EBCS EN 1995-1-1:2013

EBCS 5

Design of timber structures

Part 1-1: General –

Common rules and rules for buildings

Final Draft I

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Foreword

This Ethiopian Building Code Standard, EBCS EN 1995-1-1:2013, has been prepared by Technical Committee set up jointly by the Ministry of Urban Development and Construction(MoUDC) and Addis Ababa Institute of Technology (AAiT).

This Ethiopian Building Code Standard supersedesEBCS-5:1995.

Background of the Ethiopian Building Code Standard programme

The Structural Ethiopian Building Code Standard programme comprises the following standards generally consisting of a number of Parts:

EBCS EN 1990:2013	EBCS 0: Basis of Structural Design
EBCS EN 1991:2013	EBCS 1: Actions on structures
EBCS EN 1992:2013	EBCS 2: Design of concrete structures
EBCS EN 1993:2013	EBCS 3: Design of steel structures
EBCS EN 1994:2013	EBCS 4: Design of composite steel and concrete structures
EBCS EN 1995:2013	EBCS 5: Design of timber structures
EBCS EN 1996:2013	EBCS 6: Design of masonry structures
EBCS EN 1997:2013	EBCS 7: Geotechnical design
EBCS EN 1998:2013	EBCS 8: Design of structures for earthquake resistance

Status and field of application of Ethiopian Building Code Standards

Ethiopian Building Code Standards serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonized technical specifications for construction products (ES ENs)

The Ethiopian Building Code Standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

Additional information specific to EBCS EN 1995-1-1:2013

EBCS EN 1995:2013 describes the Principles and requirements for safety, serviceability and durability of timber structures. It is based on the limit state concept used in conjunction with a partial factor method.

For the design of new structures, EBCS EN 1995:2013 is intended to be used, for direct application, together with EBCS EN 1990:2013 and relevant Parts of EBCS EN 1991:2013.

Numerical values for partial factors and other reliability parameters are recommended as

basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and of quality management applies.

Section 1 General

1.1 Scope

1.1.1 Scope of EBCS EN 1995:2013

(1)P EBCS EN 1995:2013 applies to the design of buildings and civil engineering works in timber (solid timber, sawn, planed or in pole form, glued laminated timber or wood-based structural products, e.g. LVL) or wood-based panels jointed together with adhesives or mechanical fasteners. It complies with the principles and requirements for the safety and serviceability of structures and the basis of design and verification given in EBCS EN 1990:2013.

(2)P EBCS EN 1995:2013 is only concerned with requirements for mechanical resistance, serviceability, durability and fire resistance of timber structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

(3) EBCS EN 1995:2013 is intended to be used in conjunction with:
EBCS EN 1990:2013 Basis of design
EBCS EN 1991:2013 "Actions on structures" Ethiopian Standards for construction products relevant to timber structures
EBCS EN 1998:2013 "Design of structures for earthquake resistance", when timber structures are built in seismic regions

(4) EBCS EN 1995:2013 is subdivided into various parts:
EBCS EN 1995-1:2013 General
EBCS EN 1995-2:2013 Bridges*
(** This edition of EBCS EN 1995:2013 however does not cover EBCS EN 1995-2:2013*)

(5) EBCS EN 1995-1:2013 "General" comprises:
EBCS EN 1995-1-1:2013 General– Common rules and rules for buildings
EBCS EN 1995-1-2:2013 General rules – Structural Fire Design*
(** This edition of EBCS EN 1995-1:2013 however does not cover EBCS EN 1995-1-2:2013*)

1.1.2 Scope of EBCS EN 1995-1-1:2013

(1) EBCS EN 1995-1-1:2013 gives general design rules for timber structures together with specific design rules for buildings.

(2) The following subjects are dealt with in EBCS EN 1995-1-1:2013:

Section 1: General

Section 2: Basis of design

Section 3: Material properties

Section 4: Durability

Section 5: Basis of structural analysis

Section 6: Ultimate limit states

Section 7: Serviceability limit states

Section 8: Connections with metal fasteners

Section 9: Components and assemblies

Section 10: Structural detailing and control.

(3)P EBCS EN 1995-1-1:2013 does not cover the design of structures subject to prolonged exposure to temperatures over 60℃.

1.2 Normative references

(1) This Ethiopian Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this Ethiopian Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

ISO standards:

ISO 2081:2007 Metallic coatings - Electroplated coatings of zinc on iron or steel

ISO 2631-2:1989 Evaluation of human exposure to whole-body vibration. Part 2: Continuous and shock-induced vibrations in buildings (1 to 80 Hz)

Ethiopian Standards:

ES EN 300:1997 Oriented Strand Board (OSB) – Definition, classification and specifications

ES EN 301:1992 Adhesives, phenolic and aminoplastic for load-bearing timber structures; classification and performance requirements

ES EN 312-4:1996 Particleboards – Specifications. Part 4: Requirements for load-bearing boards for use in dry conditions

ES EN 312-5:1997 Particleboards – Specifications. Part 5: Requirements for load-bearing boards for use in humid conditions

ES EN 312-6:1996 Particleboards – Specifications. Part 6: Requirements for heavy duty load-bearing boards for use in dry conditions

ES EN 312-7:1997 Particleboards – Specifications. Part 7: Requirements for heavy duty load-bearing boards for use in humid conditions

ES EN 335-1:1992 Durability of wood and wood-based products – definition of hazard classes of biological attack. Part 1: General

ES EN 335-2:1992 Durability of wood and wood-based products – definition of hazard classes of biological attack. Part 2: Application to solid wood

ES EN 335-3:1995 Durability of wood and wood-based products – Definition of hazard classes of biological attack. Part 3: Application to wood-based panels

ES EN 350-2:1994 Durability of wood and wood-based products – Natural durability of solid wood. Part 2: Guide to natural durability and treatability of

selected wood species of importance in Ethiopia

ES EN 351-1:1995 Durability of wood and wood-based products – Preservative treated solid wood. Part 1: Classification of preservative penetration and retention

ES EN 383:1993 Timber structures – Test methods. Determination of embedding strength and foundation values for dowel type fasteners

ES EN 385:2001 Finger jointed structural timber - Performance requirements and minimum production requirements

ES EN 387:2001 Glued laminated timber – Production requirements for large finger joints. Performance requirements and minimum production requirements

ES EN 409:1993 Timber structures – Test methods. Determination of the yield moment of dowel type fasteners – Nails

ES EN 460:1994 Durability of wood and wood-based products – Natural durability of solid wood – Guide of the durability requirements for wood to be used in hazard classes

ES EN 594:1995 Timber structures – Test methods – Racking strength and stiffness of timber frame wall panels

ES EN 622-2:1997 Fibreboards – Specifications. Part 2: Requirements for hardboards

ES EN 622-3:1997 Fibreboards – Specifications. Part 3: Requirements for medium boards

ES EN 622-4:1997 Fibreboards – Specifications. Part 4: Requirements for softboards

ES EN 622-5:1997 Fibreboards – Specifications. Part 5: Requirements for dry process boards (MDF)

ES EN 636-1:1996 Plywood – Specifications. Part 1: Requirements for plywood for use in dry conditions

ES EN 636-2:1996 Plywood – Specifications. Part 2: Requirements for plywood for use in humid conditions

ES EN 636-3:1996 Plywood – Specifications. Part 3: Requirements for plywood for use in exterior conditions

ES EN 912:1999 Timber fasteners – Specifications for connectors for timber

ES EN 1075:1999 Timber structures – Test methods. Testing of joints made with punched metal plate fasteners

ES EN 1380:1999 Timber structures – Test methods – Load bearing nailed joints

ES EN 1381:1999 Timber structures – Test methods – Load bearing stapled joints

ES EN 1382:1999 Timber structures – Test methods – Withdrawal capacity of timber fasteners

ES EN 1383:1999 Timber structures – Test methods – Pull through testing of timber fasteners

EBCS EN 1990:2013 Basis of structural design

EBCS EN 1991-1-1:2013Actions on structures – Part 1-2: General actions – Densities, self-weight and imposed loads

EBCS EN 1991-1-4:2013 Actions on structures – Part 1-4: General actions – Wind loads

EBCS EN 1991-1-5:2013 Actions on structures – Part 1-5: General actions – Thermal actions

EBCS EN 1991-1-6:2013 Actions on structures – Part 1-6: General actions – Actions during execution

EBCS EN 1991-1-7:2013 Actions on structures – Part 1-7: General actions – Accidental actions due to impact and explosions

ES EN 10147:2000 Specification for continuously hot-dip zinc coated structural steel sheet and strip – Technical delivery conditions

ES EN 13271:2001 Timber fasteners – Characteristic load-carrying capacities and slip moduli for connector joints

ES EN 13986 Wood-based panels for use in construction – Characteristics, evaluation of conformity and marking

ES EN 14080 Timber structures – Glued laminated timber – Requirements

ES EN 14081-1 Timber structures – Strength graded structural timber with rectangular cross-section – Part 1, General requirements

ES EN 14250 Timber structures - Production requirements for fabricated trusses using punched metal plate fasteners

ES EN 14279 Laminated veneer lumber (LVL) – Specifications, definitions, classification and requirements

ES EN 14358 Timber structures – Fasteners and wood-based products – Calculation of characteristic 5-percentile value and acceptance criteria for a sample

ES EN 14374 Timber structures – Structural laminated veneer lumber – Requirements

ES EN 14544 Strength graded structural timber with round cross-section – Requirements

ES EN 14545 Timber structures – Connectors – Requirements

ES EN 14592 Timber structures – Fasteners – Requirements

ES EN 26891:1991 Timber structures-Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics

ES EN 28970:1991 Timber structures - Testing of joints made with mechanical fasteners; requirements for wood density (ISO 8970:1989)

1.3 Assumptions

(1)P The general assumptions of EBCS EN 1990:2013 apply.

(2) Additional requirements for structural detailing and control are given in section 10.

1.4 Distinction between Principles and Application Rules

(1)P The rules in EBCS EN 1990:2013 clause 1.4 apply.

1.5 Terms and definitions

1.5.1 General

(1)P The terms and definitions EBCS EN 1990:2013 clause 1.5 apply.

1.5.2 Additional terms and definitions used in this present standard

1.5.2.1
Characteristic value
Refer to EBCS EN 1990:2013 sub-clause 1.5.4.1.

1.5.2.2
Dowelled connection
Connection made with a circular cylindrical rod usually of steel, with or without a head, fitting tightly in prebored holes and used for transferring loads perpendicular to the dowel axis.

1.5.2.3
Equilibrium moisture content
The moisture content at which, wood neither gains nor loses moisture to the surrounding air.

1.5.2.4
Fiber saturation point
Moisture content at which, the wood cells are completely saturated.

1.5.2.5
LVL
Laminated veneer lumber, defined according to ES EN 14279 and ES EN 14374

1.5.2.6
Laminated timber deck
A plate made of abutting parallel and solid laminations connected together by nails or screws or prestressing or gluing.

1.5.2.7
Moisture content
The mass of water in wood expressed as a proportion of its oven-dry mass.

1.5.2.8
Racking
Effect caused by horizontal actions in the plane of a wall.

1.5.2.9
Stiffness property
A property used in the calculation of the deformation of the structure, such as modulus of elasticity, shear modulus, slip modulus.

1.5.2.10
Slip modulus
A property used in the calculation of the deformation between two members of a structure.

1.6 Symbols used in EBCS EN 1995-1-1:2013

For the purpose of EBCS EN 1995-1-1:2013, the following symbols apply.

Latin upper case letters

A	Cross-sectional area
A_{ef}	Effective area of the total contact surface between a punched metal plate fastener and the timber
A_f	Cross-sectional area of flange
$A_{net,t}$	Net cross-sectional area perpendicular to the grain
$A_{net,y}$	Net shear area parallel to the grain
C	Spring stiffness
$E_{0.05}$	Fifth percentile value of modulus of elasticity;
E_d	Design value of modulus of elasticity;
E_{mean}	Mean value of modulus of elasticity;
$E_{mean,fin}$	Final mean value of modulus of elasticity;
F	Force
$F_{A,Ed}$	Design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{A,\min,d}$	Minimum design force acting on a punched metal plate fastener at the centroid of the effective area
$F_{\partial,x,Ed}$	Design axial force on fastener;
$F_{\partial x,Rd}$	Design value of axial withdrawal capacity of the fastener;
$F_{\partial x,Rk}$	Characteristic axial withdrawal capacity of the fastener;
F_c	Compressive force
F_d	Design force
$F_{d,ser}$	Design force at the serviceability limit state
$F_{f,Rd}$	Design load-carrying capacity per fastener in wall diaphragm
$F_{i,c,Ed}$	Design compressive reaction force at end of shear wall
$F_{i,t,Ed}$	Design tensile reaction force at end of shear wall
$F_{i,vert,Ed}$	Vertical load on wall
$F_{i,v,Rd}$	Design racking resistance of panel *i* (in 9.2.4.2) or wall *i* (in 9.2.4.3)
$F_{l\partial}$	Lateral load
$F_{M,Ed}$	Design force from a design moment
F_t	Tensile force
$F_{v,o,Rk}$	Characteristic load-carrying capacity of a connector along the grain;
$F_{v,Ed}$	Design shear force per shear plane of fastener; Horizontal design effect on wall diaphragm
$F_{v,Rd}$	Design load-carrying capacity per shear plane per fastener; Design racking load capacity

$F_{v,Rk}$ Characteristic load-carrying capacity per shear plane per fastener
$F_{v,w,Ed}$ Design shear force acting on web;
$F_{x,Ed}$ Design value of a force in x-direction
$F_{y,Ed}$ Design value of a force in y-direction
$F_{x,Rd}$ Design value of plate capacity in x-direction;
$F_{y,Rd}$ Design value of plate capacity in y-direction;
$F_{x,Rk}$ Characteristic plate capacity in x-direction;
$F_{y,Rk}$ Characteristic plate capacity in y-direction;
$G_{0.05}$ Fifth percentile value of shear modulus
G_d Design value of shear modulus
G_{mean} Mean value of shear modulus
H Overall rise of a truss
l_f Second moment of area of flange
l_{tor} Torsional moment of inertia
l_z Second moment of area about the weak axis
K_{ser} Slip modulus
$K_{ser,fin}$ Final slip modulus
K_u Instantaneous slip modulus for ultimate limit states
$L_{net,t}$ Net width of the cross-section perpendicular to the grain
$L_{net,y}$ Net length of the fracture area in shear
$M_{A,Ed}$ Design moment acting on a punched metal plate fastener
$M_{\partial p,d}$ Design moment at apex zone
M_d Design moment
$M_{y,Rk}$ Characteristic yield moment of fastener
N Axial force
$R_{90,d}$ Design splitting capacity
$R_{90,k}$ Characteristic splitting capacity
$R_{\partial x,d}$ Design load-carrying capacity of an axially loaded connection
$R_{\partial y,d}$ Characteristic load-carrying capacity of an axially loaded connection
$R_{\partial x,\alpha,k}$ Characteristic load-carrying capacity at an angle to grain
R_d Design value of a load-carrying capacity
$R_{ef,k}$ Effective characteristic load-carrying capacity of a connection
$R_{iy,d}$ Design racking racking capacity of a wall
R_k Characteristic load-carrying capacity
$R_{sp,k}$ Characteristic splitting capacity
$R_{to,k}$ Characteristic load-carrying capacity of a toothed plate connector
$R_{y,d}$ Design racking capacity of a wall diaphragm
V Shear force; Volume

q_i V_u, V_i Shear forces in upper and lower part of beam with a hole
W_y Section modulus about axis *y*
X_d Design value of a strength property
X_k Characteristic value of a strength property

Latin lower case letters
a Distance
a_1 Spacing, parallel to grain, of fasteners within one row
a_2 Spacing, perpendicular to grain, between rows of fasteners
$a_{3,C}$ Distance between fastener and unloaded end
$a_{3,t}$ Distance between fastener and loaded end
$a_{4,c}$ Distance between fastener and unloaded edge
$a_{4,t}$ Distance between fastener and loaded edge
a_{bow} Maximum bow of truss member
$a_{bow,perm}$ Maximum permitted bow of truss member
a_{dev} Maximum deviation of truss
$a_{dev,perm}$ Maximum permitted deviation of truss
b Width
b_i Width of panel *i* (in 9.2.4.2) or wall *i* (in 9.2.4.3)
b_{net} Clear distance between studs
b_w Web width
d Diameter
d_1 Diameter of center hole of connector
d_c Connector diameter
d_{ef} Effective diameter
$f_{b,l,k}$ Characteristic embedment strength of timber member *i*
$f_{a,0,0}$ Characteristic anchorage capacity per unit area for $\alpha = 0°$ and $\beta = 0°$
$f_{\partial,90,90}$ Characteristic anchorage capacity per unit area for $\alpha = 90°$ and $\beta = 90°$
$f_{\partial,\alpha,k}$ Characteristic anchorage strength
$f_{\partial x,k}$ Characteristic withdrawal parameter for nails
$f_{c,0,d}$ Design compressive strength along the grain
$f_{c,w,d}$ Design compressive strength of web
$f_{f,c,d}$ Design compressive strength of flange
$f_{c,90,k}$ Characteristic compressive strength perpendicular to grain
$f_{f,t,d}$ Design tensile strength of flange
$f_{b,k}$ Characteristic embedment strength
$f_{head,k}$ Characteristic pull through parameter for nails
f_1 Fundamental frequency

$f_{m,k}$ Characteristic bending strength
$f_{m,y,d}$ Design bending strength about the principal y-axis
$f_{m,z,d}$ Design bending strength about the principal z-axis
$f_{m,\alpha,d}$ Design bending strength at an angle α to the grain
$f_{t,0,d}$ Design tensile strength along the grain
$f_{t,0,k}$ Characteristic tensile strength along the grain
$f_{t,90,d}$ Design tensile strength perpendicular to the grain
$f_{t,w,d}$ Design tensile strength of the web
$f_{u,k}$ Characteristic tensile strength of bolts
$f_{y,o,d}$ Design panel shear strength
$f_{y,\partial x,\alpha,k}$ Characteristic withdrawal strength at an angle to grain
$f_{y,\partial x,90,k}$ Characteristic withdrawal strength perpendicular to grain
$f_{y,d}$ Design shear strength
h Depth; Height of wall
$h_{\partial p}$ Depth of the apex zone
h_d Hole depth
h_e Embedment depth
h_e Loaded edge distance
h_{ef} Effective depth
$h_{f,c}$ Depth of compression flange
$h_{f,t}$ Depth of tension flange
h_{r1} Distance from lower edge of hole to bottom of member
h_{ru} Distance from upper edge of hole to top of member
h_w Web depth
i Notch inclination
$k_{c,y}$ or $k_{c,z}$ Instability factor
k_{crit} Factor used for lateral buckling
k_d Dimension factor for panel
k_{def} Deformation factor
k_{dis} Factor taking into account the distribution of stresses in an apex zone
$k_{f,1}k_{f,2}k_{f,3}$ Modification factors for bracing resistance
k_h Depth factor
$k_{i,q}$ Uniformly distributed load factor
k_m Factor considering re-distribution of bending stresses in a cross-section
k_{mod} Modification factor for duration of load and moisture content
k_n Sheathing material factor
k_r Reduction factor
$k_{R,red}$ Reduction factor for load-carrying capacity

k_s	Fastener spacing factor; Modification factor for spring stiffness
$k_{s,red}$	Reduction factor for spacing
k_{shape}	Factor depending on the shape of the cross-section
k_{sys}	System strength factor
k_y	Reduction factor for notched beams
k_{yol}	Volume factor
k_y or k_z	Instability factor
$\ell_{\partial,\min}$	Minimum anchorage length for a glued-in rod
ℓ	Span; contact length
ℓ_A	Support distance of a hole
ℓ_{ef}	Effective length; Effective length of distribution
ℓ_v	Distance from a hole to the end of the member
ℓ_z	Spacing between holes
m	Mass per unit area
n_{40}	Number of frequencies below 40 Hz
n_{ef}	Effective number of fasteners
p_d	Distributed load
V	Equivalent uniformly distributed load
r	Radius of curvature
s	Spacing
s_o	Basic fastener spacing
r_{in}	Inner radius
t	Thickness
t_{pen}	Penetration depth
u_{creep}	Creep deformation
u_{fin}	Final deformation
$u_{fin,G}$	Final deformation for a permanent action *G*
$u_{fin,Q,1}$	Final deformation for the leading variable action *Q*1
$u_{fin,Q,i}$	Final deformation for accompanying variable actions *Q*i
u_{inst}	Instantaneous deformation
$u_{inst,G}$	Instantaneous deformation for a permanent action *G*
$u_{inst,Q,1}$	Instantaneous deformation for the leading variable action *Q*1
$u_{inst,Q,i}$	Instantaneous deformation for accompanying variable actions *Q*i
W_c	Precamber
W_{creep}	Creep deflection
W_{fin}	Final deflection
W_{inst}	Instantaneous deflection
$W_{net,fin}$	Net final deflection
V	Unit impulse velocity response

Greek lower case letters

$\zeta\ \alpha$ Angle between the x-direction and the force for a punched metal plate; Angle between a force and the direction of grain; Angle between the direction of the load and the loaded edge (or end)

β Angle between the grain direction and the force for a punched metal plate

β_c Straightness factor

y Angle between the x-direction and the timber connection line for a punched metal plate

y_M Partial factor for material properties, also accounting for model uncertainties and dimensional variations

λ_y Slenderness ratio corresponding to bending about the y-axis

λ_z Slenderness ratio corresponding to bending about the z-axis

$\lambda_{rel,y}$ Relative slenderness ratio corresponding to bending about the y-axis

$\lambda_{rel,z}$ Relative slenderness ratio corresponding to bending about the z-axis

p_k Characteristic density

p_m Mean density

$\sigma_{c,o,d}$ Design compressive stress along the grain

$\sigma_{c,\alpha,d}$ Design compressive stress at an angle α to the grain

$\sigma_{c,c,d}$ Mean design compressive stress of flange

$\sigma_{f,c,\max,d}$ Design compressive stress of extreme fibers of flange

$\sigma_{f,t,d}$ Mean design tensile stress of flange

$\sigma_{f,t,\max,d}$ Design tensile stress of extreme fibers of flange

$\sigma_{m,crit}$ Critical bending stress

$\sigma_{m,y,d}$ Design bending stress about the principal y-axis

$\sigma_{m,z,d}$ Design bending stress about the principal z-axis

$\sigma_{m,\alpha,d}$ Design bending stress at an angle *α* to the grain

σ_N Axial stress

$\sigma_{t,o,d}$ Design tensile stress along the grain

$\sigma_{t,90,d}$ Design tensile stress perpendicular to the grain

$\sigma_{w,c,d}$ Design compressive stress of web

$\sigma_{w,t,d}$ Design tensile stress of web

T_d Design shear stress

$T_{E,d}$ Design anchorage stress from axial force

$T_{M,d}$ Design anchorage stress from moment

$T_{tor,d}$ Design shear stress from torsion

ψ_0 Factor for combination value of a variable action

ψ_2 Factor for quasi-permanent value of a variable action

ζ Modal damping ratio

Section 2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1)P The design of timber structures shall be in accordance with EBCS EN 1990:2013.

(2)P The supplementary provisions for timber structures given in this section shall also be applied.

(3) The basic requirements of EBCS EN 1990:2013section 2 are deemed to be satisfied for timber structures when limit state design, in conjunction with the partial factor method using EBCS EN 1990:2013 and EBCS EN 1991:2013 for actions and their combinations and EBCS EN 1995:2013 for resistances, rules for serviceability and durability, is applied.

2.1.2 Reliability management

(1) When different levels of reliability are required, these levels should be preferably achieved by an appropriate choice of quality management in design and execution, according to EBCS EN 1990:2013 Annex C.

2.1.3 Design working life and durability

(1) EBCS EN 1990:2013 clause 2.3 applies.

2.2 Principles of limit state design

2.2.1 General

(1)P The design models for the different limit states shall, as appropriate, take into account the following:

- different material properties (e.g. strength and stiffness);
- different time-dependent behaviour of the materials (duration of load, creep);
- different climatic conditions (temperature, moisture variations);
- different design situations (stages of construction, change of support conditions).

2.2.2 Ultimate limit states

(1)P The analysis of structures shall be carried out using the following values for stiffness properties:

- for a first order linear elastic analysis of a structure, whose distribution of internal forces is not affected by the stiffness distribution within the structure (eg. all members have the same time-dependent properties), mean values shall be used;
- for a first order linear elastic analysis of a structure, whose distribution of internal forces is affected by the stiffness distribution within the structure (eg. composite members containing materials having different time-dependent properties), final mean values adjusted to the load component causing the largest stress in relation to strength shall be used;
- for a second order linear elastic analysis of a structure, design values, not adjusted for duration of load, shall be used.

NOTE 1: For final mean values adjusted to the duration of load, see 2.3.2.2(2).

NOTE 2: For design values of stiffness properties, see 2.4.1(2)P.

(2) The slip modulus of a connection for the ultimate limit state, K_u, should be taken as: 2

$$k_u = \frac{2}{3} k_{ser} \tag{2.1}$$

where k_{ser} is the slip modulus, see 2.2.3(3)P

2.2.3 Serviceability limit states

(1)P The deformation of a structure which results from the effects of actions (such as axial and shear forces, bending moments and joint slip) and from moisture shall remain within appropriate limits, having regard to the possibility of damage to surfacing materials, ceilings, floors, partitions and finishes, and to the functional needs as well as any appearance requirements.

(2) The instantaneous deformation, u_{inst}, see figure 7.1 should be calculated for the characteristic combination of actions, EBCS EN 1990:2013, clause 6.5.3(2) a), using mean values of the appropriate moduli of elasticity, shear moduli and slip moduli.

(3) The final deformation, u_{fin}, see figure 7.1, should be calculated for the quasi-permanent combination of actions, see EBCS EN 1990:2013, clause 6.5.3(2) c).

(4) If the structure consists of members or components having different creep behaviour, the final deformation should be calculated using final mean values of the appropriate moduli of elasticity, shear moduli and slip moduli, according to 2.3.2.2(1).

(5) For structures consisting of members, components and connections with the same creep behaviour and under the assumption of a linear relationship between the actions and the corresponding deformations, as a simplification of 2.2.3(3), the final deformation, u_{fin}, may be taken as:

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + \sum u_{fin,Qi} \tag{2.2}$$

where:

$u_{fin,G} = u_{fin,G}(1 + k_{def})$ for a permanent action, G (2.3)

$u_{fin,Q,1} = u_{inst,Q,1}(1 + \psi_{2,1} k_{def})$ for the leading variable action, Q_1 (2.4)

$u_{fin,Q,i} = u_{inst,Q,i}(\psi_{0,i} + \psi_{2,1} k_{def})$ for accompanying variable actions, Q_i (i > 1) (2.5)

$u_{inst,G}, u_{inst,Q,1}, u_{inst,Q,i}$ are the instantaneous deformations for action G, Q_1, Q_i respectively;

$\psi_{2,1}, \psi_{2,i}$ are the factors for the quasi-permanent value of variable actions;

$\psi_{0,i}$ are the factors for the combination value of variable actions;

K_{def} is given in table 3.2 for timber and wood-based materials, and in 2.3.2.2 (3) and 2.3.2.2 (4) for connections.

When expressions (2.3) to (2.5) are used, the ψ_2 factors should be omitted from expressions (6.16a) and (6.16b) of EBCS EN 1990:2013.

Note: In most cases, it will be appropriate to apply the simplified method.

(6) For serviceability limit states with respect to vibrations, mean values of the appropriate stiffness moduli should be used.

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) Actions to be used in design may be obtained from the relevant parts of EBCS EN 1991:2013.

Note 1: The relevant parts of EBCS EN 1991:2013 for use in design include:

EBCS EN 1991-1-1:2013- Densities, self-weight and imposed loads
EBCS EN 1991-1-4:2013 – Wind actions
EBCS EN 1991-1-5:2013 – Thermal actions
EBCS EN 1991-1-6:2013 – Actions during execution
EBCS EN 1991-1-7:2013 – Accidental actions

(2)P Duration of load and moisture content affect the strength and stiffness properties of timber and wood-based elements and shall be taken into account in the design for mechanical resistance and serviceability.

(3)P Actions caused by the effects of moisture content changes in the timber shall be taken into account.

2.3.1.2 Load-duration classes

(1)P The load-duration classes are characterized by the effect of constant load acting for a certain period of time in the life of the structure. For a variable action the appropriate class shall be determined on the basis of an estimate of the typical variation of the load with time.

(2)P Actions shall be assigned to one of the load-duration classes given in Table 2.1

for strength and stiffness calculations.

Table 2.1 – Load-duration classes

Load-duration class	Order of accumulated duration of characteristic load
Permanent	more than 10 years
Long-term	6 months – 10 years
Medium-term	1 week – 6 months
Short-term	less than one week
Instantaneous	

NOTE: Examples of load-duration assignment are given in Table 2.2.

Table 2.2 – Examples of load-duration assignment

Load-duration class	Example of loading
Permanent	Self-weight
Long-term	Storage
Medium-term	Imposed floor load, snow
Short-term	Snow, wind
Instantaneous	Wind, accidental load

2.3.1.3 Service classes

(1)P Structures shall be assigned to one of the service classes given below:

NOTE 1: The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions.

NOTE 2: Information on the assignment of structures to service classes given in (2)P, (3)P and (4)P is given in the National annex.

(2)P Service class 1 is characterized by a moisture content in the materials corresponding to a temperature of 20℃ and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year.

NOTE: In service class 1 the average moisture content in most softwoods will not exceed 12 %.

(3)P Service class 2 is characterized by a moisture content in the materials corresponding to a temperature of 20℃ and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year.

NOTE: In service class 2 the average moisture content in most softwoods will not exceed 20 %.

(4)P Service class 3 is characterized by climatic conditions leading to higher moisture contents than in service class 2.

2.3.2 Materials and product properties

2.3.2.1 Load-duration and moisture influences on strength

(1) Modification factors for the influence of load-duration and moisture content on strength, see 2.4.1, are given in 3.1.3.

(2) Where a connection is constituted of two timber elements having different time-dependent behaviour, the calculation of the design load-carrying capacity should be made with the following modification factor k_{mod}:

$$k_{mod} = \sqrt{k_{mod,1} k_{mod,2}} \tag{2.6}$$

where $k_{\mathrm{mod,1}}$ and $k_{\mathrm{mod,2}}$ are the modification factors for the two timber elements.

2.3.2.2 Load-duration and moisture influences on deformations

(1) For serviceability limit states, if the structure consists of members or components having different time-dependent properties, the final mean value of modulus of elasticity, $E_{mean,fin}$, shear modulus $G_{mean,fin}$, and slip modulus, $k_{ser,fin}$ which are used to calculate the final deformation should be taken from the following expressions:

$$G_{mean,fin} = \frac{E_{mean}}{(1+k_{def})} \tag{2.7}$$

$$G_{mean,fin} = \frac{G_{mean}}{(1+k_{def})} \tag{2.8}$$

$$K_{ser,fin} = \frac{k_{ser}}{(1+k_{def})} \tag{2.9}$$

(2) For ultimate limit states, where the distribution of member forces and moments is affected by the stiffness distribution in the structure, the final mean value of modulus of elasticity, $E_{mean,fin}$, shear modulus $G_{mean,fin}$, and slip modulus, $k_{ser,fin}$ should be calculated from the following expressions :

$$E_{mean,fin} = \frac{E_{mean}}{(1+\psi_2 k_{def})} \tag{2.10}$$

$$G_{mean,fin} = \frac{G_{mean}}{(1+\psi_2 k_{def})} \tag{2.11}$$

$$K_{ser,fin} = \frac{K_{mean}}{(1+\psi_2 k_{def})} \tag{2.12}$$

where:

E_{mean} is the mean value of modulus of elasticity;

G_{mean} is the mean value of shear modulus;

K_{ser} is the slip modulus;

K_{def} is a factor for the evaluation of creep deformation taking into account the relevant service class;

ψ_2 is the factor for the quasi-permanent value of the action causing the largest stress in relation to the strength (if this action is a permanent action, $\psi_2$should be replaced by 1).

NOTE 1: Values of k_{def} are given in 3.1.4.

NOTE 2: Values of ψ_2 are given in EBCS EN 1990:2013.

(3) Where a connection is constituted of timber elements with the same time-dependent behaviour, the value of k_{def} should be doubled.

(4) Where a connection is constituted of two wood-based elements having different time-dependent behaviour, the calculation of the final deformation should be made with the following deformation factor k_{def} :

$$k_{def} = 2\sqrt{k_{def,1}k_{def,2}} \tag{2.13}$$

where $k_{def,1}$ and $k_{def,2}$ are the deformation factors for the two timber elements.

2.4 Verification by the partial factor method

2.4.1 Design value of material property

(1)P The design value X_d of a strength property shall be calculated as:

$$X_d = k_{\text{mod}} \frac{X_k}{\gamma_M} \tag{2.14}$$

where:

X_k is the characteristic value of a strength property;

γ_M is the partial factor for a material property;

k_{mod} is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of k_{mod} are given in 3.1.3.

NOTE 2: The recommended partial factors for material properties (γ_M) are given in Table 2.3.

Table 2.3 – Recommended partial factors γ_M for material properties and resistances

Fundamental combinations:	
Solid timber	1.3
Glued laminated timber	1.25
LVL, plywood, OSB,	1.2
Particleboards	1.3
Fiberboards, hard	1.3
Fiberboards, medium	1.3
Fiberboards, MDF	1.3
Fiberboards, soft	1.3
Connections	1.3
Punched metal plate fasteners	1.25
Accidental combinations	1.0

(2) P The design member stiffness property E_d *or* G_d shall be calculated as:

$$E_d = \frac{E_{mean}}{\gamma_M} \quad (2.15)$$

$$G_d = \frac{G_{mean}}{\gamma_M} \quad (2.16)$$

Where:

E_{mean} is the mean value of modulus of elasticity;

G_{mean} is the mean value of shear modulus.

2.4.2 Design value of geometrical data

(1) Geometrical data for cross-sections and systems may be taken as nominal values from product standards or drawings for the execution.

(2) Design values of geometrical imperfections specified in this standard comprise the effects of

- geometrical imperfections of members;
- the effects of structural imperfections from fabrication and erection;
- inhomogeneity of materials (e.g. due to knots).

2.4.3 Design resistances

(1)P The design value R_d of a resistance (load-carrying capacity) shall be calculated as:

$$R_d = k_{\mathrm{mod}} \frac{R_k}{\gamma_M} \quad (2.17)$$

where:

R_k is the characteristic value of load-carrying capacity;

γ_M is the partial factor for a material property,

k_{mod} is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of k_{mod} are given in 3.1.3.

NOTE 2: For partial factors, see 2.4.1.

2.4.4 Verification of equilibrium (EQU)

(1) The reliability format for the verification of static equilibrium given in Table A1.2 (A) in Annex A1 of EBCS EN 1990:2013 applies, where appropriate, to the design of timber structures, e.g. for the design of holding-down anchors or the verification of bearings subject to uplift from continuous beams.

Section 3 Material properties

3.1 General

3.1.1 Strength and stiffness parameters

(1)P Strength and stiffness parameters shall be determined on the basis of tests for the types of action effects to which the material will be subjected in the structure, or on the basis of comparisons with similar timber species and grades or wood-based materials, or on well-established relations between the different properties.

3.1.2 Stress-strain relations

(1)P Since the characteristic values are determined on the assumption of a linear relation between stress and strain until failure, the strength verification of individual members shall also be based on such a linear relation.

(2) For members or parts of members subjected to compression, a non-linear relationship (elastic-plastic) may be used.

3.1.3 Strength modification factors for service classes and load-duration classes

(1) The values of the modification factor k_{mod} given in Table 3.1 should be used.

(2) If a load combination consists of actions belonging to different load-duration classes a value of k_{mod} should be chosen which corresponds to the action with the shortest duration, e.g. for a combination of dead load and a short-term load, a value of k_{mod} corresponding to the short-term load should be used.

3.1.4 Deformation modification factors for service classes

(1) The values of the deformation factors k_{def} given in Table 3.2 should be used.

3.2 Solid timber

(1) P Timber members shall comply with ES EN 14081-1. Timber members with round cross-section shall comply with ES EN 14544.

NOTE: Strength classes for timber are given in ES EN 338.

(2) The effect of member size on strength may be taken into account.

(3) For rectangular solid timber with a characteristic timber density $p_k \le 700kg/m^3$, the reference depth in bending or width (maximum cross-sectional dimension) in tension is 150 mm. For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for $f_{m,k}$ and $f_{t,o,k}$ may be increased by the factor k_h, given by:

$$k_h = \min \begin{cases} \left(\frac{150}{h}\right)^{0.2} \\ 1.3 \end{cases} \tag{3.1}$$

where h is the depth for bending members or width for tension members, in mm.

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instanta-neous action
Solid timber	ES EN 14081-1	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
Glued laminated timber	ES EN 14080	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
LVL	ES EN 14374, ES EN 14279	1 2 3	0,60 0,60 0,50	0,70 0,70 0,55	0,80 0,80 0,65	0,90 0,90 0,70	1,10 1,10 0,90
Plywood	ES EN 636 Part 1, Part 2, Part 3 Part 2, Part 3 Part 3	 1 2 3	 0,60 0,60 0,50	 0,70 0,70 0,55	 0,80 0,80 0,65	 0,90 0,90 0,70	 1,10 1,10 0,90
OSB	ES EN 300 OSB/2 OSB/3, OSB/4 OSB/3, OSB/4	 1 1 2	 0,30 0,40 0,30	 0,45 0,50 0,40	 0,65 0,70 0,55	 0,85 0,90 0,70	 1,10 1,10 0,90
Particle-board	ES EN 312 Part 4, Part 5 Part 5 Part 6, Part 7 Part 7	 1 2 1 2	 0,30 0,20 0,40 0,30	 0,45 0,30 0,50 0,40	 0,65 0,45 0,70 0,55	 0,85 0,60 0,90 0,70	 1,10 0,80 1,10 0,90
Fibreboard, hard	ES EN 622-2 HB.LA, HB.HLA 1 or 2 HB.HLA1 or 2	 1 2	 0,30 0,20	 0,45 0,30	 0,65 0,45	 0,85 0,60	 1,10 0,80
Fibreboard, medium	ES EN 622-3 MBH.LA1 or 2 MBH.HLS1 or 2 MBH.HLS1 or 2	 1 1 2	 0,20 0,20 –	 0,40 0,40 –	 0,60 0,60 –	 0,80 0,80 0,45	 1,10 1,10 0,80
Fibreboard, MDF	ES EN 622-5 MDF.LA, MDF.HLS MDF.HLS	 1 2	 0,20 –	 0,40 –	 0,60 –	 0,80 0,45	 1,10 0,80

Table 3.1 – Values of k_{mod}

(4) For timber which is installed at or near its fibre saturation point, and which is likely to dry out under load, the values of k_{def} ,given in Table 3.2, should be increased by 1.0.

(5) P Finger joints shall comply with ES EN 385.

3.3 Glued laminated timber

(1) P Glued laminated timber members shall comply with ES EN 14080.

NOTE: In ES EN 1194 values of strength and stiffness properties are given for glued laminated timber allocated to strength classes, see annex D (Informative).

(2) The effect of member size on strength may be taken into account.

(3) For rectangular glued laminated timber, the reference depth in bending or width in tension is 600 mm. For depths in bending or widths in tension of glued laminated timber

less than 600 mm the characteristic values for $f_{m,k}$ and $f_{t,o,k}$ may be increased by the factor k_h, given by

$$k_h = \min \begin{cases} \left(\frac{600}{h}\right)^{0.1} \\ 1.1 \end{cases} \quad (3.2)$$

where h is the depth for bending members or width for tensile members, in mm.

(4)P Large finger joints complying with the requirements of ENV 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint.

(5)P The effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

Table 3.2- Values of k_{def} for timber and wood-based materials

Material	Standard	Service class		
		1	2	3
Solid timber	ES EN 14081-1	0.60	0.80	2.00
Glued Laminated timber	ES EN 14080	0.60	0.80	2.00
LVL	ES EN 14374, ES EN 14279	0.60	0.80	2.00
Plywood	ES EN 636 Part 1 Part 2 Part 3	 0.80 0.80 0.80	 – 1.00 1.00	 – – 2.50
OSB	ES EN 300 OSB/2 OSB/3, OSB/4	 2.25 1.50	 – 2.25	 – –
Particleboard	ES EN 312 Part 4 Part 5 Part 6 Part 7	 2.25 2.25 1.50 1.50	 – 3.00 – 2.25	 – – – –
Fibreboard, hard	ES EN 622-2 HB.LA HB.HLA1, HB.HLA2	 2.25 2.25	 – 3.00	 – –
Fibreboard, medium	ES EN 622-3 MBH.LA1, MBH.LA2 MBH.HLS1, MBH.HLS2	 3.00 3.00	 – 4,00	 – –
Fibreboard, MDF	ES EN 622-5 MDF.LA MDF.HLS	 2.25 2.25	 – 3,00	 – –

3.4 Laminated veneer lumber (LVL)

(1)P LVL structural members shall comply with ES EN 14374.

(2)P For rectangular LVL with the grain of all veneers running essentially in one direction, the effect of member size on bending and tensile strength shall be taken into account.

(3)The reference depth in bending is 300 mm. For depths in bending not equal to 300 mm the characteristic value for $f_{m,k}$ should be multiplied by the factor k_h, given by

$$k_h = \min \begin{cases} \left(\frac{300}{h}\right)^s \\ 1.2 \end{cases} \tag{3.3}$$

where:

h is the depth of the member, in mm;

s is the size effect exponent, refer to 3.4(5)P.

(4) The reference length in tension is 3000 mm. For lengths in tension not equal to 3000 mm the characteristic value for $f_{t,o,k}$ should be multiplied by the factor k_t given by

$$k_\ell = \min \begin{cases} \left(\frac{3000}{\ell}\right)^{s/2} \\ 1.1 \end{cases} \tag{3.4}$$

where ℓ is the length, in mm.

(5)P The size effect exponent *s* for LVL shall be taken as declared in accordance with ES EN 14374.

(6)P Large finger joints complying with the requirements of ENV 387 shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint.

(7)P For LVL with the grain of all veneers running essentially in one direction, the effect of member size on the tensile strength perpendicular to the grain shall be taken into account.

3.5 Wood-based panels

(1)P Wood-based panels shall comply with ES EN 13986 and LVL used as panels shall comply with ES EN 14279.

(2) The use of softboards according to ES EN 622-4 should be restricted to wind bracing and should be designed by testing.

3.6 Adhesives

(1)P Adhesives for structural purposes shall produce joints of such strength and durability that the integrity of the bond is maintained in the assigned service class throughout the expected life of the structure.

(2) Adhesives which comply with Type I specification as defined in ES EN 301 may be used in all service classes.

(3) Adhesives which comply with Type II specification as defined in ES EN 301 should only

be used in service classes 1 or 2 and not under prolonged exposure to temperatures in excess of 50℃.

3.7 Metal fasteners

(1)P Metal fasteners shall comply with ES EN 14592 and metal connectors shall comply with ES EN 14545.

Section 4 Durability

4.1 Resistance to biological organisms

(1)P Timber and wood-based materials shall either have adequate natural durability in accordance with ES EN 350-2 for the particular hazard class (defined in ES EN 335-1, ES EN 335-2 and ES EN335-3), or be given a preservative treatment selected in accordance with ES EN 351-1 and ES EN460.

NOTE 1: Preservative treatment may affect the strength and stiffness properties.

NOTE 2: Rules for specification of preservation treatments are given in ES EN 350-2 and ES EN335.

4.2 Resistance to corrosion

(1)P Metal fasteners and other structural connections shall, where necessary, either be inherently corrosion-resistant or be protected against corrosion.

(2) Examples of minimum corrosion protection or material specifications for different service classes (see 2.3.1.3) are given in Table 4.1.

Table 4.1 – Examples of minimum specifications for material protection against corrosion for fasteners (related to ES ISO 2081)

Fastener	Service Class[b]		
	1	2	3
Nails and screws with $d \leq 4$ mm	None	Fe/Zn 12ca	Fe/Zn 25ca
Bolts, dowels, nails and screws with $d > 4$ mm	None	None	Fe/Zn 25ca
Staples	Fe/Zn 12ca	Fe/Zn 12ca	Stainless steel
Punched metal plate fasteners and steel plates up to 3 mm thickness	Fe/Zn 12ca	Fe/Zn 12ca	Stainless steel
Steel plates from 3 mm up to 5 mm in thickness	None	Fe/Zn 12ca	Fe/Zn 25ca
Steel plates over 5 mm thickness	None	None	Fe/Zn 25ca
[a] If hot dip zinc coating is used, Fe/Zn 12c should be replaced by Z275 and Fe/Zn 25c by Z350 in accordance with ES EN 10147			
[b] For especially corrosive conditions consideration should be given to heavier hot dip coatings or stainless steel.			

Section 5 Basis of structural analysis

5.1 General

(1)P Calculations shall be performed using appropriate design models (supplemented, if necessary, by tests) involving all relevant variables. The models shall be sufficiently precise to predict the structural behaviour, commensurate with the standard of workmanship likely to be achieved, and with the reliability of the information on which the design is based.

(2)The global structural behaviour should be assessed by calculating the action effects with a linear material model (elastic behaviour).

(3) For structures able to redistribute the internal forces via connections of adequate ductility, elastic-plastic methods may be used for the calculation of the internal forces in the members.

(4)P The model for the calculation of internal forces in the structure or in part of it shall take into account the effects of deformations of the connections.

(5) In general, the influence of deformations in the connections should be taken into account through their stiffness (rotational or translational for instance) or through prescribed slip values as a function of the load level in the connection.

5.2 Members

(1)P The following shall be taken into account by the structural analysis:

- deviations from straightness;
- inhomogeneities of the material.

NOTE: Deviations from straightness and inhomogeneities are taken into account implicitly by the design methods given in this standard.

(2)P Reductions in the cross-sectional area shall be taken into account in the member strength verification.

(3) Reductions in the cross-sectional area may be ignored for the following cases:

- nails and screws with a diameter of 6 mm or less, driven without pre-drilling;
- holes in the compression area of members, if the holes are filled with a material of higher stiffness than the wood.

(4) When assessing the effective cross-section at a joint with multiple fasteners, all holes within a distance of half the minimum fastener spacing measured parallel to the grain from a given cross-section should be considered as occurring at that cross-section.

5.3 Connections

(1)P The load-carrying-capacity of the connections shall be verified taking into account the forces and the moments between the members determined by the global structural analysis.

(2)P The deformation of the connection shall be compatible with that assumed in the global analysis.

(3)P The analysis of a connection shall take into account the behaviour of all the elements which constitute the connection.

5.4 Assemblies

5.4.1 General

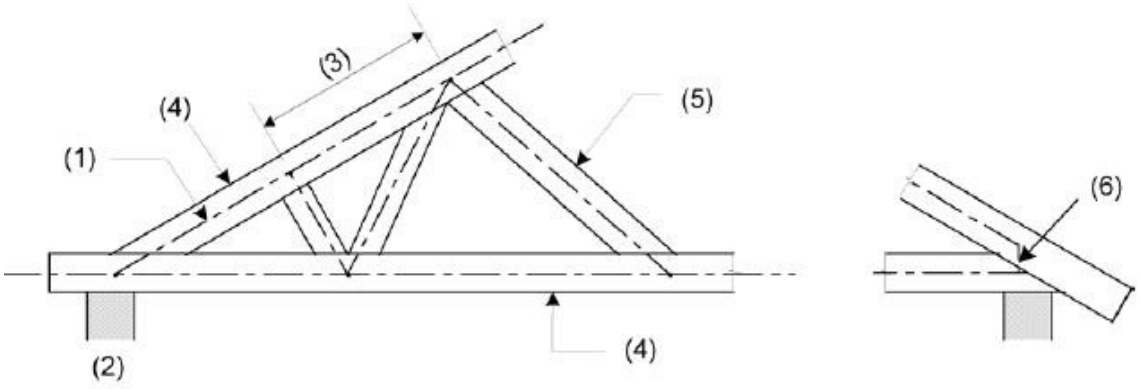
(1)P The analysis of structures shall be carried out using static models which consider to an acceptable level of accuracy the behaviour of the structure and of the supports.

(2) The analysis should be performed by frame models in accordance with 5.4.2 or by a simplified analysis in accordance with 5.4.3 for trusses with punched metal plate fasteners.

(3) Second order analysis of plane frames or arches should be performed in accordance with 5.4.4.

5.4.2 Frame structures

(1)P Frame structures shall be analysed such that the deformations of the members and joints, the influence of support eccentricities and the stiffness of the supporting structure are taken into account in the determination of the member forces and moments, see Figure 5.1 for definitions of structure configurations and model elements.



Key:
(1) System line
(2) Support
(3) Bay
(4) External member
(5) Internal member
(6) Fictitious beam element

Figure 5.1 – Examples of frame analysis model elements

(2)P In a frame analysis, the system lines for all members shall lie within the member profile. For the main members, e.g. the external members of a truss, the system lines shall coincide with the member centre-line.

(3)P If the system lines for internal members do not coincide with the centre lines, the influence of the eccentricity shall be taken into account in the strength verification of these members.

(4) Fictitious beam elements and spring elements may be used to model eccentric connections or supports. The orientation of fictitious beam elements and the location of the

spring elements should coincide as closely as possible with the actual joint configuration.

(5) In a first order linear elastic analysis, the effect of initial deformations and induced deflections may be disregarded if taken into account by the strength verification of the member.

(6) The frame analysis should be carried out using the appropriate values of member stiffness defined in 2.2.2. Fictitious beam elements should be assumed to have a stiffness corresponding to that of the actual connections.

(7) Connections may be assumed to be rotationally stiff, if their deformation has no significant effect upon the distribution of member forces and moments. Otherwise, connections may be generally assumed to be rotationally pinned.

(8) Translational slip at the joints may be disregarded for the strength verification unless it significantly affects the distribution of internal forces and moments.

(9) Splice connections used in lattice structures may be modelled as rotationally stiff if the actual rotation under load would have no significant effect upon member forces. This requirement is fulfilled if one of the following conditions is satisfied:

- The splice connection has a load-carrying capacity which corresponds to at least 1.5 times the combination of applied force and moment
- The splice connection has a load-carrying capacity which corresponds to at least the combination of applied force and moment, provided that the timber members are not subject to bending stresses which are greater than 0,3 times the member bending strength, and the assembly would be stable if all such connections acted as pins.

5.4.3 Simplified analysis of trusses with punched metal plate fasteners

(1) A simplified analysis of fully triangulated trusses should comply with the following conditions:

- there are no re-entrant angles in the external profile;
- the bearing width is situated within the length $a1$, and the distance $a2$ in Figure 5.2 is not greater than $a1/3$ or 100 mm, whichever is the greater;
- the truss height is greater than 0,15 times the span and 10 times the maximum external member depth.

(2) The axial forces in the members should be determined on the basis that every node is pin-jointed.

(3) The bending moments in single-bay members should be determined on the basis that the end nodes are pin-jointed. Bending moments in members that are continuous over several bays should be determined on the basis that the member is a beam with a simple support at each node. The effect of deflection at the nodes and partial fixity at the connections should be taken into account by a reduction of 10 % of the moments at the inner supports of the member. The inner support moments should be used to calculate the span bending moments.

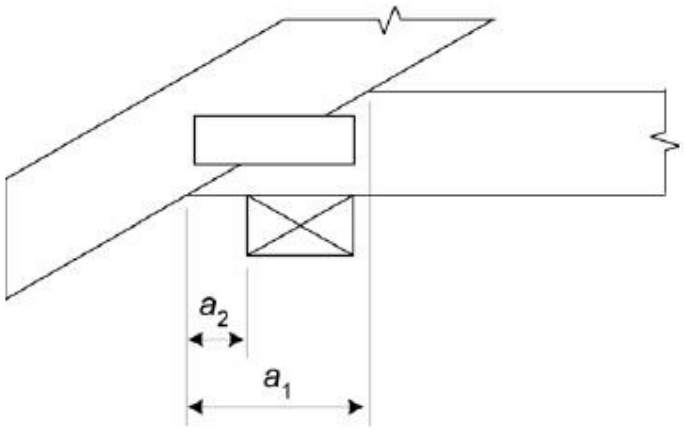


Figure 5.2 – Geometry of support

5.4.4 Plane frames and arches

(1)P The requirements of 5.2 apply. The effects of induced deflection on internal forces and moments shall be taken into account.

(2) The effects of induced deflection on internal forces and moments may be taken into account by carrying out a second order linear analysis with the following assumptions:
– the imperfect shape of the structure should be assumed to correspond to an initial deformation which is found by applying an angle ϕ of inclination to the structure or relevant parts, together with an initial sinusoidal curvature between the nodes of the structure corresponding to a maximum eccentricity *e*.

– the value of ϕ in radians should as a minimum be taken as

$$\begin{aligned} \phi &= 0.005 && for \quad h \le 5m \\ \phi &= 0.005\sqrt{5/h} && for \quad h > 5m \end{aligned} \qquad (5.1)$$

where h is the height of the structure or the length of the member, in m.

– the value of *e* should as a minimum be taken as:

$$e = 0.0025\ell \qquad (5.2)$$

Examples of assumed initial deviations in the geometry and the definition of ℓ are given in Figure 5.3.

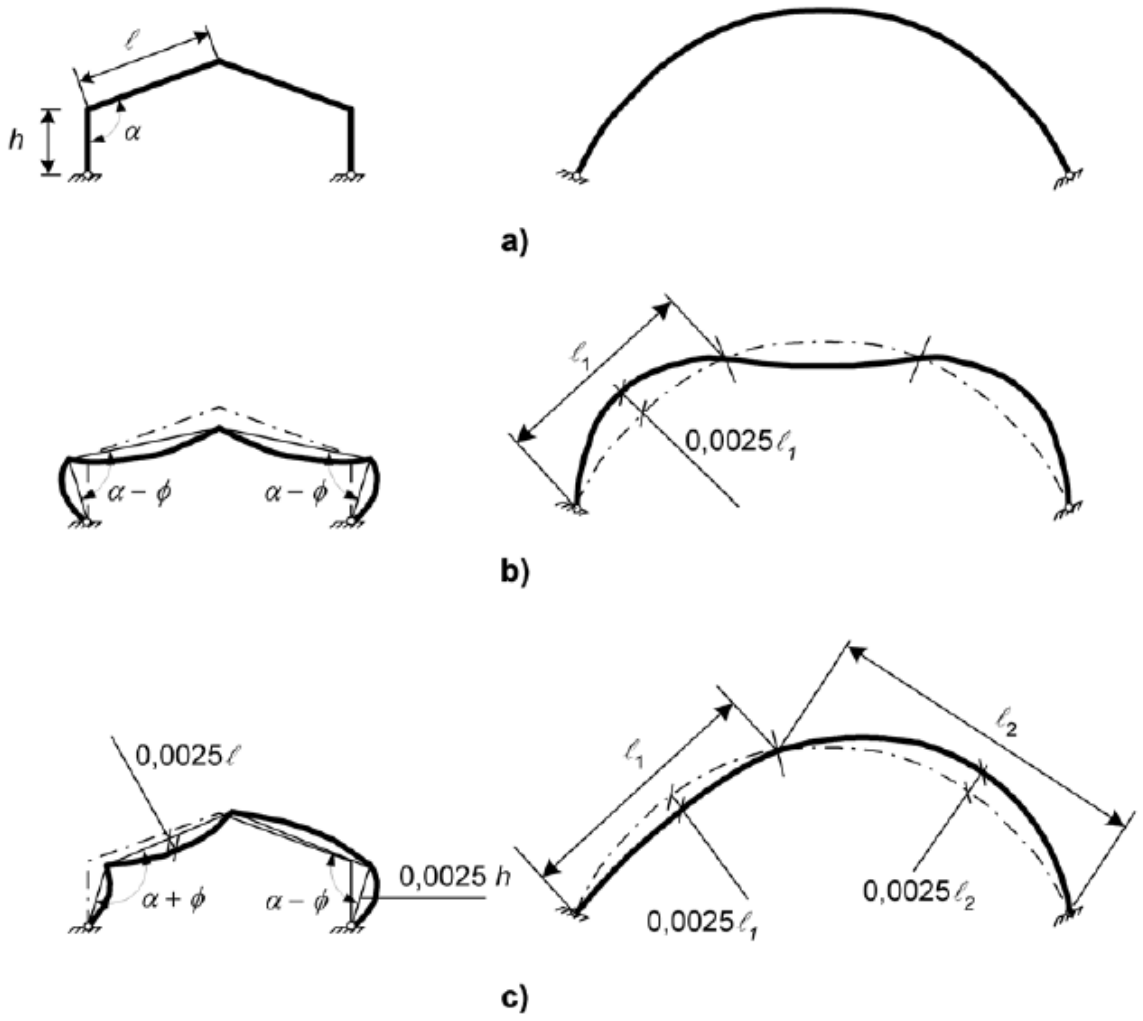


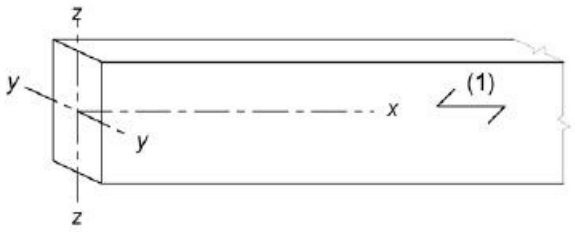
Figure 5.3 – Examples of assumed initial deviations in the geometry for a frame (a), corresponding to a symmetrical load (b) and non-symmetrical load (c)

Section 6 Ultimate limit states

6.1 Design of cross-sections subjected to stress in one principal direction

6.1.1 General

(1) Clause 6.1 applies to straight solid timber, glued laminated timber or wood-based structural products of constant cross-section, whose grain runs essentially parallel to the length of the member. The member is assumed to be subjected to stresses in the direction of only one of its principal axes (see Figure 6.1).



Key:
(1) direction of grain

Figure 6.1 – Member Axes

6.1.2 Tension parallel to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{t,0,d} \le f_{t,0,d} \quad (6.1)$$

where:

$\sigma_{t,0,d}$ is the design tensile stress along the grain;

$f_{t,0,d}$ is the design tensile strength along the grain

6.1.3 Tension perpendicular to the grain

(1)P The effect of member size shall be taken into account.

6.1.4 Compression parallel to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{c,0,d} \le f_{c,0,d} \quad (6.2)$$

where:

$\sigma_{c,0,d}$ is the design compressive stress along the grain;

$f_{c,0,d}$ is the design compressive strength along the grain.

NOTE: Rules for the instability of members are given in 6.3.

6.1.5 Compression perpendicular to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{c,90,d} \le k_{c,90} f_{c,0,d} \quad (6.3)$$

where: $\sigma_{c,90,d}$ is the design compressive stress in the contact area perpendicular to the grain $f_{c,90,d}$ is the design compressive strength perpendicular to the grain; $k_{c,90}$ is a factor taking into account the load configuration, possibility of splitting and degree of compressive deformation.

(2) The value of $k_{c,90}$ should be taken as 1.0, unless the member arrangements in the following paragraphs apply. In these cases the higher value of $k_{c,90}$ specified may be taken, up to a limiting value of $k_{c,90}$ = 4.0.

NOTE: When a higher value of $k_{c,90}$ is used, and contact extends over the full member width *b*, the resulting compressive deformation at the ultimate limit state will be approximately 10% of the member depth.

(3) For a beam member resting on supports (see Figure 6.2), the factor $k_{c,90}$ should be calculated from the following expressions:

– When the distance from the edge of a support to the end of a beam *a*, ≤ *h*/3:

$$k_{c,90} = \left(2{,}38 - \frac{\ell}{250}\right)\left(1 + \frac{h}{12\ell}\right) \tag{6.4}$$

– At internal supports:

$$k_{c,90} = \left(2{,}38 - \frac{\ell}{250}\right)\left(1 + \frac{h}{6\ell}\right) \tag{6.5}$$

where:

ℓ is the contact length in mm;

h is member depth in mm.

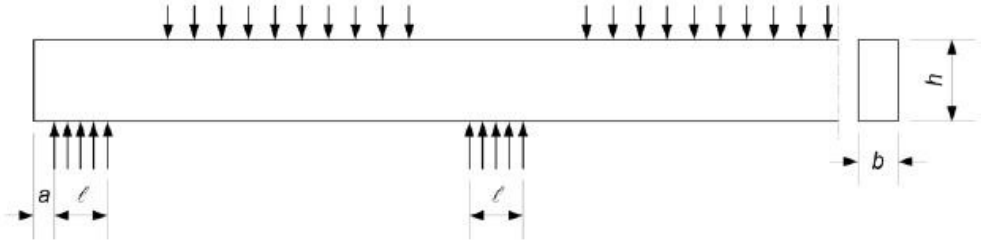


Figure 6.2 – Beam on supports

(4) For a member with a depth $h \le 2.5b$ where a concentrated force with contact over the full width *b* of the member is applied to one face directly over a continuous or discrete support on the opposite face, see Figure 6.3, the factor $k_{c,90}$ is given by:

$$k_{c,90} = \left(2.38 - \frac{\ell}{250}\right)\left(\frac{\ell_{ef}}{\ell}\right)^{0.5} \tag{6.6}$$

where:

ℓ_{ef} is the effective length of distribution, in mm, see (5) below;

ℓ is the contact length, see Figure 6.3, in mm.

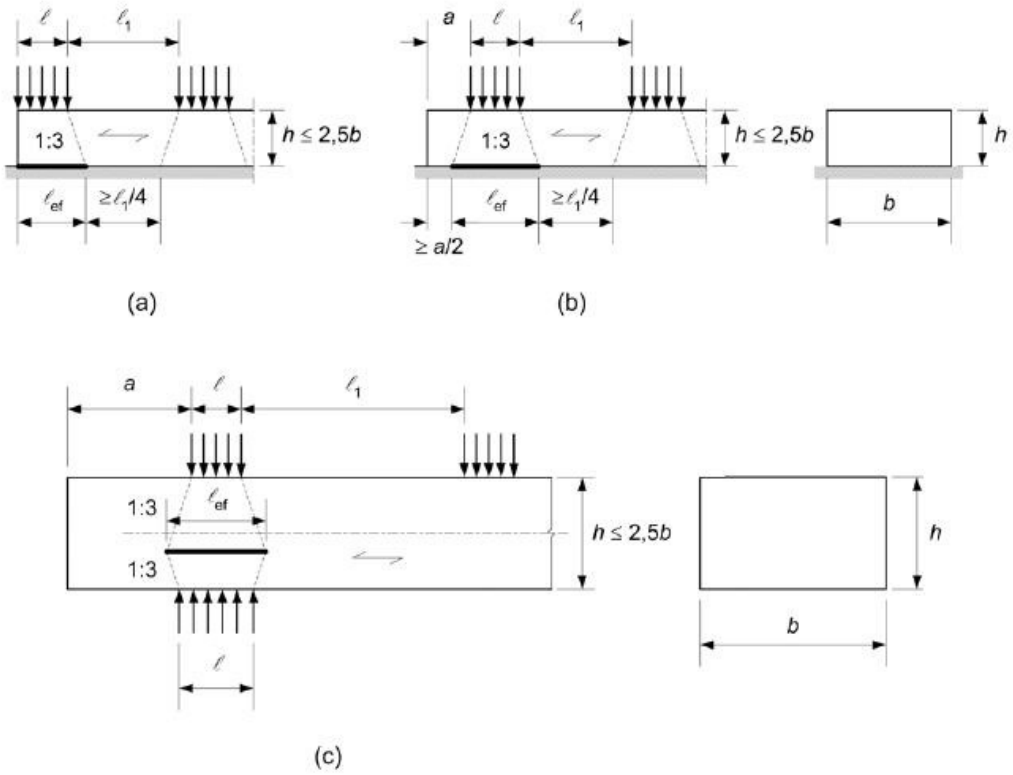


Figure 6.3 – Determination of effective lengths for a member with h/b ≤ 2.5, (a) and (b) continuous support, (c) discrete supports

(5) The effective length of distribution ℓ_{ef} should be determined from a stress dispersal line with a vertical inclination of 1:3 over the depth *h*, but curtailed by a distance of $a/2$ from any end, or a distance of $\ell_1/4$ from any adjacent compressed area, see Figure 6.3a and b. For the particular positions of forces below, the effective length is given by:

– for loads adjacent to the end of the member, see Figure 6.3a

$$\ell_{ef} = \ell + \frac{h}{3} \quad (6.7)$$

– when the distance from the edge of a concentrated load to the end of the member *a*,

$\geq \frac{2}{3}h$, see Figure 6.3b

$$\ell_{ef} = \ell + \frac{2h}{3} \quad (6.8)$$

where *h* is the depth of the member or 40 mm, whichever is the largest.

For members on discrete supports, provided that $a \geq h$ and $\ell_1 \geq 2h$, see Figure 6.3c, the effective length should be calculated as:

$$\ell_{ef} = 0.5\left(\ell + \ell_{ef} + \frac{2h}{3}\right) \quad (6.9)$$

where *h* is the depth of the member or 40 mm, whichever is the largest.

(6) For a member with a depth $h > 2.5b$ loaded with a concentrated compressive force on two opposite sides as shown in Figure 6.4b, or with a concentrated compressive force on one side and a continuous support on the other, see Figure 6.4a, the factor $k_{c,90}$ should be calculated according to expression (6.10), provided that the following conditions are fulfilled:

- the applied compressive force occurs over the full member width *b*;
- the contact length A is less than the greater of *h* or 100 mm:

$$k_{c,90} = \frac{\ell_{ef}}{\ell} \tag{6.10}$$

where:

ℓ is the contact length according to Figure 6.4;

ℓ_{ef} is the effective length of distribution according to Figure 6.4.

The effective length of distribution should not extend by more than l beyond either edge of the contact length.

(7) For members whose depth varies linearly over the support (e.g. bottom chords of trusses at the heel joint), the depth *h* should be taken as the member depth at the centreline of the support, and the effective length ℓ_{ef} should be taken as equal to the contact length ℓ.

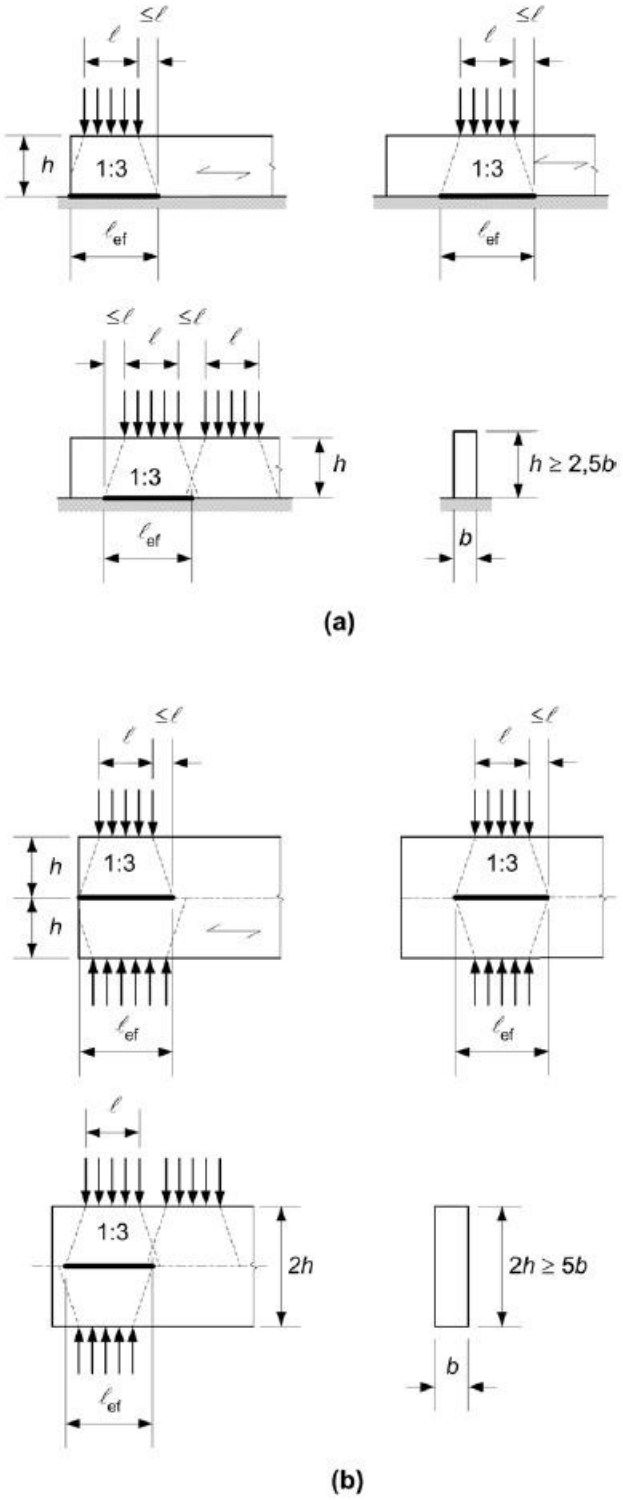


Figure 6.4 – Determination of effective lengths for a member with h/b> 2.5 on (a) a continuous support, (b) discrete supports

6.1.6 Bending

(1)P The following expressions shall be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \qquad (6.11)$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \qquad (6.12)$$

where:

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes as shown in Figure 6.1;

$f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths.

NOTE: The factor k_m makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in a cross-section.

(2) The value of the factor *k*m should be taken as follows:

For solid timber, glued laminated timber and LVL:

for rectangular sections: k_m = 0.7

for other cross-sections: k_m = 1.0

For other wood-based structural products, for all cross-sections: k_m = 1.0

(3)P A check shall also be made of the instability condition (see 6.3).

6.1.7 Shear

(1)P For shear with a stress component parallel to the grain, see Figure 6.5(a), as well as for shear with both stress components perpendicular to the grain, see Figure 6.5(b), the following expression shall be satisfied:

$$\tau_d = f_{v,d} \qquad (6.13)$$

where:

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength for the actual condition.

NOTE: The shear strength for rolling shear is approximately equal to twice the tension strength perpendicular to grain.

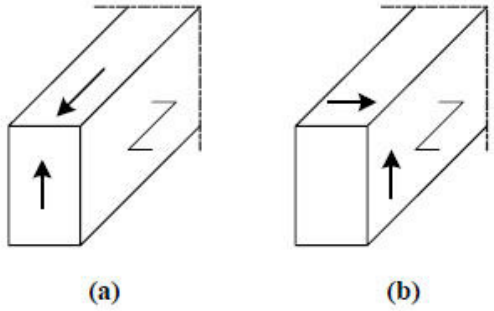


Figure 6.5 – (a) Member with a shear stress component parallel to the grain (b) Member with both stress components perpendicular to the grain (rolling shear)

(2) At supports, the contribution to the total shear force of a concentrated load *F* acting on the top side of the beam and within a distance h or h_{ef} from the edge of the support may be disregarded (see Figure 6.6). For beams with a notch at the support this reduction in the shear force applies only when the notch is on the opposite side to the support.

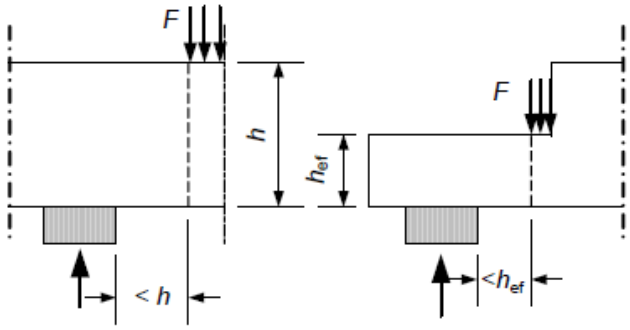


Figure 6.6 – Conditions at a support, for which the concentrated force *F* may be disregarded in the calculation of the shear force

6.1.8 Torsion

(1)P The following expression shall be satisfied:

$$\tau_{\text{tor},d} \le k_{\text{shape}} f_{v,d} \tag{6.14}$$

with

$$k_{\text{shape}} = \begin{cases} 1.2 & \text{for a circular cross section} \\ \min\left\{ 1+0.15\dfrac{h}{b} \\ 2.0 \right. & \text{for a rectangular cross section} \end{cases} \tag{6.15}$$

where:

$\tau_{tor,d}$ is the design torsional stress;

$f_{v,d}$ is the design shear strength;

k_{shape} is a factor depending on the shape of the cross-section;
h is the larger cross-sectional dimension;
b is the smaller cross-sectional dimension.

6.2 Design of cross-sections subjected to combined stresses

6.2.1 General

(1)P Clause 6.2 applies to straight solid timber, glued laminated timber or wood-based structural products of constant cross-section, whose grain runs essentially parallel to the length of the member. The member is assumed to be subjected to stresses from combined actions or to stresses acting in two or three of its principal axes.

6.2.2 Compression stresses at an angle to the grain

(1)P Interaction of compressive stresses in two or more directions shall be taken into account.

(2) The compressive stresses at an angle α to the grain, (see Figure 6.7), should satisfy the following expression:

$$\sigma_{c,\alpha,d} \le \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \qquad (6.16)$$

where:
$\sigma_{c,\alpha,d}$ is the compressive stress at an angle α to the grain;
$k_{c,90}$ is a factor given in 6..1.5 taking into account the effect of any of stresses perpendicular to the grain.

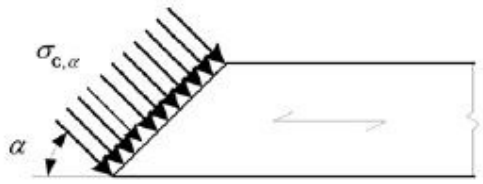


Figure 6.7 – Compressive stresses at an angle to the grain

6.2.3 Combined bending and axial tension

(1)P The following expressions shall be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,x,d}}{f_{m,x,d}} \le 1 \qquad (6.17)$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,x,d}}{f_{m,x,d}} \le 1 \quad (6.18)$$

(2) The values of k_m given in 6.1.6 apply.

6.2.4 Combined bending and axial compression

(1)P The following expressions shall be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \quad (6.19)$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \quad (6.20)$$

(2)P The values of k_m given in 6.1.6 apply.

NOTE: To check the instability condition, a method is given in 6.3.

6.3 Stability of members

6.3.1 General

(1)P The bending stresses due to initial curvature, eccentricities and induced deflection shall be taken into account, in addition to those due to any lateral load.

(2)P Column stability and lateral torsional stability shall be verified using the characteristic properties, e.g. $E_{0.05}$

(3) The stability of columns subjected to either compression or combined compression and bending should be verified in accordance with 6.3.2.

(4) The lateral torsional stability of beams subjected to either bending or combined bending and compression should be verified in accordance with 6.3.3.

6.3.2 Columns subjected to either compression or combined compression and bending

(1) The relative slenderness ratios should be taken as:

$$\lambda_{\mathrm{rel},y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \quad (6.21)$$

and

$$\lambda_{\mathrm{rel},z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \quad (6.22)$$

where:

λ_y and $\lambda_{rel,y}$ are slenderness ratios corresponding to bending about the *y*-axis (deflection in the *z*-direction);

λ_z and $\lambda_{rel,z}$ are slenderness ratios corresponding to bending about the *z*-axis (deflection in the *y*-direction);

$E_{0.05}$ is the fifth percentile value of the modulus of elasticity parallel to the grain.

(2) Where both $\lambda_{rel,z} \le 0.3$ and $\lambda_{rel,y} \le 0.3$ the stresses should satisfy the expressions (6.19) and (6.20) in 6.2.4.

(3) In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \qquad (6.23)$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \le 1 \qquad (6.24)$$

where the symbols are defined as follows: 1

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \qquad (6.25)$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \qquad (6.26)$$

$$k_y = 0.5\left(1 + \beta_c\left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right) \qquad (6.27)$$

$$k_z = 0.5\left(1 + \beta_c\left(\lambda_{rel,z} - 0.3\right) + \lambda_{rel,z}^2\right) \qquad (6.28)$$

where:

β_c is a factor for members within the straightness limits defined in Section 10:

$$\beta_c = \begin{cases} 0.2 & \text{for solid timber} \\ 0.1 & \text{for glued lainated timber and LVL} \end{cases} \qquad (6.29)$$

k_m as given in 6.1.6.

6.3.3 Beams subjected to either bending or combined bending and compression

(1) P Lateral torsional stability shall be verified both in the case where only a moment M_y

exists about the strong axis *y* and where a combination of moment M_y and compressive force N_c exists.

(2) The relative slenderness for bending should be taken as:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{6.30}$$

where $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values.

The critical bending stress should be taken as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi\sqrt{E_{0.05} I_z G_{0.05} I_{tor}}}{\ell_{ef} W_y} \tag{6.31}$$

where:

$E_{0.05}$ is the fifth percentile value of modulus of elasticity parallel to grain;

$G_{0.05}$ is the fifth percentile value of shear modulus parallel to grain;

I_z is the second moment of area about the weak axis *z.*

I_{tor} is the torsional moment of inertia;

ℓ_{ef} is the effective length of the beam, depending on the support conditions and the load configuration, according to Table 6.1:

W_y is the section modulus about the strong axis *y.*

For softwood with solid rectangular cross-section, $\sigma_{m,crit}$ should be taken as:

$$\sigma_{m,crit} = \frac{0.78 b^2}{h \ell_{ef}} E_{0.05} \tag{6.32}$$

Where:

b is the width of the beam;

h is the depth of the beam.

(3) In the case where only a moment M_y exists about the strong axis *y*, the stresses should satisfy the following expression:

$$\sigma_{m,d} \le k_{crit} f_{m,d} \tag{6.33}$$

Where:

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

Table 6.1 – Effective length as a ratio of the span

Beam type	Loading type	ℓ_{ef} / ℓ^{a}
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1.0 0.9 0.8
Cantilever	Uniformly distributed load Concentrated force at the free end	0.5 0.8
[a] The ratio between the effective length ℓ_{ef} and the span *l* is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by $2h$ and may be decreased by 0.5*h* for a load at the tension edge of the beam.		

(4) For beams with an initial lateral deviation from straightness within the limits defined in Section 10, k_{crit} may be determined from expression (6.34)

$$k_{crit} = \begin{cases} 1 & \text{for} \quad \lambda_{rel,m} \le 0.75 \\ 1.56 - 0.75\lambda_{rel,m} & \text{for} \quad 0.75 < \lambda_{rel,m} \le 1.4 \\ \dfrac{1}{\lambda_{rel,m}^2} & \text{for} \quad 1.4 < \lambda_{rel,m} \end{cases} \tag{6.34}$$

(5) The factor k_{crit} may be taken as 1.0 for a beam where lateral displacement of its compressive edge is prevented throughout its length and where torsional rotation is prevented at its supports.

(6) In the case where a combination of moment M_y about the strong axis *y* and compressive force N_c exists, the stresses should satisfy the following expression:

$$\left(\frac{\sigma_{m,d}}{k_{crit} f_{m,d}}\right)^2 + \frac{\sigma_{c,d}}{k_{c,z} f_{c,0,d}} \le 1 \tag{6.35}$$

where:

$\sigma_{m,d}$ is the design bending stress;

$\sigma_{c,d}$ is the design compressive stress;

$f_{c,0,d}$ is the design compressive strength parallel to grain;

$k_{c,z}$ is given by expression (6.26).

6.4 Design of cross-sections in members with varying cross-section or curved shape

6.4.1 General

(1)P The effects of combined axial force and bending moment shall be taken into account.

(2) The relevant parts of 6.2 and 6.3 should be verified.

(3) The stress at a cross-section from an axial force may be calculated from

$$\sigma_N = \frac{N}{A} \tag{6.36}$$

where:

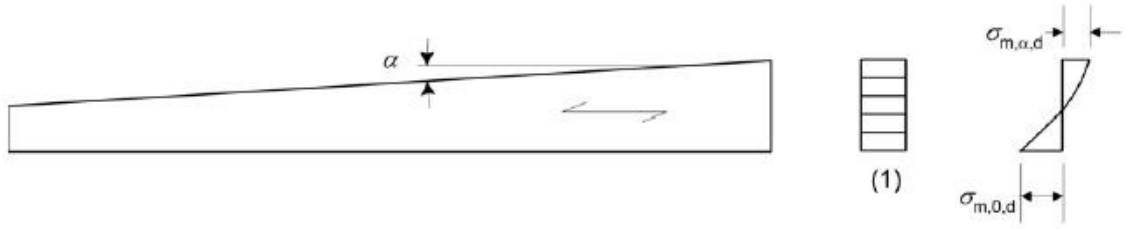
σ_N is the axial stress;

N is the axial force;

A is the area of the cross-section.

6.4.2 Single tapered beams

(1)P The influence of the taper on the bending stresses parallel to the surface shall be taken into account.



Key:
(1) cross-section

Figure 6.8 – Single tapered beam

(2) The design bending stresses, $\sigma_{m,\alpha,d}$ and $\sigma_{m,0,d}$ (see Figure 6.8) may be taken as:

$$\sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{6M_d}{bh^2} \tag{6.37}$$

At the outermost fibre of the tapered edge, the stresses should satisfy the following expression:

$$\sigma_{m,\alpha,d} \le k_{m,\alpha} f_{m,d} \tag{6.38}$$

where:

$\sigma_{m,\alpha,d}$ is the design bending stress at an angle to grain;

$f_{m,d}$ is the design bending strength;

$k_{m,\alpha}$ should be calculated as:

For tensile stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1 + \left(\frac{f_{m,d}}{0.75 f_{v,d}} \tan\alpha\right)^2 + \left(\frac{f_{m,d}}{f_{t,90,d}} \tan^2\alpha\right)^2}} \tag{6.39}$$

For compressive stresses parallel to the tapered edge:

$$k_{m,\alpha} = \frac{1}{\sqrt{1+\left(\frac{f_{m,d}}{1.5 f_{v,d}}\tan\alpha\right)^2+\left(\frac{f_{m,d}}{f_{c,90,d}}\tan^2\alpha\right)^2}} \quad (6.40)$$

6.4.3 Double tapered, curved and pitched cambered beams

(1) This clause applies only to glued laminated timber and LVL.

(2) The requirements of 6.4.2 apply to the parts of the beam which have a single taper.

(3) In the apex zone (see Figure 6.9), the bending stresses should satisfy the following expression:

$$\sigma_{m,d} \le k_r f_{m,d} \quad (6.41)$$

where k_r takes into account the strength reduction due to bending of the laminates during production.

NOTE: In curved and pitched cambered beams the apex zone extends over the curved part of the beam

(4) The apex bending stress should be calculated as follows:

$$\sigma_{m,d} = k_\ell \frac{6M_{ap,d}}{bh_{ap}^2} \quad (6.42)$$

With:

$$k_\ell = k_1 + k_2\left(\frac{h_{ap}}{r}\right) + k_3\left(\frac{h_{ap}}{r}\right)^2 + k_4\left(\frac{h_{ap}}{r}\right)^3 \quad (6.43)$$

$$k_1 = 1 + 1.4\tan\alpha_{ap} + 5.4\tan^2\alpha_{ap} \quad (6.44)$$

$$k_2 = 0.35 - 8\tan\alpha_{ap} \quad (6.45)$$

$$k_3 = 0.6 + 8.3\tan\alpha_{ap} - 7.8\tan^2\alpha_{ap} \quad (6.46)$$

$$k_4 = 6\tan^2\alpha_{ap} \quad (6.47)$$

$$r = r_{in} + 0.5 h_{ap} \quad (6.48)$$

where:

$M_{ap,d}$ is the design moment at the apex;

h_{ap} is the depth of the beam at the apex, see Figure 6.9;

b is the width of the beam;

r_{in} is the inner radius, see Figure 6.9;

α_{ap} is the angle of the taper in the middle of the apex zone, see Figure 6.9.

(5) For double tapered beams k_r = 1.0. For curved and pitched cambered beams k_r should be taken as:

$$k_r = \begin{cases} 1 & for \frac{r_{in}}{t} \geq 240 \\ 0.76 + 0.001\frac{r_{in}}{t} & for \frac{r_{in}}{t} < 240 \end{cases} \quad (6.49)$$

where

r_{in} is the inner radius, see Figure 6.9;

t is the lamination thickness.

(6) In the apex zone the greatest tensile stress perpendicular to the grain, $\sigma_{t,90,d}$, should satisfy the following expression:

$$\sigma_{t,90,d} \leq k_{dis}\, k_{vol}\, f_{t,90,d} \quad (6.50)$$

With

$$k_{vol} = \begin{cases} 1.0 & \text{for solid timber} \\ \left(\frac{V_0}{V}\right)^2 & \text{for glued laminated timber and LVL with all veneers parallel to the beam axis} \end{cases} \quad (6.51)$$

$$k_{dis} = \begin{cases} 1.4 & \text{for double tapered and curved beams} \\ 1.7 & \text{for pitched cambered beam} \end{cases} \quad (6.52)$$

where:

k_{dis} is a factor which takes into account the effect of the stress distribution in the apex zone;

k_{vol} is a volume factor;

$f_{t,90,d}$ is the design tensile strength perpendicular to the grain;

V_0 is the reference volume of 0.01m³;

V is the stressed volume of the apex zone, in m³, (see Figure 6.9) and should not be taken greater than $2V_b/3$, where V_b is the total volume of the beam.

(7) For combined tension perpendicular to grain and shear the following expression shall be satisfied:

$$\frac{\tau_d}{f_{v,d}} + \frac{\sigma_{t,90,d}}{k_{dis} k_{vol} f_{t,90,d}} \leq 1 \quad (6.53)$$

where:

τ_d is the design shear stress;

$f_{v,d}$ is the design shear strength;

$\sigma_{t,90,d}$ is the design tensile stress perpendicular to grain;

k_{dis} and k_{vol} are given in (6)

(8) The greatest tensile stress perpendicular to the grain due to the bending moment should be calculated as follows:

$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{b\,h_{ap}^2} \tag{6.54}$$

or, as an alternative to expression (6.54), as

$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{b\,h_{ap}^2} - 0.6\frac{p_d}{b} \tag{6.55}$$

where:

p_d is the uniformly distributed load acting on the top of the beam over the apex area;

b is the width of the beam;

$M_{\partial p,d}$ is the design moment at apex resulting in tensile stresses parallel to the inner curved edge;

with:

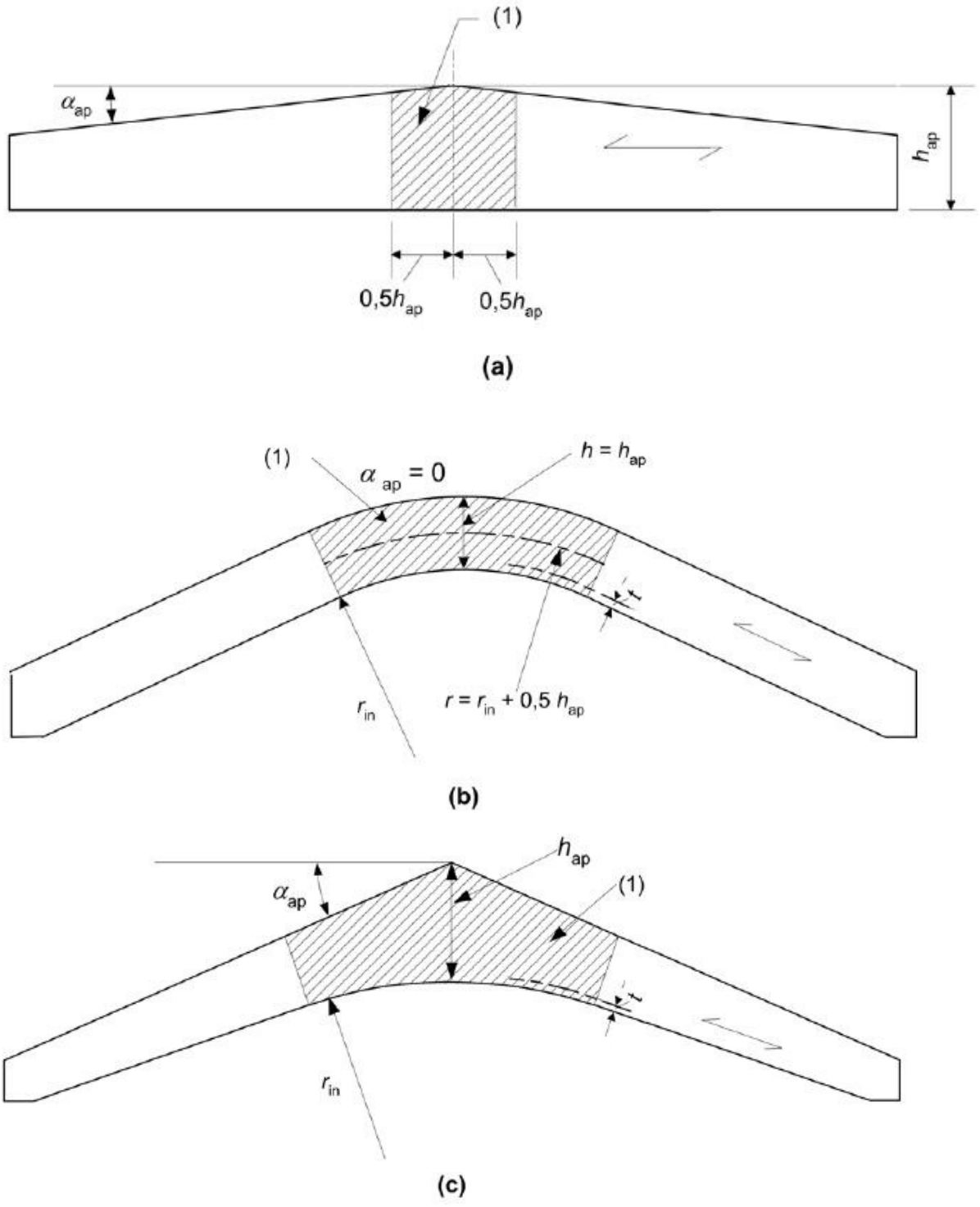
$$k_p = k_5 + k_6\left(\frac{h_{ap}}{r}\right) + k_7\left(\frac{h_{ap}}{r}\right)^2 \tag{6.56}$$

$$k_5 = 0.2\tan\alpha_{ap} \tag{6.57}$$

$$k_6 = 0.25 - 1.5\tan\alpha_{ap} + 2.6\tan^2\alpha_{ap} \tag{6.58}$$

$$k_7 = 2.1\tan\alpha_{ap} - 4\tan^2\alpha_{ap} \tag{6.59}$$

Note: The recommended expression is (6.54).



Key:
(1) Apex Zone

NOTE: In curved and pitched cambered beams the apex zone extends over the curved parts of the beam.

Figure 6.9 – Double tapered (a), curved (b) and pitched cambered (c) beams with the fibre direction parallel to the lower edge of the beam

6.5 Notched members

6.5.1 General

(1)P The effects of stress concentrations at the notch shall be taken into account in the strength verification of members.

(2) The effect of stress concentrations may be disregarded in the following cases:

- tension or compression parallel to the grain;
- bending with tensile stresses at the notch if the taper is not steeper than 1:i = 1:10, that is $i \geq 10$, see Figure 6.10a;
- bending with compressive stresses at the notch, see Figure 6.10b.

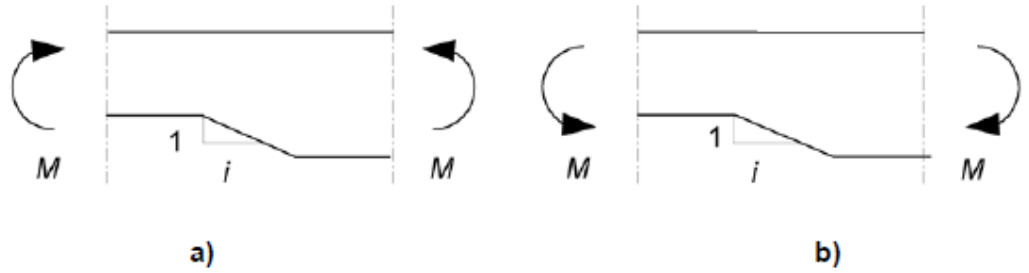


Figure 6.10 – Bending at a notch: a) with tensile stresses at the notch, b) with compressive stresses at the notch

6.5.2 Beams with a notch at the support

(1) For beams with rectangular cross-sections and where grain runs essentially parallel to the length of the member, the shear stresses at the notched support should be calculated using the effective (reduced) depth h_{ef} (see Figure 6.11).

(2) It should be verified that

$$\tau_d = \frac{1.5V}{bh_{ef}} \leq k_v f_{v,d} \tag{6.60}$$

where k_v is a reduction factor defined as follows:

- For beams notched at the opposite side to the support (see Figure 6.11b)

$$k_v = 1.0 \tag{6.61}$$

- For beams notched on the same side as the support (see Figure 6.11a)

$$k_v = \min\left\{ \begin{array}{c} 1 \\ \dfrac{k_n\left(1+\dfrac{1.1i^{1.5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)}+0.8\dfrac{x}{h}\sqrt{\dfrac{1}{\alpha}-\alpha^2}\right)} \end{array} \right. \tag{6.62}$$

Where:

i is the notch inclination (see Figure 6.11a); h is the beam depth in mm;

x is the distance from line of action of the support reaction to the corner of the notch;

$$\alpha = \frac{h_{ef}}{h}$$

$$k_n = \begin{cases} 4.5 & \text{for LVL} \\ 5 & \text{for solid timber} \\ 6.5 & \text{for glued laminated timber} \end{cases} \quad (6.63)$$

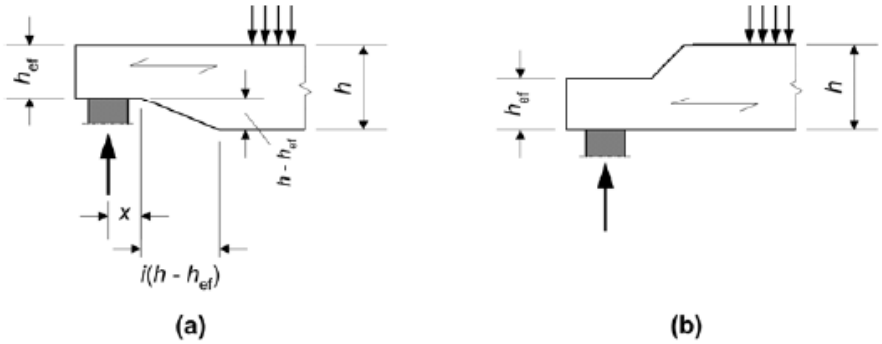


Figure 6.11 – End-notched beams

6.6 System strength

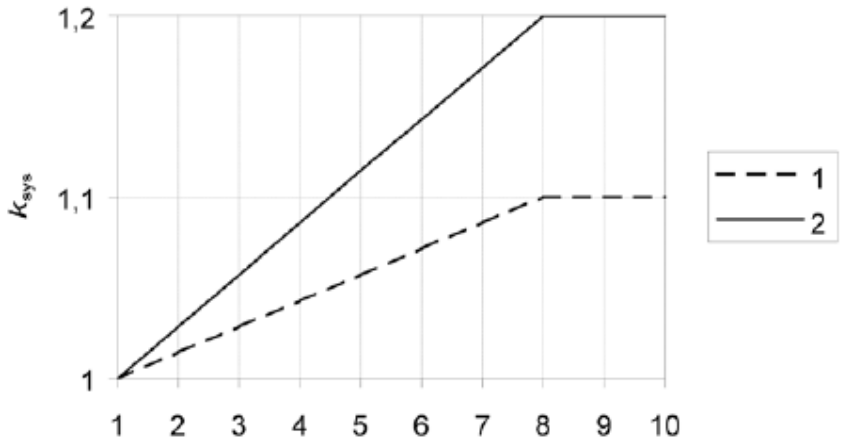
(1) When several equally spaced similar members, components or assemblies are laterally connected by a continuous load distribution system, the member strength properties may be multiplied by a system strength factor k_{sys}.

(2) Provided the continuous load-distribution system is capable of transferring the loads from one member to the neighbouring members, the factor k_{sys} should be 1.1.

(3) The strength verification of the load distribution system should be carried out assuming the loads are of short-term duration.

NOTE: For roof trusses with a maximum centre to centre distance of 1.2m it may be assumed that tiling battens, purlins or panels can transfer the load to the neighbouring trusses provided that these load-distribution members are continuous over at least two spans, and any joints are staggered.

(4) For laminated timber decks or floors the values of k_{sys} given in Figure 6.12 should be used.



Number of loaded laminations

Key:

1 Nailed or screwed laminations

2 Laminations pre-stressed or glued together

Figure 6.12 – System strength factor k_{sys} for laminated deck plates of solid timber or glued laminated members

Section 7 Serviceability limit states

7.1 Joint slip

(1) For joints made with dowel-type fasteners the slip modulus K_{ser} per shear plane per fastener under service load should be taken from Table 7.1 with ρ_m in kg/m³ and d or d_c in mm. For the definition of d_c, see ES EN 13271.

Table 7.1 – Values of K_{ser} for fasteners and connectors in N/mm in timber-to-timber and wood-based panel-to-timber connections

Fastener type	K_{ser}
Dowels Bolts with or without clearance a Screws Nails (with pre-drilling)	$\rho_m^{1.5} d / 23$
Nails (without pre-drilling)	$\rho_m^{1.5} d^{0.8} / 30$
Staples	$\rho_m^{1.5} d^{0.8} / 80$
Split-ring connectors type A according to ES EN 912 Shear-plate connectors type B according to ES EN 912	$\rho_m d_c / 2$
Toothed-plate connectors: – Connectors types C1 to C9 according to ES EN 912 – Connectors type C10 and C11 according to ES EN 912	$1.5\rho_m d_c / 4$ $\rho_m d_c / 2$
[a]The clearance should be added separately to the deformation	

(2) If the mean densities $\rho_{m,1}$ and $\rho_{m,2}$ of the two jointed wood-based members are different then ρ_m in the above expressions should be taken as

$$\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}} \tag{7.1}$$

(3) For steel-to-timber or concrete-to-timber connections, K_{ser} should be based on ρ_m for the timber member and may be multiplied by 2.0.

7.2 Limiting values for deflections of beams

(1) The components of deflection resulting from a combination of actions (see 2.2.3(5)) are shown in Figure 7.1, where the symbols are defined as follows, see 2.2.3:

- w_c is the precamber (if applied);
- w_{inst} is the instantaneous deflection;
- w_{creep} is the creep deflection;
- w_{fin} is the final deflection;
- $w_{net,fin}$ is the net final deflection.

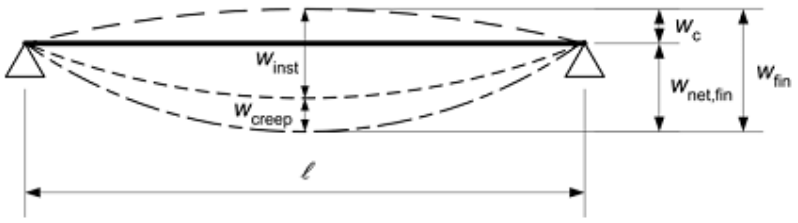


Figure 7.1 – Components of deflection

(2) The net deflection below a straight line between the supports, $w_{net,fin}$, should be taken as:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c = w_{fin} - w_c \tag{7.2}$$

NOTE: The recommended range of limiting values of deflections for beams with span l is given in Table 7.2 depending upon the level of deformation deemed to be acceptable.

Table 7.2 – Examples of limiting values for deflections of beams

	w_{inst}	$w_{inst,fin}$	w_{fin}
Beam on two supports	ℓ/300 to ℓ/500	ℓ/250 to ℓ/350	ℓ/150 to ℓ/300
Cantilevering beams	ℓ/150 to ℓ/250	ℓ/125 to ℓ/175	ℓ/75 to ℓ/150

7.3 Vibrations

7.3.1 General

(1)P It shall be ensured that the actions which can be reasonably anticipated on a member, component or structure, do not cause vibrations that can impair the function of the structure or cause unacceptable discomfort to the users.

(2) The vibration level should be estimated by measurements or by calculation taking into account the expected stiffness of the member, component or structure and the modal damping ratio.

(3) For floors, unless other values are proven to be more appropriate, a modal damping ratio of $\zeta = 0.01$ (i.e 1%) should be assumed.

7.3.2 Vibrations from machinery

(1)P Vibrations caused by rotating machinery and other operational equipment shall be limited for the unfavourable combinations of permanent load and variable loads that can be expected.

(2) For floors, acceptable levels for continuous vibration should be taken from figure 5a in Appendix A of ES ISO 2631-2 with a multiplying factor of 1,0.

7.3.3 Residential floors

(1) For residential floors with a fundamental frequency less than 8Hz $\left(f_1 \le 8Hz\right)$ a special investigation should be made.

(2) For residential floors with a fundamental frequency greater than 8Hz $\left(f_1 > 8Hz\right)$ the following requirements should be satisfied:

$$\frac{w}{F} \le a \quad \text{mm/kN} \tag{7.3}$$

and

$$v \le b^{(f_1\zeta - 1)} \quad m/\left(Ns^2\right) \tag{7.4}$$

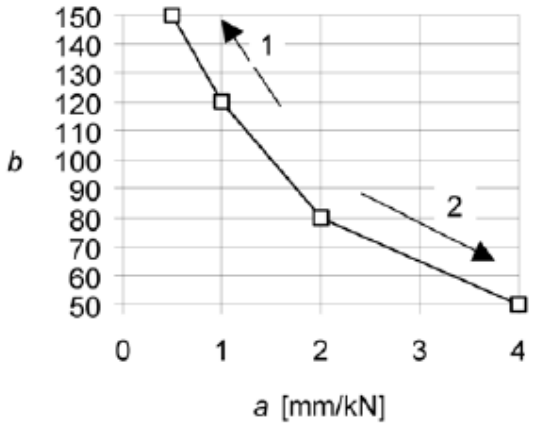
where:

w is the maximum instantaneous vertical deflection caused by a vertical concentrated static force F applied at any point on the floor, taking account of load distribution;

v is the unit impulse velocity response, i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response. Components above 40 Hz may be disregarded;

ζ is the modal damping ratio.

NOTE: The recommended range of limiting values of a and b and the recommended relationship between a and b is given in Figure 7.2.



Key:

1 Better performance
2 Poorer performance

Figure 7.2- Recommended range of and relationship between *a* and *b*

(3) The calculations in 7.3.3(2) should be made under the assumption that the floor is unloaded, i.e., only the mass corresponding to the self-weight of the floor and other permanent actions.

(4) For a rectangular floor with overall dimensions $\ell \times b$, simply supported along all four edges and with timber beams having a span ℓ, the fundamental frequency f_1 may approximately be calculated as

$$f_1 = \frac{\pi}{2\ell^2}\sqrt{\frac{(EI)_\ell}{m}} \qquad (7.5)$$

Where:

m is the mass per unit area in kg/m²;

ℓ is the floor span, in m;

$(EI)_\ell$ is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction, in Nm²/m.

(5) For a rectangular floor with overall dimensions $b \times \ell$, simply supported along all four edges, the value *v* may, as an approximation, be taken as:

$$v = \frac{4(0.4 + 0.6n_{40})}{mb\ell + 200} \qquad (7.6)$$

where:

v is the unit impulse velocity response, in m/(Ns^2);

n_{40} is the number of first-order modes with natural frequencies up to 40 Hz;

b is the floor width, in m;

m is the mass, in kg/m;

ℓ is the floor span, in m.

The value of n_{40} may be calculated from:

$$n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{b}{\ell} \right)^4 \frac{(EI)_\ell}{(EI)_b} \right\}^{0.25} \tag{7.7}$$

where $(EI)_b$ is the equivalent plate bending stiffness, in Nm^2/m, of the floor about an axis parallel to the beams, where $(EI)_b < (EI)_\ell$.

Section 8 Connections with metal fasteners

8.1 General

8.1.1 Fastener requirements

(1)P Unless rules are given in this section, the characteristic load-carrying capacity, and the stiffness of the connections shall be determined from tests according to ES EN 1075, ES EN 1380,ES EN 1381, ES EN 26891 and ES EN 28970. If the relevant standards describe tension and compression tests, the tests for the determination of the characteristic load-carrying capacity shall be performed in tension.

8.1.2 Multiple fastener connections

(1)P The arrangement and sizes of the fasteners in a connection, and the fastener spacings, edge and end distances shall be chosen so that the expected strength and stiffness can be obtained.

(2)P It shall be taken into account that the load-carrying capacity of a multiple fastener connection, consisting of fasteners of the same type and dimension, may be lower than the summation of the individual load-carrying capacities for each fastener.

(3) When a connection comprises different types of fasteners, or when the stiffness of the connections in respective shear planes of a multiple shear plane connection is different, their compatibility should be verified.

(4) For one row of fasteners parallel to the grain direction, the effective characteristic load-carrying capacity parallel to the row, $F_{v,ef,Rk}$, should be taken as:

$$F_{v,ef,Rk} = n_{ef} F_{v,Rk} \tag{8.1}$$

where:

$F_{v,ef,Rk}$ is the effective characteristic load-carrying capacity of one row of fasteners parallel to the grain;

n_{ef} is the effective number of fasteners in line parallel to the grain;

$F_{v,Rk}$ is the characteristic load-carrying capacity of each fastener parallel to the grain.

NOTE: Values of n_{ef} for rows parallel to grain are given in 8.3.1.1(8) and 8.5.1.1(5).

(5) For a force acting at an angle to the direction of the row, it should be verified that the force component parallel to the row is less than or equal to the load-carrying capacity calculated according to expression (8.1).

8.1.3 Multiple shear plane connections

(1) In multiple shear plane connections the resistance of each shear plane should be determined by assuming that each shear plane is part of a series of three-member connections.

(2) To be able to combine the resistance from individual shear planes in a multiple shear plane connection, the governing failure mode of the fasteners in the respective shear planes should be compatible with each other and should not consist of a combination of failure modes (a), (b), (g) and (h) from Figure 8.2 or modes (e), (f) and (j/l) from Figure 8.3 with the other failure modes.

8.1.4 Connection forces at an angle to the grain

(1)P When a force in a connection acts at an angle to the grain, (see Figure 8.1), the possibility of splitting caused by the tension force component, $F_{Ed}\sin\alpha$, perpendicular to the grain, shall be taken into account.

(2)P To take account of the possibility of splitting caused by the tension force component, $F_{Ed}\sin\alpha$, perpendicular to the grain, the following shall be satisfied:

$$F_{v,Ed} \le F_{90,Rd} \tag{8.2}$$

with

$$F_{v,Ed} = \max\begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases} \tag{8.3}$$

where:

$F_{90,Rd}$ is the design splitting capacity, calculated from the characteristic splitting capacity $F_{90,Rd}$ according to 2.4.3;

$F_{v,Ed,1}$, $F_{v,Ed,2}$ are the design shear forces on either side of the connection. (see Figure 8.1).

(3) For softwoods, the characteristic splitting capacity for the arrangement shown in Figure 8.1 should be taken as:

$$F_{90,Rk} = 14bw\sqrt{\frac{h_e}{\left(1-\frac{h_e}{h}\right)}} \tag{8.4}$$

where

$$w = \begin{cases} \max\left\{\left(\frac{w_{pl}}{100}\right)^{0.35} \\ 1 \end{cases} & \text{for punched metal plate fasteners} \\ 1 & \text{for all other fasteners} \end{cases} \quad (8.5)$$

and

$F_{90,Rk}$ is the characteristic splitting capacity, in N;

w is a modification factor;

h_e is the loaded edge distance to the centre of the most distant fastener or to the edge of the punched metal plate fastener, in mm;

h is the timber member height, in mm;

b is the member thickness, in mm;

w_{pl} is the width of the punched metal plate fastener parallel to the grain, in mm.

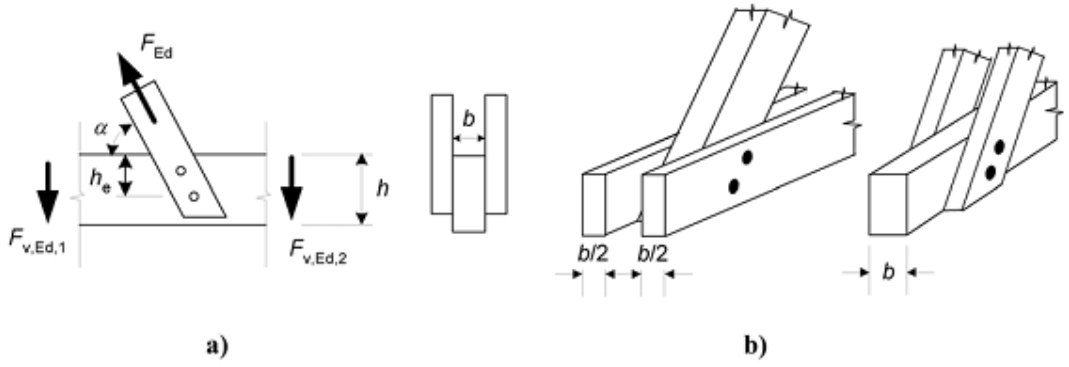


Figure 8.1 – Inclined force transmitted by a connection

8.1.5 Alternating connection forces

(1)P The characteristic load-carrying capacity of a connection shall be reduced if the connection is subject to alternating internal forces due to long-term or medium-term actions.

(2)The effect on connection strength of long-term or medium-term actions alternating between a tensile design force $F_{t,Ed}$ and a compressive design force $F_{c,Ed}$ should be taken into account by designing the connection for $\left(F_{t,Ed} + 0.5F_{c,Ed}\right)$ and $\left(F_{c,Ed} + 0.5F_{t,Ed}\right)$.

8.2 Lateral load-carrying capacity of metal dowel-type fasteners

8.2.1 General

(1)P For the determination of the characteristic load-carrying capacity of connections with metal dowel-type fasteners the contributions of the yield strength, the embedment strength, and the withdrawal strength of the fastener shall be considered.

8.2.2 Timber-to-timber and panel-to-timber connections

(1) The characteristic load-carrying capacity for nails, staples, bolts, dowels and screws per shear plane per fastener, should be taken as the minimum value found from the following expressions:

– For fasteners in single shear

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & (a) \\ f_{h,2,k} t_2 d & (b) \\ \dfrac{f_{h,1,k} t_1 d}{1+\beta} \left[\sqrt{\beta + 2\beta^2 \left[1 + \dfrac{t_2}{t_1} + \left(\dfrac{t_2}{t_1} \right)^2 \right] + \beta^3 \left(\dfrac{t_2}{t_1} \right)^2} - \beta \left(1 + \dfrac{t_2}{t_1} \right) \right] + \dfrac{F_{\text{ax,Rk}}}{4} & (c) \\ 1.05 \dfrac{f_{h,1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta) M_{y,Rk}}{f_{h,1,k} d \quad t_1^2}} - \beta \right] + \dfrac{F_{\text{ax,Rk}}}{4} & (d) \\ 1.05 \dfrac{f_{h,1,k} t_2 d}{1+2\beta} \left[\sqrt{2\beta^2 (1+\beta) + \dfrac{4\beta(1+2\beta) M_{y,Rk}}{f_{h,1,k} d \quad t_2^2}} - \beta \right] + \dfrac{F_{\text{ax,Rk}}}{4} & (e) \\ 1.15 \sqrt{\dfrac{2\beta}{1+\beta}} \sqrt{2 M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & (f) \end{cases} \quad (8.6)$$

– For fasteners in double shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & (g) \\ 0.5 f_{h,2,k} t_2 d & (h) \\ 1.05 \dfrac{f_{h,1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \dfrac{4\beta(2+\beta) M_{y,Rk}}{f_{h,1,k} d \quad t_1^2}} - \beta \right] + \dfrac{F_{\text{ax,Rk}}}{4} & (j) \\ 1.15 \sqrt{\dfrac{2\beta}{1+\beta}} \sqrt{2 M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & (k) \end{cases} \quad (8.7)$$

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \quad (8.8)$$

where:

$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;

t_i is the timber or board thickness or penetration depth, with i either 1 or 2, see also 8.3 to 8.7 ;

$f_{h,i,k}$ is the characteristic embedment strength in timber member i;

d is the fastener diameter;

$M_{y,Rk}$ is the characteristic fastener yield moment;

β is the ratio between the embedment strength of the members;

$F_{ax,Rk}$ is the characteristic axial withdrawal capacity of the fastener, see (2).

NOTE: Plasticity of joints can be assured when relatively slender fasteners are used. In that case, failure modes (f) and (k) are governing.

(2) In the expressions (8.6) and (8.7), the first term on the right hand side is the load-carrying capacity according to the Johansen yield theory, whilst the second term $F_{ax,Rk}/4$ is the contribution from the rope effect. The contribution to the load-carrying capacity due to the rope effect should be limited to following percentages of the Johansen part:

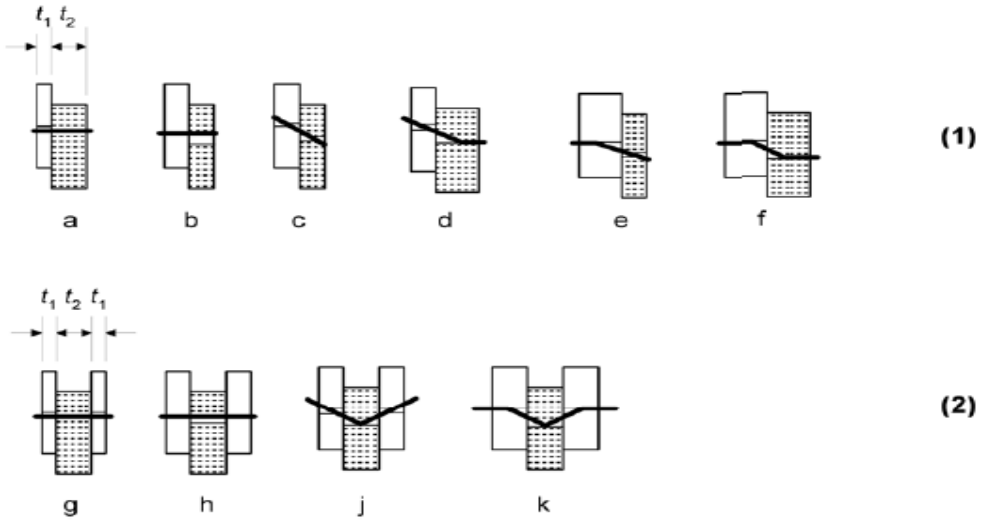
- Round nails 15 %
- Square nails 25 %
- Other nails 50 %
- Screws 100%
- Bolts 25 %
- Dowels 0 %

If $F_{ax,Rk}$ is not known then the contribution from the rope effect should be taken as zero.

For single shear fasteners the characteristic withdrawal capacity, $F_{ax,Rk}$, is taken as the lower of the capacities in the two members. The different modes of failure are illustrated in Figure 8.2. For the withdrawal capacity, $F_{ax,Rk}$, of bolts the resistance provided by the washers may be taken into account, see 8.5.2(2).

(3) If no design rules are given below, the characteristic embedment strength $F_{h,k}$ should be determined according to ES EN 383 and ES EN 14358.

(4) If no design rules are given below, the characteristic yield moment $M_{y,Rk}$ should be determined according to ES EN 409 and ES EN 14358.



Key:

(1) Single shear
(2) Double shear

NOTE: The letters correspond to the references of the expressions (8.6) and (8.7)

Figure 8.2 – Failure modes for timber and panel connections.

8.2.3 Steel-to-timber connections

(1) The characteristic load-carrying capacity of a steel-to-timber connection depends on the thickness of the steel plates. Steel plates of thickness less than or equal to 0.5*d* are classified as thin plates and steel plates of thickness greater than or equal to *d* with the tolerance on hole diameters being less than 0.1*d* are classified as thick plates. The characteristic load-carrying capacity of connections with steel plate thickness between a thin and a thick plate should be calculated by linear interpolation between the limiting thin and thick plate values.

(2)P The strength of the steel plate shall be checked.

(3) The characteristic load-carrying capacity for nails, bolts, dowels and screws per shear plane per fastener should be taken as the minimum value found from the following expressions:

– For a thin steel plate in single shear:

$$F_{v,Rk} = \min \begin{cases} 0.4 f_{h,k} t_1 d & \text{(a)} \\ 1.15\sqrt{2M_{y,Rk} f_{h,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & \text{(b)} \end{cases} \tag{8.9}$$

– For a thick steel plate in single shear:

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1 \right] + \dfrac{F_{\text{ax,Rk}}}{4} & \text{(c)} \\ 2.3\sqrt{M_{y,Rk} f_{h,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & \text{(d)} \\ f_{h,k} t_1 d & \text{(e)} \end{cases} \tag{8.10}$$

– For a steel plate of any thickness as the central member of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & (f) \\ f_{h,1,k} t_1 d \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,1,k} d t_1^2}} - 1 \right] + \dfrac{F_{\text{ax,Rk}}}{4} & (g) \\ 2.3\sqrt{M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & (h) \end{cases} \tag{8.11}$$

– For thin steel plates as the outer members of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} 0.5 f_{h,2,k} t_2 d & (j) \\ 1.15\sqrt{2M_{y,Rk} f_{h,2,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & (k) \end{cases} \quad (8.12)$$

– For thick steel plates as the outer members of a double shear connection

$$F_{v,Rk} = \min \begin{cases} 0.5 f_{h,2,k} t_2 d & (l) \\ 2.3\sqrt{M_{y,Rk} f_{h,2,k} d} + \dfrac{F_{\text{ax,Rk}}}{4} & (m) \end{cases} \quad (8.13)$$

where:

$F_{v,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener;

$f_{h,k}$ is the characteristic embedment strength in the timber member;

t_1 is the smaller of the thickness of the timber side member or the penetration depth;

t_2 is the thickness of the timber middle member;

d is the fastener diameter;

$M_{y,Rk}$ is the characteristic fastener yield moment;

$F_{\text{ax,Rk}}$ is the characteristic withdrawal capacity of the fastener.

NOTE 1: The different failure modes are illustrated in Figure 8.3

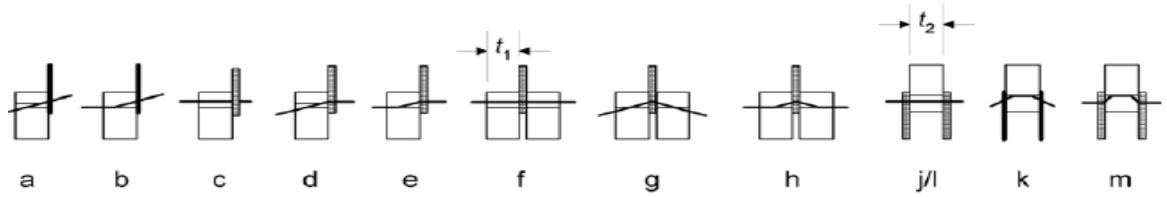


Figure 8.3 – Failure modes for steel-to-timber connection

(4) For the limitation of the rope effect $F_{\text{ax,Rk}}$ 8.2.2(2) applies.

(5)P It shall be taken into account that the load-carrying capacity of steel-to-timber connections with a loaded end may be reduced by failure along the perimeter of the fastener group.

NOTE: A method of determining the strength of the fastener group is given in Annex A (informative).

8.3 Nailed connections

8.3.1 Laterally loaded nails

8.3.1.1 General

(1) The symbols for the thicknesses in single and double shear connections (see Figure 8.4) are defined as follows:

t_1 is:

the headside thickness in a single shear connection;

the minimum of the head side timber thickness and the pointside penetration in a double shear connection;

t_2 is:

the pointside penetration in a single shear connection;

the central member thickness in a double shear connection.

(2) Timber should be pre-drilled when:

– the characteristic density of the timber is greater than 500 kg/m³;
– the diameter *d* of the nail exceeds 8 mm.

(3) For square and grooved nails, the nail diameter *d* should be taken as the side dimension.

(4) For smooth nails produced from wire with a minimum tensile strength of 600 N/mm^2, the following characteristic values for yield moment should be used:

$$M_{y,Rk} = \begin{cases} 0.3 f_u d^{2.6} & \text{for round nails} \\ 0.45 f_u d^{2.6} & \text{for square nails} \end{cases} \qquad (8.14)$$

where:

$M_{y,Rk}$ is the characteristic value for the yield moment, in Nmm;

d is the nail diameter as defined in ES EN 14592, in mm;

f_u is the tensile strength of the wire, in N/mm^2.

(5) For nails with diameters up to 8 mm, the following characteristic embedment strengths in timber and LVL apply:

– without predrilled holes

$$f_{h,k} = 0.082 \rho_k d^{-0.3} \quad \text{N/mm}^2 \qquad (8.15)$$

– with predrilled holes

$$f_{h,k} = 0.082 (1 - 0.1d) \rho_k \quad \text{N/mm}^2 \qquad (8.16)$$

where:

ρ_k is the characteristic timber density, in kg/m³;

d is the nail diameter, in mm.

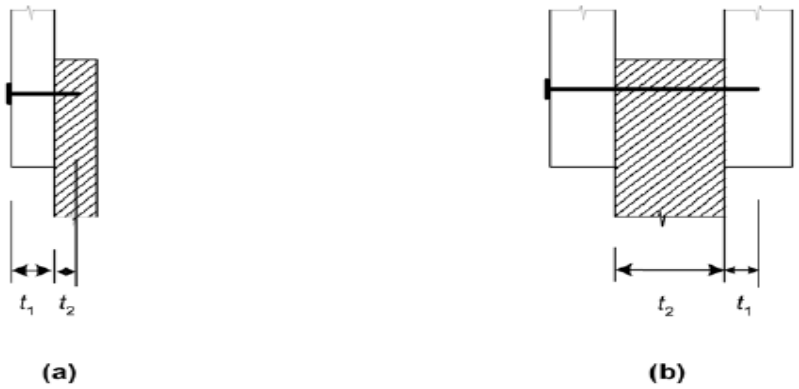


Figure 8.4 – Definitions of t_1 and t_2 (a) single shear connection, (b) double shear connection

(6) For nails with diameters greater than 8 mm the characteristic embedment strength values for bolts according to 8.5.1 apply.

(7) In a three-member connection, nails may overlap in the central member provided (t -t_2) is greater than 4d (see Figure 8.5).

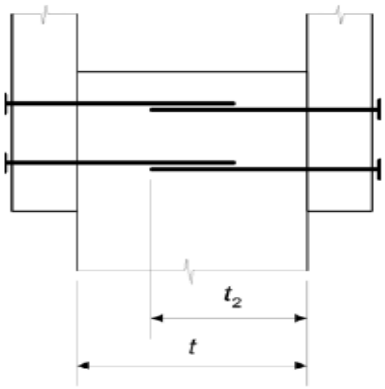


Figure 8.5 – Overlapping nails

(8) For one row of n nails parallel to the grain, unless the nails of that row are staggered perpendicular to grain by at least 1d (see figure 8.6), the load-carrying capacity parallel to the grain (see 8.1.2(4)) should be calculated using the effective number of fasteners n_{ef}, where:

$$n_{ef} = n^{k_{ef}} \tag{8.17}$$

where:

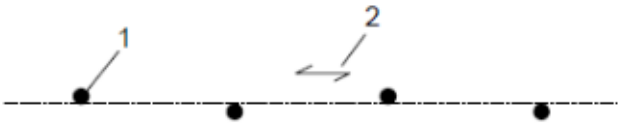
n_{ef} is the effective number of nails in the row;

n is the number of nails in a row;

k_{ef} is given in Table 8.1.

Table 8.1 – Values of k_{ef}

Spacing[a]	k_{ef}	
	Not predrilled	Predrilled
$a_1 \geq 14d$	1.0	1.0
$a_1 = 10d$	0.85	0.85
$a_1 = 7d$	0.7	0.7
$a_1 = 4d$	-	0.5
[a] For intermediate spacings, linear interpolation of k_{ef} is permitted		



Key:
1 Nail
2 Grain direction

Figure 8.6 – Nails in a row parallel to grain staggered perpendicular to grain by *d*

(9) For a force acting at an angle to the direction of the grain, it should be verified, that the force component parallel to the grain is less or equal to the load-carrying capacity calculated according to (8).

(10) There should be at least two nails in a connection.

(11) Requirements for structural detailing and control of nailed connections are given in 10.4.2.

8.3.1.2 Nailed timber-to-timber connections

(1) For smooth nails the pointside penetration length should be at least 8*d*.

(2) For nails other than smooth nails, as defined in ES EN 14592, the pointside penetration length should be at least 6*d*.

(3) Smooth nails in end grain should not be considered capable of transmitting lateral forces.

(4) As an alternative to 8.3.1.2(3), for nails in end grain the following rules apply:

- In secondary structures smooth nails may be used. The design values of the load-carrying capacity should be taken as 1/3 of the values for nails installed at right angles to the grain;
- Nails other than smooth nails, as defined in ES EN 14592, may be used in structures other than secondary structures. The design values of the load-carrying capacity should be taken as 1/3 of the values for smooth nails of equivalent diameter installed at right

angles to the grain, provided that:

- the nails are only laterally loaded;
- there are at least three nails per connection;
- the pointside penetration is at least 10*d*;
- the connection is not exposed to service class 3 conditions;
- the prescribed spacings and edge distances given in Table 8.2 are satisfied.

Note 1: An example of a secondary structure is a fascia board nailed to rafters.

Note 2: The recommended application rule is given in 8.3.1.2(3).

(5) Minimum spacings and edge and end distances are given in Table 8.2, where (see Figure 8.7):

a_1 is the spacing of nails within one row parallel to grain;

a_2 Is the spacing of rows of nails perpendicular to grain;

$a_{3,c}$ is the distance between nail and unloaded end;

$a_{3,t}$ is the distance between nail and loaded end

$a_{4,c}$ is the distance between nail and unloaded edge

$a_{4,t}$ is the distance between nail and loaded edge;

α is the angle between the force and the grain direction.

Table 8.2 – Minimum spacings and edge and end distances for nails

Spacing or distance **(see Figure 8.7)**	**Angle** α	**Minimum spacing or end/edge distance**		
		without predrilled holes		with predrilled holes
		$\rho_k \leq 420$ kg/m^3	420 kg/m^3 < $\rho_k \leq 500$ kg/m^3	
Spacing a_1 (parallel to grain)	$0° \leq \alpha \leq 360°$	d < 5 mm: $(5+5\lvert\cos\alpha\rvert)\,d$ $d \geq$ 5 mm: $(5+7\lvert\cos\alpha\rvert)\,d$	$(7+8\lvert\cos\alpha\rvert)\,d$	$(4+\lvert\cos\alpha\rvert)\,d$
Spacing $a2$ (perpendicular to grain)	$0° \leq \alpha \leq 360°$	$5d$	$7d$	$(3+\lvert\sin\alpha\rvert)\,d$
Distance $a3,t$ (loaded end)	$-90° \leq \alpha \leq 90°$	$(10+5\cos\alpha)\,d$	$(15+5\cos\alpha)\,d$	$(7+5\cos\alpha)\,d$
Distance $a3,c$ (unloaded end)	$90° \leq \alpha \leq 270°$	$10d$	$15d$	$7d$
Distance $a4,t$(loaded edge)	$0° \leq \alpha \leq 180°$	d< 5 mm: $(5+2\sin\alpha)\,d$ $d \geq$ 5 mm: $(5+5\sin\alpha)\,d$	d< 5 mm: $(7+2\sin\alpha)\,d$ $d \geq$ 5 mm: $(7+5\sin\alpha)\,d$	d< 5 mm: $(3+2\sin\alpha)\,d$ $d \geq$ 5 mm: $(3+4\sin\alpha)\,d$
Distance $a4,c$ (unloaded edge)	$180° \leq \alpha \leq 360^0$	$5d$	$7d$	$3d$

(6) Timber should be pre-drilled when the thickness of the timber members is smaller than

$$t = \max\begin{cases} 7d \\ (13d - 30)\dfrac{\rho_k}{400} \end{cases} \qquad (8.18)$$

where:

t is the minimum thickness of timber member to avoid pre-drilling, in mm;

ρ_k is the characteristic timber density in kg/m³;

d is the nail diameter, in mm.

(7) Timber of species especially sensitive to splitting should be pre-drilled when the

thickness of the timber members is smaller than

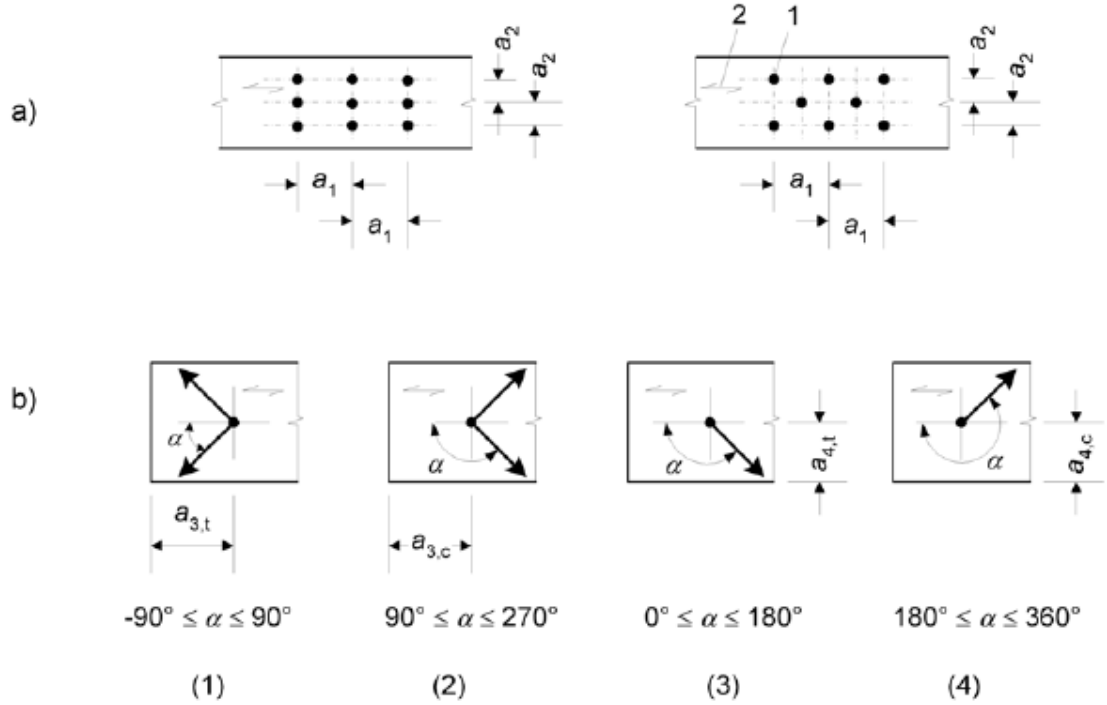
$$t = \max \begin{cases} 14d \\ (13d - 30)\dfrac{\rho_k}{200} \end{cases} \qquad (8.19)$$

Expression (8.19) may be replaced by expression (8.18) for edge distances given by:

$a_4 \geq 10d$ for $\rho_k \leq 420\,kg/m^3$

$a_4 \geq 14d$ for $420\,kg/m^3 \leq \rho_k \leq 500\,kg/m^3$

Note: Examples of species sensitive to splitting are fir (abies alba), Douglas fir (pseudotsuga menziesii) and spruce (picea abies). It is recommended to apply 8.3.1.2(7) for species fir (abies alba) and Douglas fir (pseudotsuga menziesii).



Key:
(1) Loaded end
(2) Unloaded end
(3) Loaded edge
(4) Unloaded edge
1 Fastener
2 Grain direction

Figure 8.7 – Spacings and end and edge distances
(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge and end distances

8.3.1.3 Nailed panel-to-timber connections

(1) Minimum nail spacings for all nailed panel-to-timber connections are those given in

Table 8.2, multiplied by a factor of 0.85. The end/edge distances for nails remain unchanged unless otherwise stated below.

(2) Minimum edge and end distances in plywood members should be taken as 3*d* for an unloaded edge (or end) and $(3+4\sin\alpha)d$ for a loaded edge (or end), where α is the angle between the direction of the load and the loaded edge (or end).

(3) For nails with a head diameter of at least 2*d*, the characteristic embedment strengths are as follows:

– for plywood:

$$f_{h,k} = 0.11\rho_k d^{-0.3} \quad (8.20)$$

where:

$f_{h,k}$ is the characteristic embedment strength, in N/mm^2

ρ_k is the characteristic plywood density in kg/m^3

d is the nail diameter, in mm;

– for hardboard in accordance with ES EN 622-2:

$$f_{h,k} = 30\, d^{-0.3} t^{0.6} \quad (8.21)$$

where:

$f_{h,k}$ is the characteristic embedment strength, in N/mm^2;

d is the nail diameter, in mm;

t is the panel thickness, in mm.

– for particleboard and OSB:

$$f_{h,k} = 65 d^{-0.7} t^{0.1} \quad (8.22)$$

where:

$f_{h,k}$ is the characteristic embedment strength, in N/mm^2;

d is the nail diameter, in mm;

t is the panel thickness, in mm.

8.3.1.4 Nailed steel-to-timber connections

(1) The minimum edge and end distances for nails given in Table 8.2 apply. Minimum nail spacings are those given in Table 8.2, multiplied by a factor of 0.7.

8.3.2 Axially loaded nails

(1)P Smooth nails shall not be used to resist permanent or long-term axial loading.

(2) For threaded nails, only the threaded part should be considered capable of transmitting axial load.

(3) Nails in end grain should be considered incapable of transmitting axial load.

(4) The characteristic withdrawal capacity of nails, $F_{ax,Rk}$, for nailing perpendicular to the grain (Figure 8.8 (a) and for slant nailing (Figure 8.8 (b)), should be taken as the smaller of the values found from the following expressions:

- For nails other than smooth nails, as defined in ES EN 14592:

$$F_{ax,Rk} = \begin{cases} f_{ax,k}\, d\, t_{pen} & (a) \\ f_{head,k}\, d_h^2 & (b) \end{cases} \quad (8.23)$$

– For smooth nails:

$$F_{\partial x,Rk} = \begin{cases} f_{ax,k}\, d\, t_{pen} & (a) \\ f_{ax,k}\, d\, t + f_{head,k}\, d_h^2 & (b) \end{cases} \quad (8.24)$$

where:

$f_{ax,k}$ is the characteristic pointside withdrawal strength;

$f_{head,k}$ is the characteristic headside pull-through strength;

d is the nail diameter according to 8.3.1.1;

t_{pen} is the pointside penetration length or the length of the threaded part in the pointside member;

t is the thickness of the headside member;

d_h is the nail head diameter

(5) The characteristic strengths $f_{ax,k}$ and $f_{head,k}$ should be determined by tests in accordance with ES EN 1382, ES EN 1383 and ES EN 14358 unless specified in the following.

(6) For smooth nails with a pointside penetration of at least 12*d*, the characteristic values of the withdrawal and pull-through strengths should be found from the following expressions:

$$f_{ax,k} = 20 \times 10^{-6} \rho_k^2 \quad (8.25)$$

$$f_{head,k} = 70 \times 10^{-6} \rho_k^2 \quad (8.26)$$

where:

ρ_k is the characteristic timber density in kg/m³;

(7) For smooth nails, the pointside penetration t_{pen} should be at least 8*d*. For nails with a pointside penetration smaller than 12*d* the withdrawal capacity should be multiplied by $\left(t_{pen} / 4d - 2\right)$. For threaded nails, the pointside penetration should be at least 6*d*. For nails with a pointside penetration smaller than 8*d* the withdrawal capacity should be multiplied by $\left(t_{pen} / 2d - 3\right)$.

(8) For structural timber which is installed at or near fibre saturation point, and which is likely to dry out under load, the values of $f_{ax,k}$ and $f_{head,k}$ should be multiplied by 2/3.

(9) The spacings, end and edge distances for laterally loaded nails apply to axially loaded nails.

(10) For slant nailing the distance to the loaded edge should be at least 10*d* (see Figure 8.8(b)). There should be at least two slant nails in a connection.

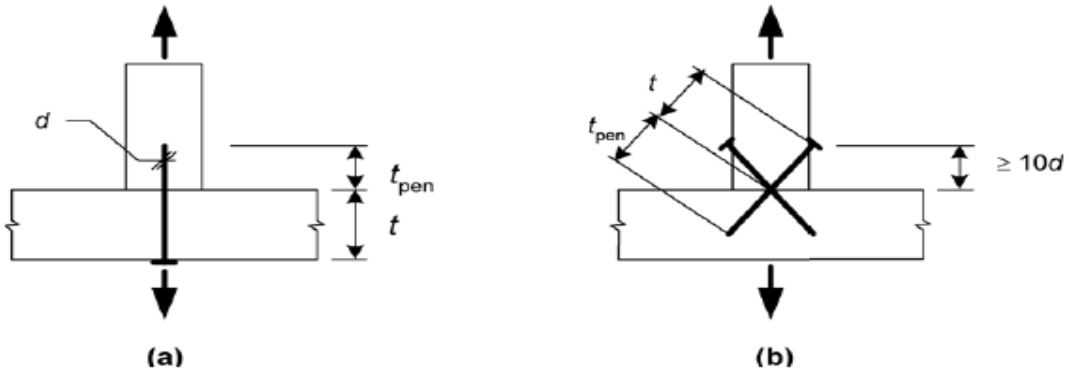


Figure 8.8 – (a) Nailing perpendicular to grain and (b) slant nailing

8.3.3 Combined laterally and axially loaded nails

(1) For connections subjected to a combination of axial load ($F_{ax,Ed}$) and lateral load ($F_{v,Ed}$) the following expressions should be satisfied:

– for smooth nails:

$$\frac{F_{ax,Ed}}{F_{ax,Rd}} + \frac{F_{v,Ed}}{F_{v,Rd}} \leq 1 \qquad (8.27)$$

– for nails other than smooth nails, as defined in ES EN 14592

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1 \qquad (8.28)$$

where:

$F_{ax,Rd}$ and $F_{v,Rd}$ are the design load-carrying capacities of the connection loaded with axial load or lateral load respectively.

8.4 Stapled connections

(1) The rules given in 8.3, except for 8.3.1.1(5) and (6) and 8.3.1.2(7), apply for round or nearly round or rectangular staples with bevelled or symmetrical pointed legs.

(2) For staples with rectangular cross-sections the diameter *d* should be taken as the square root of the product of both dimensions.

(3) The width *b* of the staple crown should be at least 6*d*, and the pointside penetration length t_2 should be at least 14*d*, see Figure 8.9.

(4) There should be at least two staples in a connection.

(5) The lateral design load-carrying capacity per staple per shear plane should be considered as equivalent to that of two nails with the staple diameter, provided that the angle between the crown and the direction of the grain of the timber under the crown is greater than 30°, see Figure 8.10. If the angle between the crown and the direction of the grain under the crown is equal to or less than 30°, then the lateral design load-carrying capacity should be multiplied by a factor of 0.7.

(6) For staples produced from wire with a minimum tensile strength of 800 N/mm², the following characteristic yield moment per leg should be used:

$$M_{y,Rk} = 240\, d^{2.6} \qquad (8.29)$$

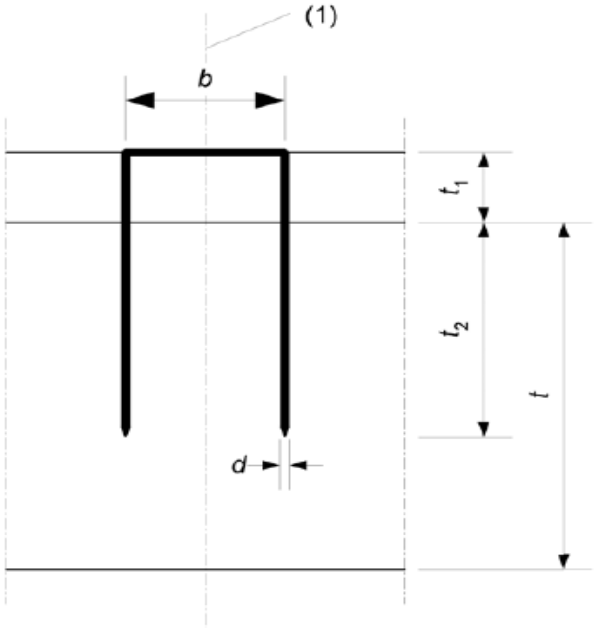
where:

$M_{y,Rk}$ is the characteristic yield moment, in Nmm;

d is the staple leg diameter, in mm.

(7) For a row of n staples parallel to the grain, the load-carrying capacity in that direction should be calculated using the effective number of fasteners n_{ef} according to 8.3.1.1(8)

(8) Minimum staple spacings, edge and end distances are given in Table 8.3, and illustrated in Figure 8.10 where Θ is the angle between the staple crown and the grain direction.



Key:
(1) staple center

Figure 8.9 – Staple dimensions

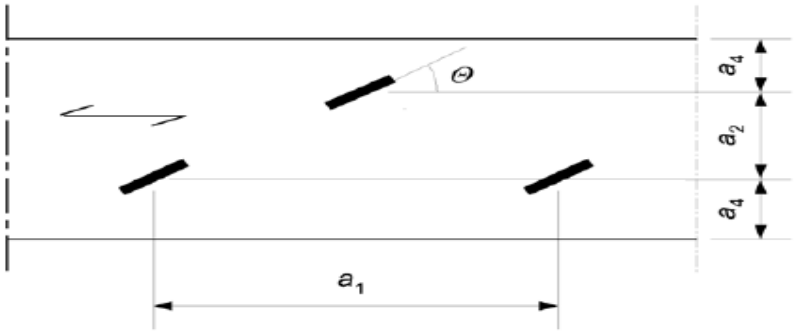


Figure 8.10 – Definition of spacing for staples

Table 8.3 – Minimum spacings and edge and end distances for staples

Spacing and edge/end distances (see Figure 8.7)	Angle	Minimum spacing or edge/end distance
a_1 (parallel to grain) for θ ≥ 30° for θ<30°	$0^0 \le \alpha \le 360^0$	 (10+5|cos α|) d (15+5|cos α|) d
a_2 (perpendicular to grain)	$0^0 \le \alpha \le 360^0$	15d
$a_{3,t}$ (loaded end)	$-90^0 \le \alpha \le 90^0$	(15+5|cos α|) d
$a_{3,c}$ (unloaded end)	$90^0 \le \alpha \le 270^0$	15d
$a_{4,t}$ (loaded edge)	$0^0 \le \alpha \le 180^0$	(15+5|sin α|) d
$a_{4,c}$ (unloaded edge)	$180^0 \le \alpha \le 360^0$	10 d

8.5 Bolted connections

8.5.1 Laterally loaded bolts

8.5.1.1 General and bolted timber-to-timber connections

(1) For bolts the following characteristic value for the yield moment should be used:

$$M_{y,Rk} = 0.3 f_{u,k} d^{2.6} \tag{8.30}$$

where:

$M_{y,Rk}$ is the characteristic value for the yield moment, in Nmm;

$f_{u,k}$ is the characteristic tensile strength, in N/mm²;

d is the bolt diameter, in mm.

(2) For bolts up to 30 mm diameter, the following characteristic embedment strength values in timber and LVL should be used, at an angle α to the grain:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \tag{8.31}$$

$$f_{h,0,k} = 0.082(1 - 0.01d)\rho_k \tag{8.32}$$

where:

$$k_{90} = \begin{cases} 1.35 + 0.015d & \text{for softwoods} \\ 1.30 + 0.015d & \text{for LVL} \\ 0.90 + 0.015d & \text{for hardwoods} \end{cases} \tag{8.33}$$

and:

$f_{h,0,k}$ is the characteristc embedment strength parallel to grain, in N/mm^2;

ρ_k is the characteristic timber density, in kg/m³;
α is the angle of the load to the grain;
d is the bolt diameter, in mm.

(3) Minimum spacings and edge and end distances should be taken from Table 8.4, with symbols illustrated in Figure 8.7.

Table 8.4 – Minimum values of spacing and edge and end distances for bolts

Spacing and end/edge distances (see Figure 8.7)	Angle	Minimum spacing or distance
a_1 (parallel to grain)	0 ° ≤ α ≤ 360 °	(4 + | cos α |) *d*
a_2 (perpendicular to grain)	0 ° ≤ α ≤ 360 °	4 *d*
$a_{3,t}$ (loaded end)	-90 ° ≤ α ≤ 90 °	max (7 *d*; 80 mm)
$a_{3,c}$ (unloaded end)	90 ° ≤ α< 150 ° 150 ° ≤ α< 210 ° 210 ° ≤ α ≤ 270 °	max [(1 + 6 sin α) *d*; 4*d*] 4 *d* max [(1 + 6 sin α) *d*; 4*d*]
$a_{4,t}$ (loaded edge)	0 ° ≤ α ≤ 180 °	max [(2 + 2 sin α) *d*; 3*d*]
$a_{4,c}$ (unloaded edge)	180 ° ≤ α ≤ 360 °	3 *d*

4) For one row of *n* bolts parallel to the grain direction, the load-carrying capacity parallel to grain, see 8.1.2(4), should be calculated using the effective number of bolts n_{ef} where:

$$n_{ef} = \min \begin{cases} n \\ n^{0.9} \sqrt[4]{\frac{a_1}{13d}} \end{cases} \quad (8.34)$$

where:

a_1 is the spacing between bolts in the grain direction:
d is the bolt diameter
n is the number of bolts in the row.

For loads perpendicular to grain, the effective number of fasteners should be taken as

$$n_{ef} = n \quad (8.35)$$

For angles 0° <α< 90° between load and grain direction, n_{ef} may be determined by linear interpolation between expressions (8.34) and (8.35).

(5) Requirements for minimum washer dimensions and thickness in relation to bolt diameter are given in 10.4.3

8.5.1.2 Bolted panel-to-timber connections

(1) For plywood the following embedment strength, in N/mm^2, should be used at all angles to the face grain:

$$f_{h,k} = 0.11(1 - 0.01d)\rho_k \quad (8.36)$$

where:

ρ_k is the characteristic plywood density, in kg/m³;

d is the bolt diameter, in mm.

(2) For particleboard and OSB the following embedment strength value, in N/mm^2, should be used at all angles to the face grain:

$$f_{h,k} = 50 d^{-0.6} t^{0.2} \quad (8.37)$$

where:

d is the bolt diameter, in mm;

t is the panel thickness, in mm.

8.5.1.3 Bolted steel-to-timber connections

(1) The rules given in 8.2.3 apply.

8.5.2 Axially loaded bolts

(1) The axial load-bearing capacity and withdrawal capacity of a bolt should be taken as the lower value of:

- the bolt tensile capacity;
- the load-bearing capacity of either the washer or (for steel-to-timber connections) the steel plate.

(2) The bearing capacity of a washer should be calculated assuming a characteristic compressive strength on the contact area of $3.0 f_{c,90,k}$.

(3) The bearing capacity per bolt of a steel plate should not exceed that of a circular washer with a diameter which is the minimum of:

- 12*t*, where *t* is the plate thickness;
- 4*d*, where *d* is the bolt diameter.

8.6 Dowelled connections

(1) The rules given in 8.5.1 except 8.5.1.1(3) apply.

(2) The dowel diameter should be greater than 6 mm and less than 30 mm.

(3) Minimum spacing and edge and end distances are given in Table 8.5, with symbols illustrated in Figure 8.7.

Table 8.5 – Minimum spacings and edge and end distances for dowels

Spacing and edge/end distances (see Figure 8.7)	Angle	Minimum spacing or edge/end distance
a_1 (parallel to grain)	$0° \le \alpha \le 360°$	$(3 + 2\lvert\cos\alpha\rvert)\ d$
a_2 (perpendicular to grain)	$0° \le \alpha \le 360°$	$3\ d$
$a_{3,t}$ (loaded end)	$-90° \le \alpha \le 900°$	max (7 d; 80 mm)
$a_{3,c}$ (unloaded end)	$90° \le \alpha < 150°$ $150° \le \alpha < 210°$ $210° \le \alpha \le 270°$	$\max(a_{3,t}\lvert\sin\alpha\rvert\ d; 3d)$ $3\ d$ $\max(a_{3,t}\lvert\sin\alpha\rvert\ d; 3d)$
$a_{4,t}$ (loaded edge)	$0° \le \alpha \le 180°$	$\max([2 + 2\sin\alpha]d; 3d)$
$a_{4,c}$ (unloaded edge)	$180° \le \alpha \le 360°$	$3\ d$

(4) Requirements for dowel hole tolerances are given in 10.4.4.

8.7 Screwed connections

8.7.1 Laterally loaded screws

(1)P The effect of the threaded part of the screw shall be taken into account in determining the load-carrying capacity, by using an effective diameter *d*ef.

(2) For smooth shank screws, where the outer thread diameter is equal to the shank diameter, the rules given in 8.2 apply, provided that:

- The effective diameter *d*ef is taken as the smooth shank diameter;
- The smooth shank penetrates into the member containing the point of the screw by not less than 4*d.*

(3) Where the conditions in (2) are not satisfied, the screw load-carrying capacity should be calculated using an effective diameter *d*ef taken as 1.1 times the thread root diameter.

(4) For smooth shank screws with a diameter $d > 6$ mm, the rules in 8.5.1 apply.

(5) For smooth shank screws with a diameter of 6 mm or less, the rules of 8.3.1 apply.

(6) Requirements for structural detailing and control of screwed joints are given in 10.4.5.

8.7.2 Axially loaded screws

(1) The following failure modes should be verified when assessing the load-carrying capacity of connections with axially loaded screws:

- the withdrawal capacity of the threaded part of the screw;
- for screws used in combination with steel plates, the tear-off capacity of the screw head should be greater than the tensile strength of the screw;
- the pull-through strength of the screw head;
- the tension strength of the screw;
- for screws used in conjunction with steel plates, failure along the circumference of a

group of screws (block shear or plug shear);

(2) Minimum spacing and edge distances for axially loaded screws should be taken from Table 8.6.

Table 8.6 – Minimum spacings and edge distances for axially loaded screws

Screws driven	Minimum spacing	Minimum edge distance
At right angle to the grain	4d	4d
In end grain	4d	2.5d

(3) The minimum pointside penetration length of the threaded part should be 6*d*.

(4) The characteristic withdrawal capacity of connections with axially loaded screws should be taken as:

$$F_{\mathrm{ax},\alpha,Rk} = n_{ef}\left(\pi d l_{ef}\right)^{0.8} f_{\mathrm{ax},\alpha,k} \tag{8.38}$$

where:

$F_{\mathrm{ax,0,Rk}}$ is the characteristic withdrawal capacity of the connection at an angle *α* to the grain;

n_{ef} is the effective number of screws;

d is the outer diameter measured on the threaded part;

l_{ef} is the pointside penetration length of the threaded part minus one screw diameter;

$f_{\mathrm{ax},\alpha,\mathrm{Rk}}$ is the characteristic withdrawal strength at an angle α to the grain.

(5) The characteristic withdrawal strength at an angle α to the grain should be taken as:

$$f_{\mathrm{ax},\alpha,k} = \frac{f_{\mathrm{ax},k}}{\sin^2\alpha + 1.5\cos^2\alpha} \tag{8.39}$$

with:

$$f_{\mathrm{ax},k} = 3.6\times10^{-3}\rho_k^{1.5} \tag{8.40}$$

where:

$f_{\mathrm{ax},\alpha,\mathrm{k}}$ is the characteristic withdrawal strength at an angle α to the grain;

$f_{\mathrm{ax,k}}$ is the characteristic withdrawal strength perpendicular to the grain;

ρ_k is the characteristic density, in kg/m^3.

NOTE: Failure modes in the steel or in the timber around the screw are brittle, i.e. with small ultimate deformation and therefore have a limited possibility for stress redistribution.

(6)P The pull-through capacity of the head shall be determined by tests, in accordance with ES EN1383.

(7) For a connection with a group of screws loaded by a force component parallel to the shank, the effective number of screws is given by:

$$n_{ef} = n^{0.9} \quad (8.41)$$

where:

n_{ef} is the effective number of screws;

n is the number of screws acting together in a connection.

8.7.3 Combined laterally and axially loaded screws

(1) For screwed connections subjected to a combination of axial load and lateral load, expression (8.28) should be satisfied.

8.8 Connections made with punched metal plate fasteners

8.8.1 General

(1)P Connections made with punched metal plate fasteners shall comprise punched metal plate fasteners of the same type, size and orientation, placed on each side of the timber members.

(2) The following rules apply only to punched metal plate fasteners with two orthogonal directions.

8.8.2 Plate geometry

(1) The symbols used to define the geometry of a punched metal plate fastener joint are given in Figure 8.11 and defined as follows:

x-direction main direction of plate;
y-direction perpendicular to the main plate direction;
α angle between the x-direction and the force (tension: 0° ≤ γ< 90°, compression: 90° ≤ γ< 180°);
β angle between the grain-direction and the force;
γ angle between the x-direction and the connection line;
A_{ef} area of the total contact surface between the plate and the timber, reduced by 5mm from the edges of the timber and by a distance in the grain direction from the end of timber equal to6 times the fastener's nominal thickness;
l dimension of the plate measured along the connection line.

8.8.3 Plate strength properties

(1)P The plate shall have characteristic values for the following properties, determined in accordance with ES EN 14545 from tests carried out in accordance with ES EN 1075:

$f_{a,0,0}$ the anchorage capacity per unit area for $\alpha = 0^0$ and $\beta = 0^0$;

$f_{a,90,90}$ the anchorage capacity per unit area for $\alpha = 90^0$ and $\beta = 90^0$;

$f_{t,0}$ the tension capacity per unit width of plate for $\alpha = 0^0$;

$f_{c,0}$ the compression capacity per unit width of plate for $\alpha = 0^0$;

$f_{v,0}$ the shear capacity per unit width of plate in the x-direction;

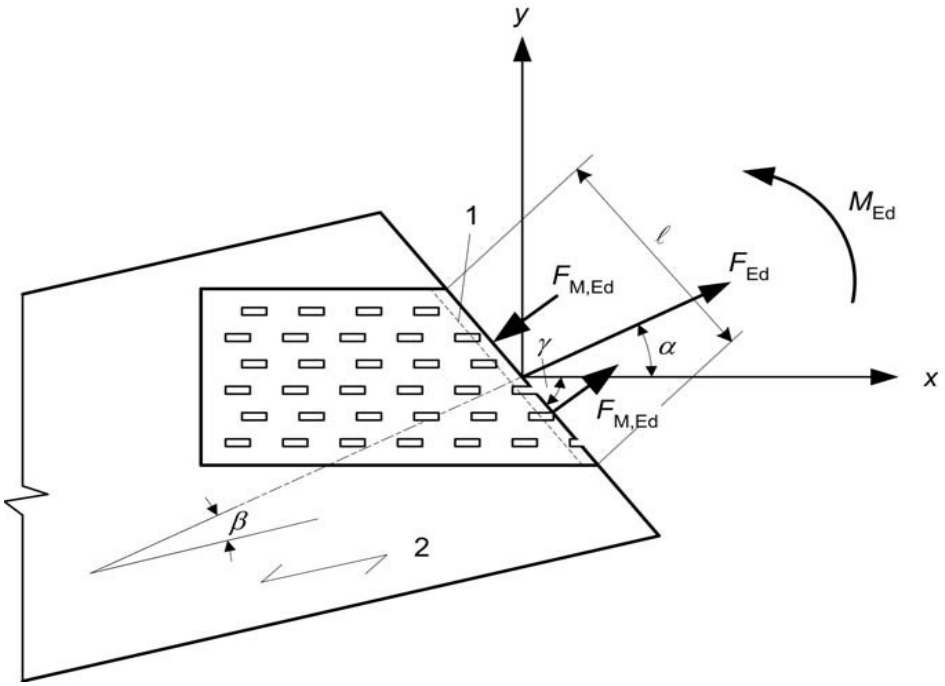
$f_{t,90}$ the tension capacity per unit width of plate for $\alpha = 90^0$;

$f_{c,90}$ the compression capacity per unit width of plate for $\alpha = 90^0$;

$f_{v,90}$ the shear capacity per unit width of plate in the y-direction;

k_1, k_2, α_0 constants.

(2)P In order to calculate the design tension, compression and shear capacities of the plate the value of k_{mod} shall be taken as 1.0.



Key:
1 Border or effective area
2 Grain direction

Figure 8.11 – Geometry of punched metal plate connection loaded by a force F_{Ed} and moment M_{Ed}

8.8.4 Plate anchorage strengths

(1) The characteristic anchorage strength per plate $f_{\text{a},\alpha,\beta,\text{k}}$ should either be derived from tests or calculated from:

$$f_{\text{a},\alpha,\beta,k} = \max \begin{cases} f_{\text{a},\alpha,0,k} - \left(f_{\text{a},\alpha,0,k} - f_{\text{a},90,90,k}\right)\dfrac{\beta}{45^o} \\ f_{\text{a},0,0,k} - \left(f_{\text{a},0,0,k} - f_{\text{a},90,90,k}\right)\sin\left(\max\left(\alpha,\beta\right)\right) \end{cases} \quad for\ \beta \le 45^o, or \qquad (8.42)$$

$$f_{\text{a},\alpha,\beta,k} = f_{\text{a},0,0,k} - \left(f_{\text{a},0,0,k} - f_{\text{a},90,90,k}\right)\sin\left(\max\left(\alpha,\beta\right)\right) \quad for\ 45^o \le \beta \le 90^o \qquad (8.43)$$

(2) The characteristic anchorage strength per plate parallel to grain should be taken as:

$$f_{a,\alpha,0,k} = \max \begin{cases} f_{a,0,0,k} + k_1 \alpha & when\ \alpha \le \alpha_o \\ f_{a,0,0,k} + k_1 \alpha_o + k_1 (\alpha - \alpha_o) & when\ \alpha_o \le \alpha \le 90^o \end{cases} \quad (8.44)$$

The constants k_1, k_2 and α_o should be determined from anchorage tests in accordance with ES EN1075 and derived in accordance with the procedure given in ES EN 14545 for the actual plate type.

8.8.5 Connection strength verification

8.8.5.1 Plate anchorage capacity

(1) The design anchorage stress $\tau_{F,d}$ on a single punched metal plate fastener imposed by a force F_{Ed} and the design anchorage stress $\tau_{M,d}$ imposed from a moment M_{Ed}, should be taken as:

$$\tau_{F,d} = \frac{F_{A,Ed}}{A_{ef}} \quad (8.45)$$

$$\tau_{M,d} = \frac{M_{A,Ed}}{W_p} \quad (8.46)$$

with:

$$W_p = \int_{A_{ef}} r\,dA \quad (8.47)$$

where:

$F_{A,Ed}$ is the design force acting on a single plate at the centroid of the effective area (i.e. half of the total force in the timber member);

$M_{A,Ed}$ is the design moment acting on a single plate on the centroid of the effective area;

d_A is the segmental area of the punched metal plate fastener;

r is the distance from the centre of gravity of the plate to the segmental plate area dA;

A_{ef} is the effective plate area.

(2) As an alternative to expression (8.47), W_p may be conservatively approximated from:

$$W_p = \frac{A_{ef} d}{4} \quad (8.48)$$

with:

$$d = \sqrt{\left(\frac{A_{ef}}{h_{ef}}\right)^2 + h_{ef}^2} \quad (8.49)$$

where: h_{ef} is the maximum height of the effective anchorage area perpendicular to the longest side.

(3) Contact pressure between timber members may be taken into account to reduce the value of F_{Ed} in compression provided that the gap between the members has an average

value, which is not greater than 1.5 mm, and a maximum value of 3 mm. In such cases the connection should be designed for a minimum compressive design force of $F_{A,Ed}/2$.

(4) Contact pressure between the timber members in chord splices in compression may be taken into account by designing the single plate for a design force, $F_{A,Ed}$, and a design moment $M_{A,Ed}$, according to the following expressions:

$$F_{A,Ed} = \sqrt{\left(\frac{F_{Ed}\cos\beta}{2} - \frac{3|M_{Ed}|}{2h}\right)^2 + \left(F_{Ed}\sin\beta\right)^2} \tag{8.50}$$

$$M_{A,Ed} = \frac{M_{Ed}}{2} \tag{8.51}$$

where:

F_{Ed} is the design axial force of the chord acting on a single plate (compression or zero);

M_{Ed} is the design moment of the chord acting on a single plate;

h is the height of the chord.

(5) The following expression should be satisfied:

$$\left(\frac{\tau_{F,d}}{f_{a,\alpha,\beta,d}}\right)^2 + \left(\frac{\tau_{M,d}}{f_{a,0,0,d}}\right)^2 \le 1 \tag{8.52}$$

8.8.5.2 Plate capacity

(1) For each joint interface, the forces in the two main directions should be taken as:

$$F_{x,Ed} = F_{Ed}\cos\alpha \pm 2F_{M,Ed}\sin\gamma \tag{8.53}$$

$$F_{y,Ed} = F_{Ed}\sin\alpha \pm 2F_{M,Ed}\cos\gamma \tag{8.54}$$

where:

F_{Ed} is the design force in a single plate (i.e. half of the total force in the timber member)

$F_{M,Ed}$ is the design force from the moment on a single plate $\left(F_{M,Ed} = 2M_{Ed}/\ell\right)$

(2) The following expression should be satisfied:

$$\left(\frac{F_{x,Ed}}{F_{x,Rd}}\right)^2 + \left(\frac{F_{y,Ed}}{F_{y,Rd}}\right)^2 \le 1 \tag{8.55}$$

where:

$F_{x,Ed}$ and $F_{y,Ed}$ are the design forces acting in the *x* and *y* direction,

$F_{x,Rd}$ and $F_{y,Rd}$ are the corresponding design values of the plate capacity. They are determined from the maximum of the characteristic capacities at sections parallel or perpendicular to the main axes, based upon the following expressions for the characteristic plate capacities in these directions

$$F_{x,Rk} = \max \begin{cases} \left| f_{n,0,k} \ell \sin\left(\gamma - \gamma_o \sin\left(2\gamma\right)\right) \right| \\ \left| f_{v,0,k} \ell \cos\gamma \right| \end{cases} \quad (8.56)$$

$$F_{y,Rk} = \max \begin{cases} \left| f_{n,90,k} \ell \cos\gamma \right| \\ k\, f_{v,90,k} \ell \sin\gamma \end{cases} \quad (8.57)$$

with

$$f_{N,0,k} = \begin{cases} f_{t,0,k} & for\ F_{x,Ed} > 0 \\ f_{c,0,k} & for\ F_{x,Ed} \le 0 \end{cases} \quad (8.58)$$

$$f_{n,90,k} = \begin{cases} f_{t,90,k} & for\ F_{y,Ed} > 0 \\ f_{c,90,k} & for\ F_{y,Ed} \le 0 \end{cases} \quad (8.59)$$

$$k = \begin{cases} 1 + k_v \sin\left(2\gamma\right) & for\ F_{x,Ed} > 0 \\ 1 & for\ F_{x,Ed} \le 0 \end{cases} \quad (8.60)$$

where γ_o and k_v are constants determined from shear tests in accordance with ES EN 1075 and derived in accordance with the procedure given in ES EN 14545 for the actual plate type.

(3) If the plate covers more than two connection lines on the member then the forces in each straight part of the connection line should be determined such that equilibrium is fulfilled and that expression (8.55) is satisfied in each straight part of the connection line. All critical sections should be taken into account.

8.9 Split ring and shear plate connectors

(1) For connections made with ring connectors of type A or shear plate connectors of type B according to ES EN 912 and ES EN 14545, and with a diameter not bigger than 200 mm, the characteristic load-carrying capacity parallel to grain, Fv,0,Rk per connector and per shear plane should be taken as:

$$F_{v,0,Rk} = \min \begin{cases} k_1 k_2 k_3 \left(35 d_c^{1.5}\right) & (a) \\ k_1 k_2 k_3 \left(31.5 d_c\right) & (b) \end{cases} \quad (8.61)$$

Where:

$F_{v,0,Rk}$ is the characteristic load-carrying capacity parallel to the grain, in N;
d_c is the connector diameter, in mm;
h_e is the embedment depth, in mm;
k_i are modification factors, with I = 1 to 4, defined below.

(2) The minimum thickness of the outer timber members should be $2.25h_e$, and of the inner timber member should be $3{,}75h_e$, where $h_c$is the embedment depth, see Figure 8.12.

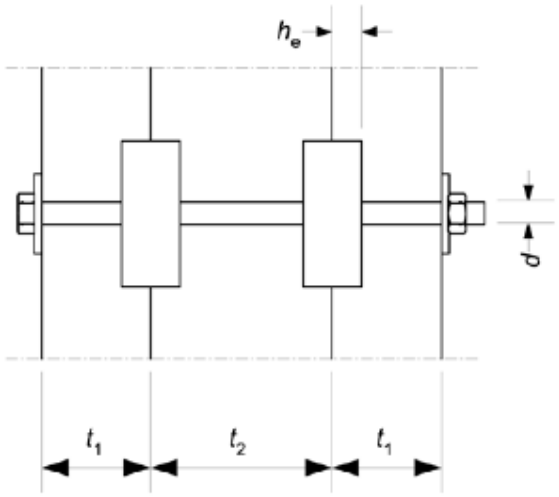


Figure 8.12 – Dimensions for connections with split ring and shear plate connectors

(3) The factor k_1 should be taken as:

$$k_1 = \min \begin{cases} 1 \\ \dfrac{t_1}{3h_e} \\ \dfrac{t_2}{5h_e} \end{cases} \qquad (8.62)$$

(4) The factor k_2 applies to a loaded end ($-30^0 \le \alpha \le 30^0$) and should be taken as:

$$k_2 = \begin{cases} k_a \\ \dfrac{a_{3,t}}{2d_c} \end{cases} \qquad (8.63)$$

where:

$$k_a = \begin{cases} 1.25 & \text{for connections with one connector per shear plane} \\ 1.0 & \text{for connections with more than one connector per shear plane} \end{cases} \qquad (8.64)$$

$\alpha_{3,t}$ is given in Table 8.7.

For other values of α, $k_2 = 1.0$

(5) The factor k_3 should be taken as:

$$k_3 = \min \begin{cases} 1.75 \\ \dfrac{\rho_k}{350} \end{cases} \quad (8.65)$$

where ρ_k is the characteristic density of the timber, in kg/m^3.

(6) The factor k_4 , which depends on the materials connected, should be taken as:

$$k_4 = \begin{cases} 1.0 & \text{for timber - to - timber connections} \\ 1.1 & \text{for steel - to - timber connections} \end{cases} \quad (8.66)$$

(7) For connections with one connector per shear plane loaded in an unloaded end situation ($150° \le \alpha \le 210°$), the condition (a) in expression (8.61) should be disregarded.

(8) For a force at an angle α to the grain, the characteristic load-carrying capacity, $F_{\alpha,Rk}$ per connector per shear plane should be calculated using the following expression:

$$F_{v,a,Rk} = \frac{F_{v,0,Rk}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad (8.67)$$

with:

$$k_{90} = 1.3 + 0.001 d_c \quad (8.68)$$

where:

$F_{v,0,Rk}$ is the characteristic load-carrying capacity of the connector for a force parallel to grain according to expression (8.61);

d_c is the connector diameter, in mm.

(9) Minimum spacing and edge and end distances are given in Table 8.7, with the symbols illustrated in Figure 8.7.

Table 8.7- Minimum spacings and edge and end distances for ring and shear plate connectors.

Spacing and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
a_1 (parallel to grain)	$0° \le \alpha \le 360°$	$(1.2 + 0.8 \lvert \cos \alpha \rvert)\, d_c$
a_2 (perpendicular to grain)	$0° \le \alpha \le 360°$	$1.2\, d_c$
$a_{3,t}$ (loaded end)	$-90° \le \alpha \le 90°$	$1.5\, d_c$
$a_{3,c}$ (unloaded end)	$90° \le \alpha < 150°$ $150° \le \alpha < 210°$ $210° \le \alpha \le 270°$	$(0.4 + 1.6 \lvert \sin \alpha \rvert)\, d_c$ $1.2\, dc$ $(0.4 + 1.6 \lvert \sin \alpha \rvert)\, d_c$
$a_{4,t}$ (loaded edge)	$0° \le \alpha \le 1800°$	$(0.6 + 0.2 \lvert \sin \alpha \rvert)\, d_c$
$a_{4,c}$ (unloaded edge)	$180° \le \alpha \le 360°$	$0.6\, d_c$

(10) When the connectors are staggered (see Figure 8.13), the minimum spacings parallel and perpendicular to the grain should comply with the following expression:

$$(k_{a1})^2 + (k_{a2})^2 \geq 1 \qquad with \quad \begin{cases} 0 \leq k_{a1} \leq 1 \\ 0 \leq k_{a2} \leq 1 \end{cases} \tag{8.69}$$

where:

k_{a1} is a reduction factor for the minimum distance $a1$ parallel to the grain;

k_{a2} is a reduction factor for the minimum distance $a2$ perpendicular to the grain.

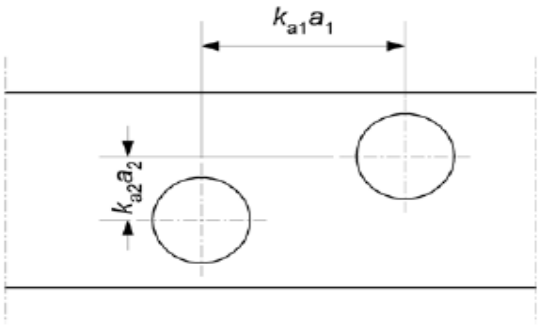


Figure 8.13 – Reduced distances for connectors

(11) The spacing parallel to grain, $k_{a1}a_1$ may further be reduced by multiplication by a factor $k_{s,red}$, with $0.5 \leq k_{s,red} \leq 1.0$, provided that the load-carrying capacity is multiplied by a factor

$$k_{R,red} = 0.2 + 0.8k_{s,red} \tag{8.70}$$

(12) For a row of connectors parallel to the grain, the load-carrying capacity in that direction should be calculated using the effective number of connectors n_{ef} where:

$$n_{ef} = 2 + \left(1 - \frac{n}{20}\right)(n-2) \tag{8.71}$$

where:

n_{ef} is the effective number of connectors;

n is the number of connectors in a line parallel to grain.

(13) Connectors should be considered as positioned parallel to the grain where $k_{a2}a_2 < 0.5\ k_{a1}a_1$.

8.10 Toothed-plate connectors

(1) The characteristic load-carrying capacity of connections made using toothed-plate connectors should be taken as the summation of the characteristic load-carrying capacity of the connectors themselves and the connecting bolts according to 8.5.

(2) The characteristic load-carrying capacity $F_{v,Rk}$ per toothed-plate connector for connectors of type C according to ES EN 912 (single-sided: type C2, C4, C7, C9, C11; double sided: type C1, C3, C5, C6, C8, C10) and ES EN 14545 should be taken as:

$$F_{v,Rk} = \begin{cases} 18 k_1 k_2 k_3 d_c^{1.5} & \text{for single-sided types} \\ 25 k_1 k_2 k_3 d_c^{1.5} & \text{for double-sided types} \end{cases} \quad (8.72)$$

where:

$F_{v,Rk}$ is the characteristic load-carrying capacity per toothed-plate connector, in N.

k_i are modification factors, with i = 1 to 3, defined below.

d_c is:

- the toothed-plate connector diameter for types C1, C2 C6 C7 C10 and C11, in mm;
- the toothed-plate connector side length for types C5,C8 and C9,in mm;
- the square of the product of both side lengths for types C3 and C4, in mm.

(3) Clause 8.9(2) applies.

(4) The factor k_1 should be taken as:

$$k_1 = \min \begin{cases} 1 \\ \dfrac{t_1}{3h_e} \\ \dfrac{t_2}{5h_e} \end{cases} \quad (8.73)$$

where:

$t1$ is the side member thickness;

$t2$ is the middle member thickness;

hc is the tooth penetration depth, in mm.

(5) The factor k_2 should be taken as:

– For types C1 to C9:

$$k_2 = \min \begin{cases} 1 \\ \dfrac{a_{3,t}}{1.5 d_c} \end{cases} \quad (8.74)$$

with

$$a_{3,t} = \max \begin{cases} 1.1 d_c \\ 7\,d \\ 80 mm \end{cases} \quad (8.75)$$

where:

d is the bolt diameter, in mm;

d_c is explained in (2) above.]

– For types C10 and C11:

$$k_2 = \min\begin{cases} 1 \\ \dfrac{a_{3,t}}{2.0 d_c} \end{cases} \tag{8.76}$$

with

$$a_{3,t} = \min\begin{cases} 1.5 d_c \\ 7\, d \\ 80 mm \end{cases} \tag{8.77}$$

where:

d is the bolt diameter in mm:
d_c is explained in (2) above.

(6) The factor k_3 should be taken as:

$$k_3 = \min\begin{cases} 1.5 \\ \dfrac{p_k}{350} \end{cases} \tag{8.78}$$

where ρ_k is the characteristic density of the timber, in kg/m^3.

(7) For toothed-plate connector types C1 to C9, minimum spacings and edge and end distances should be taken from Table 8.8, with the symbols illustrated in Figure 8.7.

(8) For toothed-plate connector types C10 and C11, minimum spacing and edge and end distances should be taken from Table 8.9, with the symbols illustrated in Figure 8.7.

(9) Where connectors of types C1, C2, C6 and C7 with circular shape are staggered, 8.9(10) applies.

(10) For bolts used with toothed-plate connectors, 10.4.3 applies.

Table 8.8 – Minimum spacings and edge and end distances for toothed-plate connector types C1 to C9.

Spacings and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
a_1 (parallel to grain)	$0° \le \alpha \le 360°$	$(1.2 + 0.3 \lvert \cos\alpha \rvert)\, d_c$
a_2 (perpendicular to grain)	$0° \le \alpha \le 360°$	$1.2\, d_c$
$a_{3,t}$ (loaded end)	$-90° \le \alpha \le 90°$	$2.0\, d_c$
$a_{3,c}$ (unloaded end)	$90° \le \alpha < 150°$ $150^0 \le \alpha < 210°$ $210° \le \alpha \le 270°$	$(0.9 + 0.6 \lvert \sin\alpha \rvert)\, d_c$ $1.2\, dc$ $(0.9 + 0.6 \lvert \sin\alpha \rvert)\, d_c$
$a_{4,t}$ (loaded edge)	$0° \le \alpha \le 180°$	$(0.6 + 0.2 \lvert \sin\alpha \rvert)\, d_c$
$a_{4,c}$ (unloaded edge)	$180° \le \alpha \le 360°$	$0.6\, d_c$

Table 8.9 – Minimum spacings and edge and end distances for toothed-plate connector types C10 and C11.

Spacings and edge/end distances (see Figure 8.7)	Angle to grain	Minimum spacings and edge/end distances
a_1 (parallel to grain)	0 ° ≤ α ≤ 360 °	(1.2 + 0.8 | cos α |) d_c
a_2 (perpendicular to grain)	0 ° ≤ α ≤ 360 °	1.2 d_c
$a_{3,t}$ (loaded end)	-90 ° ≤ α ≤ 90 °	2.0 d_c
$a_{3,c}$ (unloaded end)	90 ° ≤ α< 150 ° 150 ° ≤ α< 210 ° 210 ° ≤ α ≤ 270 °	(0.4 + 1.6 | sin α |) d_c 1.2 d_c (0.4 + 1.6 | sin α |) d_c
$a_{4,t}$ (loaded edge)	0 ° ≤ α ≤ 180 °	(0.6 + 0.2 | sin α |) d_c
$a_{4,c}$ (unloaded edge)	180 ° ≤ α ≤ 360 °	0.6 d_c

Section 9 Components and assemblies

9.1 Components

9.1.1 Glued thin-webbed beams

(1) If a linear variation of strain over the depth of the beam is assumed, the axial stresses in the wood-based flanges should satisfy the following expressions:

$$\sigma_{f,c,\max,d} \le f_{m,d} \quad (9.1)$$

$$\sigma_{f,t,\max,d} \le f_{m,d} \quad (9.2)$$

$$\sigma_{f,c,d} \le k_c f_{c,0,d} \quad (9.3)$$

$$\sigma_{f,t,d} \le f_{t,0,d} \quad (9.4)$$

where:

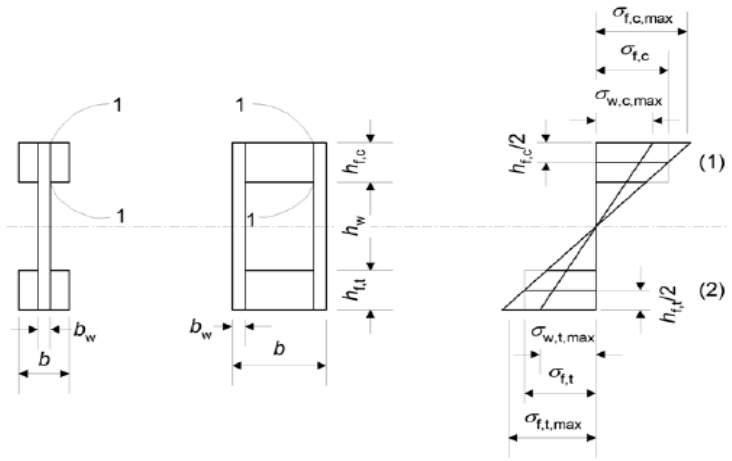
$\sigma_{f,c,\max,d}$ is the extreme fibre flange design compressive stress;

$\sigma_{f,t,\max,d}$ is the extreme fibre flange design tensile stress;

$\sigma_{f,c,d}$ is the mean flange design compressive stress;

$\sigma_{f,t,d}$ is the mean flange design tensile stress;

k_c is a factor which takes into account lateral instability.



Key:
(1) compression
(2) tension

Figure 9.1 – Thin-webbed beams

(3) The factor k_c may be determined (conservatively, especially for box beams) according to 6.3.2 with

$$\lambda_z = \sqrt{12}\left(\frac{\ell_c}{b}\right) \tag{9.5}$$

where:

ℓ_c is the distance between the sections where lateral deflection of the compressive flange is prevented;

b is given in Figure 9.1.

If a special investigation is made with respect to the lateral instability of the beam as a whole, it may be assumed that k_c= 1.0.

(4) The axial stresses in the webs should satisfy the following expressions:

$$\sigma_{w,c,d} \le f_{c,w,d} \tag{9.6}$$

$$\sigma_{w,t,d} \le f_{t,w,d} \tag{9.7}$$

where:

$\sigma_{w,c,d}$ and $\sigma_{w,t,d}$ are the design compressive and tensile stresses in the webs;

$f_{c,w,d}$ and $f_{t,w,d}$ are the design compressive and tensile bending strengths of the webs.

(5) Unless other values are given, the design in-plane bending strength of the webs should be taken as the design tensile or compressive strength.

(6)P It shall be verified that any glued splices have sufficient strength.

(7) Unless a detailed buckling analysis is made it should be verified that:

$$h_w \le 70\, b_w \tag{9.8}$$

and

$$F_{v,w,Ed} \le \begin{cases} b_w h_w \left(1 + \dfrac{0.5\left(h_{f,t} + h_{f,c}\right)}{h_w}\right) f_{v,0,d} & for\ h_w \le 35 b_w \\ 35 b_w^2 \left(1 + \dfrac{0.5(h_{f,t} + h_{f,c})}{h_w}\right) f_{v,0,d} & for\ 35 b_w \le h_w \le 70 b_w \end{cases} \tag{9.9}$$

where:

$F_{v,w,Ed}$ is the design shear force acting on each web;

h_w is the clear distance between flanges;

$h_{f,c}$ is the compressive flange depth;

$h_{f,t}$ is the tensile flange depth;

b_w is the width of each web;

$f_{v,0,d}$ is the design panel shear strength.

(8) For webs of wood-based panels, it should, for sections 1-1 in Figure 9.1, be verified that:

$$\tau_{mean,d} \ge \begin{cases} f_{v,90,d} & for\ h_f \le 4 b_{ef} \\ f_{v,90,d} \left(\dfrac{4 b_{ef}}{h_f}\right)^{0.8} & for\ h_f > 4\, b_{ef} \end{cases} \tag{9.10}$$

where:

$\tau_{mean,d}$ is the design shear stress at the sections 1-1, assuming a uniform stress distribution;

$f_{v,90,d}$ is the design planar (rolling) shear strength of the web;

h_f is either $h_{f,c}$ or $h_{f,t}$.

$$b_{ef} = \begin{cases} b_w & \text{for boxed beams} \\ b_w / 2 & \text{for I-beams} \end{cases} \tag{9.11}$$

9.1.2 Glued thin-flanged beams

(1) This clause assumes a linear variation of strain over the depth of the beam.

(2)P In the strength verification of glued thin-flanged beams, account shall be taken of the nonuniform distribution of stresses in the flanges due to shear lag and buckling.

(3) Unless a more detailed calculation is made, the assembly should be considered as a number of I-beams or U-beams (see Figure 9.2) with effective flange widths b_{ef}, as follows:

– For I-beams

$$b_{ef} = b_{c,ef} + b_w \qquad (or\ b_{t,ef} + b_w) \tag{9.12}$$

– For U-beams

$$b_{ef} = 0.5 b_{c,ef} + b_w \qquad (or\ 0.5 b_{t,ef} + b_w) \tag{9.13}$$

The values of $b_{c,ef}$ and $b_{t,ef}$ should not be greater than the maximum value calculated for shear lag from Table 9.1. In addition the value of $b_{c,ef}$ should not be greater than the maximum value calculated for plate buckling from Table 9.1.

(4) Maximum effective flange widths due to the effects of shear lag and plate buckling should be taken from Table 9.1, where l is the span of the beam.

Table 9.1 – Maximum effective flange widths due to the effects of shear lag and plate buckling

Flange material	Shear lag	Plate buckling
Plywood, with grain direction in the outer plies:		
– Parallel to the webs	0.1l	$20h_f$
– Perpendicular to the webs	0.1l	$25h_f$
Oriented strand board	0.15l	$25h_f$
Particleboard or fibreboard with random fibre orientation	0.2l	$30h_f$

(5) Unless a detailed buckling investigation is made, the unrestrained flange width should not be greater than twice the effective flange width due to plate buckling, from Table 9.1.

(6) For webs of wood-based panels, it should, for sections 1-1 of an I-shaped cross-section in Figure 9.2, be verified that:

$$\tau_{mean,d} \le \begin{cases} f_{v,90,d} & for\ b_w \le 8h_f \\ f_{v,90,d}\left(\dfrac{8h_f}{b_w}\right)^{0.8} & for\ b_w > 8h_f \end{cases} \qquad (9.14)$$

where:

$\tau_{mean,d}$ is the design shear stress at the sections 1-1, assuming a uniform stress distribution;

$f_{v,90,d}$ is the design planar (rolling) shear strength of the flange.

For section 1-1 of a U-shaped cross-section, the same expressions should be verified, but with $8h_f$ substituted by 4 h_f.

(7) The axial stresses in the flanges, based on the relevant effective flange width, should satisfy

the following expressions:

$$\sigma_{f,c,d} \le f_{f,c,d} \qquad (9.15)$$

$$\sigma_{f,t,d} \le f_{f,t,d} \qquad (9.16)$$

where:

$\sigma_{f,c,d}$ is the mean flange design compressive stress;

$\sigma_{f,t,d}$ is the mean flange design tensile stress;

$f_{f,c,d}$ is the flange design compressive strength;

$f_{f,t,d}$ is the flange design tensile strength.

(8)P It shall be verified that any glued splices have sufficient strength.

(9) The axial stresses in the wood-based webs should satisfy the expressions (9.6) to (9.7) defined
in 9.1.1

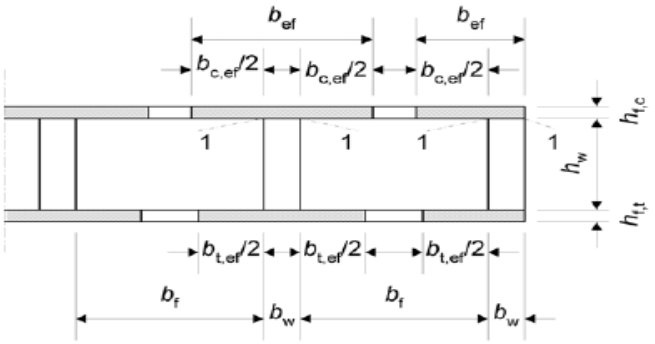


Figure 9.2 – Thin-flanged beam

9.1.3 Mechanically jointed beams

(1)P If the cross-section of a structural member is composed of several parts connected by mechanical fasteners, consideration shall be given to the influence of the slip occurring in the joints.

(2) Calculations should be carried out assuming a linear relationship between force and slip.

(3) If the spacing of the fasteners varies in the longitudinal direction according to the shear force between $s_{\min}$ and $s_{\max}$ ($\leq 4s_{\min}$), an effective spacing s_{ef} may be used as follows:

$$s_{ef} = 0.75\, s_{\min} + 0.25\, s_{\max} \qquad (9.17)$$

A method for the calculation of the load-carrying capacity of mechanically jointed beams is given in Annex B (Informative).

9.1.4 Mechanically jointed and glued columns

(1)P Deformations due to slip in joints, to shear and bending in packs, gussets, shafts and flanges, and to axial forces in the lattice shall be taken into account in the strength verification.

A method for the calculation of the load-carrying capacity of I- and box-columns, spaced columns and lattice columns is given in Annex C.

9.2 Assemblies

9.2.1 Trusses

(1) For trusses which are loaded predominantly at the nodes, the sum of the combined bending and axial compressive stress ratios given in expressions (6.19) and (6.20) should be limited to 0.9.

(2) For members in compression, the effective column length for in-plane strength verification should generally be taken as the distance between two adjacent points of contraflexure.

(3) For fully triangulated trusses, the effective column length for members in compression should be taken as the bay length, see Figure 5.1, if:

- members are only one bay long, without rigid end connections,
- members are continuous over two or more bays and are not loaded laterally

(4) When a simplified analysis of a fully triangulated truss with punched metal plate fasteners according to clause 5.4.3 has been carried out, the following effective column lengths may be assumed (see Figure 9.3)

- for continuous members without significant end moments and where the bending stresses of the lateral load are at least 40 % of the compressive stresses:
  - in an outer bay: 0.8 times the bay length;
  - in an inner bay: 0.6 the bay length;
  - at a node: 0.6 times the largest adjacent bay length;
- for continuous members with significant end moments where the bending stresses of the lateral load are at least 40 % of the compressive stresses:
  - at the beam end with moment: 0.0 (i.e. no column effect);
  - in the penultimate bay: 1.0 times bay length;
  - remaining bays and nodes: as described above for continuous beams without significant end moments;
- for all other cases 1.0 times bay length.

For the strength verification of members in compression and connections, the calculated axial forces should be increased by 10 %.

(5) When a simplified analysis is carried out for trusses which are loaded at the nodes, the tensile and compressive stress ratios as well as the connection capacity should be limited to 70 %.

(6)P A check shall be made that the lateral (out-of-plane) stability of the truss members is adequate.

(7)P The joints shall be capable of transferring the forces which may occur during handling and erection.

(8) All joints should be capable of transferring a force $F_{r,d}$ acting in any direction within the plane of the truss. $F_{r,d}$ should be assumed to be of short-term duration, acting on timber in service class 2, with the value:

$$F_{r,d} = 1.0 + 0.1L \qquad (9.18)$$

where:

$F_{r,d}$ is in kN;

L is the overall length of the truss, in m.

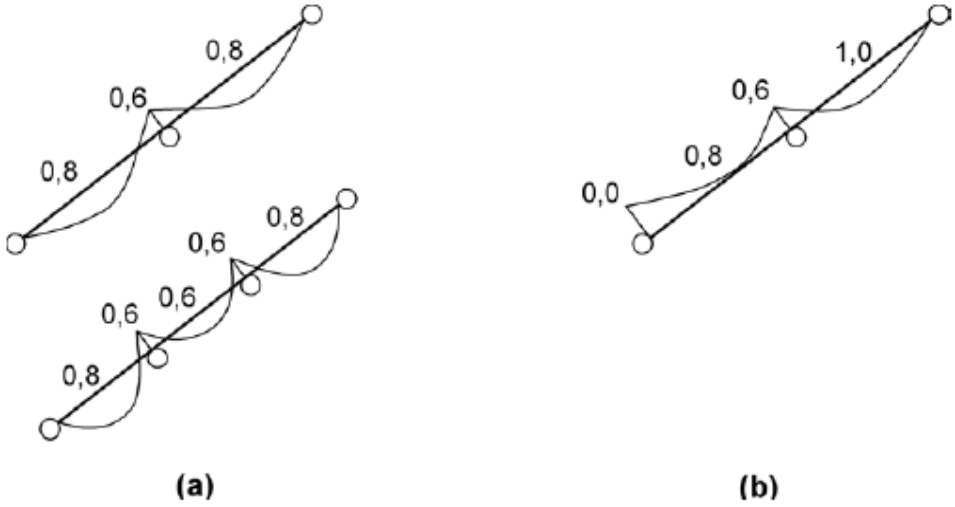


Figure 9.3 – Moment diagrams and effective lengths in compression (a) No significant end moments (b) Significant end moments

9.2.2 Trusses with punched metal plate fasteners

(1)P Trusses made with punched metal plate fasteners shall conform to the requirements of ES EN 14250.

(2) The requirements of 5.4.1 and 9.2.1 apply.

(3) For fully triangulated trusses where a small concentrated force (e.g. a man load) has a component perpendicular to the member of < 1.5kN, and where $\sigma_{c,d} < 0.4 f_{c,d}$ and $\sigma_{t,d} < 0.4 f_{t,d}$, then the requirements of 6.2.3 and 6.2.4 may be replaced by:

$$\sigma_{m,d} \le 0.75 f_{m,d} \qquad (9.19)$$

(4) The minimum overlap of the punched metal plate on any timber member should be at least equal to 40 mm or one third of the height of the timber member, whichever is the greater.

(5) Punched metal plate fasteners used in chord splices should cover at least 2/3 of the required member height.

9.2.3 Roof and floor diaphragms
9.2.3.1 General

(1) This section relates to simply supported diaphragms, such as floors or roofs, assembled from sheets of wood-based material fixed by mechanical fasteners to a timber frame.

(2) The load-carrying capacity of fasteners at sheet edges may be increased by a factor of

1.2 over the values given in Section 8.

9.2.3.2 Simplified analysis of roof and floor diaphragms.

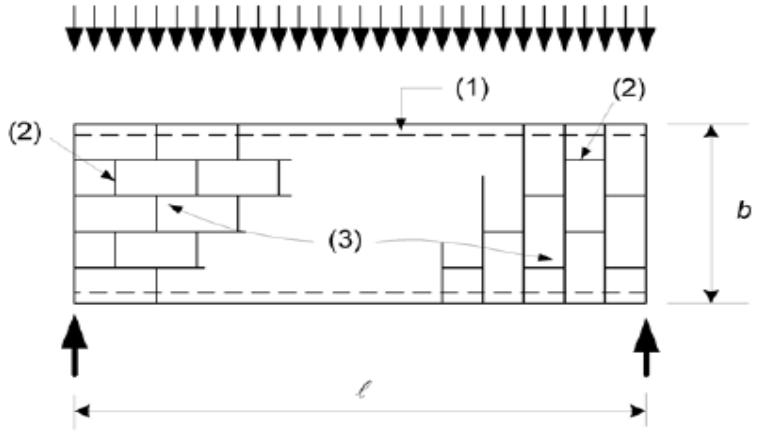
(1) For diaphragms with a uniformly distributed load (see Figure 9.4) the simplified method of analysis described in this section should be used provided that:

- the span l lies between 2*b* and 6*b*, where *b* is the diaphragm width;
- the critical ultimate design condition is failure in the fasteners (and not in the panels);
- the panels are fixed in accordance with the detailing rules in 10.8.1.

(2) Unless a more detailed analysis is made, the edge beams should be designed to resist the maximum bending moment in the diaphragm.

(3) The shear forces in the diaphragm should be assumed to be uniformly distributed over the width of the diaphragm.

(4) When the sheets are staggered, (see Figure 9.4), the nail spacings along the discontinuous panel edges may be increased by a factor of 1.5 (up to a maximum of 150 mm)without reduction of the load-carrying capacity.



Key:
(1) Edge beam
(2) Discontinuous edges
(3) Panel arrangements

Figure 9.4 – Diaphragm loading and staggered panel arrangements

9.2.4 Wall diaphragms
9.2.4.1 General

(1)P Wall diaphragms shall be designed to resist both horizontal and vertical actions imposed upon them.

(2)P The wall shall be adequately restrained to avoid overturning and sliding.

(3)P Wall diaphragms deemed to provide resistance to racking shall be stiffened in-plane by board materials, diagonal bracing or moment connections.

(4)P The racking resistance of a wall shall be determined either by test according to ES EN 594 or by calculations, employing appropriate analytical methods or design models.

(5)P The design of wall diaphragms shall take account of both the material construction and geometric make-up of the wall under consideration.

(6)P The response of wall diaphragms to actions shall be assessed to ensure the construction remains within appropriate serviceability limits.

(7) For wall diaphragms two alternative simplified methods of calculation are given in 9.2.4.2 and 9.2.4.3. The recommended procedure is method A given in 9.2.4.2.

9.2.4.2 Simplified analysis of wall diaphragms – Method A

(1) The simplified method given in this subclause should only be applied to wall diaphragms with a tie-down at their end, that is the vertical member at the end is directly connected to the construction below.

(2) The design load-carrying capacity $F_{v,Rd}$ (the design racking resistance) under a force $F_{v,Ed}$ acting at the top of a cantilevered panel secured against uplift (by vertical actions or by anchoring) should be determined using the following simplified method of analysis for walls made up of one or more panels, where each wall panel consists of a sheet fixed to one side of a timber frame, provided that:

- the spacing of fasteners is constant along the perimeter of every sheet;
- the width of each sheet is at least *h*/4.

(3) For a wall made up of several wall panels, the design racking load-carrying capacity of a wall should be calculated from

$$F_{v,Rd} = \sum F_{i,v,Rd} \qquad (9.20)$$

where $F_{i,v,Rd}$ is the design racking load-carrying capacity of the wall panel in accordance with 9.2.4.2(3) and 9.2.4.2(5).

(4) The design racking load-carrying capacity of each wall panel, $F_{i,v,Rd}$, against a force $F_{i,v,Ed}$ according to Figure 9.5 should be calculated from

$$F_{i,v,Rd} = \frac{F_{f,Rd} b_i c_i}{s} \qquad (9.21)$$

where:

$F_{f,Rd}$ is the lateral design capacity of an individual fastener;

b_i is the wall panel width;

s is the fastener spacing.

and

$$c_i = \begin{cases} 1 & \text{for} \quad b_i \geq b_0 \\ \dfrac{b_i}{b_0} & \text{for} \quad b_i < b_0 \end{cases} \tag{9.22}$$

where:

$b_0 = h/2$

h is the height of the wall.

(5) For fasteners along the edges of an individual sheet, the design lateral load-carrying capacity should be increased by a factor of 1.2 over the corresponding values given in Section 8. In determining the fastener spacing in accordance with the requirements of Section 8, the edges should be assumed to be unloaded.

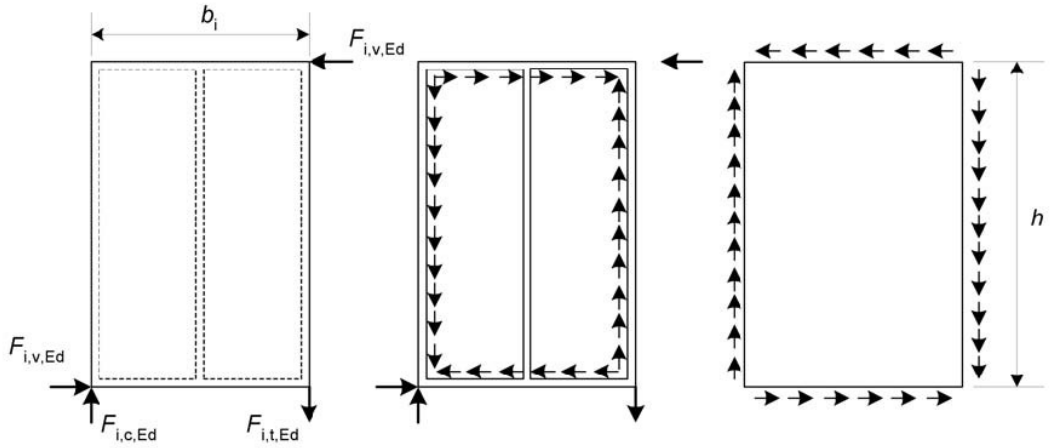


Figure 9.5 – Forces acting on:
a) wall panel;
b) framing;
c) sheet

(6) Wall panels which contain a door or window opening should not be considered to contribute to the racking load-carrying capacity.

(7) For wall panels with sheets on both sides the following rules apply:

– if the sheets and fasteners are of the same type and dimension then the total racking load-carrying capacity of the wall should be taken as the sum of the racking load-carrying capacities of the individual sides

– if different types of sheets are used, 75 % of the racking load-carrying capacity of the weaker side may, unless some other value is shown to be valid, be taken into consideration if fasteners with similar slip moduli are used. In other cases not more than 50 % should be taken into consideration.

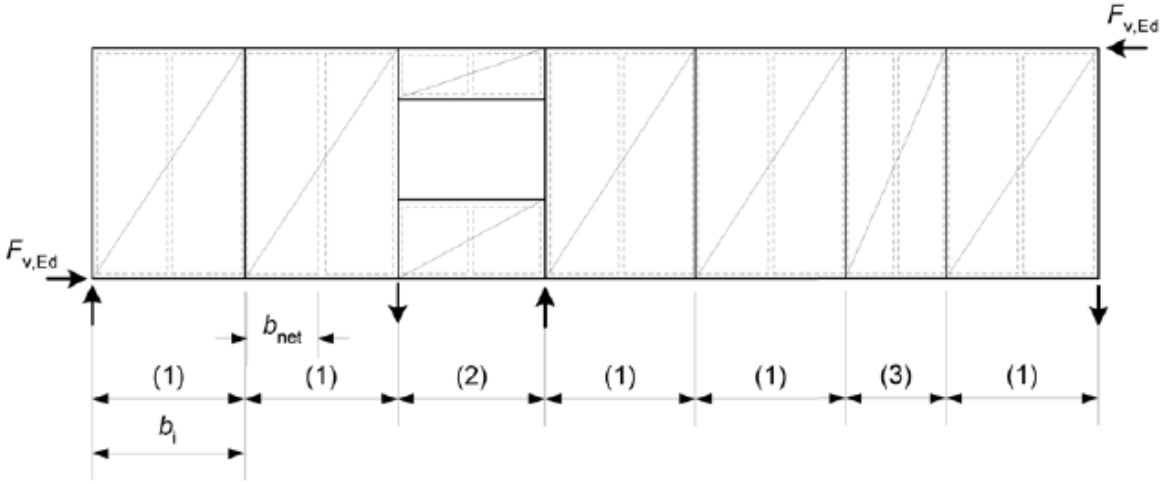
(8) The external forces $F_{i,c,Ed}$ and $F_{i,t,Ed}$ according to Figure 9.5 should be determined from

$$F_{i,c,Ed} = F_{i,t,Ed} = \frac{F_{i,v,Ed} h}{b_i} \quad (9.23)$$

where *h* is the height of the wall.

(9) These forces can either be transmitted to the sheets in the adjacent wall panel or transmitted to the construction situated above or below. When tensile forces are transmitted to the construction situated below, the panel should be anchored by stiff fasteners. Buckling of wall studs should be checked in accordance with 6.3.2. Where the ends of vertical members bear on horizontal framing members, the compression perpendicular to the grain stresses in the horizontal members should be assessed according to 6.1.5.

(10) The external forces which arise in wall panels containing door or window openings and in wall panels of smaller width, see Figure 9.6, can similarly be transmitted to the construction situated above or below.



Key:
(1) Wall panel (normal width)
(2) Wall panel with window
(3) Wall panel (smaller width)

Figure 9.6 – Example of the assembly of wall panels containing a wall panel with a window opening and a wall panel of smaller width

(11) Shear buckling of the sheet may be disregarded, provided that $\frac{b_{net}}{t} \leq 100$

where:

b_{net} is the clear distance between studs;

t is the thickness of the sheet.

(12) In order that the centre stud may be considered to constitute a support for a sheet, the spacing of fasteners in the centre stud should not be greater than twice the spacing of the fasteners along the edges of the sheet.

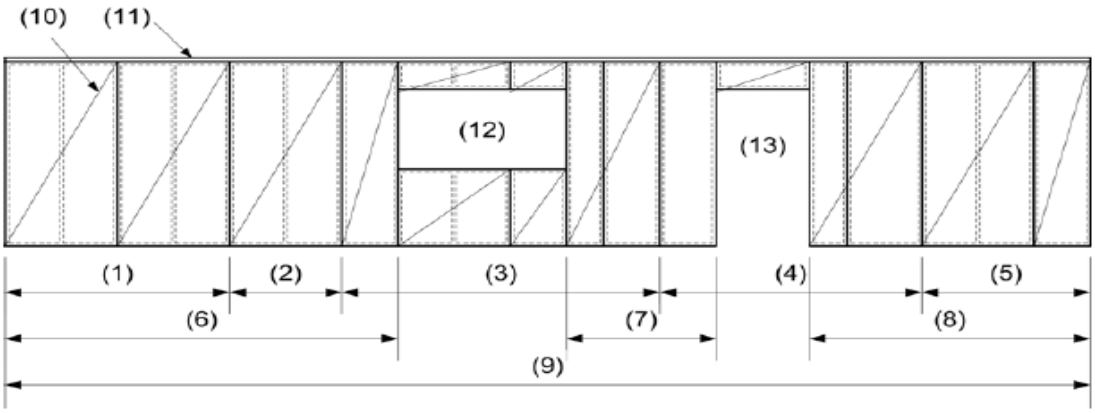
(13) Where each panel consists of a prefabricated wall element, the transfer of shear forces between the separate wall elements should be verified.

(14) In contact areas between vertical studs and horizontal timber members, compression stresses perpendicular to grain should be verified in the timber members.

9.2.4.3 Simplified analysis of wall diaphragms – Method B

9.2.4.3.1 Construction of walls and panels to meet the requirements of the simplified analysis

(1) A wall assembly (see Figure 9.7) is comprised of one or more walls with each wall formed from one or more panels, the panels being made from sheets of wood-based panel products, such as those described in 3.5, fastened to a timber frame.



Key:
(1) Wall panel 1 (2) Wall panel 2
(3) Wall panel 3 (4) Wall panel 4
(5) Wall panel 5 (6) Wall 1
(7) Wall 2 (8) Wall 3
(9) Wall assembly (10) Sheet
(11) Head binder (12) Window
(13) Door

Figure 9.7 – Example of wall assembly consisting of several wall panels

(2) For a panel to contribute to the in-plane (racking) strength of a wall the width of the panel should be at least the panel height divided by 4. The fastening of the sheets to the timber frame should be by either nails or screws and the fasteners should be equally spaced around the perimeter of the sheet. Fasteners within the perimeter of a sheet should be spaced at not more than twice the perimeter fastener spacing.

(3) Where an opening is formed in a panel, the lengths of panel on each side of the opening should be considered as separate panels.

(4) Where panels are combined to form a wall:

- the tops of individual panels should be linked by a member or construction across the panel joints;
- the required vertical connection strength between two panels should be evaluated but should have a design strength of at least 2.5 kN/m;

– the panels when joined together to form a wall should be able to resist overturning and sliding forces by either anchorage to the supporting structure or the permanent actions applied to the wall or a combination of both effects.

9.2.4.3.2 Design procedure

(1) The in-plane design shear (racking) strength $F_{v,Rd}$ against a force $F_{v,Ed}$ acting at the top of a cantilevered wall that is secured against uplift and sliding by vertical actions and/or anchorage, should be determined using the following simplified method for the wall construction defined in 9.2.4.3.1.

(2) For a wall assembly made up of several walls, the design racking strength of the wall assembly $F_{v,Rd}$ should be calculated from

$$F_{v,Rd} = \sum F_{i,v,Rd} \tag{9.24}$$

where:

$F_{i,v,Rd}$ is the design racking strength of a wall in accordance with (3) below.

(3) The design racking strength of a wall *i*, $F_{i,v,Rd}$, should be calculated from

$$F_{i,v,Rd} = \frac{F_{f,Rd}\, b_i}{s_0} k_d k_{i,q} k_s k_n \tag{9.25}$$

where:

$F_{f,Rd}$ is the lateral design capacity of an individual fastener;

b_i is the wall length, in m;

s_0 is the basic fastener spacing, see (4) below;

k_d is the dimension factor for the panel, see (4) below;

$k_{i,q}$ is the uniformly distributed load factor for wall *i*, see (4) below;

k_s is the fastener spacing factor, see (4) below;

k_n is the sheathing material factor, see (4) below.

(4) The values of s_0, k_d, $k_{i,q}$, k_s and k_n should be calculated as:

$$s_0 = \frac{9700\, d}{\rho_k} \tag{9.26}$$

where:

d is the fastener diameter, in mm;

ρ_k is the characteristic density of the timber frame;

– $$k_d = \begin{cases} \dfrac{b_i}{h} & \text{for } \dfrac{b_i}{h} \le 1.0 \quad (a) \\ \left(\dfrac{b_i}{h}\right)^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i \le 4.8m \quad (b) \\ \left(\dfrac{4.8}{h}\right)^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i > 4.8m \quad (c) \end{cases}$$ (9.27)

where

h is the height of the wall, in m;

– $$k_{i,q} = 1 + \left(0.083\, q_i - 0.0008\, q_i^2\right)\left(\frac{2.4}{b_i}\right)^{0.4}$$ (9.28)

where

q_i is the equivalent uniformly distributed vertical load acting on the wall, in kN/m, with $q_i \ge 0$, see (5) below;

– $$k_s = \frac{1}{0.86\dfrac{s}{s_0} + 0.57}$$ (9.29)

where

s is the spacing of the fasteners around the perimeter of the sheets;

– $$k_n = \begin{cases} 1.0 & \text{for sheathing on one side} \quad (a) \\ \dfrac{F_{i,v,Rd,\max} + 0.5\, F_{i,v,Rd,\min}}{F_{i,v,Rd,\max}} & \text{for sheathing on both sides} \quad (b) \end{cases}$$ (9.30)

where:

$F_{i,v,Rd,\max}$ is the design racking strength of the stronger sheathing;

$F_{i,v,Rd,\min}$ is the design racking strength of the weaker sheathing.

(5) The equivalent vertical load, q_i, used to calculate $k_{i,q}$ should be determined using only permanent actions and any net effects of wind together with the equivalent actions arising from concentrated forces, including anchorage forces, acting on the panel. For the purposes of calculating $k_{i,q}$, concentrated vertical forces should be converted into an equivalent uniformly distributed load on the assumption that the wall is a rigid body e.g. for the load $F_{i,vertEd}$ acting on the wall as shown in Figure 9.8

$$q_i = \frac{2\, a\, F_{i,vert,Ed}}{b_i^2}$$ (9.31)

where:

a is the horizontal distance from the force *F* to the leeward corner of the wall;

b is the length of the wall.

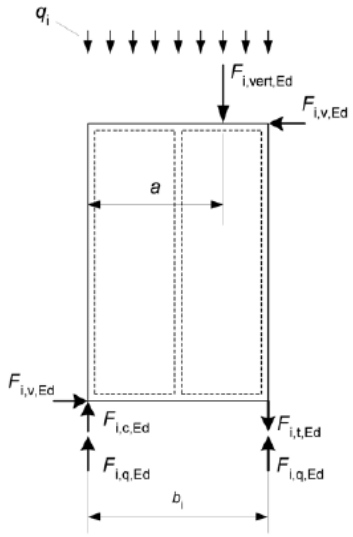


Figure 9.8 – Determination of equivalent vertical action q_i and reaction forces from vertical and horizontal actions

(6) The external forces $F_{i,c,Ed}$ and $F_{i,t,Ed}$ (see Figure 9.8) from the horizontal action $F_{i,v,Ed}$ on wall *i* should be determined from

$$F_{i,c,Ed} = F_{i,t,Ed} = \frac{F_{i,v,Ed} h}{b_i} \tag{9.32}$$

where

h is the height of the wall.

These external forces can be transmitted to either the adjacent panel through the vertical panel-to-panel connection or to the construction above or below the wall. When tensile forces are transmitted to the construction below, the panel should be anchored with stiff fasteners. Compression forces in the vertical members should be checked for buckling in accordance with 6.3.2. Where the ends of vertical members bear on horizontal framing members, the compression perpendicular to the grain stresses in the horizontal members should be assessed according to 6.1.5.

(7) The buckling of the sheets under the action of shear force $F_{v,Ed}$ may be disregarded provided

$$\frac{b_{net}}{t} \leq 100 \tag{9.33}$$

where:

b_{net} is the clear distance between vertical members of the timber frame;

t is the thickness of the sheathing.

9.2.5 Bracing

9.2.5.1 General

(1)P Structures which are not otherwise adequately stiff shall be braced to prevent instability or excessive deflection.

(2)P The stress caused by geometrical and structural imperfections, and by induced deflections (including the contribution of any joint slip) shall be taken into account.

(3)P The bracing forces shall be determined on the basis of the most unfavourable combination of structural imperfections and induced deflections.

9.2.5.2 Single members in compression

(1) For single elements in compression, requiring lateral support at intervals *a* (see Figure 9.9), the initial deviations from straightness between supports should be within *a*/500 for glued laminated or LVL members, and *a*/300 for other members.

(2) Each intermediate support should have a minimum spring stiffness *C*

$$C = k_s \frac{N_d}{a} \tag{9.34}$$

where:

k_s is a modification factor;

N_d is the mean design compressive force in the element;

a is the bay length (see Figure 9.9).

NOTE: For k_s, see note in 9.2.5.3(1)

(3) The design stabilizing force F_d at each support should be taken as:

$$F_d = \begin{cases} \dfrac{N_d}{k_{f,1}} & \text{for solid timber} \\ \dfrac{N_d}{k_{f,2}} & \text{for glued lameated timber and LVL} \end{cases} \tag{9.35}$$

where

$k_{f,1}$ and $k_{f,2}$ are modification factors.

NOTE: For $k_{f,1}$ and $k_{f,2}$, see note in 9.2.5.3(1)

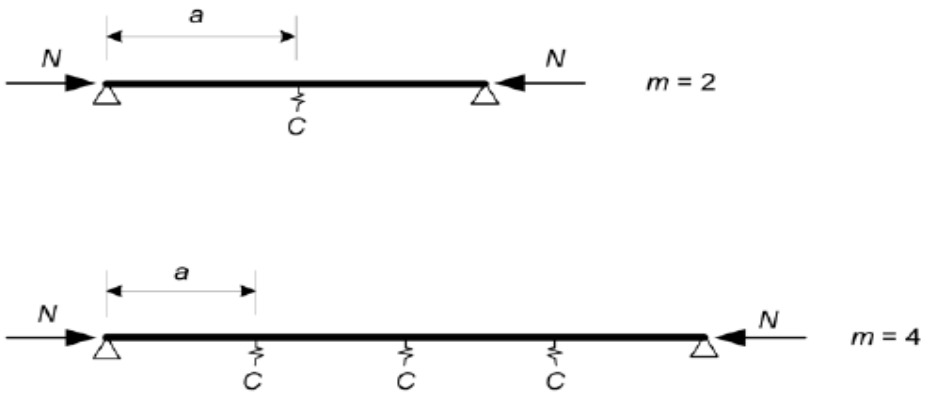


Figure 9.9 – Examples of single members in compression braced by lateral supports.

(4) The design stabilizing force F_d for the compressive edge of a rectangular beam should be determined in accordance with 9.2.5.2(3) where:

$$N_d = (1 - k_{crit}) \frac{M_d}{h} \qquad (9.36)$$

The value of k_{crit} should be determined from 6.3.3(4) for the unbraced beam, and M_d is the maximum design moment acting on the beam of depth *h*.

9.2.5.3 Bracing of beam or truss systems

(1) For a series of *n* parallel members which require lateral supports at intermediate nodes A,B, etc. (see Figure 9.10) a bracing system should be provided, which, in addition to the effects of external horizontal load (e.g. wind), should be capable of resisting an internal stability load per unit length *q*, as follows:

$$q_d = k_\ell \frac{nN_d}{k_{f,3}\ell} \qquad (9.37)$$

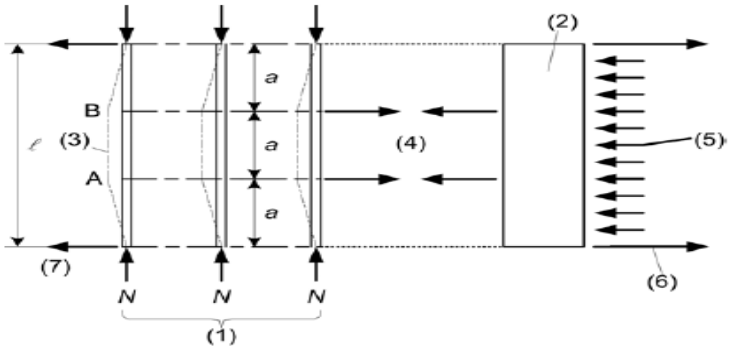
where:

$$k_{\ l} = \min \begin{cases} 1 \\ \sqrt{\dfrac{15}{l}} \end{cases} \qquad (9.38)$$

N_d is the mean design compressive force in the member;

l is the overall span of the stabilizing system, in m;

$k_{f,3}$ is a modification factor.



Key:
(1) *n* members of truss system
(2) Bracing
(3) Deflection of truss system due to imperfections and second order effects
(4) Stabilizing forces
(5) External load on bracing
(6) Reaction forces of bracing due to external loads
(7) Reaction forces of truss system due to stabilizing forces

Figure 9.10 – Beam or truss system requiring lateral supports

NOTE: The values of the modification factors k_s , $k_{f,1}$, $k_{f,2}$ and $k_{f,3}$ depend on influences such as workmanship, span etc. Ranges of values are given in Table 9.2 where the recommended values are underlined.

Table 9.2 – Recommended values of modification factors

Modification factor	Range
k_s	4 to 1
$k_{f,1}$	50 *to 80*
$k_{f,2}$	80to 100
$k_{f,3}$	30 to 80

(2) The horizontal deflection of the bracing system due to force q_d and any other external load (e.g. wind), should not exceed $\ell / 500$.

Section 10 Structural detailing and control

10.1 General

(1)P The provisions given in this section are prerequisite requirements for the design rules given in this standard to apply.

10.2 Materials

(1) The deviation from straightness measured midway between the supports should, for columns and beams where lateral instability can occur, or members in frames, be limited to 1/500 times the length of glued laminated timber or LVL members and to 1/300 times the

length of solid timber. The limitations on bow in most strength grading rules are inadequate for the selection of material for these members and particular attention should therefore be paid to their straightness.

(2) Timber and wood-based components and structural elements should not be unnecessarily exposed to climatic conditions more severe than those expected in the finished structure.

(3) Before being used in construction, timber should be dried as near as practicable to the moisture content appropriate to its climatic condition in the completed structure. If the effects of any shrinkage are not considered important, or if parts that are unacceptably damaged are replaced, higher moisture contents may be accepted during erection provided that it is ensured that the timber can dry to the desired moisture content.

10.3 Glued joints

(1) Where bond strength is a requirement for ultimate limit state design, the manufacture of glued joints should be subject to quality control, to ensure that the reliability and quality of the joint is in accordance with the technical specification.

(2) The adhesive manufacturer's recommendations with respect to mixing, environmental conditions for application and curing, moisture content of members and all factors relevant to the proper use of the adhesive should be followed.

(3) For adhesives which require a conditioning period after initial set, before attaining full strength, the application of load to the joint should be restricted for the necessary time.

10.4 Connections with mechanical fasteners

10.4.1 General

(1)P Wane, splits, knots or other defects shall be limited in the region of the connection such that the load-carrying capacity of the connection is not reduced.

10.4.2 Nails

(1) Unless otherwise specified, nails should be driven in at right angles to the grain and to such depth that the surfaces of the nail heads are flush with the timber surface.

(2) Unless otherwise specified, slant nailing should be carried out in accordance with Figure 8.8(b).

(3) The diameter of pre-drilled holes should not exceed 0,8d, where d is the nail diameter.

10.4.3 Bolts and washers

(1) Bolt holes in timber should have a diameter not more than 1 mm larger than the bolt. Bolt holes in steel plates should have a diameter not more than 2 mm or 0,1d (whichever is the greater) larger than the bolt diameter d.
(2) Washers with a side length or a diameter of at least 3d and a thickness of at least 0,3d should be used under the head and nut. Washers should have a full bearing area.

(3) Bolts and lag screws should be tightened so that the members fit closely, and they should be re-tightened if necessary when the timber has reached equilibrium moisture

content to ensure that the load-carrying capacity and stiffness of the structure is maintained.

(4) The minimum diameter requirements given in Table 10.1 apply to bolts used with timber connectors, where:

d_c is the connector diameter, in mm;

d is the bolt diameter, in mm

d_1 is the diameter of center hole of connector.

Table 10.1 – Requirements for diameters of bolts used with timber connectors

Type of connector ES EN 912	d_c	d minimum	d maximum
	mm	mm	mm
A1 – A6	≤ 130	12	24
A1, A4, A6	> 130	$0.1d_c$	24
B		d_1-1	d_1

10.4.4 Dowels

(1) The minimum dowel diameter should be 6 mm. The tolerances on the dowel diameter should be
- 0/+0.1 mm. Pre-bored holes in the timber members should have a diameter not greater than the dowel.

10.4.5 Screws

(1) For screws in softwoods with a smooth shank diameter $d \leq 6$ mm, pre-drilling is not required. For all screws in hardwoods and for screws in softwoods with a diameter $d > 6$ mm, pre-drilling is required, with the following requirements:
- The lead hole for the shank should have the same diameter as the shank and the same depth as the length of the shank
- The lead hole for the threaded portion should have a diameter of approximately 70 % of the shank diameter.

(2) For timber densities greater than 500 kg/m^3, the pre-drilling diameter should be determined by tests.

10.5 Assembly

(1) The structure should be assembled in such a way that over-stressing of its members or connections is avoided. Members which are warped, split or badly fitting at the joints should be replaced.

10.6 Transportation and erection

(1) The over-stressing of members during storage, transportation or erection should be avoided. If the structure is loaded or supported in a different manner during construction

than in the finished building the temporary condition should be considered as a relevant load case, including any possible dynamic actions. In the case of structural framework, e.g. framed arches, portal frames, special care should be taken to avoid distortion during hoisting from the horizontal to the vertical position.

10.7 Control

(1) It is assumed that a control plan comprises:
- production and workmanship control off and on site;
- control after completion of the structure.

NOTE 1: The control of the construction is assumed to include:
- preliminary tests, e.g. tests for suitability of materials and production methods;
- checking of materials and their identification e.g.:
  - for wood and wood-based materials: species, grade, marking, treatments and moisture content;
  - for glued constructions: adhesive type, production process, glue-line quality;
  - for fasteners: type, corrosive protection;
- transport, site storage and handling of materials;
- checking of correct dimensions and geometry;
- checking of assembly and erection;
- checking of structural details, e.g.:
  - number of nails, bolts etc.;
  - sizes of holes, correct pre-drilling;
  - spacings and distances to end and edge of members;
  - splitting;
- final checking of the result of the production process, e.g. by visual inspection or proof loading.

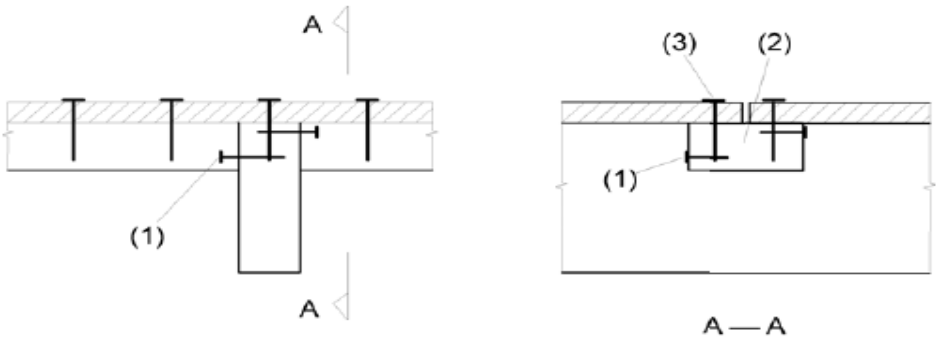
NOTE 2: A control program is assumed to specify the control measures (inspection maintenance) to be carried out in service where long-term compliance with the basic assumptions for the project is not adequately ensured.

NOTE 3: All the information required for the use in service and the maintenance of a structure is assumed to be made available to the person or authority who undertakes responsibility for the finished structure.

10.8 Special rules for diaphragm structures

10.8.1 Floor and roof diaphragms

(1) The simplified method of analysis given in 9.2.3.2 assumes that sheathing panels not supported by joists or rafters are connected to each other e.g. by means of battens as shown in Figure 10.1. Nails other than smooth nails, as defined in ES EN 14592, or screws should be used, with a maximum spacing along the edges of the sheathing panels of 150 mm. Elsewhere the maximum spacing should be 300 mm.



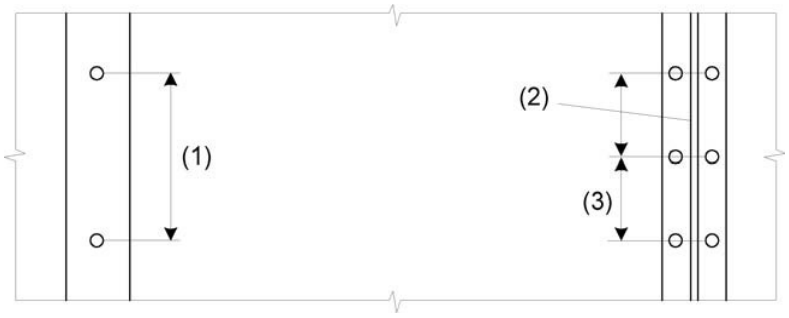
Key:

(1) Batten slant nailed to joist or rafter
(2) Batten
(3) Sheathing nailed to batten

Figure 10.1 – Example of connection of panels not supported by a joist or a rafter

10.8.2 Wall diaphragms

(1) The simplified methods of analysis given in 9.2.4.2 and 9.2.4.3 assume that panel fixings have a maximum fastener spacing along the edges of 150 mm for nails, and 200 mm for screws. On internal studs the maximum spacing should be no more than twice the spacing along the edge or 300 mm, whichever is the lesser. See Figure 10.2.



Key:
(1) Maximum nail spacing 300 mm to intermediate studs
(2) Panel edge
(3) Maximum nail spacing 150 mm

Figure 10.2 – Panel fixings

10.9 Special rules for trusses with punched metal plate fasteners

10.9.1 Fabrication

Note: Requirements for the fabrication of trusses are given in ES EN 14250.

10.9.2 Erection

(1) Trusses should be checked for straightness and vertical alignment prior to fixing the permanent bracing.

(2) When trusses are fabricated, the members should be free from distortion within the limits given in ES EN 14250. However, if members which have distorted during the period between fabrication and erection can be straightened without damage to the timber or the joints and maintained straight, the truss may be considered satisfactory for use.

(3) The maximum bow a_{bow} in any truss member after erection should be limited. Provided that it is adequately secured in the completed roof to prevent the bow from increasing, the permitted value of the maximum bow should be taken as $a_{bow,perm}$.

Note: The recommended range of $a_{bow,perm}$ is 10 to 50 mm.

(4) The maximum deviation a_{dev} of a truss from true vertical alignment after erection should be limited. The permitted value of the maximum deviation from true vertical alignment should be taken as $a_{dev,perm}$.

Note: The recommended range of $a_{dev,perm}$ is 10 to 50 mm.

Annex A (Informative): Block shear and plug shear failure at multiple dowel-type steel-to-timber connections

(1) For steel-to-timber connections comprising multiple dowel-type fasteners subjected to a force component parallel to grain near the end of the timber member, the characteristic load-carrying capacity of fracture along the perimeter of the fastener area, as shown in Figure A.1 (block shear failure) and Figure A.2 (plug shear failure), should be taken as:

$$F_{bs,Rk} = \max \begin{cases} 1.5\, A_{net,t} f_{t,0,k} \\ 0.7\, A_{net,v} f_{v,k} \end{cases} \tag{A.1}$$

With

$$A_{net,t} = L_{net,t} t_1 \tag{A.2}$$

$$A_{net,v} = \begin{cases} L_{net,v}\, t_1 \vdots & failure\ \mathrm{mod}\ es\ (e, f, j/I, k, m) \\ \dfrac{L_{net,v}}{2}\left(L_{net,t} + 2t_{ef}\right) & all\ other\ failure\ \mathrm{mod}\ es \end{cases} \tag{A.3}$$

and

$$L_{net,v} = \sum_i l_{v,i} \tag{A.4}$$

$$L_{net,t} = \sum_i l_{t,i} \tag{A.5}$$

– for thin steel plates (for failure modes given in brackets)

$$t_{ef} = \begin{cases} 0.4\, t_1 & (a) \\ 1.4\sqrt{\dfrac{M_{y,Rk}}{f_{h,k} d}} & (b) \end{cases} \tag{A.6}$$

– for thick steel plates (for failure modes given in brackets)

$$t_{ef} = \begin{cases} 2\sqrt{\dfrac{M_{y,Rk}}{f_{h,k}\, d}} & (d)(h) \\ t_1\left[\sqrt{2 + \dfrac{M_{y,Rk}}{f_{h,k} d\, t_1^2}} - 1\right] & (c)(g) \end{cases} \tag{A.7}$$

where

$F_{bs,Rk}$ is the characteristic block shear or plug shear capacity;

$A_{net,t}$ is the net cross-sectional area perpendicular to the grain;

$A_{net,v}$ is the net shear area in the parallel to grain direction;

$L_{net,t}$ is the net width of the cross-section perpendicular to the grain;

$L_{net,v}$ is the total net length of the shear fracture area;

$\ell_{v,i}, \ell_{t,i}$ are defined in figure A.1;

t_{ef} is the effective depth depending of the failure mode of the fastener, see Figure 8.3;

t_1 is the timber member thickness or penetration depth of the fastener;

$M_{y,Rk}$ is the characteristic yield moment of the fastener;

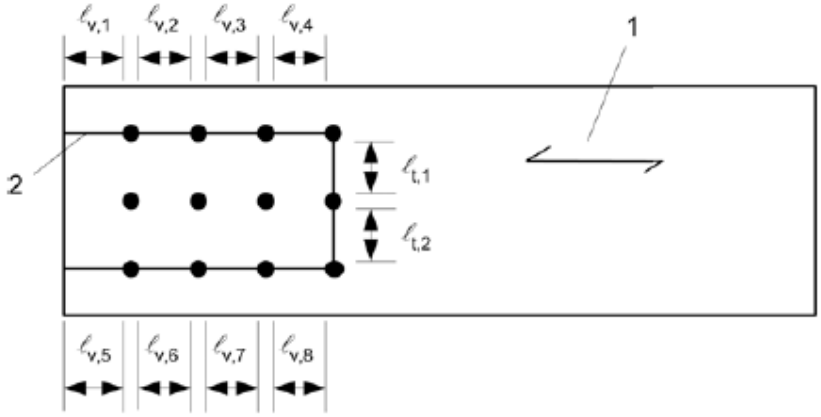
d is the fastener diameter;

$f_{t,0,k}$ is the characteristic tensile strength of the timber member;

$f_{v,k}$ is the characteristic shear strength of the timber member

$f_{h,k}$ is the characteristic embedding strength of the timber member.

NOTE: The failure modes associated with expressions (A.3), (A.6) and (A.7) are shown in Figure 8.3.



Key:

1 Grain direction

2 Fracture line

Figure A.1 – Example of block shear failure

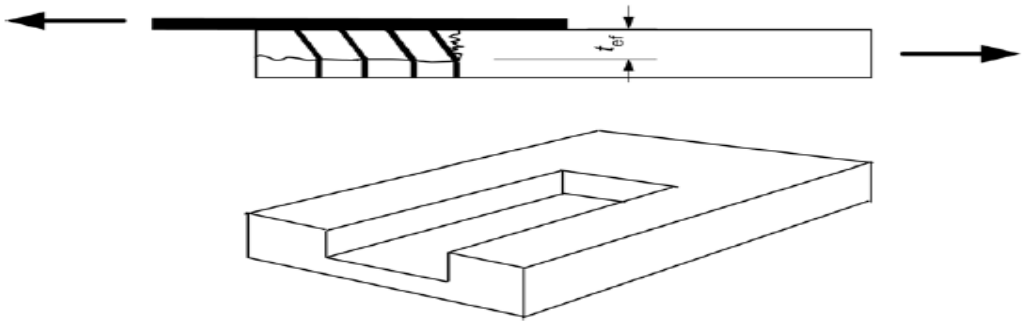


Figure A.2 – Example of plug shear failure

Annex B (Informative): Mechanically jointed beams

B.1 Simplified analysis

B.1.1 Cross-sections

(1) The cross-sections shown in Figure B.1 are considered in this annex.

B.1.2 Assumptions

(1) The design method is based on the theory of linear elasticity and the following assumptions:

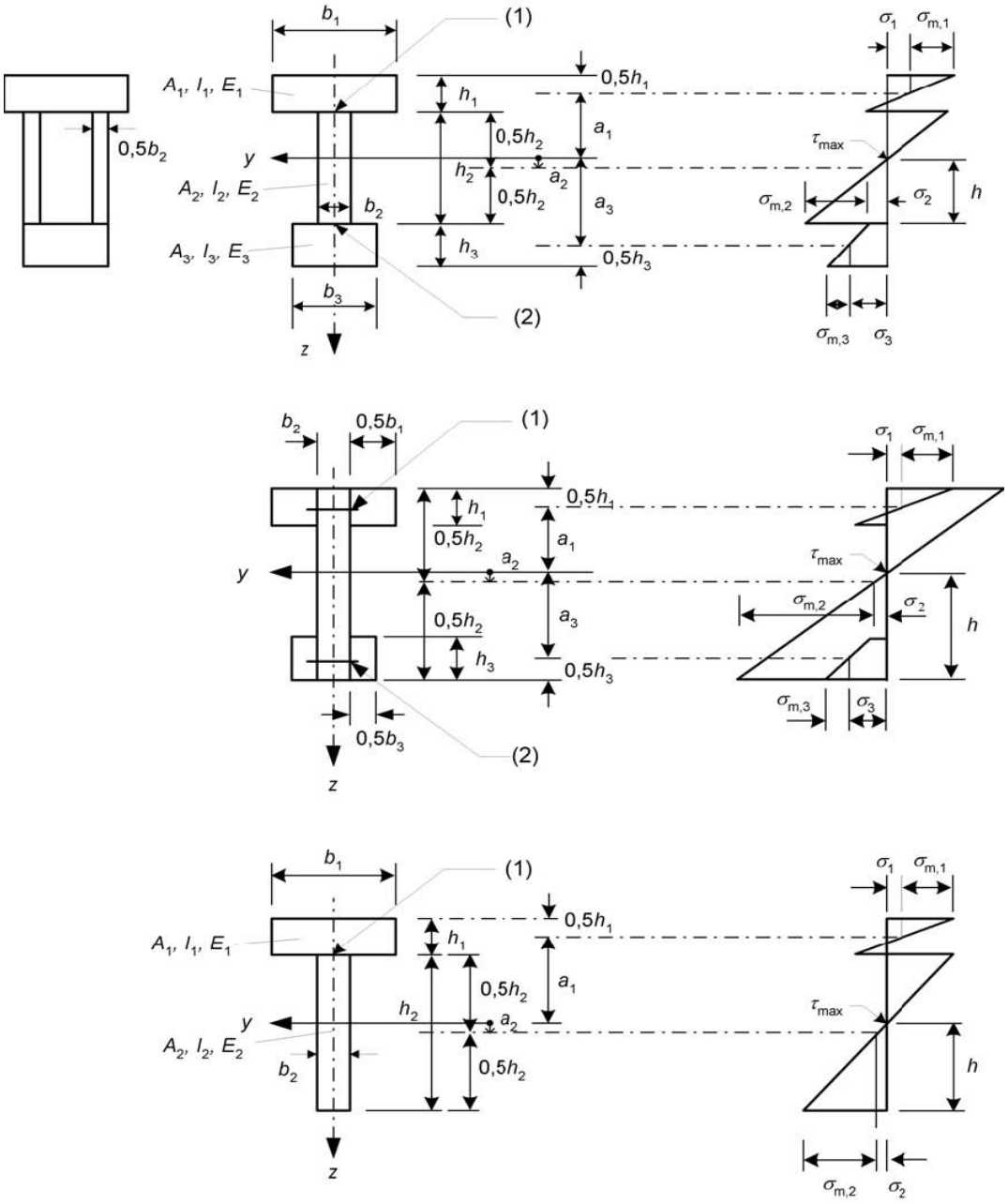
- the beams are simply supported with a span ℓ. For continuous beams the expressions may be used with ℓ equal to 0.8 of the relevant span and for cantilevered beams with ℓ equal to twice the cantilever length
- the individual parts (of wood, wood-based panels) are either full length or made with glued end joints
- the individual parts are connected to each other by mechanical fasteners with a slip modulus *K*
- the spacing *s* between the fasteners is constant or varies uniformly according to the shear force between $s_{\min}$ and $s_{\max}$, with $s_{\max} \leq 4\, s_{\min}$
- the load is acting in the z-direction giving a moment *M = M(x)* varying sinusoidally or parabolically and a shear force *V = V(x)*.

B.1.3 Spacings

(1) Where a flange consists of two parts jointed to a web or where a web consists of two parts (as in a box beam), the spacing s_i is determined by the sum of the fasteners per unit length in the two jointing planes.

B.1.4 Deflections resulting from bending moments

(1) Deflections are calculated by using an effective bending stiffness $(EI)_{ef}$ determined in accordance with B.2.



Key:

(1) spacing: s_1	slip modulus: K_1	load: F_1
(2) spacing: s_3	slip modulus: K_3	load: F_3

Figure B.1 – Cross-section (left) and distribution of bending stresses (right). All measurements are positive except for $a_2$which is taken as positive as shown.

B.2 Effective bending stiffness

(1) The effective bending stiffness should be taken as:

$$(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2) \quad \text{(B.1)}$$

Using mean values of E and where:

$$A_i = b_i h_i \quad \text{(B.2)}$$

$$I_i = \frac{b_i h_i^3}{12} \quad \text{(B.3)}$$

$$\gamma_2 = 1 \quad \text{(B.4)}$$

$$\gamma_i = \left[1 + \pi^2 E_i A_i S_i / (k_i l^2)\right]^{-1} \qquad \text{for } i = 1 \text{ and } i = 3 \quad \text{(B.5)}$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2\sum_{i=1}^{2} \gamma_i E_i A_i} \quad \text{(B.6)}$$

where the symbols are defined in Figure B.1;

$K_i = K_{ser,i}$ for the serviceability limit state calculations;

$K_i = K_{u,i}$ for the ultimate limit state calculations.

For T-sections $h_3 = 0$

B.3 Normal stresses

(1) The normal stresses should be taken as:

$$\sigma_i = \frac{\gamma_i E_i a_i M}{(EI)_{ef}} \quad \text{(B.7)}$$

$$\sigma_{m,i} = \frac{0.5 E_i h_i M}{(EI)_{ef}} \quad \text{(B.8)}$$

B.4 Maximum shear stress

(1) The maximum shear stresses occur where the normal stresses are zero. The maximum shear stresses in the web member (part 2 in Figure B.1) should be taken as:

$$\tau_{2,\max} = \frac{\gamma_3 E_3 A_3 a_3 + 0.5 E_2 b_2 b_2^2}{b_2 (EI)_{ef}} V \quad \text{(B.9)}$$

where:

i = 1 and 3, respectively;

$s_i = s_i(x)$ is the spacing of the fasteners as defined in B.1.3(1).

B.5Fastener load

(1) The load on a fastener should be taken as:

$$F_i = \frac{\gamma_i E_i A_i a_i s_i}{(EI)_{ef}} V \qquad \text{(B.10)}$$

Where:

i= 1 and 3, respectively;

$s_i = s_i(x)$is the spacing of the fasteners as defined in B.1.3(1).

Annex C (Informative): Built-up columns

C.1 General

C.1.1 Assumptions

(1) The following assumptions apply:

- the columns are simply supported with a length ℓ;
- the individual parts are full length;
- the load is an axial force F_c acting at the geometric centre of gravity, (see however C.2.3).

C.1.2 Load-carrying capacity

(1) For column deflection in the y-direction (see Figure C.1 and Figure C.3) the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual members.

(2) For column deflection in the z-direction (see Figure C.1 and Figure C.3) it should be verified that:

$$\sigma_{c,0,d} \le k_c f_{c,0,d} \qquad \text{(C.1)}$$

where:

$$\sigma_{c,0,d} = \frac{F_{c,d}}{A_{tot}} \qquad \text{(C.2)}$$

where:

A_{tot} is the total cross-sectional area;

k_c is determined in accordance with 6.3.2 but with an effective slenderness ratio λ_{ef} determined in accordance with sections C.2 - C.4.

C.2 Mechanically jointed columns

C.2.1 Effective slenderness ratio

(1) The effective slenderness ratio should be taken as:

$$\lambda_{ef} = \ell \sqrt{\frac{A_{tot}}{I_{ef}}} \qquad \text{(C.3)}$$

with

$$I_{ef} = \frac{(EI)_{ef}}{E_{mean}} \qquad \text{(C.4)}$$

where $(EI)_{ef}$ is determined in accordance with Annex B (informative).

C.2.2 Load on fasteners

(1) The load on a fastener should be determined in accordance with Annex B (informative), where

$$v_d = \begin{cases} \dfrac{F_{c,d}}{120\,k_c} & for\ \lambda_{ef} < 30 \\ \dfrac{F_{c,d}\lambda_{ef}}{3600\,k_c} & for\ 30 \le \lambda_{ef} < 60 \\ \dfrac{F_{c,d}}{60\,k_c} & for\ 60 \le \lambda_{ef} \end{cases} \quad \text{(C.5)}$$

C.2.3 Combined loads

(1) In cases where small moments (e.g. from self weight) are acting in adition to axial load, 6.3.2(3)applies.

C.3 Spaced columns with packs or gussets

C.3.1 Assumptions

(1) Columns as shown in Figure C.1 are considered, i.e. columns comprising shafts spaced by packs or gussets. The joints may be either nailed or glued or bolted with suitable connectors.

(2) The following assumptions apply:

- the cross-section is composed of two, three or four identical shafts;
- the cross-sections are symmetrical about both axes;
- the number of unrestrained bays is at least three, i.e. the shafts are at least connected at the ends and at the third points;
- the free distance *a* between the shafts is not greater than three times the shaft thickness *h* for columns with packs and not greater than 6 times the shaft thickness for columns with gussets;
- the joints, packs and gussets are designed in accordance with C.2.2;
- the pack length ℓ_2 satisfies the condition: $\ell_2 / a \ge 1.5$;
- there are at least four nails or two bolts with connectors in each shear plane. For nailed joints there are at least four nails in a row at each end in the longitudinal direction of the column;
- the gussets satisfies the condition: $\ell_2 / a \ge 2$;
- the columns are subjected to concentric axial loads.

(3) For columns with two shafts A_{tot} and I_{tot} should be calculated as

$$A_{tot} = 2A \quad \text{(C.6)}$$

$$I_{tot} = \frac{b\left[(2h+a)^3 - a^3\right]}{12} \quad \text{(C.7)}$$

(4) For columns with three shafts A_{tot} and I_{tot} should be calculated as

$$A_{tot} = 3A \quad \text{(C.8)}$$

$$I_{tot} = \frac{b\left[(3h+2a)^3 - (h+2a)^3 + h^3\right]}{12} \quad \text{(C.9)}$$

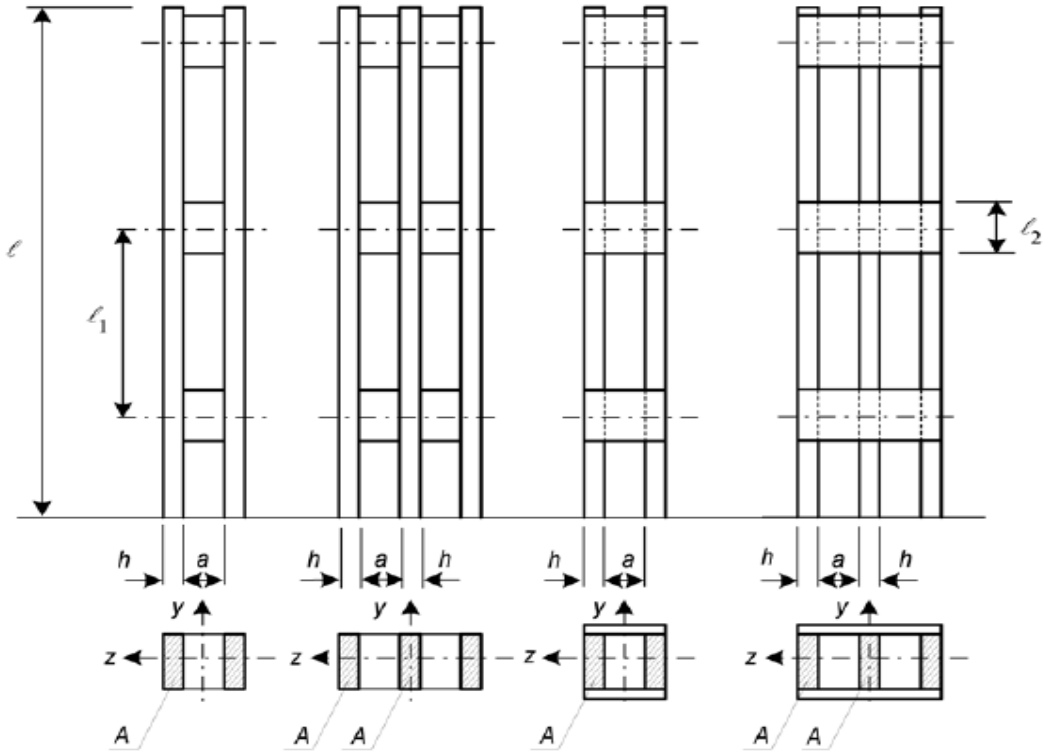


Figure C.1 – Spaced columns

C.3.2 Axial load-carrying capacity

(1) For column deflection in the y-direction (see Figure C.3) the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual members.

(2) For column deflection in the z-direction C.1.2 applies with

$$\lambda_{ef} = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \quad \text{(C.10)}$$

where:

λ is the slenderness ratio for a solid column with the same length, the same area (A_{tot}) and the same second moment of area (I_{tot}), i.e.,

$$\lambda = \ell\sqrt{A_{tot} / I_{tot}} \quad \text{(C.11)}$$

λ_1 is the slenderness ratio for the shafts and has to be set into expression (C.10) with a minimum value of at least 30, i.e.

$$\lambda_1 = \sqrt{12}\,\frac{\ell_1}{h} \quad \text{(C.12)}$$

n is the number of shafts;

η is a factor given in Table C.1.

Table C.1 – The factor η

	Packs			**Gussets**	
	Glued	Nailed	Bolted[a]	Glued	Nailed
Permanent/long-term loading	1	4	3.5	3	6
Medium/short-term loading	1	3	2.5	2	4.5
[a] with connectors					

C.3.3 Load on fasteners, gussets or packs

(1) The load on the fasteners and the gussets or packs are as shown in Figure C.2 with V_d according to section C.2.2.

(2) The shear forces on the gussets or packs, see Figure C.2, should be calculated from:

$$T_d = \frac{V_d l_1}{a_1} \quad \text{(C.13)}$$

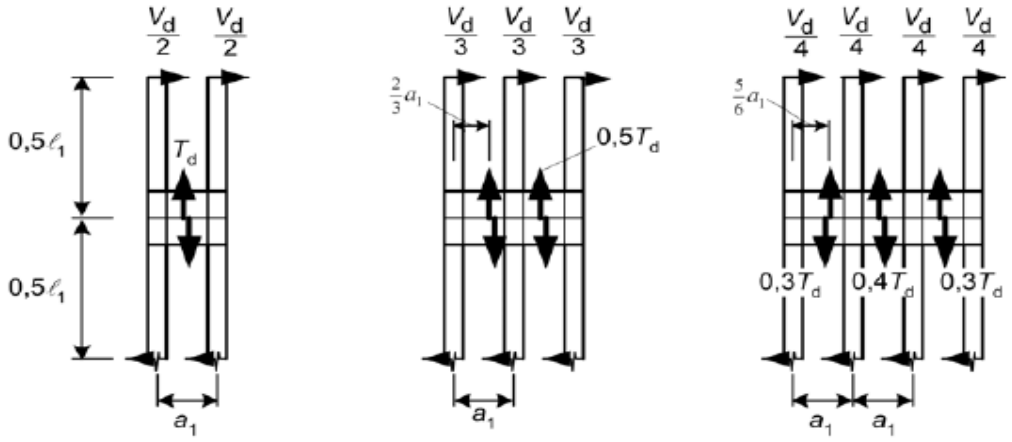


Figure C.2 – Shear force distribution and loads on gussets or packs

C.4 Lattice columns with glued or nailed joints

C.4.1 Assumptions

(1) Lattice columns with N- or V-lattice configurations and with glued or nailed joints are considered in this section, see Figure C.3.

(2) The following assumptions apply:

- the structure is symmetrical about the y- and z-axes of the cross-section. The lattice on the two sides may be staggered by a length of $l_1/2$, where l_1 is the distance between the nodes;
- there are at least three bays;
- in nailed structures there are at least four nails per shear plane in each diagonal at each nodal point; each end is braced;
- the slenderness ratio of the individual flange corresponding to the node length ℓ_1 is not greater than 60;
- no local buckling occurs in the flanges corresponding to the column length ℓ_1;
- the number of nails in the verticals (of an N-truss) is greater than $n \sin\theta$, where n is the number of nails in the diagonals and θ is the inclination of the diagonals.

C.4.2 Load-carrying capacity

- (1) For column deflection in the y-direction (see Figure C.2), the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual flanges.
- (2) For column deflection in the z-direction C.1.2 applies with

$$\lambda_{ef} = \max \begin{cases} \lambda_{tot} \sqrt{1+\mu} \\ 1.05\,\lambda_{tot} \end{cases} \qquad \text{(C.14)}$$

where:

λ_{tot} is the slenderness ratio for a solid column with the same length, the same area and the same second moment of area, i.e.

$$\lambda_{tot} \approx \frac{2\ell}{h} \tag{C.15}$$

μ takes the values given in (3) to (6) below.

(3) For a glued V-truss:

$$\mu = 4\frac{e^2 A_f}{I_f}\left(\frac{h}{\ell}\right)^2 \tag{C.16}$$

where(see Figure C.3):

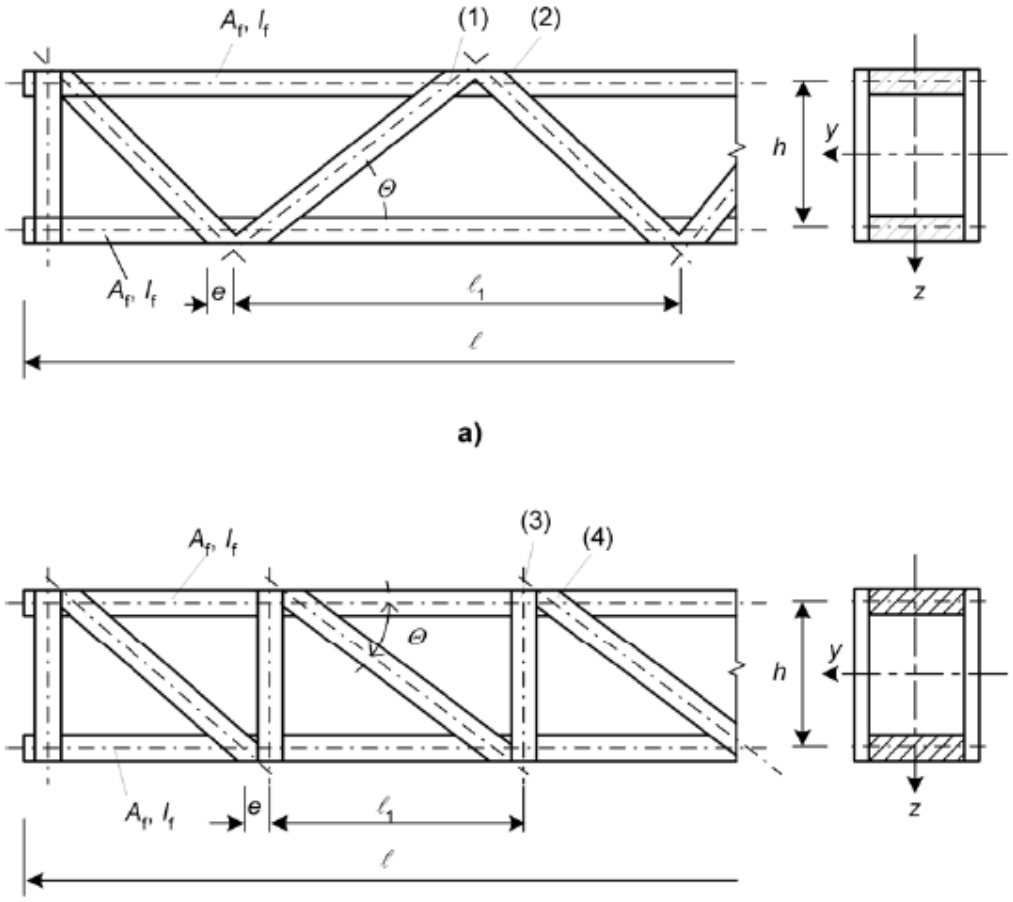
e is the eccentricity of the joints;

A_f is the area of the flange;

I_f is the second moment of area of the flange;

ℓ is the span;

h is the distance of the flanges.



Key
(1) number of nails: n
(2) number of nails: n
(3) number of nails: $\geq n \sin\theta$
(4) number of nails: n

Figure C.3 – Lattice columns: (a) V-truss, (b) N-truss

(4) For a glued n-truss:

$$\mu = \frac{e^2 A_f}{I_f}\left(\frac{h}{\ell}\right)^2 \tag{C.17}$$

(5) For a nailed V-truss:

$$\mu = 25\frac{h\, E_{mean} A_f}{\ell^2 n\, K_u \sin 2\theta} \tag{C.18}$$

where:

n is the number of nails in a diagonal. If a diagonal consists of two or more pieces, n is the sum of the nails (not the number of nails per shear plane);

E_{mean} is the mean value of modulus of elasticity; K_u is the slip modulus of one nail in the ultimate limit state.

(6) For a nailed N-truss:

$$\mu = 50 \frac{h\, E_{mean}\, A_f}{\ell^2 n\, K_u \sin 2\theta} \qquad \text{(C.19)}$$

where:

n is the number of nails in a diagonal. If a diagonal consists of two or more pieces, *n* is the sum of the nails (not the number of nails per shear plane);

K_u is the slip modulus of one nail for the ultimate limit states.

C.4.3 Shear forces

(1) C.2.2 applies.

Annex D (Informative): Bibliography

EN 338 Structural timber – Strength classes

EN 1194 Glued laminated timber – Strength classes and determination of characteristic values