$	critical bending moment about the strong y-axis
$M_{y,d}$	design value of moment about y-axis
$M_{y,Rd}$	value of moment resistance about y-axis
$M_{z,d}$	design value of moment about z-axis
$M_{z,Rd}$	value of moment resistance about z-axis
N_d	design axial force
Q	forces for design of end and intermediate restraints
$Q_{i,d}$	forces in the restraints resulting from the stiffened member
$Q_{k,j}$	characteristic value of an accompanying variable action
$Q_{k,1}$	characteristic value of the leading variable action
$Q_{S-R,d}$	forces of the connections of each to be stiffened member to the restraints
R	response factor
R_d	design value of the resistance
T_D	torsion moment
T_E	timber exposure category
V	volume
V_d	design shear force
V_{ref}	reference volume
W_{net}	section modulus of the net cross-section
W_{res}	section modulus of the residual cross-section
W_y	section modulus for bending about the strong y-axis

Latin lower case letters

a	distance parallel to grain from the line of action of the support reaction to the corner of the notch
a_{ecc}	factor for the eccentric position of the load
a_R	spacing of the restraint members
a_{rms}	root mean square value of acceleration
a_z	the distance from the centroid of the cross-section to the point of the applied load
$a_{z,w}$	axial stiffness of diagonal including connection stiffness at both ends
$a_{z,y}$	number of diagonals of the bracing
a_1	spacing of the reinforcement parallel to grain at the height of the beam axis
a_1	factor for the moment shape distribution
a_2	factor for the position of the load, depending on the moment distribution and boundary conditions
b	width of a cross-section
c_b	factor accounting for the boundary conditions for in-plane buckling
d	diameter of the circular hole
d_{hole}	applicable diameter of the hole
e	imperfection from straightness or flatness
e	eccentricity of the centre of the hole to the neutral axis of the member
$e_{0,y}$	initial imperfection alongside the y-axis
$e_{0,z}$	initial imperfection alongside the z-axis
$f_{c,0,d}$	design compressive strength parallel to grain
$f_{c,0,k}$	characteristic compressive strength parallel to grain
$f_{c,90,d}$	design compressive strength perpendicular to grain
f_d	design value of a strength property

f_k	characteristic value of a strength property
$f_{m,d}$	design bending strength
$f_{m,k}$	characteristic bending strength
$f_{m,y,d}$	design bending strength for bending about the y-axis
$f_{m,z,d}$	design bending strength for bending about the z-axis
$f_{m,0,d}$	design bending strength parallel to grain in plane
$f_{m,90,d}$	design bending strength perpendicular to grain in plane
$f_{t,0,d}$	design tensile strength parallel to grain
$f_{t,0,k}$	characteristic tensile strength parallel to grain
$f_{t,90,d}$	design tensile strength perpendicular to grain
$f_{v,d}$	design shear strength
$f_{v,k}$	characteristic shear strength
$f_{v,k,ref}$	reference characteristic shear strength of the material
f_w	walking frequency
f_1	floor fundamental frequency
$f_{1,beam,1}$	fundamental frequency of the supporting beam on one side span end
$f_{1,beam,2}$	fundamental frequency of the supporting beam on the other side span end
$f_{1,rigid}$	fundamental frequency when supported on rigid supports
h	height of the beam
h_{ap}	height of the beam at the apex
h_d	height of the hole
h_{ef}	effective height of the notched part
h_h	height of the rectangular hole
h_{ref}	reference height
h_{res}	distance from the edge of the hole to the edge of the member
h_{rl}	height of the beam below the hole
h_{rp}	height of the plane reinforcement above and below the holes edges
h_{ru}	height of the beam above the hole
i	notch inclination
i_{bow}	bow imperfection
i_{dev}	deviation imperfection
k_{cr}	factor for the effects of cracks
k_{creep}	factor to account for the effects of creep
$k_{c,y/z}$	factor for the effect of imperfections, for buckling about y- and z-axis, respectively
$k_{c,\alpha}$	load arrangement factor
$k_{c,1}$	factor considering the effect the difference in moisture content in Service Class 1 and Service Class 2 on the compressive strength
$k_{c,90}$	load arrangement factor
k_{def}	factor for the evaluation of creep deformation taking into account the relevant service class
k_{diam}	factor for the stress distribution and the location of the crack onset
k_{dis}	factor for the effect of the stress distribution in the apex zone
$k_{e,1}$	frequency factor considering double span floor
$k_{e,2}$	frequency factor considering the effect of transverse floor stiffness
k_h	depth modification factor
$k_{h,v}$	depth modification factor for shear strength
k_{imp}	factor accounting for the higher modes in the transient response
k_{ka}	factor for the distribution of tensile stresses perpendicular to grain along the beam axis
k_l	factor taking into account the length of the member
k_l	factor for the increased bending stresses in the apex zone
k_m	factor for the effect of imperfections for lateral torsional buckling

$k_{m,\alpha}$ factor accounting for the stress combination at the tapered edge
k_{mod} modification factor taking into account the duration of the load and the moisture content
k_n material constant factor
k_p factor accounting for the material behaviour and the degree of compressive deformation perpendicular to the grain
k_p factor for the tensile stresses perpendicular to grain from the deviation of bending stresses in the apex zone
k_r factor for the strength reduction due to bending of the laminations during production
k_{rad} factor for the effect of the corner radius on the shear stress
k_{red} factor considering the low probability of having the lowest strength over the width at the point of maximum stress
k_{res} factor accounting for higher modes of vibration
k_{shape} factor for the shape of the cross-section
k_{sim} Factor accounting for the likelihood of identical random imperfections occurring simultaneously
k_{space} factor accounting for the possibility of placing up to 3 circular holes at close spacing
k_{sys} system strength factor
k_v adjustment factor for shear strength
k_{var} factor accounting for the effects of service conditions variations
$k_{v,n}$ factor for the effect of the notch
k_{vol} factor for the volume effect
$k_{vol,t}$ factor for the volume effects
$k_{y,d}$ equivalent distributed spring stiffness per unit length
$k_{\alpha/\beta}$ factors accounting for the notch geometry
k_τ factor for the increased shear stress
$k_{\tau,c/t}$ factor for the effect of compressive/tensile stresses perpendicular to the grain on the shear strength
$k_{1/2/3/4}$ modification factors
$k_{5/6/7}$ modification factors
l length of the member
l_A distance between edge of hole and the line of the support reaction
l_c length of the contact area of the applied force
l_D length of the diagonal bracing
l_{ef} effective spreading length of the compressive stress in grain direction
$l_{ef,d}$ equivalent length of the to be stiffened member
l_h width of the rectangular hole
$l_{k,y/z}$ effective length of the member, in the y-axis or z-axis buckling mode, respectively
$l_{m,ef}$ equivalent member length concerning bending
l_r effective anchorage length
$l_{R,ef}$ effective length of the restraint
$l_{t,90}$ length under tensile stresses perpendicular to grain
$l_{t,90,M}$ distribution length for tensile stresses perpendicular to grain from the transfer of bending stresses
$l_{t,90,V}$ distribution length for tensile stresses perpendicular to grain from the transfer of shear stresses
l_v distance between end of beam and edge of hole
$l_{y/z,ef}$ effective length of the member, in the y-axis or z-axis buckling mode, respectively
l_z distance between the closest edges of two holes in a beam

l_2	the sorter span of a two span floor
m	number of half-sine waves
m	floor mass per unit area
n	number of primary systems in a row
n_B	number of fasteners in the connection of the interior restraint to the bracing
n_C	number of fasteners in the connection of the member to the interior restraint
n_D	number of fasteners in the connection of the diagonals
n_{R-B}	number of fasteners in the connection restraint to the bracing
n_{S-R}	number of fasteners in the connection restraint to the member to be stiffened
p	factor accounting for the probabilistic effects on combined bending and compressive strength of the cross-section
p_d	uniformly distributed load
q_d	acting force on the bracing as half sine-wave shaped distributed loading
q_z	equally distributed load with respect to L
r	radius of the curved part of the beam
r	corner radius of the rectangular hole
r_{in}	inner radius of the curved part of the beam
s	height of the pitch cambered beam
t	lamination thickness
v	deflection
v_{rms}	root mean square value of velocity
$v_{tot,peak}$	total peak velocity response
$v_{1,peak}$	peak velocity response
w	deflection
$w_{beam,1}$	deflection of the supporting beam 1 caused by a vertical static point load of $0{,}5$ kN
$w_{beam,2}$	deflection of the supporting beam 2 caused by a vertical static point load of $0{,}5$ kN
w_c	precamber
w_{creep}	creep deflection
w_{fin}	final deflection
w_{inst}	instantaneous deflection
w_{lim}	limit deflection
$w_{lim,max}$	upper deflection limit
w_{max}	max deflection taking into account the precamber
$w_{net,fin}$	net final deflection
w_{rigid}	deflection of the floor between rigid supports caused by a vertical static point load of 1 kN
w_{sys}	deflection of the floor under the load induced by the floor mass m
w_{tot}	total deflection
w_1	initial part of the deflection under permanent (including quasi-permanent) loads
w_{1kN}	maximum deflection due to a vertical static point-load $F = 1$ kN at the mid-span
w_2	long-term part of the deflection under permanent loads including the quasi-permanent part of variable actions
w_3	instantaneous deflection due to variable actions excluding their quasi-permanent parts

Greek lower case letters

α	angle
α	ratio of the effective height of the notched part to the beam height
α_{ap}	angle of the taper in the middle of the apex zone
α_d	critical factor
α_d	parameter
α^*	parameter
β	ratio between the beam height and the distance from the line of support reaction to the corner of the notch
β_c	material specific imperfection factor for members subjected to compression
$\beta_{c,y}$	imperfection factor for y-axis buckling
$\beta_{c,z/t}$	imperfection factor accounting for the effects of lateral imperfection $e_{0,y}$ on compressive or tensile stresses, respectively
$\beta_{c,0}$	initial imperfection factor for members subjected to axial buckling
β_m	material specific imperfection factor for members subjected to bending
$\beta_{m,0}$	initial imperfection factor for members subjected to bending
β_θ	factor accounting for twist imperfection
γ_M	partial factor for strength properties
γ_R	partial factor for resistance properties
δ	Dischinger-coefficient
δ_y	factor for the shape of y-axis bending moment
ζ	modal damping ratio
η	intermediate factor
θ	twist angle of the cross-section
θ_0	initial twist imperfection
$\kappa_{c,y}$	factor accounting for second order effects for y-axis buckling
$\kappa_{c/t,z}$	factors accounting for second order effects on axial stresses, when bending respectively is acting in combination with compressive or tensile stress
$\kappa_{m,c/t}$	factors for second order effects on bending stresses, when bending respectively is acting in combination with compressive or tensile stress
$\lambda_{c,y/z,rel}$	relative slenderness ratios for y- and z-axis buckling mode, respectively
$\lambda_{c,rel,0}$	threshold above which member is to be verified against buckling
$\lambda_{m,rel}$	relative slenderness ratio for bending
$\lambda_{m,rel,0}$	threshold above which member is to be verified against lateral torsional buckling
$\lambda_{rel,c,y,d}$	design relative slenderness ratio for y-axis buckling
$\lambda_{rel,c/t,z,d}$	design relative slenderness ratio for the z-axis, for compressive or tensile stress
$\lambda_{rel,m,d}$	design relative slenderness ratio for bending
$\lambda_{rel,y/z}$	relative slenderness ratios corresponding to bending about y- and z-axis, respectively
$\lambda_{y/z}$	slenderness ratios corresponding to bending about y- and z-axis, respectively
μ	resonant buildup factor
$\sigma_{crit,y}$	critical stresses for y-axis buckling mode
$\sigma_{crit,z}$	critical stresses for z-axis buckling mode
$\sigma_{c,0,d}$	design compressive stress parallel to grain
$\sigma_{c,90,d}$	design compressive stress perpendicular to grain
$\sigma_{c,\alpha,d}$	design compressive stress at an angle α to the grain
$\sigma_{m,crit}$	critical stress for bending
$\sigma_{m,y,crit}$	critical stress for y-axis bending

$\sigma_{m,y,d}$	design bending stress about y-axis
$\sigma_{m,z,d}$	design bending stress about z-axis
$\sigma_{m,\alpha,d}$	design bending stress at an angle α to grain
$\sigma_{m,0,d}$	design bending stress parallel to grain
$\sigma_{t,0,d}$	design tensile stress parallel to grain
$\sigma_{t,90,d}$	design tensile stress perpendicular to grain
$\sigma_{t,\alpha,d}$	design tensile stress at an angle α to the grain
$\sigma_{y/z,crit}$	critical stress for y- and z-axis buckling mode, respectively
τ_d	design shear stress
$\tau_{max,d}$	maximum design shear stress
$\tau_{tor,d}$	design torsional stress
ϕ	derivation angle from the vertical
$\phi_{c,y}$	intermediate parameter for the calculation of $k_{c,y}$
ϕ_m	intermediate parameter for the calculation of k_m
ψ_G	share of permanent loads in the considered load combination
$\psi_{0,j}$	combination factor applied to an accompanying variable action to determine its combination value
$\psi_{2,j}$	combination factor applied to an accompanying variable action to determine its quasi-permanent value
$\psi_{2,1}$	combination factor applied to the leading variable action to determine its quasi-permanent value

1. Introduction

The objective of this thesis is to review "CEN/TC 250/SC 5 N 1489: Consolidated draft prEN 1995-1-1 with markups" [1] (hereafter referred to as EN1995-1-1-Draft). EN1995-1-1-Draft [1] will be compared to "Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings" [2] (hereafter referred to as EN1995-1-1).

1.1 Second generation Eurocodes

The ones responsible for developing and defining standards at the European level is the European Committee for Standardisation (CEN) [3]. In December 2012, the European Commission had finalised Mandate M/515, inviting CEN to develop a work program for the second generation of Eurocodes [3]. The mandate asked for more user-friendly approach [4] and inclusion of state-of-the-art [3] in the Eurocodes. In May 2013 CEN responded to Mandate M/515 with a plan towards the second generation of Eurocodes [5].

1.1.1 The Eurocode 5 standardisation committee

Inside CEN there are several technical committees (TC) in charge of different topics. The TC in charge of the design rules of civil engineering structures and common buildings is CEN/TC 250. Inside CEN/TC 250 there are different subcommittees (SC). Each of the Eurocodes has a different SC, Eurocode 5 is under SC5. The different SC have several working groups (WG). Different WG covers different subjects. Table 1.1 shows the different topics of the WG in CEN/TC 250/SC 5. Project teams (PT) work closely with a WG and are responsible for the writing process. Experts from different countries participate in the different WG and PT. [3]

Table 1.1: Working groups of CEN/TC 250/SC 5 and their respective subjects [3]

Working group	Subject
WG 1	Cross Laminated Timber (CLT)
WG 2	Timber Concrete Composite (TCC)
WG 3	Cluster: Racking strength, floor vibrations, stability of members etc.
WG 4	Fire
WG 5	Connections
WG 6	Bridges
WG 7	Reinforcement
WG 8	Seismic design
WG 9	Execution
WG 10	Basis of design and materials

Different PT and WG have made additions/changes to the EN1995-1-1-Draft [1]. Each TC and WG have their own colour in which they write in. The different colours make it possible to trace the different additions/changes back to the corresponding PT or WG. Working groups 1,3,5,7,9 and 10 have made additions/changes to EN1995-1-1-Draft [1]. Project teams 1,3 and 5 have also made additions/changes to EN1995-1-1-Draft [1].

1.2 Thesis question

What are the differences between EN1995-1-1 [2] and EN1995-1-1-Draft [1]? How do these differences affect the design of timber structures?

To answer this, a comparison study is performed on EN1995-1-1 [2] and EN1995-1-1-Draft [1]. This includes some case studies. This thesis will not cover the entire EN1995-1-1-Draft [1], only chapters 4-9. Some of the topics in these chapters are also not covered. Topics not covered are rules for CLT and reinforcement, except for reinforcement for special members. The cases in this thesis focuses on Solid Timber (ST), of grade C24, and Glue Laminated Timber (GLT), of grade GL30c. These material grades was chosen because these are some of the most commonly used materials in construction in Norway.

1.3 Thesis structure

This thesis is structured by topic. For each topic, there is a comparison of the rules in EN1995-1-1 [2] and EN1995-1-1-Draft and a discussion of the additions/changes. Some of the topics will also have a section with case studies. The different topics covered in this thesis are the basis of design, structural analysis, cross-section verifications, stability of members, members with special geometry, and serviceability limit state design. After all the different topics are presented and discussed, a shared conclusion is drawn.

2. Basis of design

This chapter includes the three chapters "Basis of design", "Materials" and "Durability" from EN1995-1-1 [2] and EN1995-1-1-Draft [1].

EN1995-1-1-Draft [1] includes information about more materials than EN1995-1-1 [2], most notably cross laminated timber (CLT). With several groups and subgroups for different materials, EN1995-1-1-Draft [1] has included Table 2.1 as an overview of the different abbreviations. In this thesis, the same abbreviations as those presented in Table 2.1 are used. EN1995-1-1 [2] does not have an overview like this, but EN1995-1-1 [2] uses the abbreviations less than EN1995-1-1-Draft [1].

Table 2.1: Products and materials with their abbreviations used in EN1995-1-1-Draft [1]

<table>
<tr><th>No.</th><th colspan="2">Groups and Subgroups</th><th>Product</th><th>Abbreviation</th><th>hEN or EAD</th></tr>
<tr><td>1</td><td rowspan="10">Solid wood based (SWB)</td><td rowspan="4">Structural timber (ST)</td><td>Strength graded structural softwood timber with rectangular cross-section</td><td>ST-c [b]</td><td>EN 14081-1, EN 1912</td></tr>
<tr><td>2</td><td>Strength graded structural hardwood timber with rectangular cross-section</td><td>ST-d [b]</td><td>EN 14081-1, EN 1912</td></tr>
<tr><td>3</td><td>Structural finger jointed timber</td><td>FST [c]</td><td>EN 15497</td></tr>
<tr><td>4</td><td>Glued solid timber</td><td>GST [c]</td><td>EN 14080</td></tr>
<tr><td>5</td><td rowspan="4">Paral. lami. timber (PL)</td><td>Glued laminated timber made of softwoods</td><td>GLT-c</td><td>EN 14080</td></tr>
<tr><td>6</td><td>Block glued glulam</td><td>BGLT</td><td>EN 14080</td></tr>
<tr><td>7</td><td>Glued laminated timber made of hardwoods</td><td>GLT-d</td><td>EAD</td></tr>
<tr><td>8</td><td>Single layered solid wood panel</td><td>SWP-P</td><td>EN 13353</td></tr>
<tr><td>9</td><td rowspan="2">Cross layered timber (CL)</td><td>Cross laminated timber</td><td>CLT</td><td>EAD 130005-00-0304</td></tr>
<tr><td>10</td><td>multi-layered solid wood panel</td><td>SWP-C</td><td>EN 13353</td></tr>
</table>

Continues on the next page

11	Veneer-based (VB) [a,c]	Laminated veneer lumber (LVL)	LVL-P	Softwood LVL with parallel veneers	LVL-P-c	EN 14374
				Hardwood LVL with parallel veneers	LVL-P-d	
12				Hardwood Glued LVL with parallel veneers	GLVL-P-c	EAD 130337-00-0304
				Hardwood Glued LVL with parallel veneers	GLVL-P-d	EAD 130010-01-0304
13			LVL-C	Softwood LVL with crossband veneers	LVL-C-c	EN 14374
				Hardwood LVL with crossband veneers	LVL-C-d	
14				Softwood Glued LVL with crossband veneers	GLVL-C-c	
				Softwood Glued LVL with crossband veneers	GLVL-C-d	ETA
		PLY		Softwood Plywood	PLY-c	EN 13986 and EN 636
15				Hardwood Plywood	PLY-d	EN 13986 and EN 636
16	Strand based (SB) [c]			Oriented strand board	OSB	EN 13986 and EN 300
17				Laminated strand lumber	LSL	EAD 130308-00-0304
18	Wood-fibre-based (WFB) [a,c]			Fibreboard, hard	HB	EN 622-2
19				Fibreboard, medium	MB	EN 622-3
20				Softboard	SB	EN 622-4
21	Wood-particle-based (WPB) [a,c]			Resinoid-bonded particle board	RPB	EN 13986 and EN 312
22				Cement bonded particle board	CPB	EN 13986 and EN 634-2
23	Gypsum-based (GYB)			Gypsum plasterboards	GPB	EN 520
24				Gypsum fibreboards	GFB	EN 15283-2

[a] wood based panels
[b] wood
[c] wood based products

2.1 EN1995-1-1 vs EN1995-1-1-Draft

The three chapters “Basis of design”, “Materials” and “Durability” in EN1995-1-1 [2] are 12 pages in total. The corresponding chapters in EN1995-1-1-Draft [1] are 29 pages in total. This means that EN1995-1-1-Draft [1] has more than double the amount of information. Some additions/changes are minor, while some are more significant. In the following sections, the most significant changes and additions are presented.

2.1.1 Basis of design

Some sections of EN1995-1-1-Draft [1] are new and some are larger than those of EN1995-1-1 [2]. The section about service classes (2.3.1.3 in EN1995-1-1 [2], 4.3.1.4 in EN1995-1-1-Draft [1]) is larger in EN1995-1-1-Draft [1]. In EN1995-1-1 [2] there are 3 service classes (SC), SC 1 with an upper limit of 65% saturation, SC 2 with an upper limit of 85% saturation, and SC 3 with a saturation greater than 85%. The Table 2.2 from EN1995-1-1-Draft [1] shows that service classes 1 and 2 are the same as in EN1995-1-1 [2] while service class 3 has an upper limit of 95% saturation, and EN1995-1-1-Draft [1] has also added service class 4 which is saturated. EN1995-1-1-Draft [1] also includes examples of structures assigned to different service classes. Figure 2.1 is new to EN1995-1-1-Draft [1], it illustrates how the moisture content varies in a cross-section, with high relative humidity in the middle figure and low relative humidity in the right figure.

Table 2.2: Service classes (SC) [1]

Relative humidity of surrounding air at temperature of 20°C	**Service class (SC)**			
	1	2	3	4
Upper limit (Corresponding representative moisture content of SWB-c)	65 % (12 %)	85 % (20 %)	95 % (24 %)	- (saturated)
Yearly average (Corresponding representative moisture content of SWB-c)	50 % (10 %)	75 % (16 %)	85 % (18 %)	- (saturated)

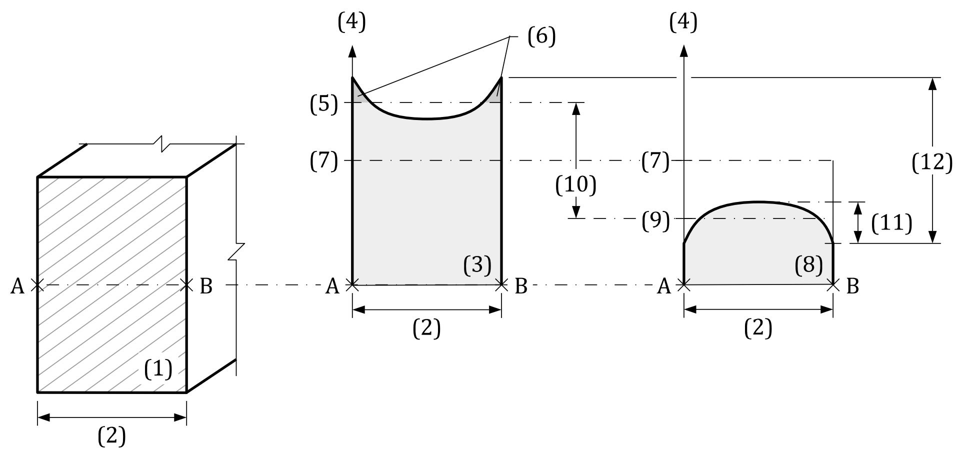


Figure 2.1: Moisture content along the width or depth of a timber cross-section with high and low relative humidity [1]

The formula for the design values have been altered in EN1995-1-1-Draft [1]. EN1995-1-1-Draft [1] have added Πk_i to Equation 2.1, which EN1995-1-1 [2] did not have. Πk_i is the sum of the applicable modification factors except k_{mod}. Some of these modification factors are included in Table 2.3. EN1995-1-1-Draft [1] also changed the symbol for the strength property from $X_{d/k}$ to $f_{d/k}$, while the resistant property still uses the symbol $R_{d/k}$.

$$f_d = k_{mod} \Pi k_i \frac{f_k}{\gamma_M} \tag{2.1}$$

Table 2.3: Selection of applicable modification factors [1]

k_i	Type of factor
$k_{c,90}$	Load arrangement factor
k_{cr}	Factor for effects of cracks
k_h	Depth modification factor
k_{sys}	System strength factor

EN1995-1-1-Draft [1] has implemented two different signs for the partial factors γ_M and γ_R, γ_M for the strength property and γ_R for the resistance property, while EN1995-1-1 [2] only uses γ_M for both properties. The factor values are the same in both versions. There are two new sections in EN1995-1-1-Draft [1]. The first section "4.4 Stiffness values for structural analysis" covers which stiffness value, mean stiffness value or 5th-percentile characteristic value, is to be used in different types of structural analysis. The second section "4.6 Design of connections" covers what forces should be used in the analysis and the load distribution in a connection. This topic is not covered in EN1995-1-1 [2].

2.1.2 Materials/Material properties

The additional information in EN1995-1-1-Draft [1] is mainly due to three things. First, the inclusion of more timber products in the tables. The second reason is because glued solid timber, cross laminated timber, and glued laminated veneer lumber each got their own section. And lastly because EN1995-1-1-Draft [1] has added a new section with shrinkage and swelling values. Some of these values are presented in Table 2.4.

Table 2.4: Shrinkage/swelling value, in % for an average difference of material moisture content of 1% when below the fibre saturation point [1]

Material	**Perpendicular** to grain a)	**Parallel** to grain a)
Softwood	0,25 b)	0,01
Hardwood	0,28-0,45 b)	0,01
Plywood	0,32	0,02
CLT	0,24	0,02-0,04 c)

a) or plane of panel
b) Simplified value averaged from higher shrinkage/swelling value for tangential direction and lower value for radial direction.
c) Swelling and shrinkage values in plane depend on the layup. For typical layups, the lower value refers to the x-direction and the higher to the y-direction.

Due to the introduction of service class 4, as mentioned in Section 4.1.1, the modification factor, k_{mod}, for service class 3 has been altered in EN1995-1-1-Draft [1]. The new k_{mod} factors for a selection of the materials is shown in Table 2.5.

Table 2.5: Values of k_{mod} [1]

Material	Service class	Load-duration of action				
		Permanent	Long term	Medium term	Short term	Instan-taneous
ST	1 and 2	0,60	0,70	0,80	0,90	1,10
	3	0,55	0,60	0,70	0,80	1,00
	4	0,50	0,55	0,65	0,70	0,90
GLT, LVL, PLY	1 and 2	0,60	0,70	0,80	0,90	1,10
	3	0,55	0,60	0,70	0,80	1,00
CLT	1 and 2	0,60	0,70	0,80	0,90	1,10

Table 2.6: Values of k_{def} [1]

Material	Service class			
	1	2	3	4
ST	0,60	0,80	2,00	2,00
GLT	0,60	0,80	2,00	-
LVL-P	0,60	0,80	2,00	-
PLY	0,80	1,00	2,50	-
CLT	0,80	1,00	-	-

Table 2.5 and Table 2.6 show that structural timber is the only material to be used in service class 4. While glue laminated timber, laminated veneer lumber and plywood can be used in service class 3, cross laminated timber can only be used in service class 1 and 2.

The formulation in EN1995-1-1-Draft [1] of when to use the depth modification factor, k_h, for glue laminated timber is changed from *"For depths in bending or widths in tension of glued laminated timber* ***less*** *than 600 mm the characteristic values for $f_{m,k}$ and $f_{t,0,k}$* ***may*** *be increased by the factor k_h"*[2] to *"Depths of glued laminated timber members* ***other*** *than 600 mm subjected to bending the characteristic 5th-percentile value of bending strength $f_{m,k}$* ***shall*** *be multiplied by the factor k_h"*[1]. How the modification factor k_h changes depending on the height of the member and the word *shall* is presented in Figure 2.2, for a selection of materials.

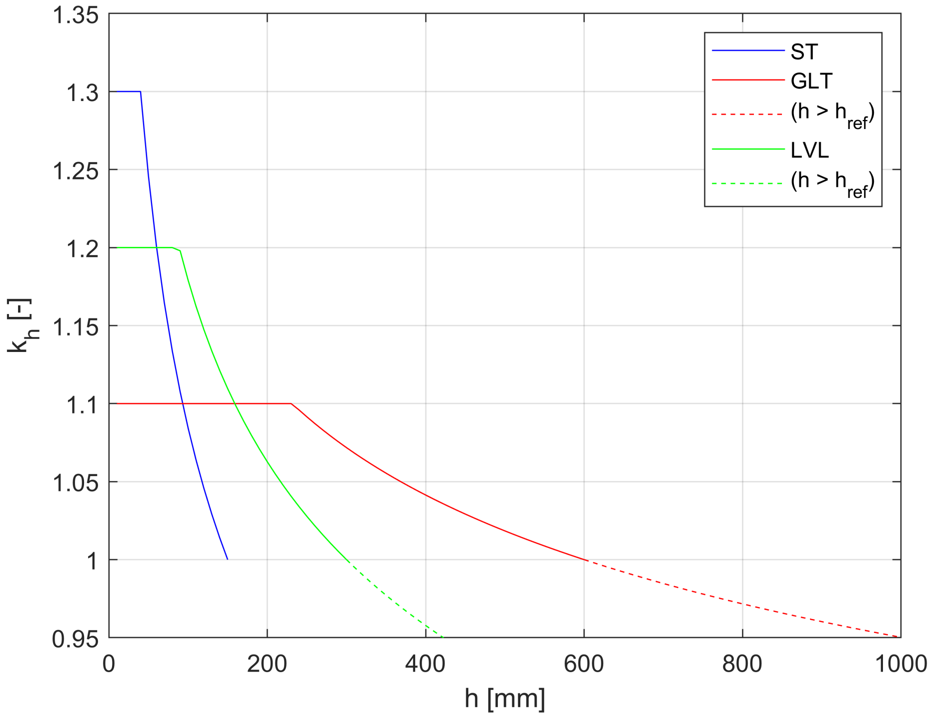


Figure 2.2: Factor k_h, for structural timber (ST), glue laminated timber (GLT), and laminated veneer lumber (LVL), depending on the height of the cross-section

2.1.3 Durability

Chapter 4 of EN1995-1-1 [2] refers only to other standards on the requirements for resistance to biological organisms. It also barley mentions resistance to corrosion of metal fasteners. EN1995-1-1-Draft [1] Chapter 6 goes into the details and requirements to resist biological attack. To avoid wood-destroying fungi, the timber members must have an appropriate moisture content at installation, max 20%, and be protected against increased moisture content in transport, storage and use. EN1995-1-1-Draft [1] has added Figure 2.3 to visualise how members can be protected from rain. All members should also have sufficient durability throughout the design service life from natural ingredients, layup, production, or by use of preservative treatment.

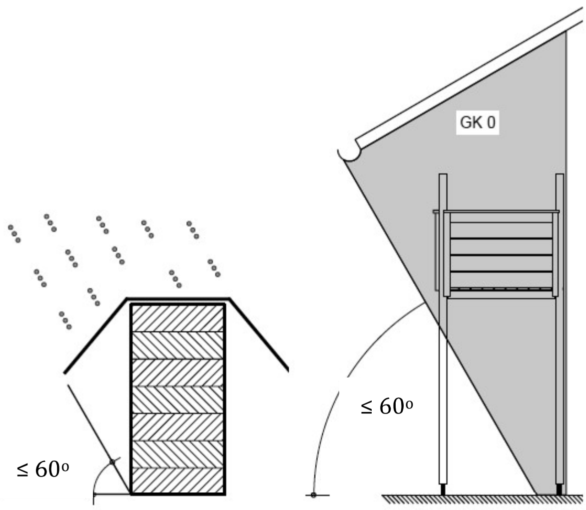


Figure 2.3: Examples for protection of members by rain shadow [1]

EN1995-1-1-Draft [1] also goes into greater detail about corrosion resistance. Metal fasteners have two different exposure categories, timber exposure category T_E and the atmospheric exposure category C_E. The timber exposure category depends on the conditions of the timber. Table 2.7 shows the different definitions of the timber exposure categories and the minimum requirement for the fasteners according to them. The atmospheric exposure category depends on the climatic condition in which the fasteners are located, indoor/outdoor, heated/unheated, and how they are protected from rain. The minimum corrosion resistance class for stainless steel connections with respect to atmospheric exposure category is determined according to EN 1993-1-4:2006/A1:2015.

Table 2.7: Definition of timber exposure categories and examples of resistance classes [1]

Timber exposure category	$\boldsymbol{T_E 1}$	$\boldsymbol{T_E 2}$	$\boldsymbol{T_E 3}$	$\boldsymbol{T_E 4}$	$\boldsymbol{T_E 5}$
Average yearly moisture content (MC)	$\leq$ 10 %	$\leq$ 16 %	16 % $<$ MC $\leq$ 20 %		$>$ 20 %
Service class	SC 1	SC 2	SC 3		SC 4
pH-value of wood species	Any	Any	$>$ 4	$\leq$ 4	Any
Treatment of timber	Any	Any	Untreated	Treated/any	Any
Minimum zinc thickness	-	10 µm	20 µm	55 µm	n.a.
Minimum corrosion resistance class (CRC)	-	CRC I	CRC II	CRC III	CRC III

2.2 Discussion

EN1995-1-1-Draft [1] have material specific parameters for more different materials than EN1995-1-1 [2]. This makes the EN1995-1-1-Draft [1] applicable for more cases with new the materials.

The change in wording on when to use the k_h value for GLT is a significant change, since in EN1995-1-1 [2] k_h was only used when it was favourable and optional. In EN1995-1-1-Draft [1] it is now mandatory to use k_h even when this is unfavourable.

The EN1995-1-1-Draft [1] includes more figures and tables than EN1995-1-1 [2]. This makes EN1995-1-1-Draft [1] clearer and easier to use. Pictures are good for visualisation, and tables give a good overview.

By including more of the rules regarding durability in the EN1995-1-1-Draft [1], and not only referring to other standards, the EN1995-1-1-Draft [1] may be more user-friendly than EN19951-1-1 [2]. The chapter on durability becomes a more relevant part of EN1995-1-1-Draft [1] compared to the chapter in EN1995-1-1 [2].

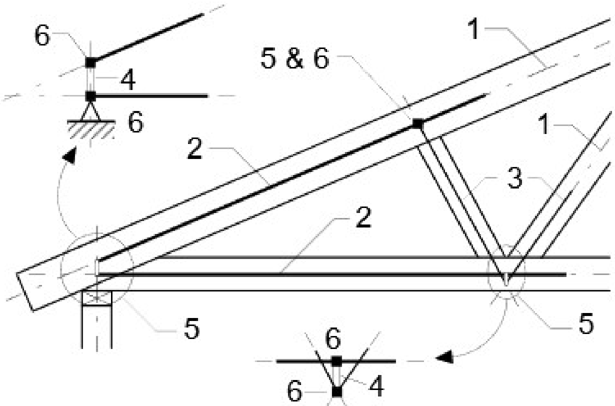
3. Structural analysis

3.1 EN1995-1-1 vs EN1995-1-1-Draft

EN1995-1-1-Draft [1] cover most of the same rules for structural analysis as EN1995-1-1 [2]. In addition to the rules covered by EN1995-1-1 [2], EN1995-1-1-Draft [1] covers more rules for the structural analysis on both member and global level.

3.1.1 Member and global analysis, imperfections and assemblies

Both EN1995-1-1 [2] and EN1995-1-1-Draft [1] states that the system lines shall lie within the member profile. Both also agree that if the system lines do not coincide with the member lines that the eccentricities can be modelled by a fictitious element, as seen in Figure 3.1.



1	System line	4	Fictitious rigid link or stiff beam element
2	Beam member	5	Joint
3	Bar member	6	Nodal point

Figure 3.1: Position of centroidal axes of members (of e.g. trusses) and definitions [1]

EN1995-1-1-Draft [1] have some additional requirements for trusses. All connections should be capable of transferring a load, $F_{r,d}$, given in Equation 3.1, where $F_{r,d}$ is in kN and L is the overall length of the truss in meters. If trusses are loaded predominantly at the nodes, then the sum combined bending and axial compression ratios should be limited to 0,9 instead of 1.

$$F_{r,d} = 1{,}0 + 0{,}1L \tag{3.1}$$

EN1995-1-1-Draft [1] have included rules for minimum values for imperfections in members. The sway of a member, ϕ, in radians, is given in Equation 3.2, where h is the length of the member in meter or height of the structure. The deviation from straightness, e, can be calculated by Equation 3.3, with l as the length of the member, and b the width of the member. The twist imperfection, θ, is calculated according to Equation 3.4, with h as the height of the structure or length of the member.

$$\phi = \begin{cases} \dfrac{1}{200} & \text{for } h \leq 5 \text{ m} \\ \dfrac{1}{200}\sqrt{\frac{5}{h}} & \text{for } h > 5 \text{ m} \end{cases} \tag{3.2}$$

$$e = \begin{cases} \left.\begin{matrix} \dfrac{l}{400} \\ l < 50b : \dfrac{b}{8} \end{matrix}\right\} & \text{for ST} \\ \dfrac{l}{1000} & \text{for GLT} \end{cases} \tag{3.3}$$

$$\theta = e/h \tag{3.4}$$

3.1.2 Braced structures

EN1995-1-1-Draft [1] have added a new section about the stiffness and action effects of braced structures in the structural analysis chapter. EN1995-1-1 [2] have a small section about bracing at the end of chapter 9 "Components and assemblies".

For a braced system, the global imperfection factors may be determined from Table 3.1. The deviation of imperfection, i_{dev}, and the bow imperfection, i_{bow}, is shown in Figure 3.2.

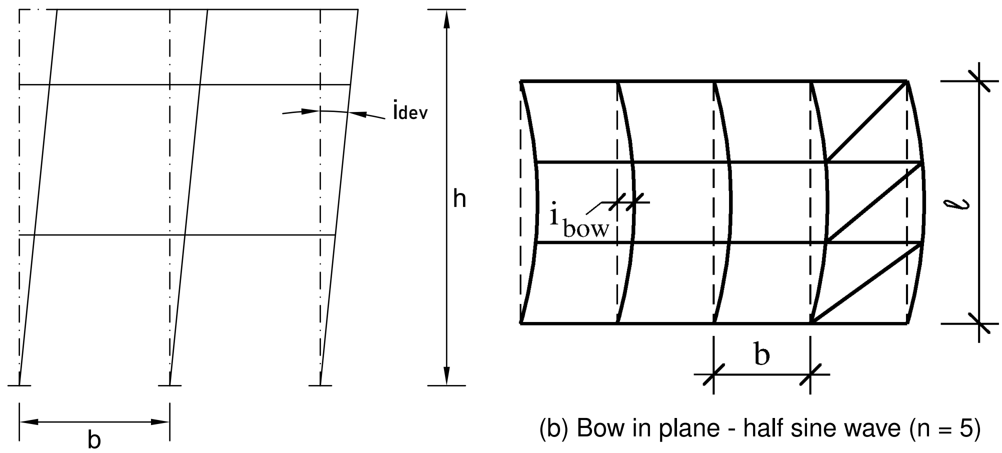
Table 3.1: Equivalent imperfections of bracing system [1]

Deviation imperfection	$$i_{dev} = k_{sim}\phi \tag{3.5}$$
Bow imperfection	$$i_{bow} = k_{sim}e \tag{3.6}$$
Factor accounting for the likelihood of identical random imperfections occurring simultaneously	$$k_{sim} = \sqrt{\frac{1}{2}\left(1 + \frac{1}{n}\right)} \tag{3.7}$$ $$\sqrt{\frac{1}{2}} \leq k_{sim} \leq 1 \tag{3.8}$$ n is the number of primary systems in a row

For a member stiffened by three elastic restraints on elastic bracing, as shown in Figure 3.3, Equation 3.9 may be used to calculate the equivalent spring stiffness, $K_{y,d}$, of the restraint. Equation 3.9 only apply for members with the same cross-section, A, and the same type of fasteners with stiffness K_d. The effective length of the restraint, $l_{R,ef}$, which takes into account the increase of normal force along its length, is calculated by Equation 3.10.

$$\frac{1}{K_{y,d}} = \frac{1}{K_d}\left(\frac{1{,}5}{n_C} + \frac{1{,}5}{n_B}nk_{sim} + \frac{1}{n_D}\frac{l_D^2}{b^2}nk_{sim}\right) + \left(1{,}5l_{R,ef} + \frac{l_D^3}{2b^2}\right) \tag{3.9}$$

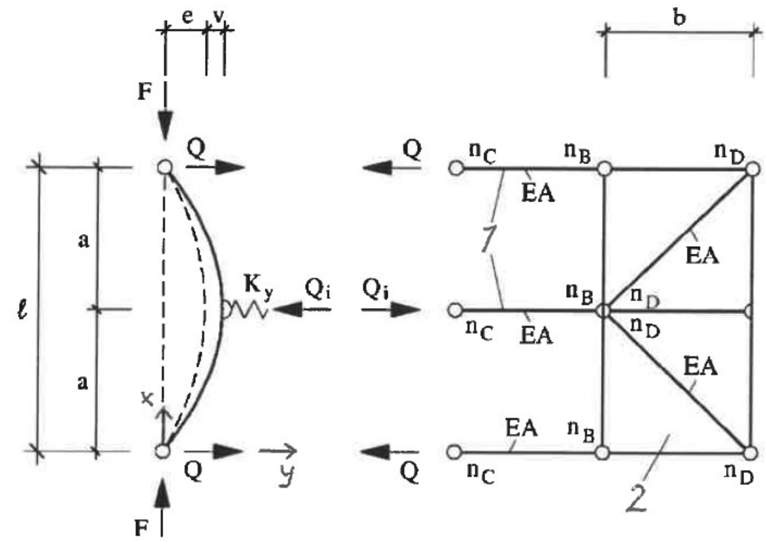
$$l_{R,ef} = \frac{n}{2}(n+1)b \tag{3.10}$$



(b) Bow in plane - half sine wave (n = 5)

(a) Sway in plane (n = 3)

Figure 3.2: Equivalent sway and bow imperfections of a set of *n* primary systems [1]



1	Restraint	n_C	Number of fasteners in the connection of the member to the interior restraint
2	Bracing or secondary system	n_B	Number of fasteners in the connection of the interior restraint to the bracing
1+2	Stiffening	n_D	Number of fasteners in the connection of the diagonals
l_D	Length of the diagonal of the bracing	n	Number of members to be stiffened by the bracing

Figure 3.3: Member in compression braced by lateral supports on elastic bracing ($n = 1$) [1]

The forces $Q_{i,d}$ and $Q_d = Q_{i,d}/2$ of the connections in Figure 3.3 can be determined according to Equation 3.11, where v is the peak value of the elastic deformation.

$$Q_{i,d} \cong 5\frac{e+v}{l}F_d \tag{3.11}$$

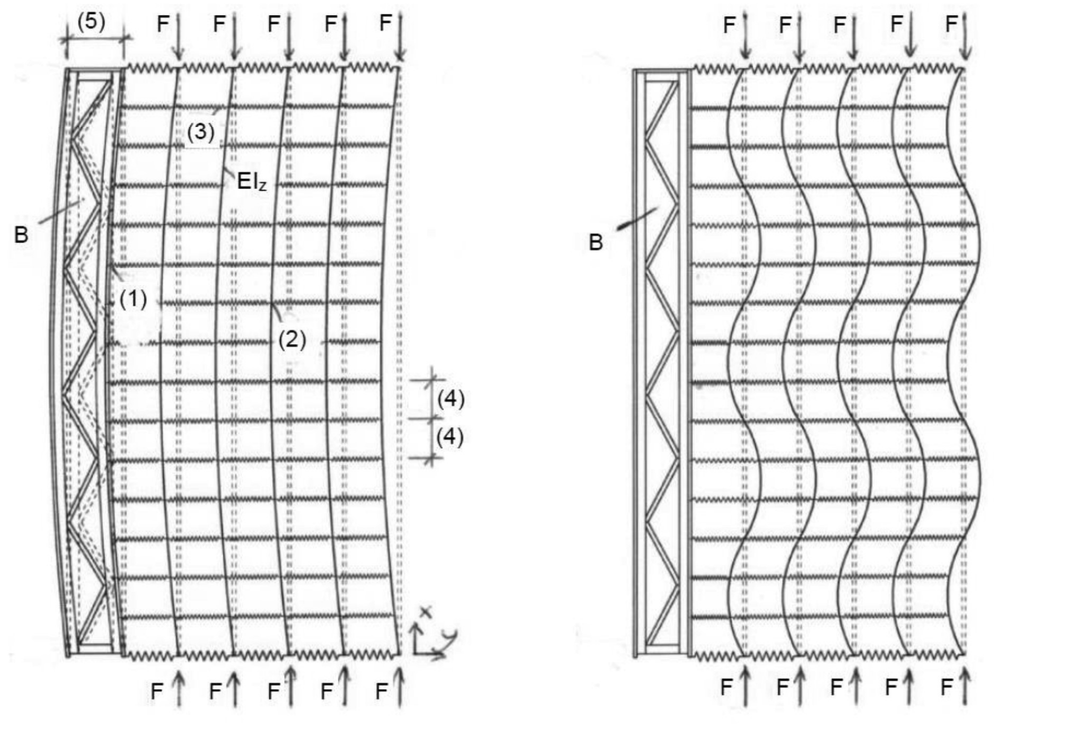
The force $Q_{i,d}$ may be estimated as $Q_{i,d} \cong \frac{F_d}{50}$ if $e \leq l/400$ and the peak deflection, v, satisfy the criteria in Equation 3.12.

$$v = \frac{Q_{i,d}}{K_{y,d}} \leq \frac{l}{500} \tag{3.12}$$

For a system with up to 5 members which need to be stiffened on one side of the bracing system, then $Q_{i,d}$ and $Q_d = Q_{i,d}/2$ can be calculated by Equation 3.13.

$$Q_{i,d} = k_{sim}\frac{nF_d}{50} \tag{3.13}$$

A member needed to be stiffened which has many restraints, see Figure 3.4, where all restraint have the same cross-section A and stiffness K_d, the equivalent distributed spring stiffness per unit length $k_{y,d}$ may be assumed according to the equations in Table 3.2. Different forces acting in the system can also be calculated according to Equation 3.25 and Equation 3.20 or Equation 3.21 in Table 3.2.



a) half sine-wave bow (m = 1) **b)** multiple sine-wave bow (m > 1)

(1) K_{R-B}, stiffness of connection bracing to restraint

(2) K_{S-R}, stiffness of connection of the restraint to the member needed to be stiffened

(3) E_0A, axial stiffness of restraint

(4) a_R, spacing of restraints

(5) Width of bracing

B Bending stiffness of bracing

Figure 3.4: Members in compression braced by lateral supports on elastic bracing ($n = 5$) [1]

Table 3.2: Equations for members with many restraints along their length [1]

Equivalent distributed spring stiffness per unit length	$$\frac{1}{k_{y,d}} = a_R\left(\frac{1}{K_{R-B}} + \frac{1}{K_R} + \frac{1}{K_{S-R}}\right) \tag{3.14}$$
Stiffness of connection restraint to bracing	$$K_{R-B} = \frac{n_{R-B}K_d}{nk_{sim}} \tag{3.15}$$
Stiffness of the restraints	$$K_R = \frac{E_{0,d}A}{l_{R,ef}} \tag{3.16}$$ ($l_{R,ef}$ from Equation 3.10)
Stiffness of connection of the restraint to the member needed to be stiffened	$$K_{S-R} = n_{S-R}K_d \tag{3.17}$$
Force between the restraint and the member to be stiffened	$$Q_{S-R,d} = \frac{\alpha_d}{1-\alpha_d}\frac{l_{ef,d}}{400}k_{y,d}a_R \tag{3.18}$$
Effective length	$$l_{ef,d} = \pi\sqrt{\frac{6E_{0,d}I_z}{F_d + \sqrt{F_d^2 + 12E_{0,d}I_z k_{y,d}}}} \tag{3.19}$$
Peak value of the distributed loading at mid height acting on the bracing	$$q_d = k_{sim}\frac{\pi^2}{l^2}\frac{e}{1-\alpha^*}nF_d \tag{3.20}$$ May instead be estimated as: $$q_d = k_{sim}\frac{8(e+v)}{l^2}nF_d \neq \frac{Q_i}{l/m} \tag{3.21}$$ (where m is the number of sine-wave bows)
α parameters	$$\alpha_d = \frac{F_d}{\frac{\pi^2 E_{0,d}I_z}{l_{ef,d}^2} + \frac{l_{ef,d}^2 k_{y,d}}{\pi^2}} \tag{3.22}$$ $$\alpha^* = \left(\frac{\pi^2}{l^2 k_{y,d}} + \frac{l^2}{\pi^2 B}\right)(F_d - F_E) \tag{3.23}$$ (B is the bending stiffness of the bracing system from Equation 3.26)
Critical Euler load	$$F_d > F_E = \frac{\pi^2 E_{0,d}I_z}{l^2} \tag{3.24}$$
Forces for the members (where n is the number of members to be stiffened by the bracing)	$$Q = \begin{cases} q_d\frac{l}{2} & \text{for adjacent member to bracing} \\ \frac{1}{n}q_d\frac{l}{2} & \text{for farthest member to bracing} \end{cases} \tag{3.25}$$

EN1995-1-1-Draft [1] includes rules for bracing against lateral torsional buckling (LTB). Figure 3.5 shows a model of two members stiffened against lateral torsional buckling with elastic restraints on elastic bracing. The equations for bending members stiffened by elastic restraints on elastic bracing, in Table 3.3, only apply when all bracing diagonals have the same cross-section A_D and connection stiffness K_d. The deflection from Equation 3.28 neglects the bending stiffness of bending members about the z-axis and assumes posts of bracing and their connections to be rigid. The torsional moment from Equation 3.29 applies for a beam with a constant line load at its top and with max $e = l/400$ and $v = l/500$ at the location for the torsional moment.

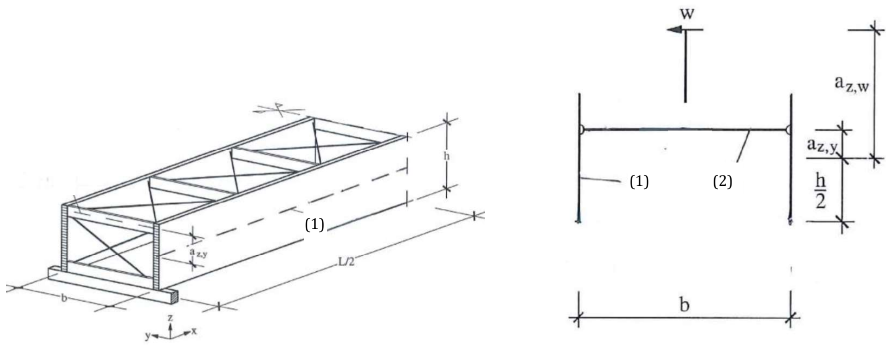


Figure 3.5: Structural model of 2 members stiffened against LTB [1]

Table 3.3: Equations for bending members stiffened by elastic restraints on elastic bracing [1]

Equivalent bracing stiffness	$$\frac{1}{B_d} = \frac{2}{b^2}\frac{1}{E_{0,d}A} + \frac{2}{b^2}\frac{1}{E_{0,d}I_y}(2a_{z,y} - a_{z,w})a_{z,y} + \frac{48}{5E_D A_D L^2}\frac{1}{\sin^2\alpha\cos\alpha} \quad (3.26)$$
α factor	$$\alpha = \arctan\frac{b}{L/n} \quad (3.27)$$
Horizontal deflection	$$v = \frac{5}{384}\frac{wL^4}{B_d} \quad (3.28)$$
Design torsional moment	$$T_d = \frac{M_{y,d}}{80} \quad (3.29)$$
Force in the compression chord of the bending member	$$F_d = \frac{M_d}{h} \quad (3.30)$$
Bracing force caused by n bending members ($n \leq 10$)	$$q_d = k_{sim}\frac{8(k_l e + v)}{l^2}nF_d \quad (3.31)$$
Factor taking into account the length of the member	$$k_l = \min\left\{1; \sqrt{\frac{15}{l}}\right\} \quad (3.32)$$

The last bracing case presented in EN1995-1-1-Draft [1] is bracing for pitched cambered beams. The calculation model is shown in Figure 3.6. If all bracing diagonals have the same cross-section A_D and type of fasteners with stiffness K_d, then the equations in Table 3.4 may be used to calculate the equivalent bracing stiffness and the horizontal deflection.

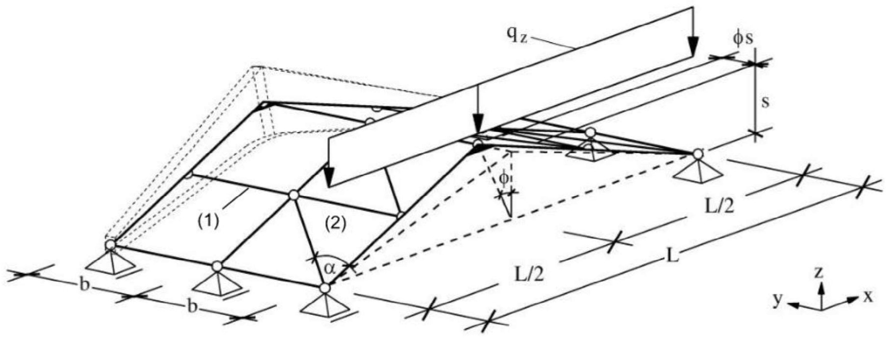


Figure 3.6: Calculation model for braced pitched cambered beams [1]

Table 3.4: Equations for braced pitched cambered beams [1]

Equivalent bracing stiffness	$$\frac{1}{B_d} = \frac{1}{48}\frac{s^2}{b^2}\frac{1}{E_{0,d}I_{y,ef}} + \frac{48}{5E_D A_D L^2}\frac{1}{\sin^2\alpha\cos\alpha} \quad (3.33)$$ (α as in Equation 3.27)
Horizontal deflection	$$v = \frac{5}{384}\frac{\phi q_z L^4}{B_d} \quad (3.34)$$

3.2 Discussion

EN1995-1-1-Draft [1] has changed the minimum value of the deviation for straights, e, to be used in the calculations of the members. The value in EN1995-1-1-Draft [1] is less strict than that in EN1995-1-1 [2].

Bracing is an important part of the design to get a stable structure. The inclusion of design rules related to bracing in the EN1995-1-1-Draft [1] is good. Several different bracing systems are included. This may give the user a better fit between the design case and the rules. The calculations themselves are a bit confusing and it feels a bit unclear where the application of the calculations is to be used.

4. Cross-section verifications

The most loaded cross sections must be checked in the ultimate limit state (ULS). Different design checks must be performed depending on the loading of the member. In this chapter, verifications for tension, compression, bending, shear, torsion, and combination of these are presented.

4.1 EN1995-1-1 vs EN1995-1-1-Draft

4.1.1 Tension and compression

In Table 4.1 the different equations for the verification of tension and compression according to EN1995-1-1-Draft [1] are presented.

Table 4.1: Verifications of tension or compression [1]

Tension parallel to grain	$$\sigma_{t,0,d} \leq k_l f_{t,0,d} \tag{4.1}$$
Tension perpendicular to grain	Rules in section 8.3 [1] apply
Tension at an angle to grain	$$\sigma_{t,\alpha,d} \leq \frac{f_{t,0,d} f_{t,90,d}}{f_{t,0,d}\sin^2\alpha + f_{t,90,d}\cos^2\alpha} \tag{4.2}$$
Compression parallel to grain	$$\sigma_{c,0,d} \leq k_{c,1} f_{c,0,d} \tag{4.3}$$
Compression perpendicular to grain	$$\sigma_{c,90,d} \leq k_p k_{c,90} f_{c,90,d} \tag{4.4}$$
Compression at an angle to the grain	$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,\alpha f_{c,90,d}}}\sin^2\alpha + \cos^2\alpha} \tag{4.5}$$

For tension parallel to the grain Equation 4.1 must be satisfied. In EN1995-1-1-Draft [1], the length modification factor, k_l, is new. Compression parallel to grain must be verified by Equation 4.3. The factor $k_{c,1}$ is introduced in EN1995-1-1-Draft [1] to consider that the compressive strength is affected by the moisture content. For ST and GLT in SC 1 $k_{c,1} = 1{,}2$, and for LVL in SC 2 and 3 $k_{c,1} = 0{,}83$.

EN1995-1-1 [2] does not have any verification of tension at an angle to the grain, but EN1995-1-1-Draft [1] introduces Equation 4.2. For compression at an angle to the grain, Equation 4.5 should be satisfied. From EN1995-1-1 [2] to EN1995-1-1-Draft [1] $k_{c,90}$ have been switched out with $k_{c,\alpha}$. Equation 4.6 is used to find $k_{c,\alpha}$.

$$k_{c,\alpha} = \sqrt{\tan(90 - \alpha) + 1} \tag{4.6}$$

If there is tension perpendicular to the grain, this verification is carried out according to the rules of Section 8.3 of the EN1995-1-1-Draft [1]. For compression perpendicular to the grain Equation 4.4 must be satisfied. The k_p factor is new to EN1995-1-1-Draft [1] and considers the deformation of the material and the importance of a member in a system. For ST, GLT, and CLT see Table 4.2 for the different values of k_p.

Table 4.2: k_p for ST, GLT, and CLT [1]

	Deformation	k_p
Case A a)	2,5%	1,4
Case B b)	10%	2,1
Case C c)	20%	2,7

a) Deformation leads to unacceptable damages to other components or result in member or system instability. e.g. for ST when $h > 5b$.
b) Deformation has no significant effect on member or system instability.
c) Failure of member do not lead to failure in the structure.

How to determine the load arrangement factor, $k_{c,90}$, has changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. In EN1995-1-1 [2] $k_{c,90}$ is constant but varies from 1,0 to 1,75 depending on the load situation and the material. In EN1995-1-1-Draft [1] $k_{c,90}$ may vary between 1,0 and 4,0, and is calculated by Equation 4.7. In Equation 4.7 $k_{c,90}$ is a ratio between the effective spreading length, l_{ef}, of the compressive stress and the contact length, l_c, of the applied force.

$$k_{c,90} = \sqrt{\frac{l_{ef}}{l_c}} \leq 4{,}0 \tag{4.7}$$

4.1.2 Bending

For bending Equation 4.8 and Equation 4.9 must be satisfied according to EN1995-1-1-Draft [1]. The equations are the same as in EN1995-1-1 [2]. The only difference is that the symbol for the reduction factor k_m in EN1995-1-1 [2] has changed to k_{red} in EN1995-1-1-Draft [1]. $k_m = k_{red} = 0{,}7$ for rectangular sections and $k_m = k_{red} = 1{,}0$ for other cross-sections.

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.8}$$

$$k_{red}\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.9}$$

EN1995-1-1 [2] does not have any verification for the bending stresses at an angle to the grain in plane, but EN1995-1-1-Draft [1] introduces Equation 4.10 for bending at an angle to the grain.

$$\sigma_{m,\alpha,d} \leq \frac{f_{m,0,d} f_{m,90,d}}{f_{m,0,d}\sin^2\alpha + f_{m,90,d}\cos^2\alpha} \tag{4.10}$$

4.1.3 Combined bending and tension/compression

For combined bending and axial tension, the verification is the same in both EN1995-1-1 [2] and EN1995-1-1-Draft [1], both of which require Equation 4.11 and Equation 4.12 to be satisfied. Regarding bending, the factor k_m [2] has changed to k_{red} [1].

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.11}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_{red}\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.12}$$

For axial compression and bending Equation 4.13 and Equation 4.14 must be satisfied according to the EN1995-1-1-Draft [1]. For rectangular cross-sections $p = 2$, for other cross-sections $p = 1$. The verification in EN1995-1-1 [2] is almost the same, but it uses $p = 2$ for all cases.

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^p + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.13}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^p + k_{red}\frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{4.14}$$

4.1.4 Shear

In EN1995-1-1 [2], shear verification is similar to Equation 4.15, which is used in EN-1991-1-1-Draft, but the factor k_v is not included in EN1995-1-1 [2]. k_v is an adjustment factor and is calculated according to Equation 4.16. In Equation 4.16 $k_{h,v}$ is the depth modification factor and is calculated according to Equation 4.17. $f_{v,k}$ is the characteristic shear strength of the material. $f_{v,k,ref}$ is the reference characteristic shear strength and it is 2,0 for ST and 2,5 for GLT. k_{var} is a factor that accounts for service conditions such as cracks, frequent severe moisture, or excessive drying. This factor k_{var} has not yet been determined in the EN1995-1-1-Draft [1].

$$\tau_d \leq k_v f_{v,d} \tag{4.15}$$

$$k_v = \text{min}\left\{k_{h,v}k_{var}\frac{f_{v,k,ref}}{f_{v,k}}; 1{,}0\right\} \tag{4.16}$$

$$k_{h,v} = \begin{cases} \text{min}\left\{\left(\dfrac{150\text{ mm}}{h}\right)^{0{,}2}; 1{,}3\right\} & \geq 1{,}0 \quad \text{for ST} \\ \text{min}\left\{\left(\dfrac{600\text{ mm}}{h}\right)^{0{,}1}; 1{,}1\right\} & \geq 1{,}0 \quad \text{for GLT} \end{cases} \tag{4.17}$$

Both EN1995-1-1 [2] and EN1995-1-1-Draft [1] allow a reduction of the shear force close to the supports if the beam is supported over the full member width. The shear force can be calculated at a distance e as shown in Figure 4.1 a) and b). EN1995-1-1-Draft [1] also included the case of reinforcement at the support or the force is transferred by connections at the end grain, as shown in Figure 4.1 c) and d), where the reduction in total shear should not be applied. EN1995-1-1 [2] does not mention anything about the case of reinforcement at the support.

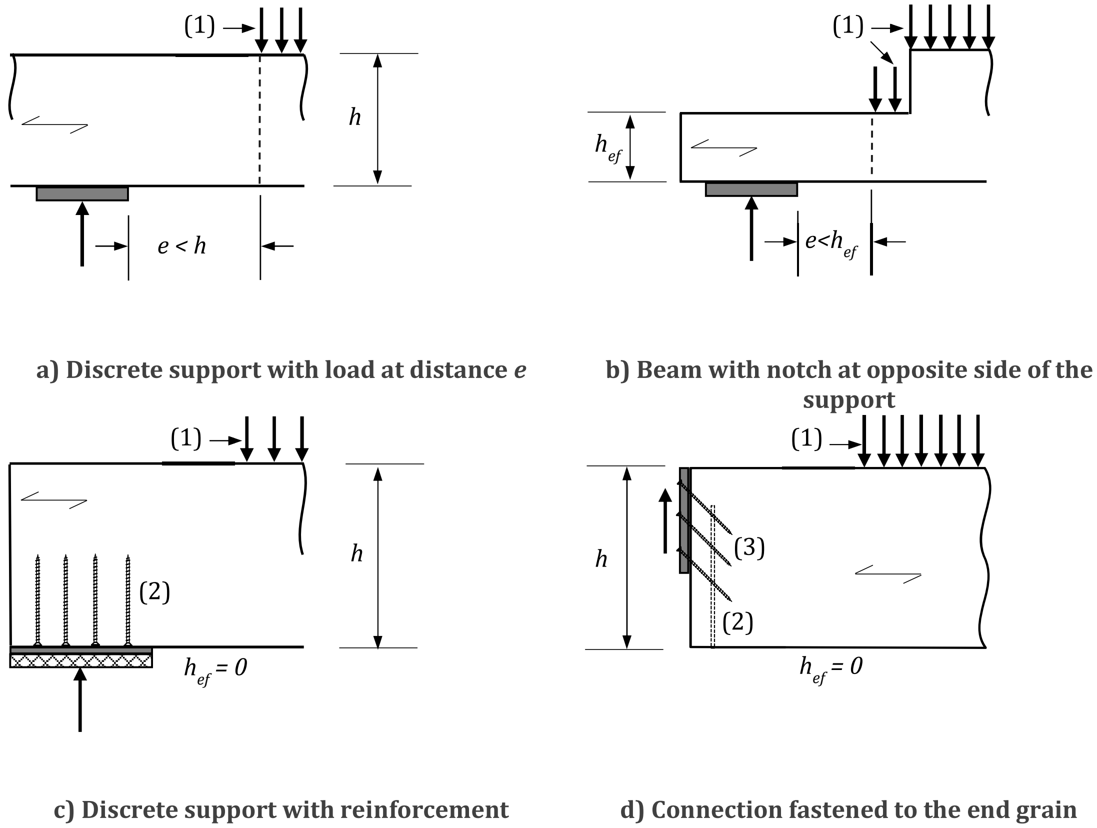


Figure 4.1: Conditions at support to determine the effective shear force [1]
(1) Concentrated or distributed load, (2) Reinforcement, (3) Connector or group of fasteners

Two major changes from EN1995-1-1 [2] to EN1995-1-1-Draft [1] is the inclusion of verification for combined shear from two axis bending, Equation 4.18, and combined shear and compression/tension perpendicular to grain. For compression perpendicular to the grain in combination with shear Equation 4.19 must be satisfied. With increased $\sigma_{c,90,d}$, the τ_c can be increased compared to shear alone. For tension perpendicular to the grain Equation 4.20 must be satisfied. For increased $\sigma_{t,90,d}$, τ_c must be reduced compared to shear alone. The volume effect factor, $k_{vol,t}$, used in Equation 4.20 is calculated according to Equation 4.21. EN1995-1-1-Draft [1] does not give any guidance on what the volume, V, in Equation 4.21 should be.

$$\left(\frac{\tau_{y,d}}{k_v f_{v,d}}\right)^2 + \left(\frac{\tau_{z,d}}{k_v f_{v,d}}\right)^2 \leq 1 \tag{4.18}$$

$$\frac{\tau_d}{f_{v,d}} - 0{,}25\frac{\sigma_{c,90,d}}{f_{c,90,d}} \leq 1 \tag{4.19}$$

$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{vol,t} f_{t,90,d}} \leq 1 \tag{4.20}$$

$$k_{vol,t} \begin{cases} 1{,}0 & \text{for ST} \\ \left(\dfrac{V_{ref}}{V}\right)^{0{,}2} & \text{with } V_{ref} = 0{,}01 \text{ m}^3\text{, for GLT and LVL} \end{cases} \tag{4.21}$$

4.1.5 Torsion

For shear stresses from torsion Equation 4.22 must be satisfied according to EN1995-1-1-Draft [1]. k_{shape} is a factor that accounts for the shape of the cross-section, determined by Equation 4.23. In EN1995-1-1 [2] the k_v factor in Equation 4.22 is not included, and for rectangular cross-sections the maximum value for k_{shape} is 1,3.

$$\tau_{tor,d} \leq k_{shape} k_v f_{v,d} \tag{4.22}$$

$$k_{shape} = \begin{cases} 1{,}2 & \text{for a circular cross-section} \\ \text{min}\left(1 + 0{,}05\dfrac{h}{b}; 2\right) & \text{for a rectangular cross-section} \end{cases} \tag{4.23}$$

If there is torsion in combination with bending, then the EN1995-1-1-Draft [1] requires Equation 4.24 to be satisfied. Equation 4.24 is a combination of Equation 4.18 for two axis bending and Equation 4.22 for torsion. EN1995-1-1 [2] does not have a design check for combined bending and torsion.

$$\frac{\tau_{tor,d}}{k_{shape} k_v f_{v,d}} + \left(\frac{\tau_{y,d}}{k_v f_{v,d}}\right)^2 + \left(\frac{\tau_{z,d}}{k_v f_{v,d}}\right)^2 \leq 1 \tag{4.24}$$

4.2 Example cases

4.2.1 Stresses at an angle to the grain

By using Equation 4.2 and Equation 4.5 it is possible to plot how the design strength changes depending on the angle between the stresses and the grain in a member. Figure 4.2 presents how the design strength varies for C24 for both tension and compression at an angle to the grain.

In Figure 4.2 the graphs are marked for stresses at angles of 0°, 45° and 90° to the grain. It is clear from the figure that the tensile strength is reduced faster than the compressive strength when the angle of the stresses increases. Compared to the strength parallel to the grain (0°), the tensile strength is halved at a angle of just 10° to the grain, while the compressive strength is halved at a 30° angle to the grain.

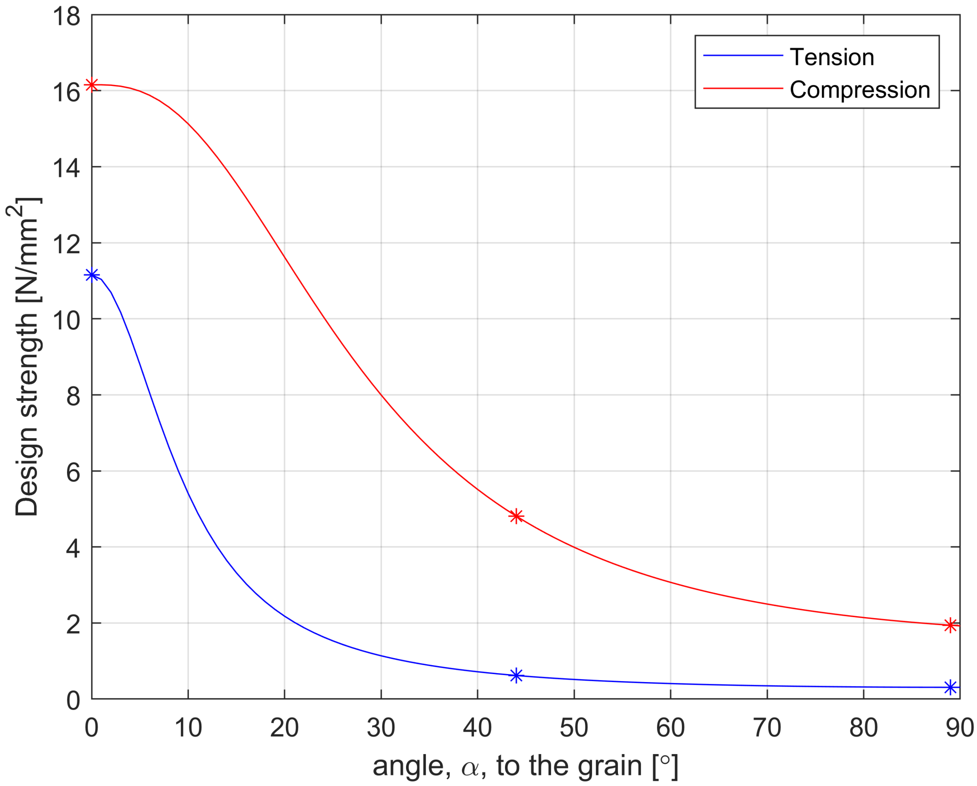
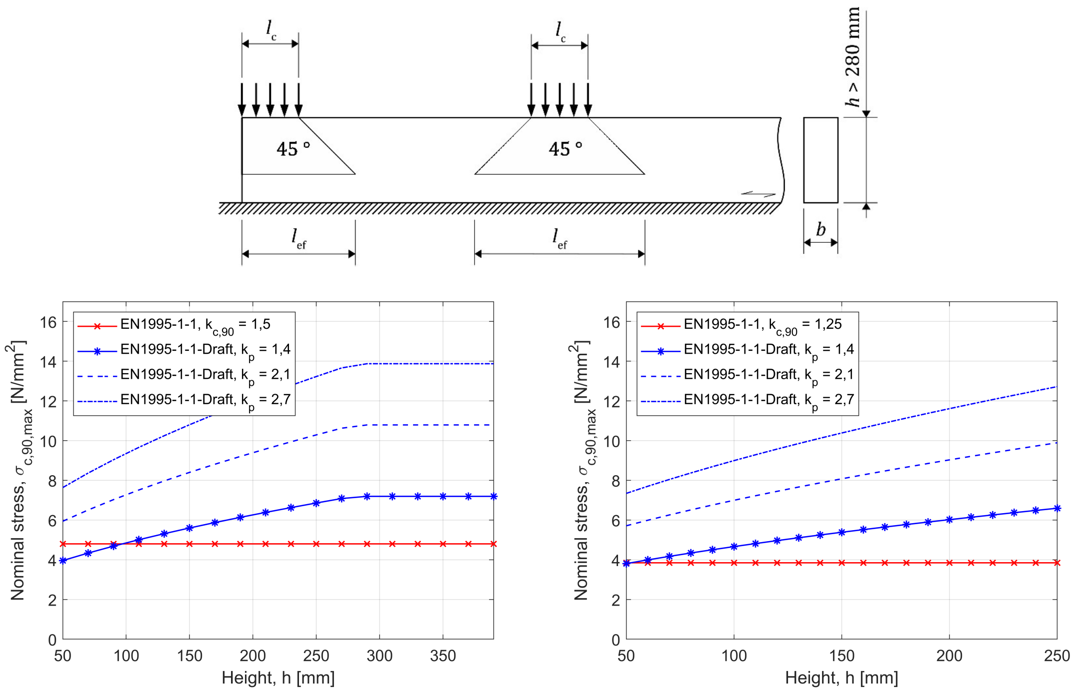


Figure 4.2: Design strength of ST (C24, $k_{mod} = 1{,}0$) for tension and compression at an angle to the grain

4.2.2 Compression perpendicular to grain

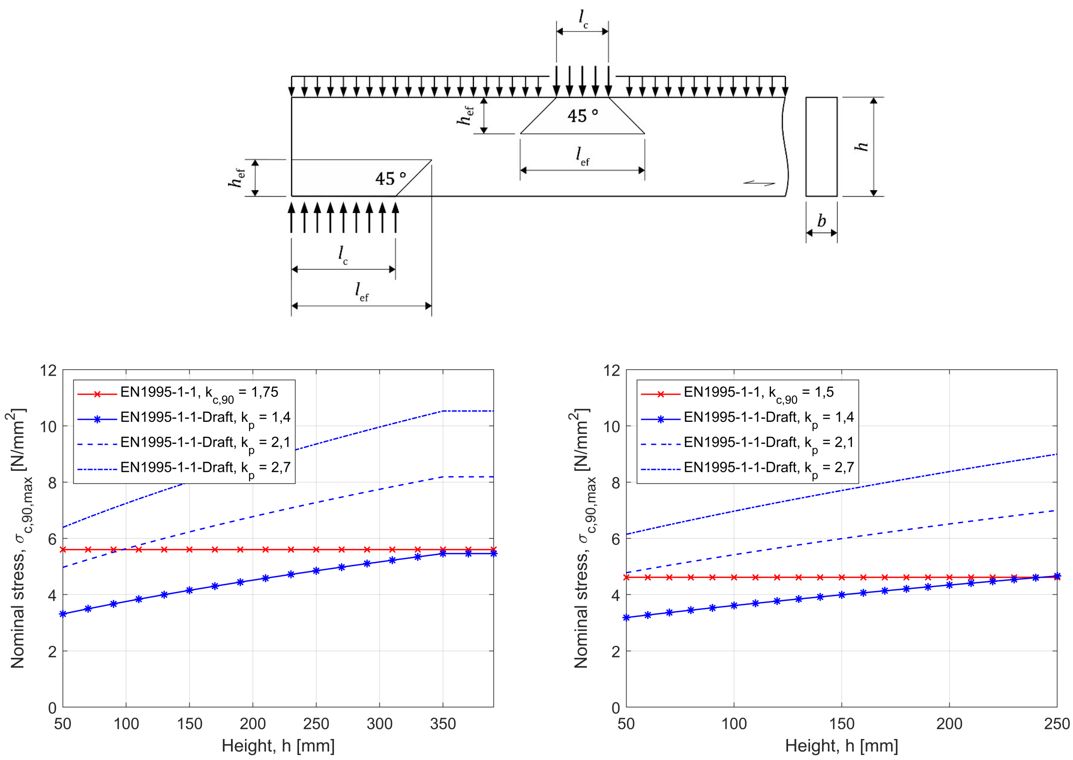
To visualise the differences the calculations have on the design resistance, two cases are tested. First case, with a load on a continuous support, Figure 4.3. The second case has a load or a discrete support at the middle of a beam, Figure 4.4. Both cases are for structural timber in strength class C24 and glue laminated timber in strength class GL30c, with $k_{mod} = 1{,}0$, a load length of 100 mm and a varying height.

In Figure 4.3 and Figure 4.4 the red curve corresponds to the rules of EN1995-1-1 [2], where the $k_{c,90}$ value is used. The blue curves correspond to the rules in EN1995-1-1-Draft [1]. Each of the blue curves is based on a different value of k_p. A higher value of k_p is used for the less critical members of a structure.



(a) GLT, GL30c, $b = 115$ mm (b) ST, C24, $b = 48$ mm

Figure 4.3: Max design load at members on continuous support, with $l_c = 100$ mm, according to EN1995-1-1 [2] and EN1995-1-1-Draft



(a) GLT, GL30c, $b = 115$ mm (b) ST, C24, $b = 48$ mm

Figure 4.4: Max design load at members on discreet support, with $l_c = 100$ mm, according to EN1995-1-1 [2] and EN1995-1-1-Draft

4.2.3 Shear

The effect tension/compression has on the verifications in Equation 4.19 and Equation 4.20 is visualised in Figure 4.5.

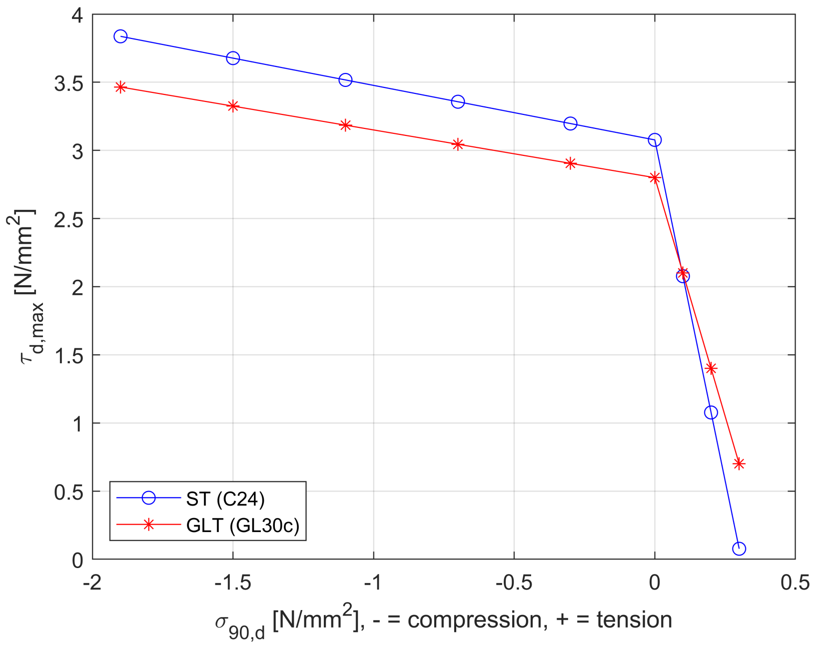


Figure 4.5: Max design shear, for ST (C24) and GLT (GL30c) (with $k_{vol,t} = 1{,}0$), with varying tension/compression perpendicular to grain

The experimental results in Figure 4.6 (the dots) from R. Steiger and E. Gehri [6], show that shear strength falls sharply if the member is subjected to tension perpendicular to the grain. When the member is subjected to increased compression perpendicular to the grain, the shear strength increases. Both Figure 4.5 and Figure 4.6 show that the increase in shear strength due to compression is less than the reduction due to tension.

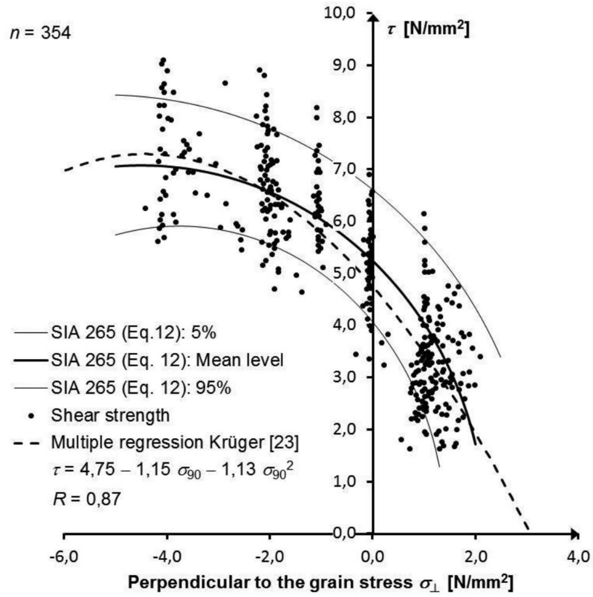


Figure 4.6: Results of combined shear and tension/compression [6]

4.3 Discussion

EN1995-1-1-Draft [1] introduced Equation 4.2 for tension at an angle to the grain. An example of how the angle affects the design strength is presented in Figure 4.2. Timber has a low strength for tension perpendicular to the grain, and Figure 4.2 shows that the design strength is immediately reduced as soon as the tensile stresses no longer act parallel to the grain.

The rules in EN1995-1-1-Draft [1] allow for more stresses perpendicular to the grain in compression, in less important members of a structure. In Figure 4.3 and Figure 4.4 the differences in the maximum stress perpendicular to the grain by the different k_p values from Table 4.2 are shown. A member in case C with $k_p = 2{,}7$ can be designed to take twice the amount of stresses perpendicular to the grain than a similar member in case A with $k_p = 1{,}4$. For a member with continuous support, as shown in Figure 4.3, the rules in EN1995-1-1-Draft [1] are less strict than in EN1995-1-1 [2]. For a member with discreet support, as shown in Figure 4.4, the rules for the most critical members (Case A: $k_p = 1{,}4$) according to EN1995-1-1-Draft [1] are stricter than the rules in EN1995-1-1 [2]. For a less critical member, the rules are less strict. The only guidance EN1995-1-1-Draft [1] provides on how to determine the k_p factor for a member is by how important the member is in the structure.

The use of the adjustment factor k_v in Equation 4.15 for shear gives some odd results if $k_{h,v} k_{var} \dfrac{f_{v,k,ref}}{f_{v,k}} < 1$. Combining Equation 4.15, Equation 4.16 and Equation 2.1 gives $\tau_d \leq f_{v,d}$ if $k_{h,v} k_{var} \dfrac{f_{v,k,ref}}{f_{v,k}} \geq 1$. If $k_{h,v} k_{var} \dfrac{f_{v,k,ref}}{f_{v,k}} < 1$ then the shear capacity is determined by the reference characteristic strength, $f_{v,k,ref}$, rather than the characteristic shear strength of the material, as shown in Equation 4.25. This means that the shear strength of a material does not matter for the shear capacity. Solid timber of grade C24 has $f_{v,k} = 4{,}0$ Mpa, while EN1995-1-1-Draft [1] gives $f_{v,k,ref} = 2{,}0$ Mpa for solid timber. This reduces the shear strength by half. If the k_{var} factor, which is not yet determined, does not depend on the strength classification of the material, then the shear verification will be independent of the strength of the material and only depends on the type of material.

$$\tau_d \leq \left(k_{h,v} k_{var} \frac{f_{v,k,ref}}{f_{v,k}} \right) \left(k_{mod} \Pi k_i \frac{f_{v,k}}{\gamma_M} \right) = k_{h,v} k_{var} k_{mod} \Pi k_i \frac{f_{v,k,ref}}{\gamma_M} \tag{4.25}$$

Compression / tension perpendicular to the grain will affect the shear properties of a member, as shown in the experimental results in Figure 4.6. The linear approach of Equation 4.19 and Equation 4.20 does not fit exactly with the results from Figure 4.6. Figure 4.6 shows a spread of the results, so any formula will never give exact results compared to reality. Although Equation 4.19 and Equation 4.20 do not represent the exact increase / reduction in shear, it gives a conservative approximation of the correlation between shear and tension / compression perpendicular to the grain.

Most of the cross-section verifications are the same, or with just small changes, e.g. a symbol is changed, from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. EN1995-1-1-Draft [1] has included some new verifications to cases not included in EN1995-1-1 [2], e.g. tension at an angle to the grain and a combination of shear and stresses perpendicular to grain, as discussed above.

5. Stability of members

Buckling is a vital part of the design of a member. EN1995-1-1 [2] and EN1995-1-1-Draft [1] have rules for lateral flexural buckling and lateral torsional buckling (LTB). Buckling problems can be reduced by bracing against it. Bracing affects the effective length of a member. Annex D in EN1995-1-1-Draft [1] provides rules on how to calculate the effective length for different bracings. The effective lengths presented in this chapter apply for members without bracing.

5.1 EN1995-1-1 VS. EN1995-1-1-Draft

EN1995-1-1 [2] has one set of verification rules for buckling, while EN1995-1-1-Draft [1] currently has two different sets, the k-method and the κ-method. In both EN1995-1-1 [2] and EN1995-1-1-Draft [1] (k-method) the lateral flexural buckling can be calculated for both y- and z-direction, and the formulas are the same for both directions, only the subscripts change depending on the direction.

5.1.1 Simplified verifications, k_c and k_m methods

The equations to calculate relative slenderness changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. In EN1995-1-1 [2] Equation 5.1 is used to calculate the relative slenderness. Equation 5.2 is the general formula to calculate the slenderness ratio. This equation is not present in EN1995-1-1 [2]. Combining Equation 5.1 and Equation 5.2 gives Equation 5.5. In EN1995-1-1-Draft [1], Equation 5.1 shall be used to calculate the relative slenderness. The critical stress, $\sigma_{y,crit}$, is calculated according to Equation 5.4. Equation 5.6 is a combination of Equation 5.3 and Equation 5.4. Equation 5.5 and Equation 5.6 is the same equation, with only some notation differences, so even though the equation for relative slenderness has changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1], the relative slenderness is still the same.

Table 5.1: Calculation of relative slenderness according to EN1995-1-1 [2] and EN1995-1-1-Draft [1]

EN1995-1-1 [2]	EN1995-1-1-Draft [1]
$$\lambda_{rel,y/z} = \frac{\lambda_{y/z}}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \quad (5.1)$$ $$\lambda_{y/z} = \frac{l_{k,y/z}}{\sqrt{\frac{I_{y/z}}{A}}} \quad (5.2)$$	$$\lambda_{c,y/z,rel} = \sqrt{\frac{f_{c,0,k}}{\sigma_{y/z,crit}}} \quad (5.3)$$ $$\sigma_{y/z,crit} = \frac{\pi^2}{A}\frac{E_{0,05}I_{y/z}}{l^2_{y/z,ef}} \quad (5.4)$$
$$\lambda_{rel,y/z} = \frac{l_{k,y/z}}{\pi}\sqrt{\frac{f_{c,0,k}A}{E_{0,05}I_{y/z}}} \quad (5.5)$$	$$\lambda_{c,y/z,rel} = \frac{l_{y/z,ef}}{\pi}\sqrt{\frac{f_{c,0,k}A}{E_{0,05}I_{y/z}}} \quad (5.6)$$

In EN1995-1-1-Draft [1], the effective length, $l_{ef,y/z}$, of the compressed member is calculated according to Equation 5.7. Since EN1995-1-1 [2] does not include the formula for the slenderness ratio, it also does not include guidance on the effective length for lateral flexural buckling. For unbraced members, some typical equivalent length factors, c_b, are given in Table 5.2. The first column is from Annex D of EN1995-1-1-Draft [1], while the last two are values from "Limtreboka" [7].

$$l_{ef,y/z} = c_b l \tag{5.7}$$

Table 5.2: Typical c_b factor for unbraced members [1]

Boundary conditions	c_b [1]	Theoretical c_b [7, p. 72]	Recommended c_b [7, p. 72]
Pinned - Pinned	1,00	1,00	1,00
Fixed - Pinned	0,70	0,70	0,85
Fixed - Fixed	0,50	0,50	0,7
Cantilever	2,00	2,00	2,25

Table 5.3 provides the limit values for the relative slenderness and β_0 factors used in the calculations of the k method in Table 5.4. If both $\lambda_{c,y,rel} \leq \lambda_{c,rel,0}$ and $\lambda_{c,z,rel} \leq \lambda_{c,rel,0}$ then there is no need to check for lateral flexural buckling, only combined bending and compression with Equation 4.13 and Equation 4.14. If $\lambda_{c,y,rel} > \lambda_{c,rel,0}$ or $\lambda_{c,z,rel} > \lambda_{c,rel,0}$ then the calculations in Table 5.4 must be performed. $k_{c,y}/k_{c,z}$ has to be calculated with Equation 5.16, where $\phi_{c,y}/\phi_{c,z}$ is calculated by Equation 5.18, β_c is calculated by Equation 5.20 and k_{creep} by Equation 5.28. In EN1995-1-1 [2] β_c is a constant depending on the material, while in EN1995-1-1-Draft [1] β_c is calculated considering the effect of creep on the initial imperfections.

Table 5.3: Parameters for the k_c and k_m methods [1]

Material	$\boldsymbol{\lambda_{c,rel,0}}$	$\boldsymbol{\beta_{c,0}}$	$\boldsymbol{\lambda_{m,rel,0}}$	$\boldsymbol{\beta_{m,0}}$
ST	0,30	0,20	0,30	0,15
GLT	0,30	0,10	0,55	0,10
LVL	0,30	0,10	0,55	0,10

From EN1995-1-1 [2] to EN1995-1-1-Draft [1] two new checks have been introduced, Equation 5.10 and Equation 5.11. Both checks must be satisfied to use the k_c method for buckling, otherwise the κ_c method in Annex E of EN1995-1-1-Draft [1] must be used instead. If both equations are fulfilled, then Equation 5.14 and the equivalent equation for the z-direction are the necessary design checks for lateral flexural buckling.

For LTB, the method and equations in EN1995-1-1-Draft [1] are almost the same as those for lateral flexural buckling. The relative slenderness is calculated according to Equation 5.22, where the critical stress for bending $\sigma_{m,crit}$ is given by Equation 5.23. These equations to find the relative slenderness, from EN1995-1-1-Draft [1], are the same as in EN1995-1-1 [2]. EN1995-1-1 [2] has a simplified equation for $\sigma_{m,crit}$. This simplified equation is a non-conservative simplification, and it is not included in EN1995-1-1-Draft [1]. If Equation 5.9 is fulfilled, then there is no need for further verification of LTB. If Equation 5.9 is not fulfilled, then the rest of the calculations in Table 5.4 must be calculated. The rest of the LTB verification in EN1995-1-1-Draft [1] is different from that in EN1995-1-1 [2]. To use the k_m method as shown in Table 5.4, the member can **not** be subjected to an axial force, in addition to fulfilling Equation 5.12 and Equation 5.13. This is the reason Equation 5.15 does not have an axial force component.

$$\lambda_{m,rel} = \sqrt{\frac{f_{m,y,k}}{\sigma_{m,crit}}} \tag{5.22}$$

Table 5.4: Calculations of lateral flexural and torsional buckling, for k_c and k_m methods [1]

	Lateral flexural buckling, k_c method (y-direction)	**Lateral torsional buckling, k_m method**
Check if buckling check is necessary, if satisfied no buckling check	$$\lambda_{c,y,rel} \leq \lambda_{c,rel,0} \tag{5.8}$$	$$\lambda_{m,rel} \leq \lambda_{m,rel,0} \tag{5.9}$$
Check if k_c/k_m method can be used	$$k_{c,y}\lambda_{c,y,rel}^2 \leq 0{,}5 \tag{5.10}$$ $$\frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 0{,}25 \tag{5.11}$$	$$k_m\lambda_{m,rel}^4 \leq 0{,}5 \tag{5.12}$$ $$\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 0{,}25 \tag{5.13}$$
Verification for buckling	$$\frac{\sigma_{c,0,d}}{k_{c,y}f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{5.14}$$	$$\max\left\{\frac{\sigma_{m,y,d}}{k_m f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}}; k_{red}\frac{\sigma_{m,y,d}}{k_m f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}}\right\} \leq 1 \tag{5.15}$$
Factor accounting for the effects of imperfections	$$k_{c,y} = \frac{1}{\phi_{c,y} + \sqrt{\phi_{c,y}^2 - \lambda_{c,y,rel}^2}} \tag{5.16}$$	$$k_m = \frac{1}{\phi_m + \sqrt{\phi_m^2 - \lambda_{m,rel}^2}} \tag{5.17}$$
Intermediate parameter	$$\phi_{c,y} = 0{,}5[1 + \beta_c(\lambda_{c,y,rel} - \lambda_{c,rel,0}) + \lambda_{c,y,rel}^2] \tag{5.18}$$	$$\phi_m = 0{,}5[1 + \beta_m(\lambda_{m,rel} - \lambda_{m,rel,0}) + \lambda_{m,rel}^2] \tag{5.19}$$
Material-specific imperfection factor	$$\beta_c = \beta_{c,0}k_{creep} \tag{5.20}$$	$$\beta_m = \beta_{m,0}k_{creep} \tag{5.21}$$

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi}{W_y l_{m,ef}}\sqrt{E_{0,05}I_z G_{0,05}I_{tor}} \tag{5.23}$$

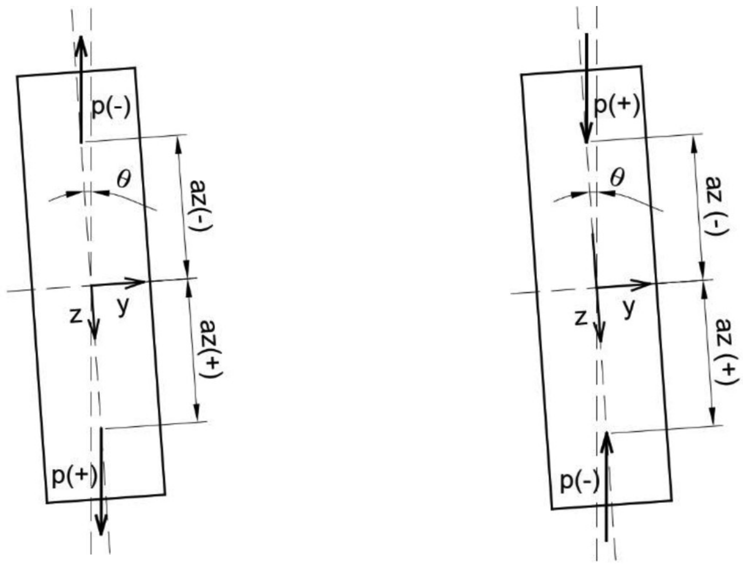
The effective length of the bent member, $l_{m,ef}$, is calculated according to Annex D in EN1995-1-1-Draft [1]. For unbraced members, Equation 5.24 can be used to calculate $l_{m,ef}$. a_1 is the moment distribution factor, and a selection of cases with the corresponding factor a_1 can be found in Table 5.5. The different load cases in Table 5.5 are shown in Figure 5.2. a_{ecc} is the eccentricity factor considering the distance of the load from the centroid of the cross-section of the member. a_{ecc} is calculated with Equation 5.25, where η is found by Equation 5.26. The torsional moment of inertia, I_x, for a rectangular cross-section is calculated from Equation 5.27. a_z is the distance from the load to the centroid of the cross-section. Figure 5.1 shows how a_z may be positive or negative depending on the position and direction of the load. If the load is on the neutral axis of the cross-section, then $a_{ecc} = 1$.

$$l_{m,ef} = \frac{l}{a_1 a_{ecc}} \tag{5.24}$$

$$a_{ecc} = \eta + \sqrt{\eta^2 + 1} \tag{5.25}$$

$$\eta = a_2 \frac{a_z}{l} \sqrt{\frac{E_0 I_z}{G_0 I_x}} \tag{5.26}$$

$$I_x = \frac{b^3 h}{3}(1 - 0{,}63\frac{b}{h}) \tag{5.27}$$



(a) Stabilising configuration

(b) Destabilising configuration

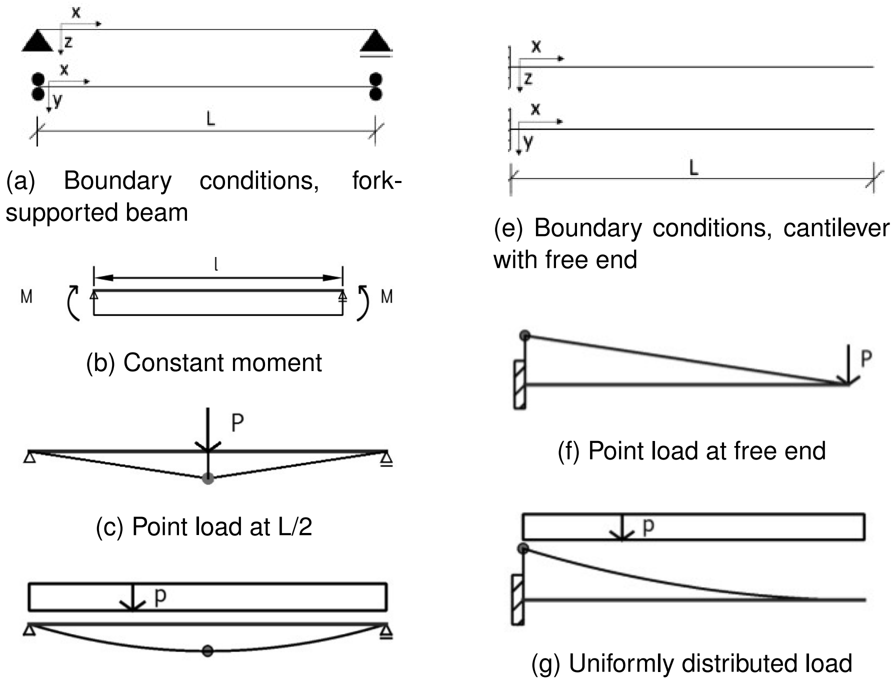
Figure 5.1: How the direction and position of the load, p, determines a_z [1]

Table 5.5: Factors for the effective length of the bending member for different load cases [1], $\frac{l_{ef}}{l}$ factors from EN1995-1-1 [2]

Beam type	**Load type**	a_1	a_2	$\frac{l_{ef}}{l}$ [2]
Forked supports, both ends (a)	Constant moment (b)	1,00	0	1,0
	Point load at L/2 (c)	1,35	1,74	0,8
	Uniformly distributed load (d)	1,13	1,44	0,9
Cantilever (fixed + free end) (e)	Point load at free end (f)	1,28	1,5	0,8
	Uniformly distributed load (g)	2,05	2,61	0,5

The factor to account for creep effects, k_{creep}, is the same for both lateral flexural buckling and LTB in EN1995-1-1-Draft [1]. EN1995-1-1 [2] does not include creep effects in buckling verifications. k_{creep} is calculated by Equation 5.28, where ψ_G is the share of permanent load and k_{def} is the deformation factor given in Table 2.6.

$$k_{creep} = 1 + \psi_G k_{def} \tag{5.28}$$



(a) Boundary conditions, fork-supported beam

(b) Constant moment

(c) Point load at L/2

(d) Uniformly distributed load

(e) Boundary conditions, cantilever with free end

(f) Point load at free end

(g) Uniformly distributed load

Figure 5.2: Different load cases [1]

5.1.2 Methods with lager scope of applications, κ_c and κ_m methods

EN1995-1-1-Draft [1] gives three conditions that must be satisfied to use the κ-methods provided in Annex E in EN1995-1-1-Draft [1]. Firstly, the cross section of the member must be rectangular, with $h \geq b$. Second, if a member has bi-axial bending, then Equation 5.29 must be satisfied. Lastly, the member must satisfy the design rules for SLS.

$$\frac{M_{y,d}}{M_{y,Rd}} \geq \frac{M_{z,d}}{M_{z,Rd}} \tag{5.29}$$

As mentioned above, if Equation 5.10 and Equation 5.11 are not fulfilled simultaneously, then the κ_c method must be used instead of the k_c method. In the κ_c method only buckling about y-direction is verified. Equations related to the verification of lateral flexural buckling are presented in Table 5.7.

The factor for the shape of the bending moment about the y-axis, δ_y, from Equation 5.33, is the Dischinger-coefficient, δ. δ is given in Annex C in EN1995-1-1-Draft [1]. A selection of δ values is presented in Table 5.6. The imperfection along the z-axis, $e_{0,z}$, may be calculated from Equation 3.3 as a minimum value.

Table 5.6: Dischinger-coefficient δ [1]

Type of loading	δ
Only axial load	0
Axial load + uniformly distributed load over the length of the member	$+0{,}0324$
Axial load + point load at the middle of the member	$-0{,}189$

Table 5.7: Calculations for lateral flexural buckling, by κ_c method [1]

Relative slenderness ratio	$$\lambda_{rel,c,y,d} = \sqrt{\frac{f_{c,0,d}}{\sigma_{y,crit}}} \quad (5.30)$$
Check if buckling check is necessary, if satisfied no buckling check	$$\frac{\sigma_{c,0,d}}{f_{c,0,d}}\lambda^2_{rel,c,y,d} \leq 0{,}1 \quad (5.31)$$
Verification for buckling	$$\frac{\sigma_{c,0,d}}{f_{c,0,d}}\kappa_{c,y} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.32)$$
Factor accounting for the second order effects	$$\kappa_{c,y} = \frac{\sigma_{c,0,d}}{f_{c,0,d}} + \beta_{c,y} + \lambda^2_{rel,c,y,d}\left[1 - \left(\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 - \delta_y\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}}\right)\right] \quad (5.33)$$
β factor	$$\beta_{c,y} = \frac{f_{c,0,d}}{f_{m,y,d}}\frac{6}{h}e_{0,z} \quad (5.34)$$

In Table 5.8 the different equations used to check for LTB with the κ-method are presented. As Table 5.8 shows, the equations differ depending on whether the member is under compression or tension. The equations look almost identical for tension and compression, but they are different. The differences between them are marked in red to help better see where the equations differ from each other. The design relative slenderness, $\lambda_{rel,m,d}$, is calculated from Equation 5.35. $\sigma_{m,y,crit}$ is the same as that used in the k-method and is calculated from Equation 5.23. In addition to the equations in Table 5.8, the two β-factors from Equation 5.36 and Equation 5.37 are used to calculate the κ-factors.

$$\lambda_{rel,m,d} = \sqrt{\frac{f_{m,y,d}}{\sigma_{m,y,crit}}} \quad (5.35)$$

$$\beta_\theta = 1 + \theta_0\frac{h}{b} \quad (5.36)$$

$$\beta_m = \frac{f_{m,y,d}}{G_{05}}\frac{h^2}{2b^3}e_{0,y} \quad (5.37)$$

Table 5.8: Calculations for lateral torsional buckling, for the κ_m method [1]

	κ_m method (bending + compression)	**κ_m method (bending + tension)**
Relative slenderness ratios ($\lambda_{rel,m,d}$ se Eq. 5.35)	$\lambda_{rel,c,z,d} = \sqrt{\frac{f_{c,0,d}}{\sigma_{z,crit}}}$ (5.38)	$\lambda_{rel,t,z,d} = \sqrt{\frac{f_{t,0,d}}{\sigma_{z,crit}}}$ (5.39)
Check if buckling check is necessary, if satisfied no buckling check	$\left(\frac{\sigma_{m,y,d}}{f_{m,y,d}}\right)^2 \lambda^4_{rel,m,d} + \frac{\sigma_{c,0,d}}{f_{c,0,d}} \lambda^2_{rel,c,z,d} \leq 0{,}1$ (5.40)	$\left(\frac{\sigma_{m,y,d}}{f_{m,y,d}}\right)^2 \lambda^4_{rel,m,d} - \frac{\sigma_{t,0,d}}{f_{t,0,d}} \lambda^2_{rel,t,z,d} \leq 0{,}1$ (5.41)
Verification for buckling	$\frac{\sigma_{c,0,d}}{f_{c,0,d}} \kappa_{c,z} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \kappa_{m,c} + k_{red} \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$ (5.42)	$\frac{\sigma_{t,0,d}}{f_{t,0,d}} \kappa_{c,z} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \kappa_{m,t} + k_{red} \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$ (5.43)
Factor accounting for the second order effects	$\kappa_{c,z} = \frac{\sigma_{c,0,d}}{f_{c,0,d}} + \beta_{c,z} + \lambda^2_{rel,c,z,d} \left[1 - \left(\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red} \delta_y \frac{\sigma_{m,z,d}}{f_{m,z,d}}\right)\right]$ (5.44) $\kappa_{m,c} = \beta_\theta + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \left[\beta_m + \lambda^4_{rel,m,d} \left(\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 - \left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}}\right)\right)\right]$ (5.45)	$\kappa_{t,z} = 1 + \beta_{t,z} - \lambda^2_{rel,t,z,d} \left[1 - \left(\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} - k_{red} \delta_y \frac{\sigma_{m,z,d}}{f_{m,z,d}}\right)\right]$ (5.46) $\kappa_{m,t} = \beta_\theta + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \left[\beta_m + \lambda^4_{rel,m,d} \left(1 - \left(\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}}\right)\right)\right]$ (5.47)
β factors (β_θ se Eq. 5.36, β_m se Eq. 5.37)	$\beta_{c,z} = \frac{f_{c,0,d}}{f_{m,z,d}} \frac{6}{b} e_{0,y}$ (5.48)	$\beta_{t,z} = \frac{f_{t,0,d}}{f_{m,z,d}} \frac{6}{b} e_{0,y}$ (5.49)

5.1.3 Proposal for an alternative method in prEN 1995-1-1, Clause 8.2

An alternative method [8] to the buckling checks in the EN1995-1-1-Draft [1] has been proposed by CEN/TC 250/SC 5. In this proposal [8], if $\alpha_d > 0{,}09$ then a stability check must be performed. The critical factor α_d is calculated according to Equation 5.50. A value of $\alpha_d = 0{,}09$ corresponds to a slenderness ratio $\lambda_{c,rel} \cong 0{,}30$ or $\lambda_{m,rel} \cong 0{,}55$ [8].

$$\alpha_d = \begin{cases} \dfrac{\sigma_{c,0,d}}{\sigma_{crit,y}} & \text{for major (y) axis lateral buckling} \\ \dfrac{\sigma_{c,0,d}}{\sigma_{crit,z}} & \text{for minor (z) axis lateral buckling} \\ \dfrac{\sigma^2_{m,y,d}}{\sigma^2_{m,crit}} + \dfrac{\sigma_{c,0,d}}{\sigma_{crit,z}} & \text{for LTB with compressive stress} \\ \dfrac{\sigma^2_{m,y,d}}{\sigma^2_{m,crit}} - \dfrac{\sigma_{t,0,d}}{\sigma_{crit,z}} & \text{for LTB with tensile stress} \end{cases} \tag{5.50}$$

The proposal [8] has split the requirement into two, one for major axis buckling and one for the LTB and minor axis buckling. In Table 5.9 the verification formulas required for major axis buckling is presented. Verifications for minor axis buckling and LTB are presented in Table 5.10.

Table 5.9: Calculations for major axis buckling ($\kappa_{c,y}$) [8]

Verification for major axis buckling	$\dfrac{\sigma_{c,0,d}}{f_{c,0,d}}\kappa_{c,z} + max\left\{\dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + k_{red}\dfrac{\sigma_{m,z,d}}{f_{m,z,d}}; k_{red}\dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + \dfrac{\sigma_{m,z,d}}{f_{m,z,d}}\right\} \leq 1$ (5.51)
Factor accounting for the second order effects	$\kappa_{c,y} = \dfrac{\sigma_{c,0,d}}{f_{c,0,d}} + \beta_{c,y} + \lambda^2_{c,y,rel}$ (5.52)
Relative slenderness ratio	$\lambda_{c,y,rel} = \sqrt{\dfrac{f_{c,0,d}}{\sigma_{y,crit}}}$ (5.53)
Imperfection factor	$\beta_{c,y} = 6\dfrac{e_{0,z}}{h}\dfrac{f_{c,0,d}}{f_{m,y,d}}(1 + \psi_G k_{def})$ (5.54)

Table 5.10: Calculations for minor axis buckling ($\kappa_{c,z}$) and LTB (κ_m)[8]

Verification for LTB, with compressive axial stress	$$\frac{\sigma_{c,0,d}}{f_{c,0,d}}\kappa_{c,y} + max\left\{\frac{\sigma_{m,y,d}}{f_{m,y,d}}\kappa_m + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}};\ k_{red}\frac{\sigma_{m,y,d}}{f_{m,y,d}}\kappa_m + \frac{\sigma_{m,z,d}}{f_{m,z,d}}\right\} \leq 1 \quad (5.55)$$	
Verification for LTB, with tensile axial stress	$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + max\left\{\frac{\sigma_{m,y,d}}{f_{m,y,d}}\kappa_m + k_{red}\frac{\sigma_{m,z,d}}{f_{m,z,d}};\ k_{red}\frac{\sigma_{m,y,d}}{f_{m,y,d}}\kappa_m + \frac{\sigma_{m,z,d}}{f_{m,z,d}}\right\} \leq 1 \quad (5.56)$$	
Factors accounting for the second order effects	$$\kappa_{c,z} = \frac{\sigma_{c,0,d}}{f_{c,0,d}} + \beta_{c,z} + \lambda^2_{c,z,rel} \quad (5.57)$$	$$\kappa_m = 1 + \beta_m + 0{,}25\lambda^4_{m,rel} \quad (5.58)$$
Relative slenderness	$$\lambda_{c,z,rel} = \sqrt{\frac{f_{c,0,d}}{\sigma_{z,crit}}} \quad (5.59)$$	$$\lambda_{m,rel} = \sqrt{\frac{f_{m,y,d}}{\sigma_{m,crit}}} \quad (5.60)$$
Imperfection factors	$$\beta_{c,z} = 6\frac{e_{0,y}}{b}\frac{f_{c,0,d}}{f_{m,z,d}}(1+\psi_G k_{def}) \quad (5.61)$$	$$\beta_m = \left[\frac{h^2 e_{0,y}}{2b^3}\frac{f_{m,y,d}}{G_{0,05}} + \theta_0\frac{h}{b}\right](1+\psi_G k_{def}) \quad (5.62)$$

5.2 Example cases

5.2.1 Lateral flexural buckling

Figure 5.4 to Figure 5.11 shows how different parameters affect the design check for lateral flexural buckling. The constant parameters for each figure is in the corresponding figure caption. The doted lines represent the rules in EN1995-1-1 [2], while the solid lines with stars represent the results from calculating with the rules in EN1995-1-1-Draft [1].

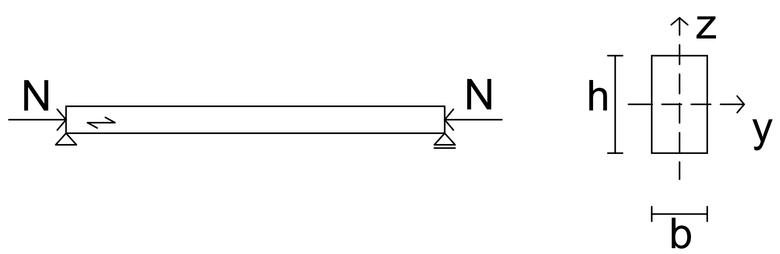


Figure 5.3: Lateral flexural buckling case

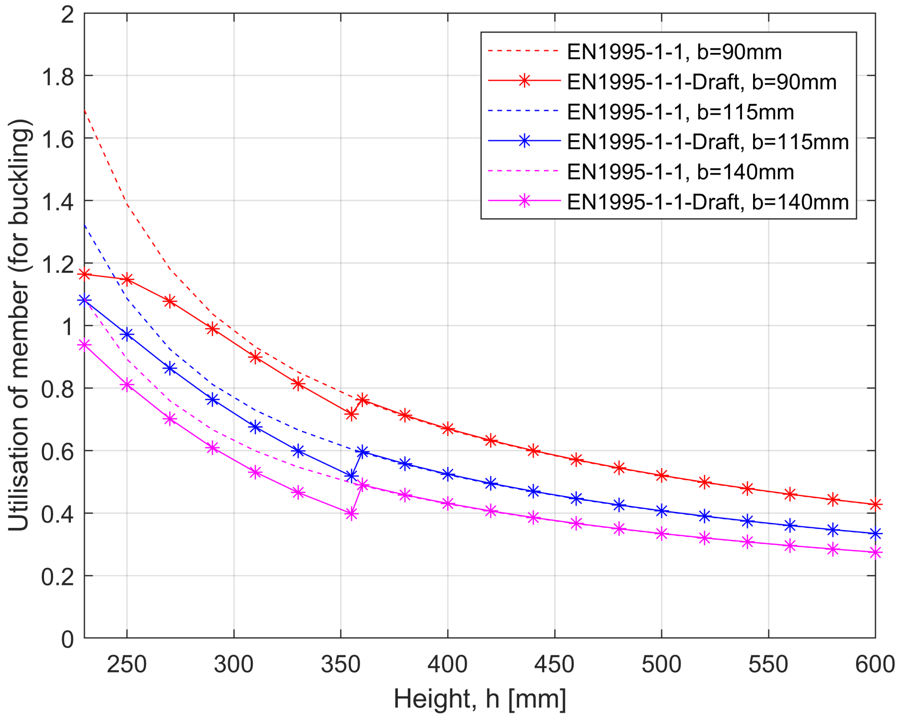


Figure 5.4: Buckling check of GL30c for different widths, with $N_d = 400$ kN, $l = 5m$, $\psi_G = 0{,}1$ and varying height

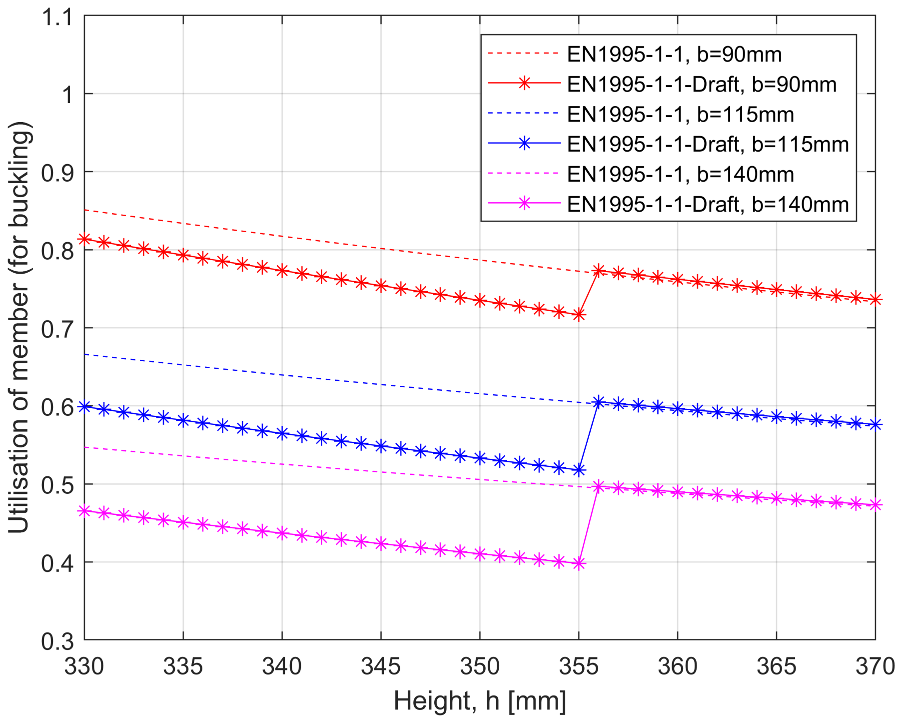


Figure 5.5: Detail of buckling check of GL30c for different widths, with $N_d = 400$ kN, $l = 5m$, $\psi_G = 0{,}1$ and varying height

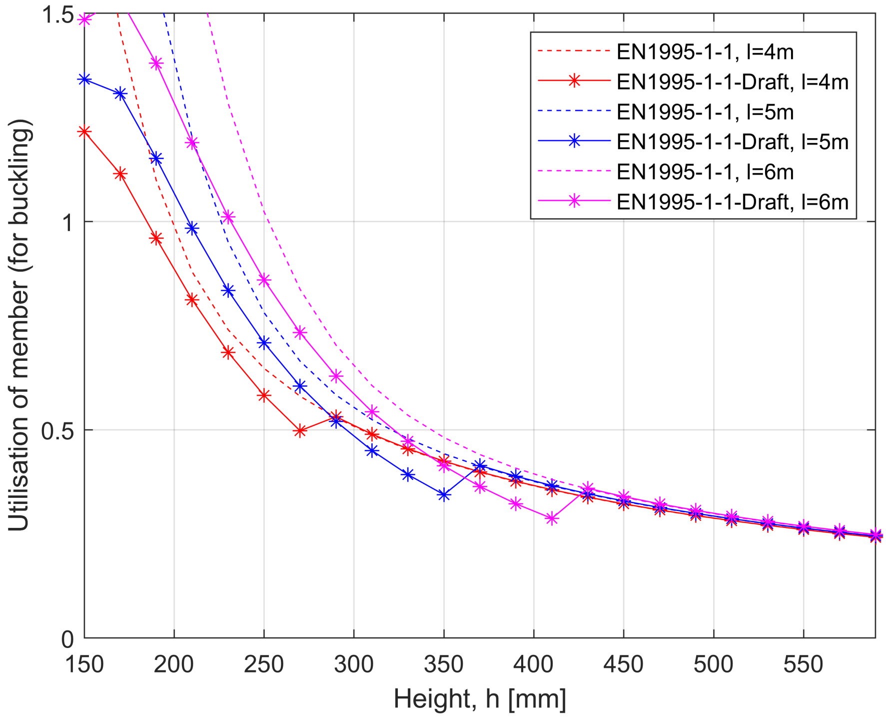


Figure 5.6: Buckling check of GL30c for different lengths, with $b = 140$ mm, $N_d = 350$ kN, $\psi_G = 0{,}1$ and varying height

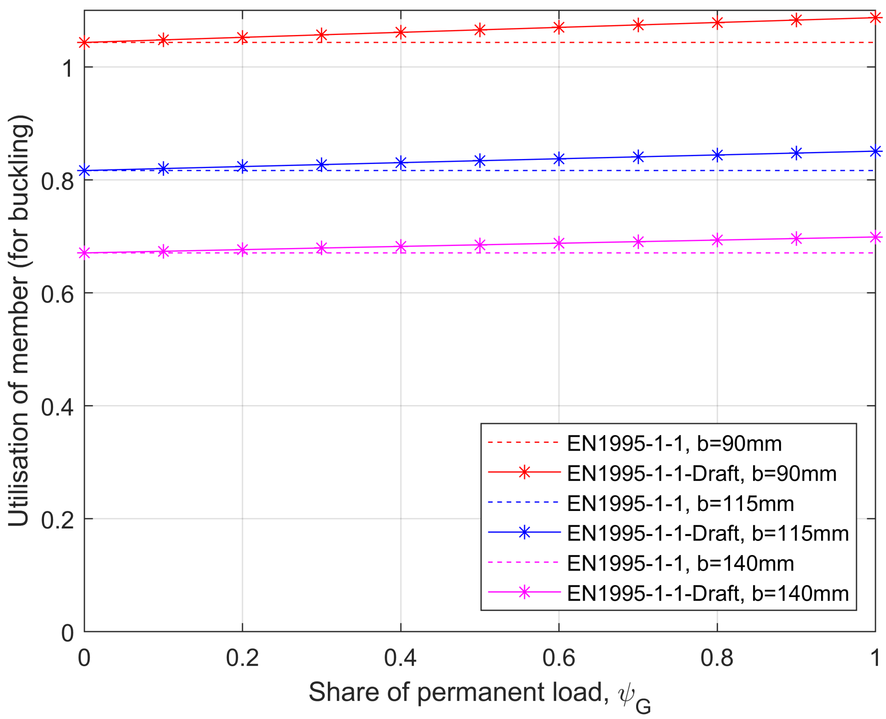


Figure 5.7: Buckling check of GL30c for different widths, with $h = 360$ mm, $l = 5$ m, $N_d = 550$ kN and varying ψ_G

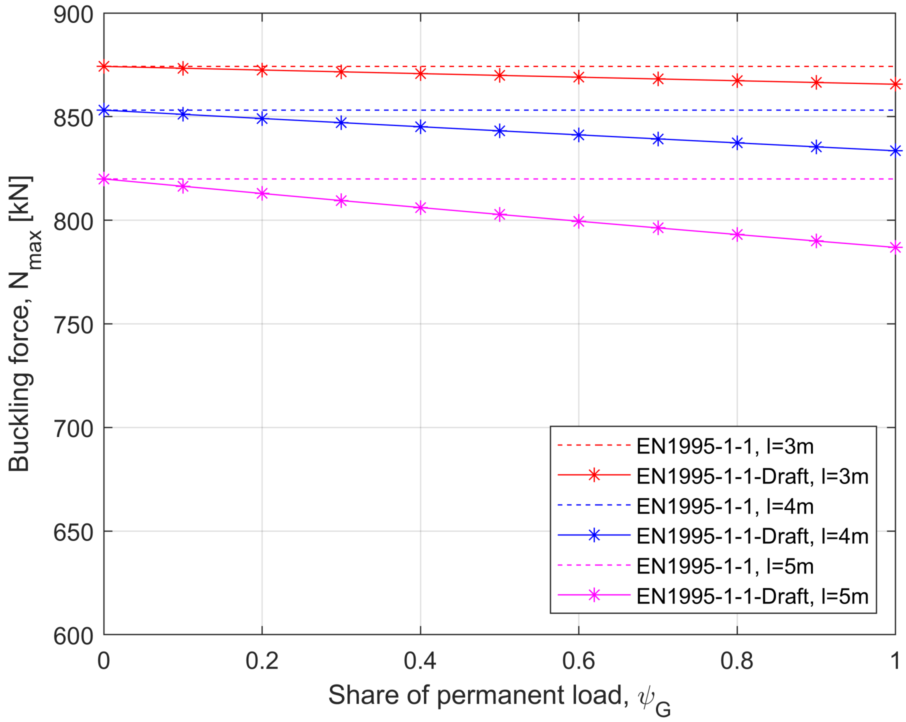


Figure 5.8: Max load of GL30c according to buckling checks for different lengths, with $b = 140$ mm, $h = 360$ mm and varying ψ_G

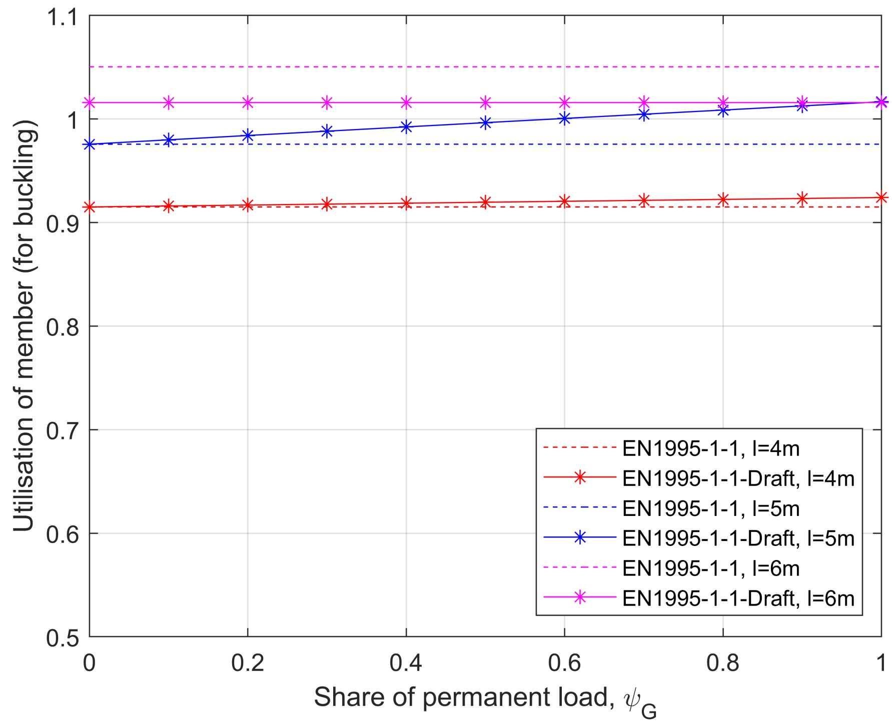


Figure 5.9: Buckling check of GL30c for different lengths, with $b = 140$ mm, $h = 360$ mm, $N_d = 800$ kN and varying ψ_G

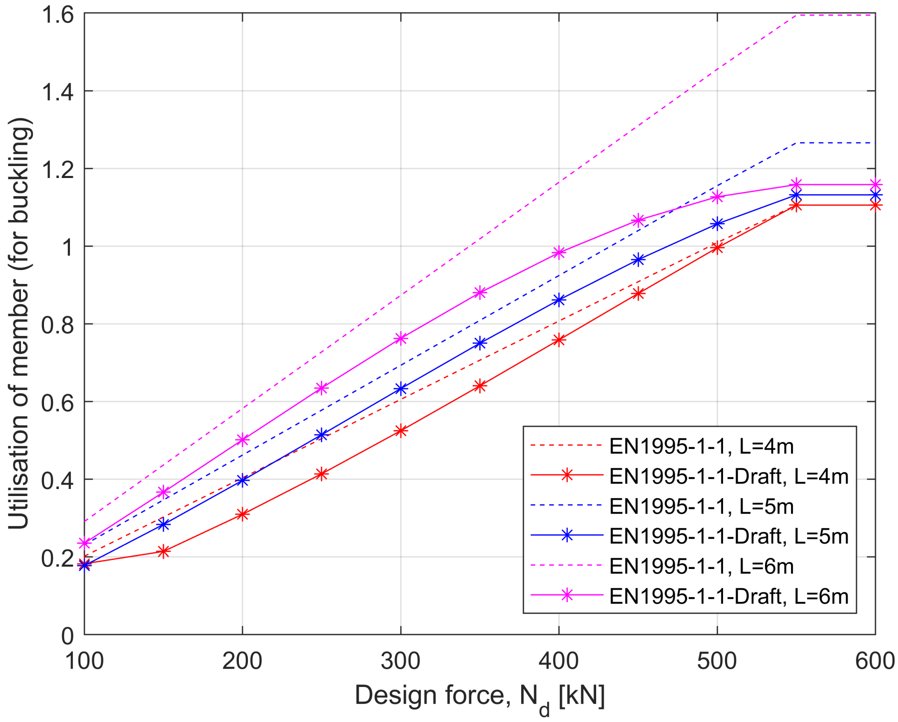


Figure 5.10: Buckling check of GL30c for different lengths, with $h = 270$ mm, $b = 115$ mm, $\psi_G = 0{,}1$ and varying load

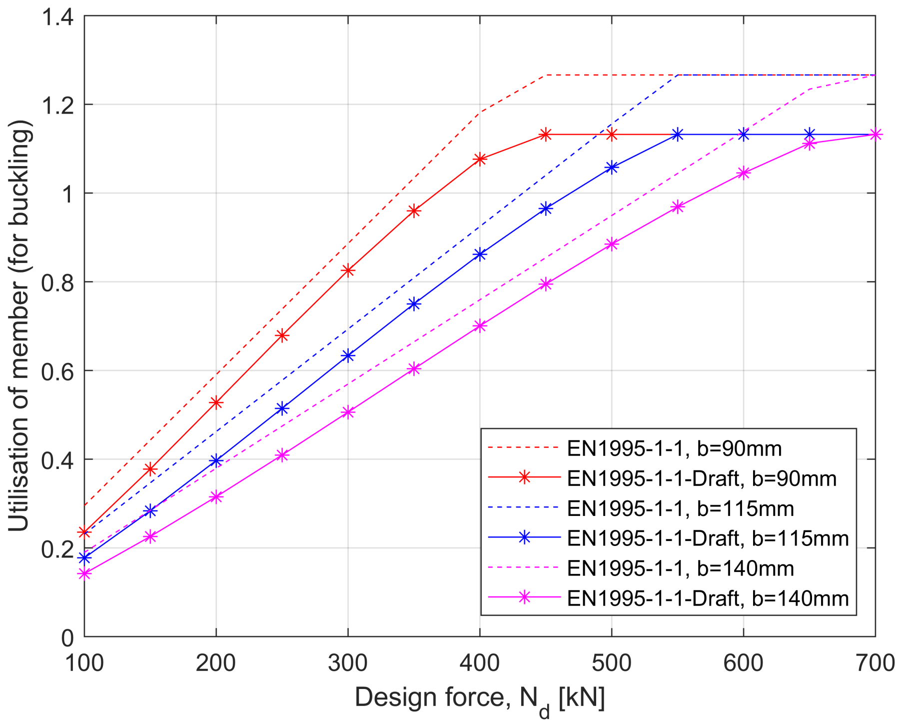


Figure 5.11: Buckling check of GL30c for different widths, with $h = 270$ mm, $l = 5$ m, $\psi_G = 0{,}1$ and varying load

Figure 5.12 shows the difference between EN1995-1-1 [2], EN1995-1-1-Draft [1], and the new proposal [8] for buckling verifications. Both the EN1995-1-1-Draft [1] and the new proposal [8] have a small jump when the calculations switch between no bucking risk and risk of buckling based on the relative slenderness of the beam. In this case, the calculations for EN1995-1-1 [2] never reach the point where there is no risk of buckling.

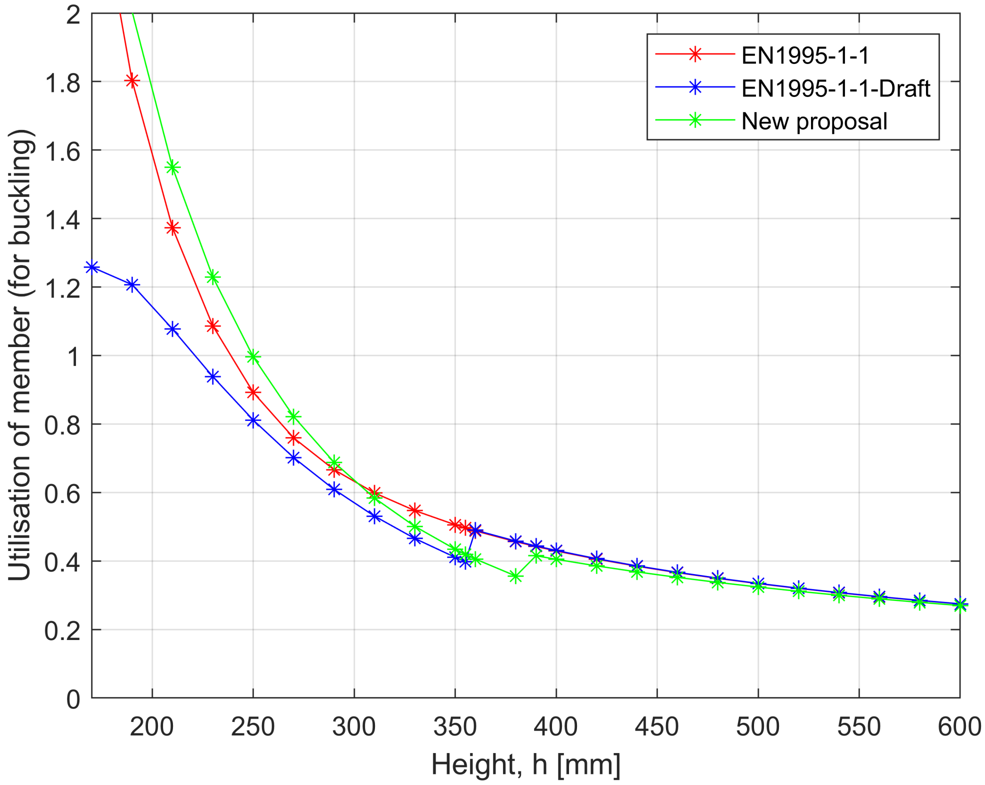


Figure 5.12: Buckling check of GL30c for varying height, with $b = 140$ mm, $N_d = 400$ kN, $l = 5m$ and $\psi_G = 0{,}1$

5.2.2 Lateral torsional buckling

The position of the load on a member is important and affects the effective bending length, $l_{m,ef}$. Figure 5.13 shows how the different placement of the load affects $l_{m,ef}$ in both EN1995-1-1 [2] and EN1995-1-1-Draft [1]. For each load case, there are six bars, two blue, two red, and two green. The blue bar is for loading placed at the neutral axis of the cross-section. The red and green bars represent a destabilising or stabilising load at the edge of the beam, as shown in Figure 5.1. The first bar in each colour is $l_{m,ef}$ according to the rules of EN1995-1-1 [2], while the second bar is for the rules in EN1995-1-1-Draft [1].

With the exception of case b), Figure 5.13 shows that EN1995-1-1-Draft [1] gives a shorter $l_{m,ef}$ than EN1995-1-1 [2]. This means that the effective moment length is a less conservative value in EN1995-1-1-Draft [1] than it is in EN1995-1-1 [2]. For case b) (constant moment) EN1995-1-1-Draft [1], the length is constant. This is because $a_2 = 0$ for this case, as shown in Table 5.5. EN1995-1-1 [2] allows for an increase / decrease of $l_{m,ef}$ for all cases.

To investigate how different factors affect the LTB design check, a reference case was used. The parameters of the reference case are shown in Table 5.11. The load is assumed to be at the centre of the cross-section. For each figure, a different parameter is changed, while the other factor remains the same as in the reference case.

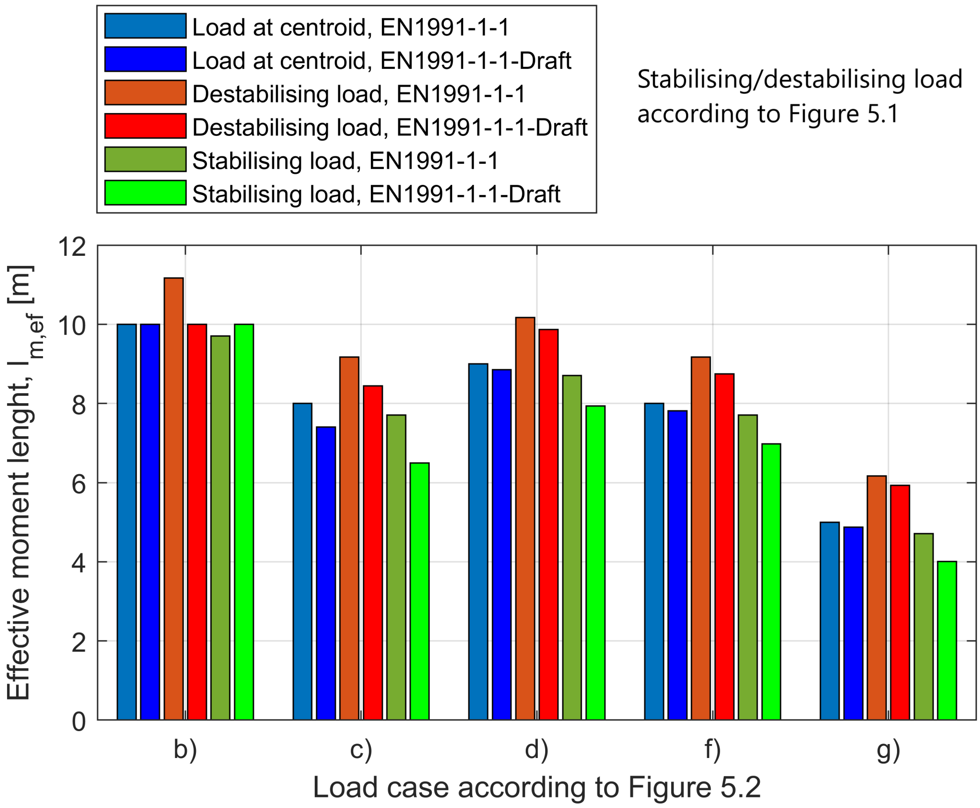


Figure 5.13: Effective moment length for the different load cases in Figure 5.2. The stabilising/destabilising load is placed at the top/bottom of the beam, with the following dimensions: $b = 90$ mm, $h = 585$ mm and $l = 10$ m

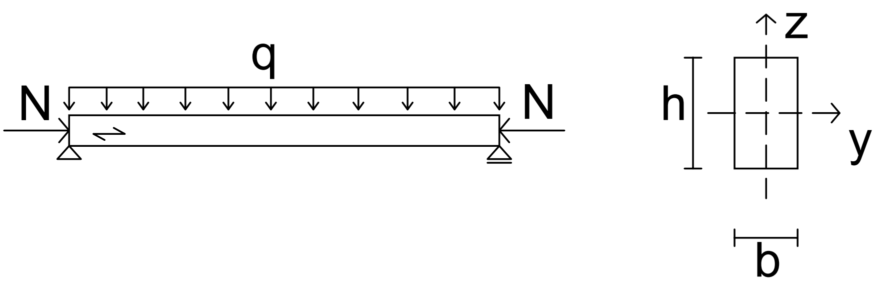


Figure 5.14: Loading case for lateral torsional buckling

Table 5.11: Dimensions, loads, and factors of the reference case for checking LTB

Material	GL30c
Height, h	630 mm
Width, b	90 mm
Lenght, l	7 m
Distributed load, q	10 N/mm
Axial load, N	10 kN
Initial imperfection, e_0	l/1000
Share of permanent loads, ψ_G	0,1
Load case, according to Figure 5.2	d)
Position of the load, a_z	0 mm

Each figure has two plots for EN1995-1-1 [2], the red lines, where one is for the simplified calculation of σ_m and the other uses the same formula as for EN1995-1-1-Draft [1]. EN1995-1-1-Draft [1] also has two plots, the blue lines, where one represents the design check for LTB presented in the EN1995-1-1-Draft [1], while the other presents the maximum value of the design check for LTB (Equation 5.42) and the combination of bending and axial compression (Equation 4.13). The last plot, the green line, is of the new proposal [8] for LTB check to EN1995-1-1-Draft [1].

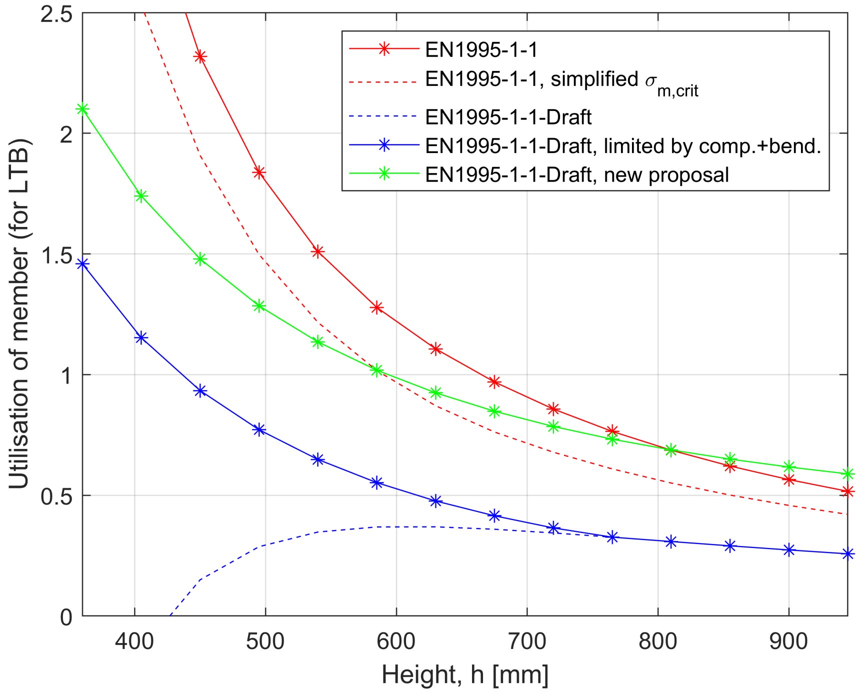


Figure 5.15: LTB check of GL30c for varying height

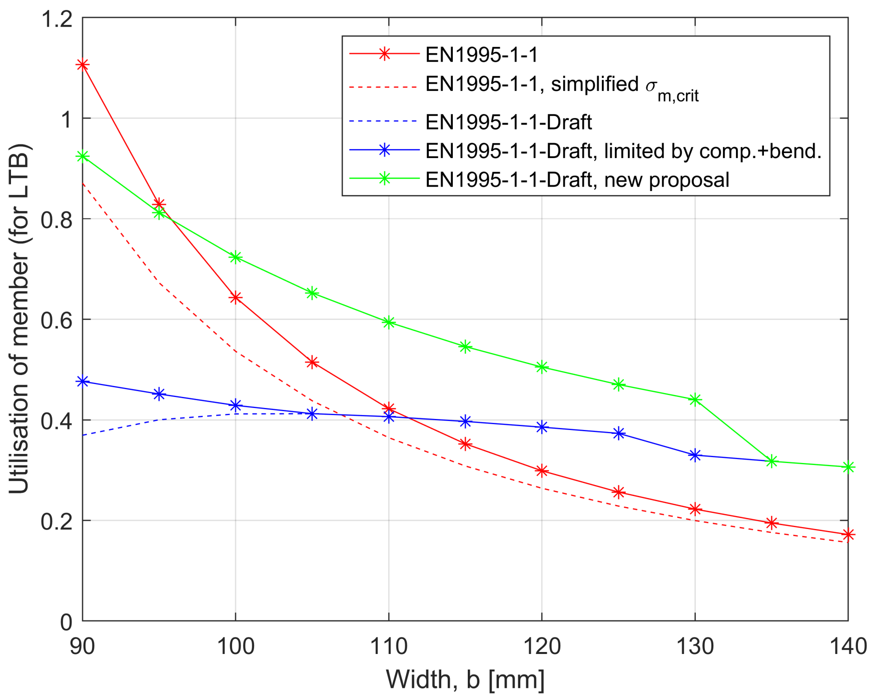


Figure 5.16: LTB check of GL30c for varying width

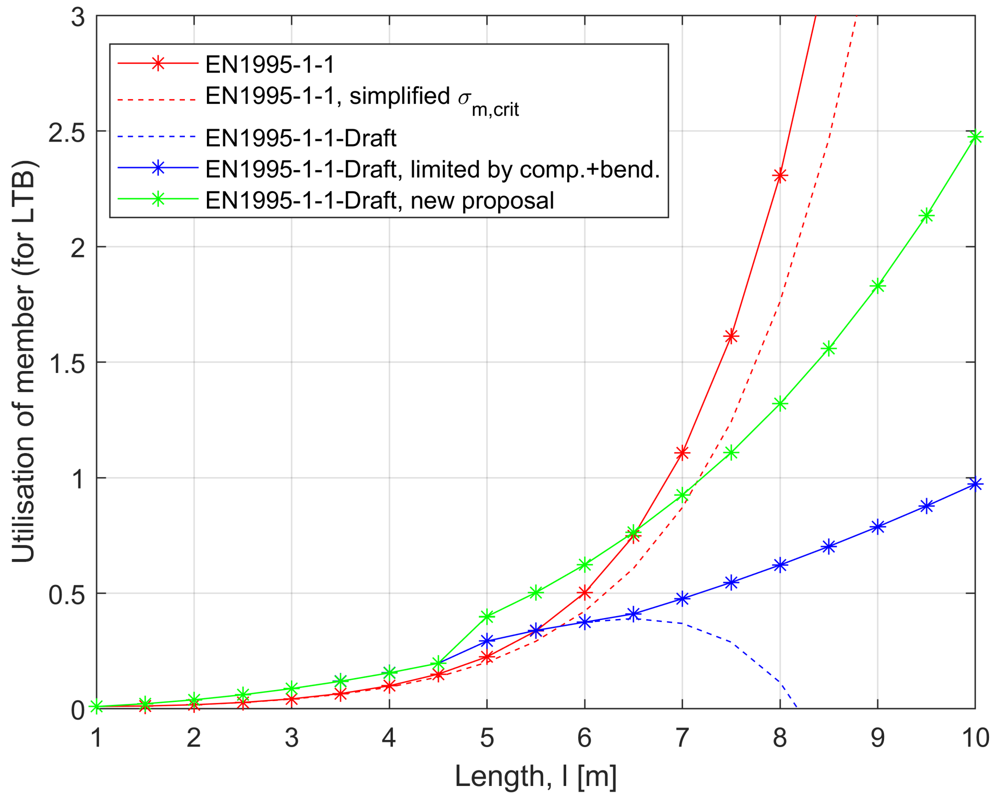


Figure 5.17: LTB check of GL30c for varying length

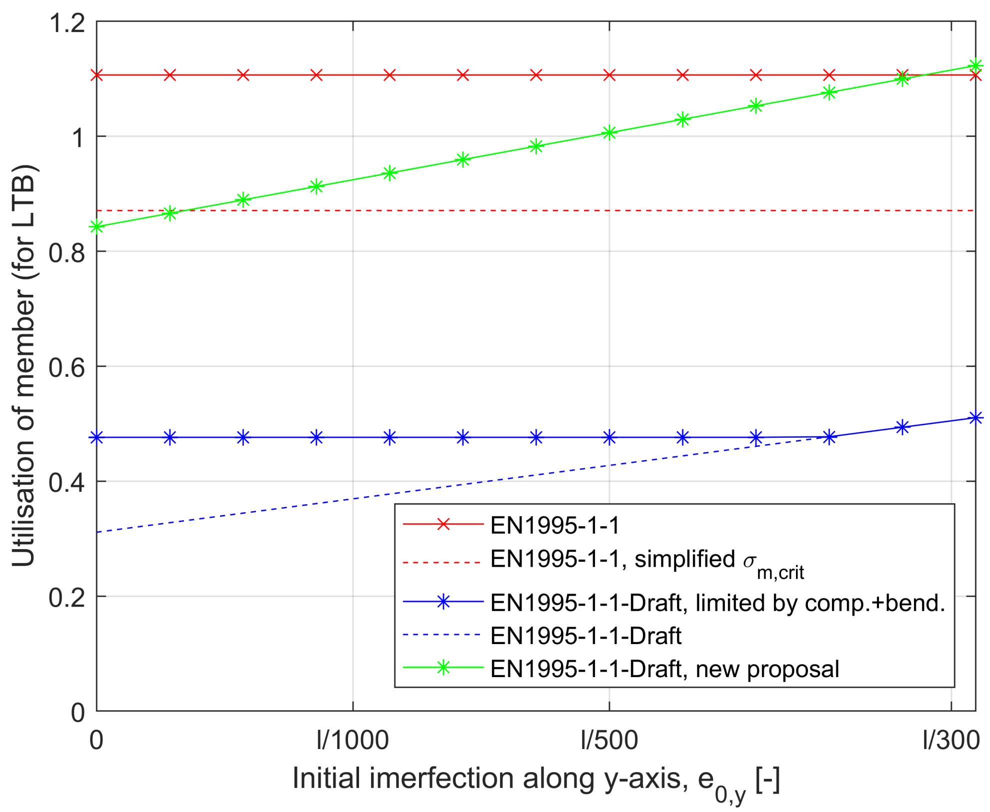


Figure 5.18: LTB check of GL30c for varying e_0

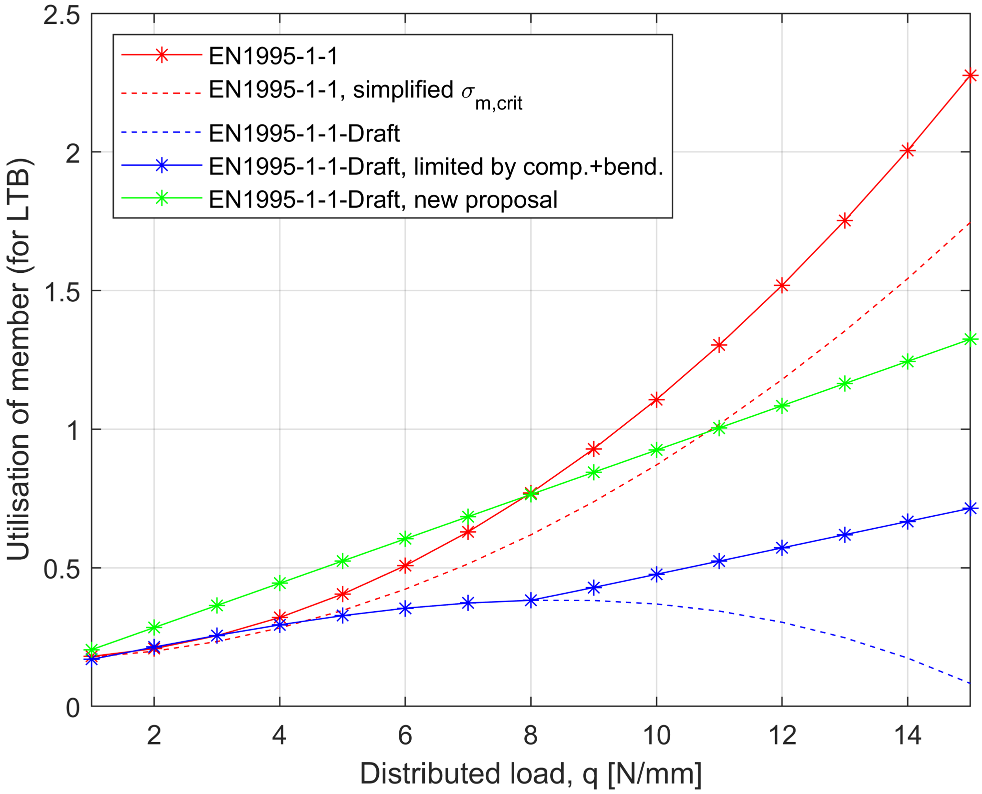


Figure 5.19: LTB check of GL30c for varying distributed load

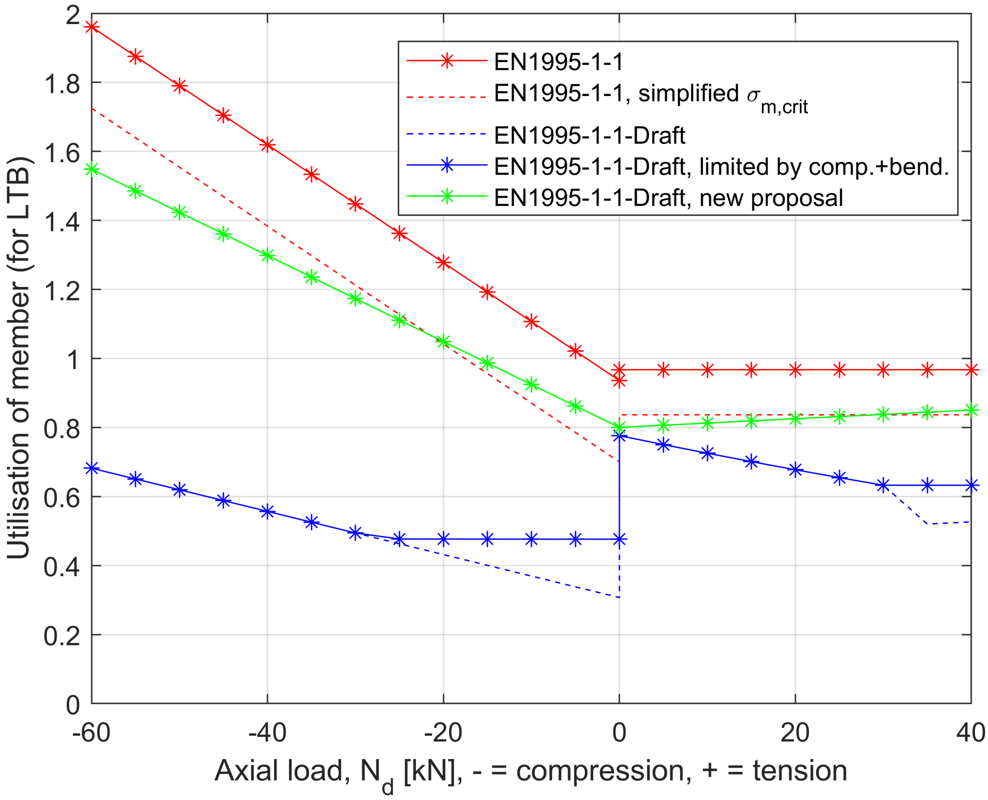


Figure 5.20: LTB check of GL30c for varying axial load

In all figures, the blue lines representing the verification according to EN1995-1-1-Draft [1] are generally less conservative. The verification according to the new proposal to EN1995-1-1-Draft [8] (green line) is generally closer to the red lines (EN1995-1-1 [2]) than the blue lines (EN1995-1-1-Draft [1]). The blue dashed line, which follows the rules for LTB in EN1995-1-1-Draft [1], starts to decrease in Figure 5.17 and Figure 5.19 as the length of the beam or the load on the beam increases.

5.3 Discussion

The design check for lateral flexural buckling according to EN1995-1-1 [2] is close to the design check for EN1995-1-1 [2], as can be seen in Figure 5.6 to Figure 5.11. The design checks from EN1995-1-1-Draft [1] are slightly less conservative than those from EN1995-1-1 [2], but not much. Figure 5.12 shows that the verification for major axis bucking in the new proposal for buckling [8], EN1995-1-1-Draft [1] and EN1995-1-1 [2] gives a similar design check.

For LTB, the rules in EN1995-1-1-Draft [1] may result in some strange results, as shown in Figure 5.17 and Figure 5.19. The case of this problem is probably the highlighted part of Equation 5.45, below. The highlighted term will always be negative, so when $\lambda_{rel,m,d}$ increases, $\kappa_{m,c}$ decreases. If β_m is smaller than the rest of the term inside the square bracket, then everything, except β_θ, becomes a negative term. Then if β_θ is smaller than the negative second part of Equation 5.45 then $\kappa_{m,c}$ becomes negative. This is probably what happened in Figure 5.17 and Figure 5.19. In LTB verification, Equation 5.42, $\kappa_{m,c}$ is multiplied by $\frac{\sigma_{m,y,d}}{f_{m,y,d}}$, so that if $\kappa_{m,c}$ is negative, then the LTB verification may become negative. For increased saifty due to the risk of LTB $\kappa_{m,c}$ should be greater than 1. If $\kappa_{m,c}$ is below 1 then the check for LTB becomes less strict than the verification for a member not at risk for LTB.

$$\kappa_{m,c} = \beta_\theta + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \left[\beta_m + \lambda_{rel,m,d}^4 \left(\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 - \left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \right) \right) \right] \tag{5.45}$$

The new proposal for buckling [8] does not have the same issue as EN1995-1-1-Draft [1] has for LTB. The equations are similar to the equations for the κ_m-method in EN1995-1-1-Draft [1], but without the negative terms the κ_m-method has. The new proposal [8] has also combined some of the factors so there is less variables to calculate for the verification check. By looking at Figure 5.15 to Figure 5.20 the new proposal [8] seems to fit better with what is expected, and are mor conservative than EN1995-1-1-Draft [1] and closer to the results from EN1995-1-1 [2].

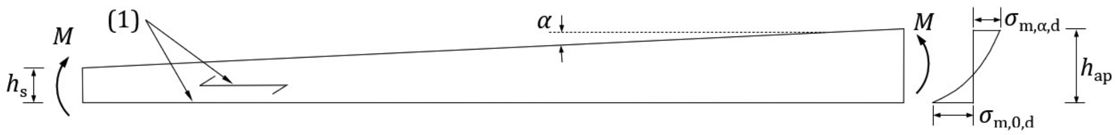
6. Members with special geometry

Members which are not have a constant straight cross section requires additional verifications. Stress consecrations is one type of problem these members may have. This requires a verification to ensure that the stresses do not exceeds the stress limits of the material. Another problem is that the shape of the member causes tension perpendicular to the grain, which also must be verified.

6.1 EN1995-1-1 vs EN1995-1-1-Draft

Both EN1995-1-1 [2] and EN1995-1-1-Draft [1] have verification rules for single tapered beams, double tapered beams, curved beams, pitch cambered beams and notched members at support. EN1995-1-1-Draft [1] have in addition rules for reinforcement of these members and verification rules for members with holes, including reinforcement.

6.1.1 Members with varying cross-section or curved shape



(1) (Recommended) grain direction

Figure 6.1: Single tapered beam with notations [1]

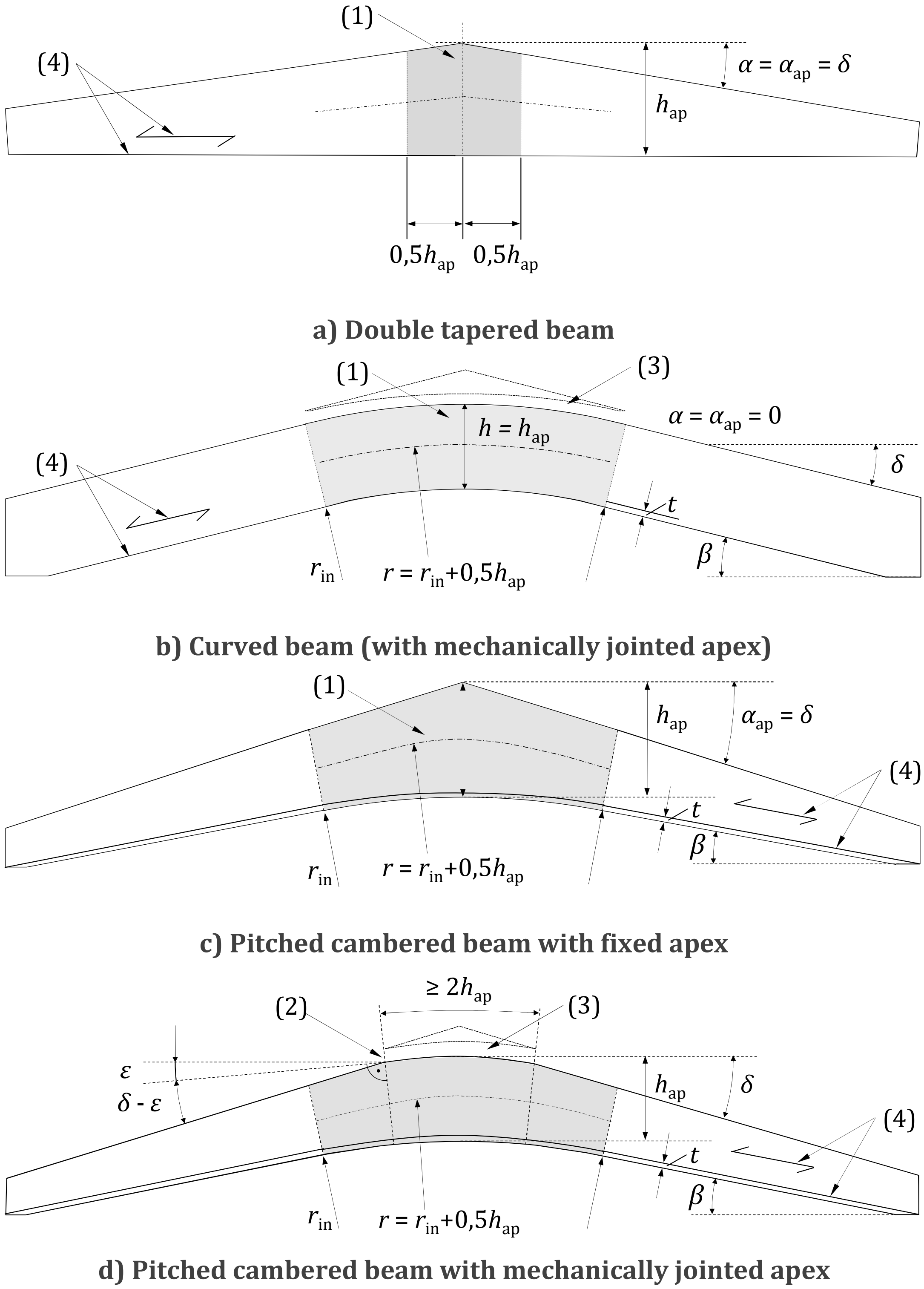
For single tapered beams the rules are almost identical in EN1995-1-1 [2] and EN1995-1-1-Draft [1]. The rules in EN1995-1-1-Draft [1] is presented in Table 6.1, with α shown in Figure 6.1. The only difference from EN1995-1-1 [2] to EN1995-1-1-Draft [1] is the introduction of $k_{\tau,t}$ and $k_{\tau,c}$ as factors depending on the material. In EN1995-1-1 [2] these factors are the same as the factors for GLT in Equation 6.5 and Equation 6.6.

Table 6.1: Calculations for single tapered beams [1]

Design check	$$\sigma_{m,\alpha,d} \leq k_{m,\alpha} f_{m,d} \quad (6.1)$$
Design bending stresses	$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{6M_d}{bh^2} \quad (6.2)$$
Factor to account for the stress combination at the tapered edge (Equation 6.3 for tensile and Equation 6.4 compressive stresses parallel to the tapered edge)	$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{k_{\tau,t} f_{v,d}} \tan\alpha\right)^2 + \left(\frac{f_{m,d}}{f_{t,90,d}} \tan^2\alpha\right)^2}} \quad (6.3)$$ $$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{k_{\tau,c} f_{v,d}} \tan\alpha\right)^2 + \left(\frac{f_{m,d}}{f_{c,90,d}} \tan^2\alpha\right)^2}} \quad (6.4)$$
Factors for the effect of stresses perpendicular to grain on the shear strength	$$k_{\tau,t} = \begin{cases} 0{,}75 & \text{GLT and LVL-P from softwood} \\ 1{,}0 & \text{LVL-C from softwood in edgewise bending} \end{cases} \quad (6.5)$$ $$k_{\tau,c} = \begin{cases} 1{,}5 & \text{GLT and LVL-P from softwood} \\ 1{,}0 & \text{LVL-C from softwood in edgewise bending} \end{cases} \quad (6.6)$$

According to EN1995-1-1-Draft [1] the calculations in Table 6.2 and Table 6.3 applies to double tapered, curved and pitched cambered beams. The angle of the tapper, α_{ap}, the height of the apex zone, h_{ap} and the inner radius of the curved part of the beam, r_{in}, are shown in Figure 6.2.

The calculations in Table 6.2 are the same as the ones in EN1995-1-1 [2]. For tensile stresses perpendicular to grain, Table 6.3, the two factors k_{dis} and k_{vol} are changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. The k_{dis} values are lower in EN1995-1-1-Draft [1] than in EN1995-1-1 [2]. k_{vol} are calculated by the volume of the apex zone in EN1995-1-1 [2], while in EN1995-1-1-Draft [1] it is calculated by the height of the apex zone. The calculation of the tensile stress perpendicular to grain in EN1995-1-1 [2] is the same as Equation 6.19 but with the option to include an additional term "$-0{,}6\frac{p_d}{b}$", where p_d is the uniformly distributed load acting on the top of the apex zone. This optional term, "$-0{,}6\frac{p_d}{b}$", is not present in EN1995-1-1-Draft [1].



(1) Apex zone (grey)
(2) Secondary apex (inflection point)
(3) Mechanically jointed apex (no glued joint)
(4) (Recommended) grain direction

Figure 6.2: Double tapered, curved and pitched cambered beams with notations [1]

Table 6.2: Calculations for bending stresses in the apex zone [1]

Check of bending stress	$$\sigma_{m,d} \leq k_r f_{m,d} \qquad (6.7)$$
Bending stress at apex	$$\sigma_{m,d} = k_l \frac{6M_{ap,d}}{bh_{ap}^2} \qquad (6.8)$$
Factor for increased bending stresses in the apex zone	$$k_l = k_1 + k_2 \frac{h_{ap}}{r} + k_3 \frac{h_{ap}}{r}^2 + k_4 \frac{h_{ap}}{r}^3 \qquad (6.9)$$
Radius of the curved part of the beam	$$r = r_{in} + 0{,}5 h_{ap} \qquad (6.10)$$
Modification factors	$$k_1 = 1 + 1{,}4 \tan \alpha_{ap} + 5{,}4 \tan^2 \alpha_{ap} \qquad (6.11)$$ $$k_2 = 0{,}35 - 8 \tan \alpha_{ap} \qquad (6.12)$$ $$k_3 = 0{,}6 + 8{,}3 \tan \alpha_{ap} - 7{,}8 \tan^2 \alpha_{ap} \qquad (6.13)$$ $$k_4 = 6 \tan^2 \alpha_{ap} \qquad (6.14)$$
Factor for strength reduction due to bending of the laminations during production	$$k_r = \begin{cases} 1 & \text{for} \frac{r_{in}}{t} \geq 240 \\ 0{,}76 + 0{,}001 \frac{r_{in}}{t} & \text{for} \frac{r_{in}}{t} < 240 \end{cases} \qquad (6.15)$$

EN1995-1-1-Draft [1] includes rules for reinforcement of double tapered, curved and pitched cambered beams, while EN1995-1-1 [2] does not cover any rules for reinforcement. According to EN1995-1-1-Draft [1] Equation 6.16 is to be used to calculate the design tensile force, $F_{t,90,Ed}$ for the reinforcement. The spacing of the reinforcement, a_1, is measured parallel to the grain at the height of the beam axis. The factor for distribution of tensile stresses perpendicular to grain, k_{ka}, is 1,0 for curved beams and for double tapered and cambered beams with the **inner** quarters exposed to tensile stresses perpendicular to grain. The k_{ka} factor is 0,67 for double tapered and cambered beams with the **outer** quarters exposed to tensile stresses perpendicular to grain.

$$F_{t,90,Ed} = k_{ka} \sigma_{t,90,d} b a_1 \qquad (6.16)$$

Table 6.3: Calculations for tensile stresses perpendicular to grain in the apex zone [1]

Check of tensile stresses perpendicular to grain	$$\sigma_{t,90,d} \leq k_{dis} k_{vol} f_{t,90,d} \quad (6.17)$$
Check for combined shear and tension perpendicular to grain	$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{dis} k_{vol} f_{t,90,d}} \leq 1 \quad (6.18)$$
Tensile stress perpendicular to grain at apex	$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{bh_{ap}^2} \quad (6.19)$$
Factor for tensile stresses perpendicular to grain in the apex zone	$$k_p = k_5 + k_6 \frac{h_{ap}}{r} + k_7 \frac{h_{ap}}{r}^2 \quad (6.20)$$
Radius of the curved part of the beam	$$r = r_{in} + 0{,}5h_{ap} \quad (6.10)$$
Modification factors	$$k_5 = 0{,}2 \tan \alpha_{ap} \quad (6.21)$$ $$k_6 = 0{,}25 - 1{,}5 \tan \alpha_{ap} + 2{,}6 \tan^2 \alpha_{ap} \quad (6.22)$$ $$k_7 = 2{,}1 \tan \alpha_{ap} - 4 \tan^2 \alpha_{ap} \quad (6.23)$$
Factor for stress distribution in the apex zone	$$k_{dis} = \begin{cases} 1{,}3 & \text{for double tapered and pitched cambered beams} \\ 1{,}15 & \text{for curved beams} \end{cases} \quad (6.24)$$
Factor for the volume effect	$$k_{vol} = \left(\frac{h_{ref}}{h_{ap}}\right)^{0{,}3} \quad (6.25)$$ $$h_{ref} = 600 \text{ mm}$$

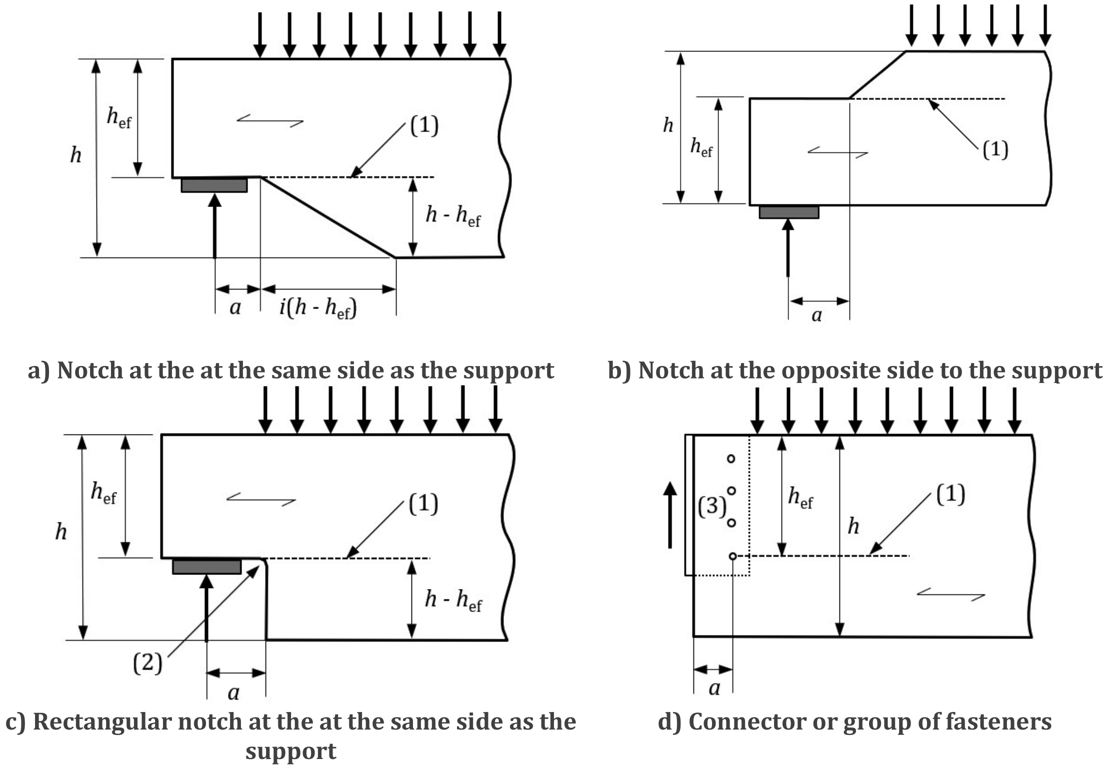
6.1.2 Notched members

The rules for notched member without reinforcement are almost identical in EN1995-1-1 [2] and EN1995-1-1-Draft [1]. The rules in EN1995-1-1-Draft [1] are in Table 6.4. The only difference in EN1995-1-1 [2] is Equation 6.26. In EN1995-1-1 [2] the notation for reduction factor is k_v instead of $k_{v,n}$. EN1995-1-1 [2] does not include the adjustment factor for shear strength, k_v, and the application of this factor is still in discussion in EN1995-1-1-Draft [1]. Notch inclination, i, the distance from line of the action to the corner of the notch, a, the beam height, h, and the effective height of the notched part, h_{ef}, are presented in Figure 6.3 for different end-notched beams.

Table 6.4: Calculations for end-notched beams [1]

Design check for notched support	$$\tau_d = \frac{1{,}5 V_d}{b h_{ef}} \leq k_{v,n} k_v f_{v,d} \qquad (6.26)$$
Factor for the effect of the notch	For beams notched at the opposite side to the support: $$k_{v,n} = 1{,}0 \qquad (6.27)$$ For beams notched at the opposite side to the support: $$k_{v,n} = \text{min}\left(1; \frac{k_n\left(1+\frac{1{,}1 i^{1{,}5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha\left(1-\alpha\right)}+0{,}8\frac{a}{h}\sqrt{\frac{1}{\alpha}-\alpha^2}\right)}\right) \qquad (6.28)$$
Constant material factor	$$k_n = \begin{cases} 4{,}5 & \text{for LVL-P from softwood in edgewise bending} \\ 5 & \text{for ST, FST and GST from softwood} \\ 6{,}5 & \text{for GLT and BGLT from softwood} \end{cases} \qquad (6.29)$$
Ratio of the effective height of the notched part to the beam height	$$\alpha = \frac{h_{ef}}{h} \qquad (6.30)$$

EN1995-1-1 [2] does not cover any rules about reinforcement at notched ends. In EN1995-1-1-Draft [1] the rules in Table 6.5 apply. If both $\alpha \leq 0{,}6$ and $\beta \leq 0{,}2$ then $k_\alpha k_\beta$ in Equation 6.31 may be taken as $k_\alpha k_\beta = 1{,}3$ without further verification.



(1) Possible crack line
(2) Rounded corner r $\geq$ 20 mm (recommendation)
(3) Connector or fasteners

Figure 6.3: End-notched beams with notations [1]

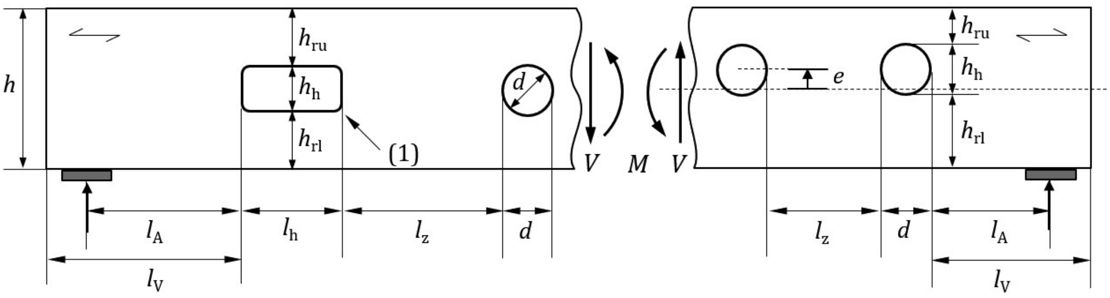
Table 6.5: Calculations for end-notched beams with reinforcement [1]

Design tensile force on reinforcement	$F_{t,90,Ed} = k_\alpha k_\beta V_d \left[3\,(1-\alpha)^2 - 2\,(1-\alpha)^3\right]$ (6.31)
Factors accounting for notch geometry	$k_\alpha = 0{,}9 + 0{,}5\,(2\alpha - 1)^2$ (6.32) $k_\beta = 1 + 2\beta$ (6.33)
Ratio between the beam height and the distance from the line of support reaction to the corner of the notch	$\beta = \dfrac{a}{h}$ (6.34)

6.1.3 Members with holes

EN1995-1-1 [2] does not cover rules for member with holes. All the rules covered in this section is new to EN1995-1-1-Draft [1]. If a hole in a beam is larger or equal to min$\{50$ mm$; 0{,}1h\}$ then the beam has to be checked for stress consecrations around the hole. These types of holes should not be placed in unreinforced zones with tensile stresses perpendicular to grain. The holes should preferably be placed in the centre of the neutral axis of the member. EN1995-1-1-Draft [1] gives the necessary equations for eccentric arrangement of holes.

The minimum distances in a beam with holes are given in Table 6.6. The maximum dimensions for a rectangular or circular hole are given in Table 6.7. Both the different dimensions of the beam and the hole is visualized in Figure 6.4. The necessary verification and calculations for unreinforced beams are in Table 6.8. A hole in a member affects the bending and shear stresses. The stresses around the hole is calculated using the equations in Table 6.9.



(1) Rounded corner: corner radius $r \geq 20$ mm, if $h_h \leq 200$ mm
corner radius $r \geq 40$ mm, if $h_h > 200$ mm

Figure 6.4: Rectangular and circular holes in members subjected to bending, with notations [1]

Table 6.6: Minimum distances of unreinforced and reinforced holes in beams with rectangular cross-section [1]

	Minimum distances			
	End	Spacing	Support	Edges
Unreinforced	$l_v \geq h$ a)	Individual hole: $l_z \geq 1{,}5h$, at least 300 mm	$l_A \geq h/2$	$h_{ru} \geq 0{,}15h$, at least one lamination b)
		Group of circular holes: $l_z \geq d$		$h_{rl} \geq 0{,}2h$, at least 1,5 laminations b)
Reinforced	$l_v \geq h$ a)	Individual hole: $l_z \geq 1{,}0h$, at least 300 mm	$l_A \geq h/2$	$h_{ru} \geq 0{,}15h$, at least one lamination b)
		Group of circular holes: $l_z \geq d$		$h_{rl} \geq 0{,}2h$, at least 1,5 laminations b)

a) In applications with permanently dry or frequently changing climate, l_v should be increased so that $l_v \geq 1{,}5h$

b) For LVL-P: $h_{ru} \geq 40$ mm and $h_{rl} \geq 60$ mm

Table 6.7: Maximum dimensions of unreinforced and reinforced holes in beams with rectangular cross-section [1]

	Maximum dimensions		
	Rectangular holes		Circular holes
Unreinforced	$l_h/h_h \leq 2{,}5$	$h_h \leq 0{,}2h$	For $e \leq \pm 0{,}1h$: $d \leq 0{,}3h$
	$l_h \leq 0{,}5h$		For $e > \pm 0{,}1h$: $d \leq 0{,}2h$
Reinforced	$l_h/h_h \leq 2{,}5$	$h_h \leq 0{,}3h$ a)	$d \leq 0{,}3h$ a)
	$l_h \leq h$	$h_h \leq 0{,}4h$ b)	$d \leq 0{,}4h$ b)

a) For holes with internal dowel-type reinforcement
b) For holes with plane external reinforcement

Table 6.8: Calculations for unreinforced holes in rectangular beams [1]

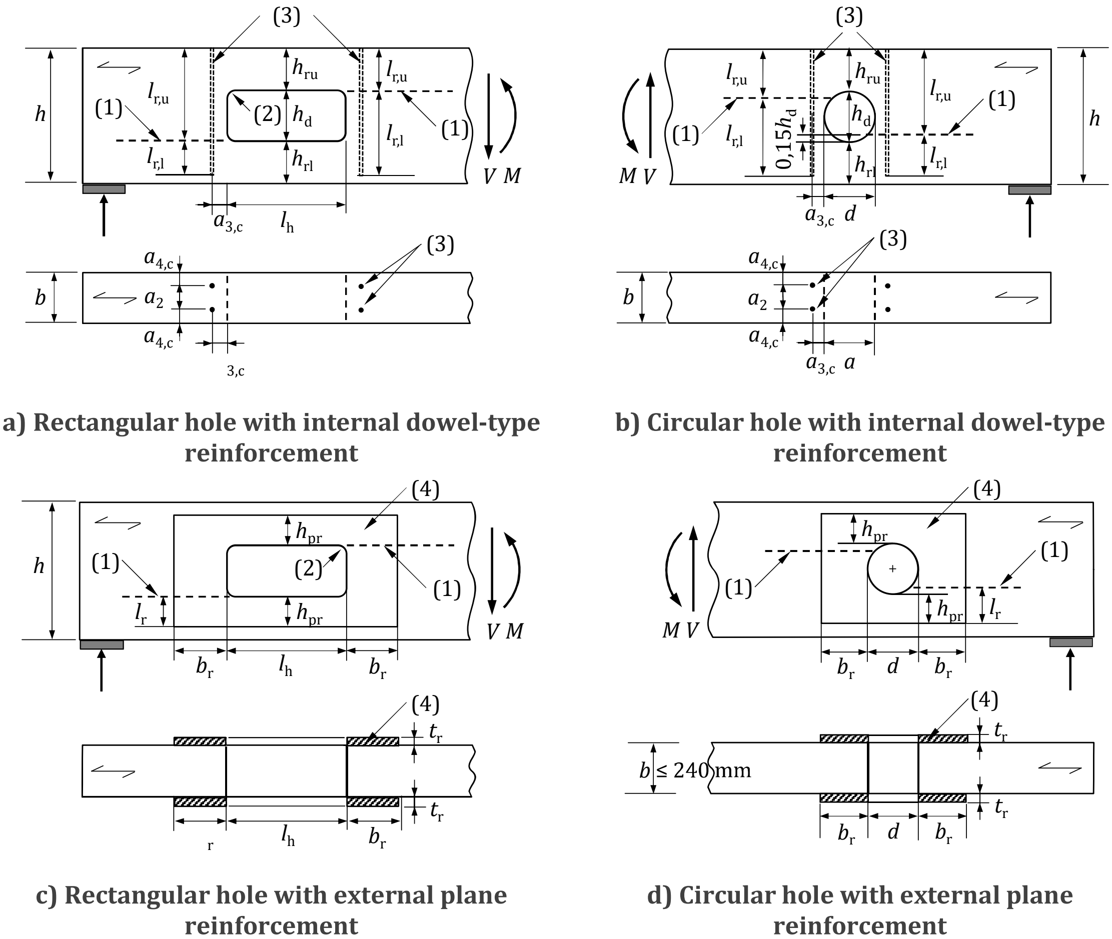
Design check	$$\frac{\frac{F_{t,90,V,Ed}}{l_{t,90,V}} + \frac{F_{t,90,M,Ed}}{l_{t,90,M}}}{0{,}5bk_{vol}k_{space}f_{t,90,d}} \leq 1{,}0 \quad (6.35)$$
Design tensile force perpendicular to grain from the transfer of shear/bending stress around the hole	$$F_{t,90,V,Ed} = \frac{V_d 0{,}7d_{hole}}{4h}\left[3 - \left(\frac{0{,}7d_{hole}}{h}\right)^2\right] k_{diam} \quad (6.36)$$ $$F_{t,90,M,Ed} = 0{,}09\frac{M_d}{h}\left(\frac{d_{hole}}{h}\right)^2 \quad (6.37)$$
Distribution length for tensile stresses perpendicular to grain from the transfer of shear/bending stresses	$$l_{t,90,V} = 1{,}3d_{hole} \quad (6.38)$$ $$l_{t,90,M} = 0{,}8d_{hole} \quad (6.39)$$
Factor to account for the stress distribution and the location onset	$$k_{diam} = 1{,}1 + 1{,}3\left[\frac{d_{hole}}{h} - \left(\frac{d_{hole}}{h}\right)^2\right] \quad (6.40)$$
Factor to account for the possibility of placing up to 3 circular holes at closer spacing	For individual holes: $k_{space} = 1{,}0$ For members with groups of holes: $$k_{space} = \min \begin{Bmatrix} 1 \\ 1 - 0{,}2\frac{1{,}5h - l_z}{1{,}5h} \\ 1 - 0{,}4\frac{5d - l_z}{5d} \end{Bmatrix} \quad (6.41)$$ if the holes have the same diameter, eccentricity and spacing parallel to grain, and l_z fulfils the following: $d \leq l_z \leq 1{,}5h$, for unreinforced holes $d \leq l_z \leq 1{,}0h$, for reinforced holes

Factor to account for volume effects	$$k_{vol} = \left(\frac{V_{ref}}{0{,}25bd_{hole}^2}\right)^{0{,}2} \quad (6.42)$$ $V_{ref} = 0{,}01$ m^3
The applicable diameter of the hole	For circular holes: $d_{hole} = d$ For rectangular holes: $d_{hole} = k_{shape}k_h$
Factor to account for the shape of the rectangular hole	If $r/h_h \geq 0{,}1$ and $0 \leq \frac{\lvert V_d \rvert h}{\lvert M_d \rvert} \leq 1$ then: $$k_{shape} = 1{,}25 + 0{,}3\frac{l_h}{h_h}\left[4\frac{V_dh}{M_d} - 3\left(\frac{V_dh}{M_d}\right)^2\right] \quad (6.43)$$

Table 6.9: Calculations for bending and shear stresses in cross-section with holes [1]

Design bending stress for circular holes	$$\sigma_{m,d} = \frac{M_d}{W_{net}} \quad (6.44)$$
Design bending stress for rectangular holes	$$\sigma_{m,d} = \frac{M_d}{W_{net}} + \frac{M_{res,d}}{W_{res}} \quad (6.45)$$
Design bending moment from the frame action around the hole	$$M_{res,d} = \frac{V_d}{2}\frac{l_h}{2} \quad (6.46)$$
Section modulus of the residual cross-section	$$W_{res} = \frac{bh_{res}^2}{6} \quad (6.47)$$
Distance from hole to edge	$$h_{res} = \frac{h - h_h}{2} \quad (6.48)$$
Maximum design shear stress	$$\tau_{max,d} = k_\tau \frac{1{,}5V_d}{b(h - h_h)} \quad (6.49)$$
Factor to account for increased shear stress	$$k_\tau = k_{rad}\left(1 + \frac{l_h}{h}\right)\left(\frac{l_h}{h}\right)^{0{,}2} \quad (6.50)$$
Factor to account for the effect of corner radius on shear stress	$k_{rad} = 1{,}8$, for members of GLT and LVL-P in edgewise bending
Height of the hole	$h_h = 0{,}7d$, for circular holes For rectangular holes, see Figure 6.4

For members with reinforcement the minimum distances in Table 6.6 and maximum dimensions in Table 6.7 apply. Members with reinforcement have some additional requirements presented in Table 6.10. The design tensile force for the reinforcement is calculated according to Equation 6.51, with $F_{t,90,V,Ed}$, $F_{t,90,M,Ed}$ and k_{space} from Table 6.8. The different dimensions of reinforced beams are presented in Figure 6.5.



(1) Possible crack line
(2) Rounded corner: corner radius $r \geq 20$ mm, if $h_h \leq 200$ mm
corner radius $r \geq 40$ mm, if $h_h > 200$ mm
(3) Internal reinforcement
(4) External reinforcement

Figure 6.5: Reinforcement of members with holes, with notations [1]

Table 6.10: Calculations for reinforcement of members with holes [1]

Design tensile force for the reinforcement	$$F_{t,90,Ed} = \frac{F_{t,90,V,Ed} + F_{t,90,M,Ed}}{k_{space}} \quad (6.51)$$
Effective anchorage length	For internal dowel-type reinforcement: $l_r = h_{rl}$ or h_{ru} for rectangular holes $l_r = h_{rl} + 0{,}15h_h$ or $h_{ru} + 0{,}15h_h$ for circular holes For plane reinforcement: $l_r = h_{rp}$ for rectangular holes $l_r = h_{rp} + 0{,}15h_h$ for circular holes
Limit to the width of plane reinforcement	$$0{,}25l_h \leq b_r \leq 0{,}6l_{t,90} \quad (6.52)$$
Length under tensile stress perpendicular to grain	$$l_{t,90} = 0{,}5(h_h + h) \quad (6.53)$$
Limit to the height of plane reinforcement above/below a hole	$$h_{rp} \geq \text{max}\{80 \text{ mm}; 0{,}25l_h\} \quad (6.54)$$

6.2 Discussion

EN1995-1-1-Draft [1] has not made significant changes to the rules included in EN1995-1-1 [2]. This may indicate that the rules in EN1995-1-1 [2] was a good estimate of reality with sufficient safety margins, so no change was needed.

The addition of rules for members with holes in EN1995-1-1-Draft [1] is important. This is because members with holes have stress concentrations around the hole. These stress concentrations are important to consider when designing a member with one or more holes.

The calculations in EN1995-1-1-Draft [1] are supported by explanatory text and figures that make the rules easy to understand. With as much text as EN1995-1-1-Draft [1] has included, the rules may seem a little daunting, but they reduce the risk of ambiguities.

In Figure 6.4 h_h is used for the height of the hole, while in Figure 6.5 h_d is used for the same dimension. The reinforcement part of the section about members with holes is from a different working group from most of the rest of the section about members with holes. This fact may explain why two different symbols are used for the same measurement. As the EN1995-1-1-Draft [1] is a draft, some of these types of inconsistencies are expected.

7. Serviceability limit states

7.1 EN1995-1-1 vs EN1995-1-1-Draft

EN1995-1-1 [2] have a section about the slip modulus for fasteners and connectors. In EN1995-1-1-Draft [1] this information is moved to the chapter about connections.

7.1.1 Deformations

Both EN1995-1-1 [2] and EN1995-1-1-Draft [1] cover deflections of members in serviceability limit state design. The notations of the deflection have changed, see Table 7.1, and the formula for max deflection from EN1995-1-1 [2], Equation 7.1, has been split into Equation 7.2 and Equation 7.3 in EN1995-1-1-Draft [1]. The different types of deflection are visualized in Figure 7.1. EN1995-1-1 [2] have limit values for w_{inst}, $w_{net,fin}$ and w_{fin}. The verification for deflection has changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. The limit values that apply for EN1995-1-1-Draft [1] are given in prEN1990:2021 [9], see Table 7.2. The limits in EN1995-1-1-Draft [1] are more detailed than the limits in EN1995-1-1 [2].

Table 7.1: Notations and equations for deflections [1]

	EN1995-1-1	**EN1995-1-1-Draft**
Max deflection, equations	$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \quad (7.1)$	$w_{max} = w_{tot} - w_c \quad (7.2)$ $w_{tot} = w_1 + w_2 + w_3 \quad (7.3)$
Initial/instant. deflection	w_{inst}	w_1, for the permanent part of loads w_3, for the variable part of the load
Long term deflection	w_{creep}	w_2
Total deflection	w_{fin}	w_{tot}
Precamber	w_c	w_c
Final deflection (max deflection)	$w_{net,fin}$	w_{max}

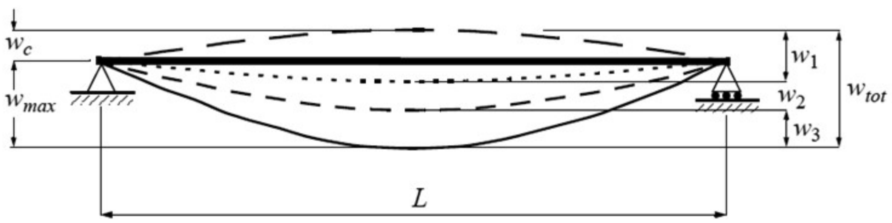


Figure 7.1: Vertical deformations, with notations [1]

Table 7.2: Vertical deflection limits for non-industrial buildings from prEN1990:2021 [9]

Serviceability criteria	**Limiting damage to elements other than structural**[a]	**Comfort of users**	**Appearance**
Combination of actions to be considered	**Characteristic, Formula (8.29)**	**Frequent, Formula (8.30)**	**Quasi-permanent, Formula (8.31)**
Not accessible roof	Roofing rigid roofing: $w_2+w_3 \leq L/250$ resilient roofing: $w_2+w_3 \leq L/125$ Ceiling plastered ceiling: $w_2+w_3 \leq L/350$ false ceiling: $w_2+w_3 \leq L/250$	$w_2+w_3 \leq L/300$	$w_1+w_2-w_c \leq L/250$
Floor, accessible roof	Internal partition walls not reinforced: — partitions of brittle material or non-flexible: $w_2+w_3 \leq L/500$ — partitions of non-brittle materials: $w_{max} \leq L/400$ reinforced walls: $w_2+w_3 \leq L/350$ removable walls: $w_2+w_3 \leq L/250$ Flooring: — tiles rigidly fixed: $w_2+w_3 \leq L/500$ — small tiles[b] or deflection not fully transmitted: $w_2+w_3 \leq L/350$ — resilient flooring: $w_2+w_3 \leq L/250$ Ceiling plastered ceiling: $w_2+w_3 \leq L/350$ false ceiling: $w_2+w_3 \leq L/250$	$w_2+w_3 \leq L/300$	$w_1+w_2-w_c \leq L/250$
Structural frames	Windows: — no loose joints (no clearance between glass and frame): $w_2+w_3 \leq L/1000$ — with loose joints: $w_2+w_3 \leq L/350$		

[a] L = span (or, for cantilever, twice the length); w_1, w_2, w_3, w_{max} are defined in Figure A.1.1.

[b] Small tiles: sides less than 10 cm.

EN1995-1-1 [2] does not include more details about how to calculate the different deflections, except for describing the type of deflection. EN1995-1-1-Draft [1] on the other hand gives the different load combinations for structures with the same and different creep behavior in the materials. The different combinations are listed in Table 7.3. For structures with the same creep behaviour w_2 may be calculated from Equation 7.4. The combination factors, ψ_i, is defined in EN1990:2021 [9].

$$w_2 = w_1 k_{def} \tag{7.4}$$

Table 7.3: Relevant load for the deflection for different SLS load combinations [1]

Load combination	Deflection	Load
Same creep behaviour		
Characteristic	w_{tot}	$$\sum F_d = \sum_{i\geq1} G_{k,i}(1+k_{def})+Q_{k,1}(1+\psi_{2,1}k_{def})+ \sum_{j>1} Q_{k,j}(\psi_{0,j}+\psi_{2,j}k_{def}) \quad (7.5)$$
Frequent	w_{tot}	$$\sum F_d = \sum_{i\geq1} G_{k,i}(1+k_{def})+Q_{k,1}(\psi_{1,1}+ \psi_{2,1}k_{def})+\sum_{j>1} \psi_{2,j}Q_{k,j}(1+k_{def}) \quad (7.6)$$
Quasi-permanent	w_{tot}	$$\sum F_d = \sum_{i\geq1} G_{k,i}(1+k_{def})+ \sum_{j\geq1} \psi_{2,j}Q_{k,j}(1+k_{def}) \quad (7.7)$$
Different creep behaviour		
Characteristic	w_1+w_2	$$\sum F_d = \sum_{i\geq1} G_{k,i}+\psi_{2,1}Q_{k,1}+\sum_{j>1} \psi_{2,j}Q_{k,j} \quad (7.8)$$
	w_3	$$\sum F_d = (1-\psi_{2,1})Q_{k,1}+\sum_{j>1}(\psi_{0,j}-\psi_{2,j})Q_{k,j} \quad (7.9)$$
Frequent	w_1+w_2	$$\sum F_d = \sum_{i\geq1} G_{k,i}+\psi_{2,1}Q_{k,1}+\sum_{j>1} \psi_{2,j}Q_{k,j} \quad (7.10)$$
	w_3	$$\sum F_d = (\psi_{1,1}-\psi_{2,1})Q_{k,1} \quad (7.11)$$
Quasi-permanent	w_1+w_2	$$\sum F_d = \sum_{i\geq1} G_{k,i}+\sum_{j\geq1} \psi_{2,j}Q_{k,j} \quad (7.12)$$

7.1.2 Vibrations

Both EN1995-1-1 [2] and EN1995-1-1-Draft [1] have rules for vibration in structures. Only Equation 7.14 is similar in EN1995-1-1 [2] and EN1995-1-1-Draft [1]. The rules in EN1995-1-1-Draft [1] are more detailed and complex than the ones in EN1995-1-1 [2]. EN1995-1-1-Draft [1] provides thee different verification methods which may be used to verify for vibrations in a structure. The first method is verification by in-situ measurements or measurements on test floors. The second verification method for vibration is to do the calculations provided in EN1995-1-1-Draft [1] and compare the results to the requirements for the vibrations, listed in Table 7.5. The last verification method is doing a thorough dynamic analysis, e.g. by finite element method, to verify that the criteria for the corresponding floor performance level, from EN1995-1-1-Draft [1], is satisfied. The floor performance level determines the vibration criteria in Table 7.5. A list of recommended floor performance levels, based on the use category and whether the quality or economy aspect has more emphasis, is presented in Table 7.4.

Table 7.4: Recommended floor performance levels [1]

Use category	Quality choice	Base choice	Economy choice
A (residential)			
- multi-family block	levels I, II, III	level IV	level V
- single family house	levels I, II, III, IV	level V	level VI
B (office)	levels I, II, III	level IV	level V

Table 7.5: Floor vibration criteria according to the floor performance level [1]

Criteria		Floor performance level					
		I	II	III	IV	V	VI
Response factor, R		4	8	12	24	36	48
Upper deflection limit, $w_{lim,max}$ [mm]		0,25	0,25	0,5	1,0	1,5	2,0
Stiffness criteria for all floors, w_{1kN} [mm]	$\leq$	w_{lim} calculated with Equation 7.13					
Frequency criteria for all floors, f_1 [Hz]	$\geq$	4,5					
Acceleration criteria for resonant vibration ($f_1 < f_{1,lim}$) design situations a_{rms} [m/s^2]	$\leq$	0,005 R					
Velocity criteria for all floors, v_{rms} [m/s]	$\leq$	0,0001 R					

$$\begin{aligned} w_{lim} &= w_{lim,max} && \text{when } w_{lim,max} \leq 0{,}5 \\ 0{,}5 \leq w_{lim} &= \frac{150R}{L} \leq w_{lim,max} && \text{when } w_{lim,max} > 0{,}5 \end{aligned} \tag{7.13}$$

EN1995-1-1-Draft [1] has many different equations for vibration verification. The different equations are presented in Table 7.7 to Table 7.10. Several different variables and constants are used in the equations in Table 7.7 to Table 7.10, these variables and factors are presented in Table 7.6.

Table 7.6: Variables and constants used in vibration verification [1]

ζ	Modal damping - $\zeta = 0{,}02$ for joisted floors - $\zeta = 0{,}025$ for timber-concrete, rib type and slab type floors - $\zeta = 0{,}03$ for joisted floors with a floating floor layer - $\zeta = 0{,}04$ for timber-concrete, rib type and slab type floors with a floating floor layer
f_w	Walking frequency - $f_w \geq 1{,}5$ Hz for residential floors - $f_w \geq 2{,}0$ Hz for floors in other use categories - $f_w \geq 2{,}5$ Hz for all floors and use categories, if it is possible to walk 10 m unobstructed in one direction
L	Floor span length (longer span in case of double span floor), in m
B	Floor width, in m
m	Floor mass per unit area, in kg/m^3
$(EI)_L$	Bending stiffness along the floor span, in Nm^2/m
$(EI)_T$	Bending stiffness transverse to the floor span, in Nm^2/m
$(EI)_{ST}$	Bending stiffness of discrete bending member at midspan transverse to floor span, in Nm^2

Table 7.7: Equations for fundamental frequency f_1 [1]

Fundamental frequency	If floor is on rigid support and mainly subjected to uniform loading: $$f_1 = k_{e,1} k_{e,2} \frac{\pi}{2L^2} \sqrt{\frac{(EI)_L}{m}} \quad (7.14)$$ Else: $$f_1 = k_{e,1} k_{e,2} \frac{18}{\sqrt{w_{sys}}} \quad (7.15)$$ If single span floor is supported elastically on a beam on one or both sides: $$f_1 = \sqrt{\frac{1}{\frac{1}{f_{1,rigid}^2} + \frac{1}{3f_{1,beam,1}^2} + \frac{1}{3f_{1,beam,2}^2}}} \quad (7.16)$$
Frequency factor considering double span floor	Single span: $k_{e,1} = 1$ Double span: $k_{e,1}$ given in Table 7.11
Frequency factor considering the effect of transverse floor stiffness	Single span: $k_{e,2} = 1$ Double span: $$k_{e,2} = \sqrt{1 + \frac{\left(\frac{L}{B}\right)^4 (EI)_T}{(EI)_L}} \quad (7.17)$$
Deflection of the floor	- if w_{sys} is calculated by including the effect of transverse bending stiffness: $k_{e,2} = 1$ - if w_{sys} is calculated by assuming flexible supports: Equation 7.16 may be neglected

Table 7.8: Equations for deflection w_{1kN} [1]

Maximum deflection (due to a vertical static point-load F = 1 kN at the mid-span)	$$w_{1kN} = \frac{FL^3}{48(EI)_L B_{ef}} \quad (7.18)$$ For a floor on flexible supports, as shown in Figure 7.2: $$w_{1kN} = 0{,}5 w_{beam,1} + 0{,}5 w_{beam,2} + w_{rigid} \quad (7.19)$$
Effective width	Floor with uniform transverse bending stiffness: $$B_{ef} = \text{min}\left\{0{,}95L\left(\frac{(EI)_T}{(EI)_L}\right)^{0{,}25}; B\right\} \quad (7.20)$$ Floor with discrete bending member mechanically connected joists at mid-span: $$B_{ef} = \text{min}\left\{1{,}07L^{0{,}75}\left(\frac{(EI)_{ST} + 0{,}63L(EI)_T}{(EI)_L}\right)^{0{,}25}; B\right\} \quad (7.21)$$

Table 7.9: Equations for acceleration a_{rms} [1]

Root mean square value of **acceleration**	$$a_{rms} = \frac{k_{res}\mu F_h}{2\sqrt{2}\zeta M^*} \quad (7.22)$$
Resonant build up factor	May use $\mu = 0{,}4$ If walker can walk 10 m unobstructed: $\mu = 0{,}8$ is recommended
Vertical force caused by the weight of a walking person	Should be taken as: $F_h = 50N$
Factor accounting for the effect of higher modes on vibrations	$$k_{res} = \text{max}\left\{0{,}192\frac{B}{L}\left(\frac{(EI)_L}{(EI)_T}\right)^{0{,}25}; 1{,}0\right\} \quad (7.23)$$
Modal mass	$$M^* = \frac{mLB}{4} \quad (7.24)$$

Table 7.10: Equations for velocity v_{rms} [1]

Root mean square value of **velocity**	$$v_{rms} = v_{tot,peak}(0{,}65 - 0{,}01 f_1)(1{,}22 - 11{,}0\zeta)\eta \quad (7.25)$$
Total velocity peak response	$$v_{tot,peak} = k_{imp} v_{1,peak} \quad (7.26)$$
Peak velocity response for the fundamental mode	$$v_{1,peak} = k_{red} \frac{I_m}{(M^* + 70\text{ kg})} \quad (7.27)$$
Mean modal impulse	$$I_m = \frac{42 f_w^{1{,}43}}{f_1^{1{,}3}} \quad (7.28)$$
Factor accounting for higher modes in the transient response	$$k_{imp} = \text{max}\left\{0{,}48\frac{B}{L}\left(\frac{(EI)_L}{(EI)_T}\right)^{0{,}25}; 1{,}0\right\} \quad (7.29)$$
Factor η	If: $1{,}0 \leq k_{imp} \leq 1{,}9$ (joisted floors): $1{,}0 \leq k_{imp} \leq 1{,}9$ (all other floors): $$\eta = 1{,}35 - 0{,}4 k_{imp} \quad (7.30)$$ Else: $\eta = 0{,}59$ (joisted floors) $\eta = 0{,}67$ (all other floors)

Table 7.11: Factor $k_{e,1}$ [1]

l_2/L a)	1,0	0,9	0,8	0,7	0,6	0,5	0,4	0,3	0,2
$k_{e,1}$ b)	1,00	1,09	1,16	1,21	1,25	1,28	1,32	1,36	1,41

a) L is the longer span, as used in Equation 7.14.
l_2 is the shorter span of a two-span floor, in m.

b) Intermediate values may be obtained by linear interpolation.

1 Floor span, L
2 Deflection w_{rigid}
3 Deflection $w_{beam,1}$
4 Deflection $w_{beam,2}$

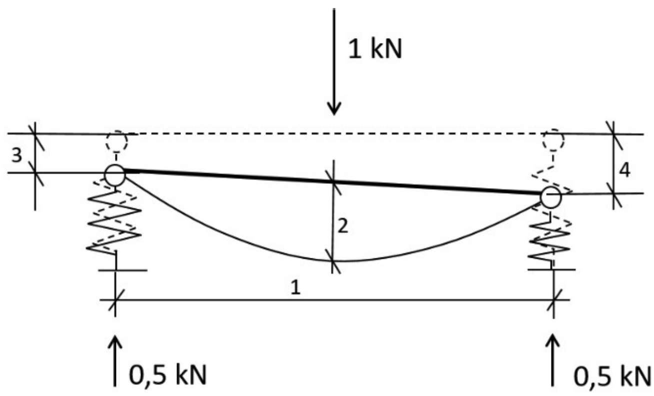


Figure 7.2: Deflection of an elastically supported floor under a vertical static point load [1]

7.2 Discussion

The calculation of the deflection of members does not have a significant change from EN1995-1-1 [2] to EN1995-1-1-Draft [1]. The additional information and the detailed limits make the verification for deflection less open to interpretation. This leaves less room for mistakes. The changes from EN1995-1-1 [2] to EN1995-1-1-Draft [1] are so minor that it is easy to understand the changes.

More calculations are needed to verify vibrations in EN1995-1-1-Draft [1] than in EN1995-1-1 [2]. The difference between the calculations in EN1995-1-1-Draft [1] and EN1995-1-1 [2] are so large that it is hard to compare the rules. It seems that the EN1995-1-1-Draft [1] has substituted the entire section on vibration. Almost all of the different factors and variables needed for the verifications have values or are well explained. The equation for fundamental frequency for a beam, needed in Equation 7.16, is not present in EN1995-1-1-Draft [1].

8. Example cases covering multiple topics

In this chapter some example cases covering topics from different sections is presented. The result of the calculations from EN1995-1-1 [2] and EN1995-1-1-Draft [1] is compared.

8.1 Simply supported beam at an angle

The case presented below is from the 2020 examination paper for TKT4211 "Timber Structures 1" [10] at NTNU. The results based on the rules in EN1995-1-1 [2] is from "Exam Spring 2020 – Solution Proposal" [11].

In this case a beam is simply supported at an angle, as seen in Figure 8.1. Out of plane displacements is prevented at the supports. The beam is also laterally supported out of plane at the mid-span on the top of the beam. The beam is loaded as shown in Figure 8.1, with the characteristic values for the loads listed below along with the other parameters for the beam.

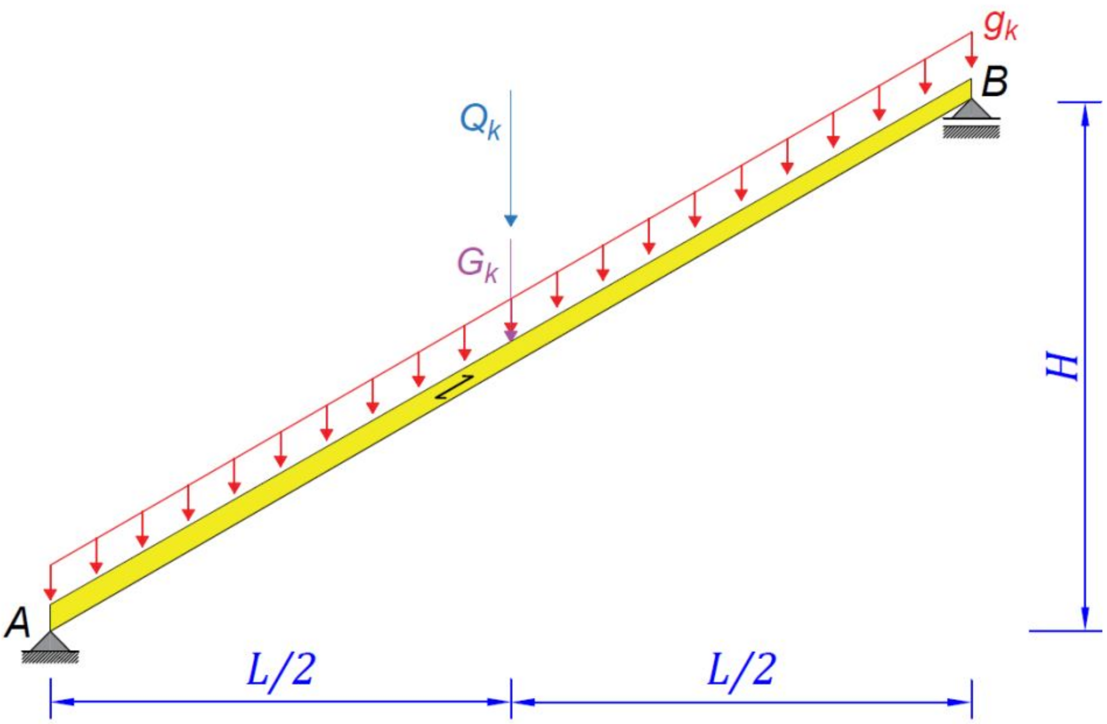


Figure 8.1: Simply supported beam subjected to permanent and variable loads [10]

Parameters used in the case:

Length: $L = 12$ m

Height: $H = 4$ m

Permanent line load: $g_k = 2$ kN/m

Permanent concentrated force: $G_k = 24$ kN

Medium-term variable concentrated load: $Q_k = 50$ kN

Cross-section:

Width: $b = 160$ mm

Height: $h = 990$ mm

Design strengths:

Strength class GL28h

$\left.\begin{array}{l}\text{Service class 1}\\ \text{Medium-term load}\end{array}\right\}k_{mod} = 0{,}8$

$\gamma_M = 1{,}15$ (Norwegian national annex [2])

$f_{m,d} = 19{,}48$ MPa

$f_{v,d} = 2{,}43$ MPa

$f_{c,0,d} = 19{,}48$ MPa

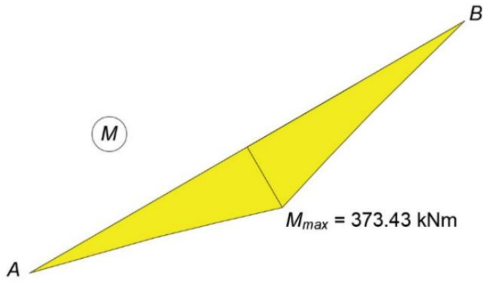
$f_{t,0,d} = 15{,}51$ MPa

The tasks to be performed:

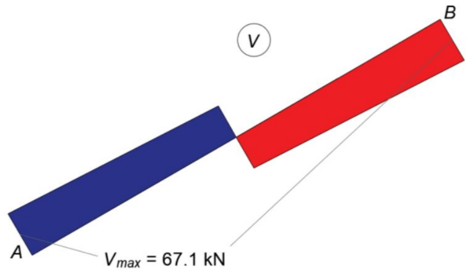
a) Find a cross-section height which gives a maximum design moment between 70% and 80% of the design bending resistance. The width of the cross-section is $b = 160$ mm.

b) Preform all necessary design checks.

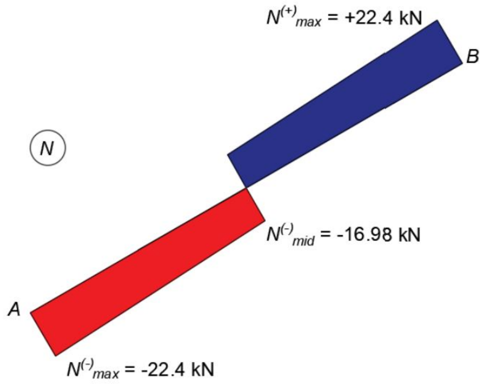
c) Determine the instantaneous and final deflections of the beam for the characteristic load combination. Compare the results with the corresponding requirements (in EN1995-1-1 [2] or EN1995-1-1-Draft [1]).



(a) Moment force diagram [11]



(b) Shear force diagram [11]



(c) Axial force diagram [11]

Figure 8.2: Force diagrams for ULS combination of loads

a)

A height of $h = 990$ mm gives a utilization of bending strength between 70% and 80% for the case for EN1995-1-1 [2] and EN1995-1-1-Draft [1]. The utilization is a bit higher for calculations by EN1995-1-1-Draft [1] due to the inclusion of the depth modification factor, k_h, for the bending strength of the beam. The utilization according to both EN1995-1-1 [2] and EN1995-1-1-Draft [1] is shown in Table 8.1.

b)

The necessary design checks for the beam are compared in Table 8.1. Assumptions/simplifications made in both calculations:

- No reduction of the shear close to the supports performed.

- Calculate bending + compression/tension with max values for compression/tension instead of the value at the mid-span, the difference between the values is small and the compression/tension part of the check is small, so the difference is negligible.
- Simplify the beam model for out of plane buckling: assumes it is simply supported with a length of $l/2$.

Table 8.1: Comparing results from task b) [1]

Design check ($\leq 1 \rightarrow$ OK)	**Equation**	**EN1995-1-1 [2], [11]**	**EN1995-1-1-Draft [1]**
Bending	Equation 4.8	0,73	0,77
Shear	Equation 4.15	0,33	0,38 a)
Compression parallel	Equation 4.3	0,007	0,006
Tension parallel	Equation 4.1	0,009	0,009
Bending + Compression	Equation 4.13	0,73	0,77
Bending + Tension	Equation 4.11	0,74	0,78
Axial buckling (about y-axis)	Equation 5.14	0,74	b)
Axial buckling (about x-axis)		0,54	b)
Lateral torsional buckling	Equation 5.15	0,88	c)

a) the variable k_{var} is not yet determined, assumes $k_{var} = 1$.
b) The kappa method had to be used, so no verification for buckling about z-axis. $\frac{\sigma_{c,0,d}}{f_{c,0,d}} \lambda^2_{rel,c,y,d} = 0{,}003 \leq 0{,}1$ so no buckling check is necessary for buckling about y-axis.
c) Equation 5.40 and Equation 5.40 is fulfilled, so no further check for lateral torsional buckling required.

c)

The serviceability loads perpendicular to the beam axis is the concentrated permanent load $G_{SLS} = 22{,}77$ kN, the concentrated variable load $Q_{SLS} = 47{,}43$ kN and the distributed permanent load $g_{SLS} = 1{,}90$ kN/m.

Table 8.2: Comparing results from task c) [1]

	EN1995-1-1 [2]	**EN1995-1-1-Draft** [1]
Inst. deflection (w_{inst} / $(w_1 + w_3)$) [mm] Requirement [mm]	25,07 25,29 to 42,16	25,07 -
Final deflection (w_{fin} / w_{tot}) [mm] Requirement [mm]	34,22 42,16 to 84,33	34,22 -
Max deflection without inst. deflection from permanent part of load ($w_2 + w_3$) [mm] Requirement [mm]	- -	18,97 50,60 a)

a) Assuming the beam is part of a non-accessible rigid roofing

The deflection requirements in EN1995-1-1-Draft [1] are structured different than the requirements in EN1995-1-1 [2]. The requirements are therefor hard to compare.

9. Conclusion

This thesis aimed to fined out what the differences between EN1995-1-1 [2] and EN1995-1-1-Draft [1] are, and how these differences affect the design of timber structures.

The most significant changes from EN1995-1-1 [2] to EN1995-1-1-Draft [1] found in this thesis include the following:

- In EN1995-1-1-Draft [1], the depth modification factor, k_h, is mandatory to use for GLT, and it applies for all heights of a beam, not only when $h < h_{ref} = 600$ mm. In EN1995-1-1 [2] k_h is mandatory to use and is only used when $h < h_{ref} = 600$ mm.
- EN1995-1-1-Draft [1] includes more design checks for combined stresses than EN1995-1-1 [2] has. This includes the combined shear stress and tensile or compressive stresses perpendicular to the grain, the combined shear stresses from two axis bending, and the combined torsion and bending shear stresses.
- The biggest changes in EN1995-1-1-Draft [1] compared to EN1995-1-1 [2] for the verification of compression perpendicular to the grain are the inclusion of the k_p value and that the verification is no longer constant for a change in the height of the member.
- The k_c method for lateral flexural buckling in EN1995-1-1-Draft [1] does not have major differences from the buckling verification in EN1995-1-1 [2]. The κ_c method is new to EN1995-1-1-Draft [1] and the verification differs slightly from the buckling verification in EN1995-1-1 [2]. The new proposal [8] for the rules for major axis buckling is somewhere between the rules for buckling in EN1995-1-1 [2] and the κ_c method in the EN1995-1-1-Draft [1].
- For the LTB verification rules in EN1995-1-1-Draft [1] is quite different from the rules in EN1995-1-1 [2]. The k_m-method in EN1995-1-1-Draft [1] is quite limited as it cannot be used if there are any axial forces in the member. The κ_m method in EN1995-1-1-Draft [1] is problematic, as in some cases the utilisation goes below 0. The new proposal [8] for minor axis buckling and LTB verifications solves this problem.
- Both the rules for members with varying cross-section and notched members at the support are almost identical in EN1995-1-1-Draft [1] and EN1995-1-1 [2]. EN1995-1-1-Draft [1] has added rules for members with holes and reinforcement for special members, which EN1995-1-1 [2] does not include.
- The calculations for deflections are the same in both EN1995-1-1 [2] and EN1995-1-1-Draft [1] except for notation changes. On the other hand, the verification for the deflections is completely changed.
- Vibration calculation and verification are completely changed from EN1995-1-1 [2] to EN1995-1-1-Draft [1].

Other differences between EN1995-1-1 [2] and EN1995-1-1-Draft [1] of less importance to the design include the following:

- EN1995-1-1-Draft [1] has included more materials in the tables for material-specific factors, e.g. k_{def} and k_{mod}, and thus made it possible to use the design rules on materials that are not present in EN1995-1-1 [2].

- EN1995-1-1-Draft [1] includes rules regarding the durability of timber members and metal fasteners. EN1995-1-1 [2], on the other hand, refers mainly to other standards for the durability of the timber members and its metal fasteners.
- EN1995-1-1-Draft [1] goes in more detail into the structural analysis than EN1995-1-1 [2] does.
- EN1995-1-1-Draft [1] has more information about every subject included in EN1995-1-1 [2].

9.1 Further work

This thesis does not cover all of the EN1995-1-1-Draft [1], and some of the content was not fully explored. Further work that may be interesting and relevant includes the following:

- Investigate how the new topics in the EN1995-1-1-Draft [1] compare to the literature on the topics and experimental results.
- The proposed rules for CLT.
- The proposed rules for connections and compare them to the rules in EN1995-1-1 [2].
- More in depth about the proposed rules for reinforcement.
- The rules for composite members.

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