prEN 1995-1-1

Eurocode 5
Design of timber structures

Part 1.1
General Rules
General rules and rules for buildings

Final Draft (Stage 34)
# Final draft of EN 1995-1-1

## Foreword

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Foreword

This European Standard EN 1995-1-1, Eurocode : Design of timber structures, Part 1.1 : General Rules, General rules and rules for buildings, has been prepared on behalf of Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

The text of the draft standard was submitted to the formal vote and was approved by CEN as EN 1995-1-1 on YYYY-MM-DD.

No existing European Standard is superseded.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement¹ between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Eurocodes with the provisions of all the Council’s Directives and/or Commission’s Decisions.

¹ Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

<table>
<thead>
<tr>
<th>Year</th>
<th>Eurocode</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990</td>
<td>EN 1990</td>
<td>Eurocode : Basis of Structural Design</td>
</tr>
<tr>
<td>1991</td>
<td>EN 1991</td>
<td>Eurocode 1: Actions on structures</td>
</tr>
<tr>
<td>1993</td>
<td>EN 1993</td>
<td>Eurocode 3: Design of steel structures</td>
</tr>
<tr>
<td>1994</td>
<td>EN 1994</td>
<td>Eurocode 4: Design of composite steel and concrete structures</td>
</tr>
<tr>
<td>1995</td>
<td>EN 1995</td>
<td>Eurocode 5: Design of timber structures</td>
</tr>
<tr>
<td>1996</td>
<td>EN 1996</td>
<td>Eurocode 6: Design of masonry structures</td>
</tr>
<tr>
<td>1997</td>
<td>EN 1997</td>
<td>Eurocode 7: Geotechnical design</td>
</tr>
<tr>
<td>1999</td>
<td>EN 1999</td>
<td>Eurocode 9: Design of aluminium structures</td>
</tr>
</tbody>
</table>

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

**Status and field of application of Eurocodes**

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:


- as a basis for specifying contracts for construction works and related engineering services ;

- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode 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.

**National Standards implementing Eurocodes**

\[^{2}\] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

\[^{3}\] According to Art. 12 of the CPD the interpretative documents shall :

- a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
- b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
- c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.
The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e.:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode,
- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

**Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products**

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

**Additional information specific to EN 1995-1-1**

EN 1995 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, EN 1995 is intended to be used, for direct application, together with EN 1990 and relevant Parts of EN 1991.

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. When EN 1995 is used as a base document by other CEN/TCs the same values need to be taken.

**National annex for EN 1995-1-1**

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1995 should have a National annex giving advice on all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1995-1-1 through clauses:

- 2.4.1 (1)P Partial factors for material properties
- 2.3.1.2 (2)P Assignment of loads to load duration classes
- 7.2(2) Limiting values for deflections
- 7.3.3(2) Limiting values for vibrations
- 7.3.3(5) Design methods for floors in vibration.

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4 see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.
SECTION 1  GENERAL

1.1  SCOPE

1.1.1  SCOPE OF EUROCODE 5

(1) Eurocode 5 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 for example 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 their design and verification that are given in prEN 1990 – Basis of structural design.

(2) Eurocode 5 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) Eurocode 5 is intended to be used in conjunction with:
   EN 1990 “Basis of structural design”
   EN 1991 “Actions on structures”
   EN’s for construction products relevant for timber structures
   EN 1998 “Design of structures for earthquake resistance”, when timber structures are built in seismic regions

(4) Eurocode 5 is subdivided in various parts:
   EN 1995-1  General rules
   EN 1995-2  Bridges

(5) EN 1995-1 “General rules” comprises:
   EN 1995-1-1 General rules - General rules and rules for buildings
   EN 1995-1-2 General rules - Structural Fire Design


1.1.2  SCOPE OF EN 1995-1-1

(1) Part 1.1 of Eurocode 5 gives general design rules for timber structures which are referenced in Part 2 for timber bridge design.

(2) The following subjects are dealt with in Part 1.1:
   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) SECTION 1 and SECTION 2 provide additional clauses to those given in prEN 1990 “Basis of structural design”.

(4) SECTION 3 deals with material properties.
(5) SECTION 4 gives general rules for durability.

(6) SECTION 5 refers to the basis of structural analysis.

(7) SECTION 6 gives detailed rules for the design of cross sections and members in the ultimate limit state.

(8) SECTION 7 gives detailed rules for serviceability.

(9) SECTION 8 gives detailed rules for connections.

(10) SECTION 9 gives detailed rules for components and assemblies

(11) SECTION 10 gives rules for structural detailing and control.

1.2 NORMATIVE REFERENCES

(1) The following normative documents contain provisions which, through references in this text, constitute provisions of this European standard. For dated references, subsequent amendments to or revisions of any of these publications do not apply. However, parties to agreements based on this European standard are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references the latest edition of the normative document referred to applies.

ISO standards

ISO 2081 Metallic coatings. Electroplated coatings of zinc on iron or steel

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

European Standards

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

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

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

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

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

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

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

EN 335-2 Durability of wood and wood-based products - definition of hazard classes of biological attack. Part 2: Application to solid wood
<table>
<thead>
<tr>
<th>Standard Code</th>
<th>Description</th>
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<tr>
<td>EN 335-3</td>
<td>Durability of wood and wood-based products – Definition of hazard classes of biological attack Part 3: Application to wood based panels</td>
</tr>
<tr>
<td>EN 350-2</td>
<td>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 Europe</td>
</tr>
<tr>
<td>EN 351-1</td>
<td>Durability of wood and wood-based products – Preservative treated solid wood. Part 1: Classification of preservative penetration and retention</td>
</tr>
<tr>
<td>EN 383</td>
<td>Timber structures - Test methods. Determination of embedding strength and foundation values for dowel type fasteners</td>
</tr>
<tr>
<td>EN 385</td>
<td>Finger jointed structural timber. Performance requirements and minimum production requirements</td>
</tr>
<tr>
<td>ENV 387</td>
<td>Glued laminated timber - Production requirements for large finger joints. Performance requirements and minimum product requirements</td>
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<tr>
<td>EN 409</td>
<td>Timber structures - Test methods. Determination of the yield moment of dowel type fasteners – Nails</td>
</tr>
<tr>
<td>EN 460</td>
<td>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</td>
</tr>
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<td>EN 594</td>
<td>Test method for shear walls</td>
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<td>EN 622-2</td>
<td>Fibreboards - Specifications - Part 2: Requirements for Hardboards</td>
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<td>Fibreboards - Specifications. Part 3: Requirements for medium boards</td>
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<td>Fibreboards - Specifications. Part 4: Requirements for soft boards</td>
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<td>EN 622-5</td>
<td>Fibreboards - Specifications. Part 5: Requirements for dry process boards</td>
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<td>EN 636-1</td>
<td>Plywood - Specifications. Part 1: Requirements for plywood for use in dry conditions</td>
</tr>
<tr>
<td>EN 636-2</td>
<td>Plywood - Specifications. Part 2: Requirements for plywood for use in humid conditions</td>
</tr>
<tr>
<td>EN 636-3</td>
<td>Plywood - Specifications. Part 3: Requirements for plywood for use in exterior conditions</td>
</tr>
<tr>
<td>EN 912</td>
<td>Timber fasteners – Specifications for connectors for timber</td>
</tr>
<tr>
<td>EN TC 124-1.3</td>
<td>Timber structures. Production requirements for fabricated trusses using punched metal plate fasteners</td>
</tr>
<tr>
<td>EN 1075</td>
<td>Timber structures - Test methods. Testing of joints made with punched metal plate fasteners</td>
</tr>
</tbody>
</table>
2.2 Assumptions

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

2.3 Distinction between principles and application rules

(1)P The rules in EN 1990 clause 1.4 apply.
1.3 DEFINITIONS

(1) The definitions of EN 1990 clause 1.5 apply.

(2) The following terms are used in Part 1-1 of EN 1995-1-1 with the following meanings:

1.3.1 Balanced plywood
A plywood in which the outer and inner plies are symmetrical about the centre plane with respect to thickness and species.

1.3.2 Characteristic value
The characteristic value is normally that value which has a prescribed probability of not being attained in a hypothetical unlimited test series, i.e., a fractile in the distribution of the property. The characteristic value is called a lower or upper characteristic value if the prescribed value is less or greater than 0.50 respectively.

1.3.3 Dowel
Circular cylindrical rod usually of steel fitting tightly in prebored holes and used for transferring loads perpendicular to the dowel axis.

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

1.3.5 Fibre saturation point
Moisture content at which the wood cells are completely saturated.

1.3.6 LVL
Laminated Veneer Lumber, defined according to prEN 124-aaa

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

1.3.8 Laminated timber floor
as laminated timber deck

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

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

1.3.11 Stiffness property
A property used in the calculation of the deformation of the structure, e.g. modulus of elasticity, shear modulus.

1.3.12 Slip modulus
A property used in the calculation of the deformation between two members of a structure.
1.4 SYMBOLS USED IN PART 1-1 OF EN 1995

For the purpose of this Part 1-1 of EN 1995, the following symbols apply.

<table>
<thead>
<tr>
<th>LATIN UPPER CASE LETTERS</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Cross-section</td>
</tr>
<tr>
<td>$A_{ef}$</td>
<td>Area of the total contact surface between a punched metal plate fastener and the timber</td>
</tr>
<tr>
<td>$A_{net,t}$</td>
<td>Net cross section perpendicular to the grain</td>
</tr>
<tr>
<td>$A_{net,v}$</td>
<td>Net shear area parallel to the grain</td>
</tr>
<tr>
<td>$E_{0,05}$</td>
<td>Fifth percentile value of a stiffness property</td>
</tr>
<tr>
<td>$E_d$</td>
<td>Design member stiffness</td>
</tr>
<tr>
<td>$E_{mean}$</td>
<td>Instantaneous mean value of a stiffness property</td>
</tr>
<tr>
<td>$E_{mean,fin}$</td>
<td>Mean value of stiffness property used to calculate the final state of a structure</td>
</tr>
<tr>
<td>$F$</td>
<td>Force</td>
</tr>
<tr>
<td>$F_{A,d}$</td>
<td>Design force acting on a punched metal plate fastener at the centroid of the effective area</td>
</tr>
<tr>
<td>$F_{A,min,d}$</td>
<td>Minimum design force acting on a punched metal plate fastener at the centroid of the effective area</td>
</tr>
<tr>
<td>$F_{ax}$</td>
<td>Axial load</td>
</tr>
<tr>
<td>$F_c$</td>
<td>Compressive force</td>
</tr>
<tr>
<td>$F_d$</td>
<td>Design force</td>
</tr>
<tr>
<td>$F_{d,ser}$</td>
<td>Design force at the serviceability limit state</td>
</tr>
<tr>
<td>$F_{la}$</td>
<td>Lateral load</td>
</tr>
<tr>
<td>$F_{M,d}$</td>
<td>Design force from a design moment</td>
</tr>
<tr>
<td>$F_i$</td>
<td>Tensile force</td>
</tr>
<tr>
<td>$F_{x,d}$</td>
<td>Design value of a force in x direction</td>
</tr>
<tr>
<td>$F_{y,d}$</td>
<td>Design value of a force in y direction</td>
</tr>
<tr>
<td>$G_{0,05}$</td>
<td>Fifth percentile value of shear modulus</td>
</tr>
<tr>
<td>$H$</td>
<td>Overall rise of a trussed rafter</td>
</tr>
<tr>
<td>$I_{ax}$</td>
<td>Torsional moment of inertia</td>
</tr>
<tr>
<td>$I_z$</td>
<td>Second moment of area about the weak axis</td>
</tr>
<tr>
<td>$K_{ser}$</td>
<td>Instantaneous slip modulus</td>
</tr>
<tr>
<td>$K_u$</td>
<td>Instantaneous slip modulus for ultimate limit states</td>
</tr>
<tr>
<td>$L_{net,t}$</td>
<td>Net width of the cross section perpendicular to the grain</td>
</tr>
<tr>
<td>$L_{net,v}$</td>
<td>Net length of the fracture area in shear</td>
</tr>
<tr>
<td>$M_{A,d}$</td>
<td>Design moment acting on a punched metal plate fastener</td>
</tr>
<tr>
<td>$M_{sp,d}$</td>
<td>Design moment at apex zone</td>
</tr>
<tr>
<td>$M_d$</td>
<td>Design moment</td>
</tr>
<tr>
<td>$M_{y,k}$</td>
<td>Characteristic fastener yield moment</td>
</tr>
<tr>
<td>$N$</td>
<td>Axial force</td>
</tr>
<tr>
<td>$R_{90,d}$</td>
<td>Design splitting capacity</td>
</tr>
<tr>
<td>$R_{90,k}$</td>
<td>Characteristic splitting capacity</td>
</tr>
<tr>
<td>$R_{ax,d}$</td>
<td>Design load carrying capacity of a connection axially loaded</td>
</tr>
<tr>
<td>$R_{ax,k}$</td>
<td>Characteristic load carrying capacity</td>
</tr>
<tr>
<td>$R_{ax,a,k}$</td>
<td>Characteristic load carrying capacity at an angle to grain</td>
</tr>
<tr>
<td>$R_d$</td>
<td>Design value of a load carrying capacity</td>
</tr>
<tr>
<td>$R_{ef,k}$</td>
<td>Effective characteristic load carrying capacity of a connection</td>
</tr>
<tr>
<td>$R_{lk}$</td>
<td>Characteristic load-carrying capacity per fastener</td>
</tr>
<tr>
<td>$R_{la,d}$</td>
<td>Design racking load-carrying capacity of a wall</td>
</tr>
<tr>
<td>$R_k$</td>
<td>Characteristic load carrying capacity</td>
</tr>
<tr>
<td>$R_{la,d}$</td>
<td>Design load carrying capacity of a connection laterally loaded</td>
</tr>
<tr>
<td>$R_{sp,k}$</td>
<td>Characteristic splitting capacity</td>
</tr>
<tr>
<td>$R_{to,k}$</td>
<td>Characteristic load carrying capacity of a toothed plate connector</td>
</tr>
<tr>
<td>$R_{v,d}$</td>
<td>Design load carrying capacity of a wall diaphragm</td>
</tr>
<tr>
<td>$R_{x,d}$</td>
<td>Design value of a plate capacity in x direction</td>
</tr>
<tr>
<td>$R_{y,d}$</td>
<td>Design value of a plate capacity in y direction</td>
</tr>
<tr>
<td>$V$</td>
<td>Volume</td>
</tr>
<tr>
<td>$W_y$</td>
<td>Moment of resistance about the strong axis</td>
</tr>
<tr>
<td>$X_d$</td>
<td>Design value of a strength property</td>
</tr>
<tr>
<td>$X_k$</td>
<td>Characteristic value of a strength property</td>
</tr>
</tbody>
</table>
LATIN LOWER CASE LETTERS

- $a_1$: Spacing distance in grain direction
- $a_2$: Distance
- $a_{3,c}$: End distance to unloaded end
- $a_{3,t}$: End distance to loaded end
- $a_{4,c}$: Edge distance to unloaded edge
- $a_{4,t}$: Edge distance to loaded edge
- $b_{net}$: Clear spacing between studs
- $b_w$: Web width
- $d$: Diameter
- $d_c$: Connector diameter
- $d_{ef}$: Effective diameter
- $f_{h,i,k}$: Characteristic embedment strength of timber member $i$
- $f_{a,0,0}$: Anchorage capacity per unit area for $\alpha = 0^\circ$ and $\beta = 0^\circ$
- $f_{a,90,90}$: Anchorage capacity per unit area for $\alpha = 90^\circ$ and $\beta = 90^\circ$
- $f_{a,\alpha,\beta,k}$: Characteristic anchorage strength
- $f_{a,k}$: Characteristic withdrawal parameter for nails
- $f_{c,d}$: Design compressive strength along the grain
- $f_{c,w,d}$: Design compressive strength of the web
- $f_{c,e,d}$: Flange design compressive strength
- $f_{s,d}$: Flange design tensile strength
- $f_{b,k}$: Characteristic embedment strength
- $f_{ead,k}$: Characteristic pull through parameter for nails
- $f_i$: 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
- $f_{m,a,d}$: Design bending strength at an angle $\alpha$ to the grain
- $f_{t,0,d}$: Design 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
- $f_{v,d}$: Design shear strength
- $h$: Depth
- $h_{ap}$: Depth of the apex zone
- $h_d$: Hole depth
- $h_e$: Embedment depth
- $h_l$: Loaded edge distance
- $h_{ed}$: Effective depth
- $h_{1,c}$: Compressive flange depth
- $h_{1,t}$: Tensile flange depth
- $h_{ld}$: Lower edge distance of a hole
- $h_{ua}$: Upper edge distance of a hole
- $h_w$: Web depth
- $i$: Notch inclination
- $k_{c,y}$ or $z$: Instability factor
- $k_{cal}$: Calibration factor
- $k_{crit}$: Factor used for lateral buckling
- $k_{def}$: Deformation factor
- $k_{ds}$: Factor taking into account the distribution of stresses in an apex zone
- $k_h$: Depth factor
- $k_m$: Factor considering redistribution of bending stresses in a cross-section
- $k_{mod}$: Modification factor
- $k_t$: Reduction factor
\( k_{\text{shape}} \)  
Factor depending on the shape of the cross section

\( k_{\text{sys}} \)  
System strength factor

\( k_i \)  
Reduction factor for notched beams

\( k_{\text{vol}} \)  
Volume factor

\( k_{y,\text{in}z} \)  
Instability factor

\( \ell_{\text{a,min}} \)  
Minimum anchorage length for a glued-in rod

\( \ell \)  
Span

\( \ell_{\text{A}} \)  
Support distance of a hole

\( \ell_{\text{ef}} \)  
Effective length

\( \ell_{\text{v}} \)  
End distance of a hole

\( \ell_{\text{z}} \)  
Spacing distance between holes

\( m \)  
Number of fastener rows parallel to the grain; Mass per unit area

\( n_{\text{gf}} \)  
Number of frequencies below 40 Hz

\( n_{\text{ef}} \)  
Effective number of fasteners

\( p_a \)  
Distributed load

\( r \)  
Radius of curvature

\( r_{\text{in}} \)  
Inner radius

\( t \)  
Thickness

\( t_{\text{pen}} \)  
Penetration depth

\( u_0 \)  
Precamber

\( u_t \)  
Instantaneous deflection

\( u_{\text{creep}} \)  
Creep deflection

\( u_{\text{fin}} \)  
Final deflection

\( u_{\text{inst}} \)  
Instantaneous deformation

\( u_{\text{net}} \)  
Net deflection

\( u_{\text{net,fin}} \)  
Net final deflection

\( u_{\text{ser}} \)  
Slip at the serviceability limit state

\( u_u \)  
Slip at the ultimate limit state

\( v \)  
Unit impulse velocity response

**GREEK LOWER CASE SYMBOLS**

\( \alpha \)  
Angle between the x-direction and the force for a punched metal plate  
Angle to the direction of grain

\( \beta \)  
Angle between the grain direction and the force for a punched metal plate

\( \beta_\ell \)  
Straightness factor

\( \gamma \)  
Angle between the x-direction and the timber connection line for a punched metal plate

\( \gamma_{\text{st}} \)  
Partial factor

\( \lambda_y \)  
Slenderness ratio corresponding to bending about the y-axis

\( \lambda_z \)  
Slenderness ratio corresponding to bending about the z-axis

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

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

\( \rho_\ell \)  
Characteristic density

\( \rho_m \)  
Mean density

\( \sigma_{\ell,0,d} \)  
Design compressive stress along the grain

\( \sigma_{\ell,\alpha,d} \)  
Design compressive stress at an angle \( \alpha \) to the grain

\( \sigma_{t,0,d} \)  
Mean flange design compressive stress

\( \sigma_{t,\alpha,d} \)  
Mean flange design tensile stress

\( \sigma_{t,\alpha,\text{max},d} \)  
Extreme fibre flange design compressive stress

\( \sigma_{t,\alpha,\text{max},d} \)  
Extreme fibre flange design tensile stress

\( \sigma_{\text{crit}} \)  
Critical bending stress

\( \sigma_{\ell,y,d} \)  
Design bending stress about the principal y-axis

\( \sigma_{\ell,z,d} \)  
Design bending stress about the principal z-axis

\( \sigma_{\ell,\alpha,d} \)  
Design bending stress at an angle \( \alpha \) to the grain

\( \sigma_{\ell} \)  
Axial stress

\( \sigma_{t,0,d} \)  
Design tensile stress along the grain

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

\( \sigma_{w,\ell,d} \)  
Design compressive stress of the web
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{w,d}$</td>
<td>Design tensile stress of the web</td>
</tr>
<tr>
<td>$\tau_d$</td>
<td>Design shear stress</td>
</tr>
<tr>
<td>$\tau_F,d$</td>
<td>Design anchorage stress from axial force</td>
</tr>
<tr>
<td>$\tau_M,d$</td>
<td>Design anchorage stress from moment</td>
</tr>
<tr>
<td>$\tau_{tor,d}$</td>
<td>Design shear stress from torsion</td>
</tr>
<tr>
<td>$\psi_2$</td>
<td>Factor for quasi-permanent value of a variable action</td>
</tr>
<tr>
<td>$\zeta$</td>
<td>Modal damping ratio</td>
</tr>
</tbody>
</table>
SECTION 2  BASIS OF DESIGN

2.1  REQUIREMENTS

2.1.1  Basic requirements

(1) The design of timber structures shall be in accordance with the general rules given in EN 1990.

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

(3) The basic requirements of EN 1990 section 2 are deemed to be satisfied for timber structures when limit state design, in conjunction with the partial factor method using EN 1990 and EN 1991 for actions and their combinations and EN 1995 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 EN 1990 Annex C.

2.1.3  Design working life and durability

(1) EN 1990 clause 2.3 applies.

2.2  PRINCIPLES OF LIMIT STATE DESIGN

2.2.1  General

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

- different material properties (e.g. modulus of elasticity, strength and failure mode)
- different time dependent behaviour of the materials (creep)
- different climatic conditions for the materials (temperature, moisture variations)
- different design situations (stages of construction, change of support conditions)

2.2.2  Ultimate limit states

(1) Where a structural analysis is carried out, the stiffness properties shall be:

- the mean values for a first order linear elastic stress analysis if the members have the same time dependent (creep) properties
- the final mean values adjusted to the duration of the largest load component, as defined in 2.3.2.2(1), where the distribution of member forces and moments is affected by the degree of deformation in the structure
- the design values, see 2.4.1(2), for a second order linear elastic analysis

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

\[
K_u = \frac{2}{3} K_{ser}
\]

where:

\( K_{ser} \) is the instantaneous slip modulus, see 2.2.3(3)

(3) The slip in a connection for a design force \( F_d \) should be taken as \( u_{as} \), given by:
2.2.3 Serviceability limit states

(1) 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, \( u_{\text{inst}} \), under an action should be calculated using the mean value of the appropriate instantaneous stiffness and slip moduli.

(3) The slip modulus \( K_{\text{ser}} \) for the serviceability limit states shall be determined by testing according to the method for determining \( k_s (= K_{\text{ser}}) \) given in EN 26891, or by calculation according to 7.1

(4) The final deformation for each action, \( u_{\text{fin}} \), for members and connections should be calculated as:

\[
 u_{\text{fin}} = u_{\text{inst}} + u_{\text{creep}} = u_{\text{inst}} \left(1 + \psi_2 k_{\text{def}}\right)
\]

(5) If the structure consists of members or components having different creep behaviour, the final deformation should be calculated as the sum of the individual deformation contributions.

(6) The deformation from a combination of actions should be calculated as the combination of the contributions from the individual actions. The possibility of having simultaneous occurrence of two variable loads may be taken into account by \( \psi_6 \) factors (see EN 1990).

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) 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.

2.3.1.2 Load-duration classes

(1) The load-duration classes are characterised by the effect of a 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 interaction between the typical variation of the load with time.

\[
 u_u = 1.5 u_{\text{set}} \quad \text{with} \quad u_{\text{set}} = \frac{F_d}{K_{\text{ser}}}
\]

where

- \( u_u \) is the slip at the ultimate limit state
- \( u_{\text{set}} \) is the slip at the serviceability limit state
for strength and stiffness calculations.

### Table 2.1 - Load-duration classes

<table>
<thead>
<tr>
<th>Load-duration class</th>
<th>Order of accumulated duration of characteristic load</th>
<th>Examples of loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>more than 10 years</td>
<td>self weight</td>
</tr>
<tr>
<td>Long-term</td>
<td>6 months – 10 years</td>
<td>storage</td>
</tr>
<tr>
<td>Medium-term</td>
<td>1 week – 6 months</td>
<td>imposed floor load, snow</td>
</tr>
<tr>
<td>Short-term</td>
<td>less than one week</td>
<td>snow, wind</td>
</tr>
<tr>
<td>Instantaneous</td>
<td></td>
<td>wind and accidental load</td>
</tr>
</tbody>
</table>

**NOTE:** Since climatic loads (snow, wind) vary between countries, information on their load duration assignment may be specified in a National Annex.

#### 2.3.1.3 Service classes

(1) 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 may be given in the National Annex.

(2) Service class 1: is characterised by a moisture content in the materials corresponding to a temperature of 20°C 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) Service class 2: is characterised by a moisture content in the materials corresponding to a temperature of 20°C 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) Service class 3: 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, 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 $k_{mod}$:

\[
  k_{mod} = \sqrt{k_{mod,1} \cdot k_{mod,2}} \tag{2.4}
\]

where:

- $k_{mod,1}$ and $k_{mod,2}$ are the modification factors for the two timber elements.
2.3.2.2 Load-duration and moisture influences on deformations

(1) The final mean value \( E_{\text{mean,fin}} \) of a stiffness property should be taken as:

\[
E_{\text{mean,fin}} = \frac{E_{\text{mean}}}{(1 + \psi_2 k_{\text{def}})}
\]

where:
- \( E_{\text{mean}} \) is the mean value of a stiffness property of timber or of connection, determined by standardized tests
- \( k_{\text{def}} \) is a deformation factor taking into account the effect on the stiffness parameters of the load and the moisture content in the structure.
- \( \psi_2 \) is a factor for the quasi-permanent value of a variable action. For permanent actions, \( \psi_2 \) should be taken equal to 1.0

NOTE: Values of \( k_{\text{def}} \) are given in 3.1.4

(2) For connections, the deformation factor \( k_{\text{def}} \) should be doubled.

(3) Where a connection is constituted of two timber elements having different time dependent behaviour, the calculation of the final deformation should be made with the following deformation factor \( k_{\text{def}} \):

\[
k_{\text{def}} = 2.0 \sqrt{k_{\text{def,1}} k_{\text{def,2}}}
\]

where:
- \( k_{\text{def,1}} \) and \( k_{\text{def,2}} \) 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) The design value \( X_d \) of a strength property shall be calculated as:

\[
X_d = k_{\text{mod}} \frac{X_k}{\gamma_M}
\]

where:
- \( X_k \) is the characteristic value of a strength property
- \( \gamma_M \) is the partial factor for a material property, specified in National Annexes
- \( k_{\text{mod}} \) is a modification factor taking into account the effect of the duration of load and moisture content.

NOTE 1: Values of \( k_{\text{mod}} \) are given in 3.1.3.

NOTE 2: The recommended partial factors for material properties (\( \gamma_M \)) are given in Table 2.2. Information on Nationally determined parameters may be found in a National Annex.
Table 2.2 – Recommended partial factors for material properties ($\gamma_M$)

<table>
<thead>
<tr>
<th>Fundamental combinations:</th>
<th>1.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>solid timber</td>
<td>1.3</td>
</tr>
<tr>
<td>glued laminated timber</td>
<td>1.25</td>
</tr>
<tr>
<td>LVL, plywood, OSB, particle boards, fibre boards</td>
<td>1.2</td>
</tr>
<tr>
<td>Other wood-based materials</td>
<td>1.3</td>
</tr>
<tr>
<td>Connections</td>
<td>1.3</td>
</tr>
<tr>
<td>Punched metal plate fasteners (plate capacity verification only)</td>
<td>1.1</td>
</tr>
<tr>
<td>accidental combinations</td>
<td>1.0</td>
</tr>
</tbody>
</table>

(2) The design member stiffness property $E_d$ should be calculated as:

$$E_d = \frac{E_{0.05}}{\gamma_M}$$  \hspace{1cm} (2.8)

where:

$E_{0.05}$ is the fifth percentile value of a stiffness property, determined by standard tests.

### 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 hEN 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) The design value $R_d$ of a resistance (load carrying capacity) shall be calculated as:

$$R_d = k_{mod} \frac{R_k}{\gamma_M}$$  \hspace{1cm} (2.9)

where:

- $R_k$ is the characteristic value of a load carrying capacity
- $\gamma_M$ 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**: Values of $k_{mod}$ are given in 3.1.3.
2.4.4 Verification of equilibrium (EQU)

(1) The reliability format for the verification of static equilibrium in Table 1.2 (A) in Annex A of EN 1990 also applies to design situations equivalent to (EQU), e.g. for the design of hold down anchors or the verification of uplift of bearings of continuous beams.

2.5 DESIGN ASSISTED BY TESTING

(1) When resistances $R_k$ for new products are to be determined from tests, the procedure specified in this standard should be considered.
SECTION 3 MATERIAL PROPERTIES

3.1 GENERAL

3.1.1 STRENGTH AND STIFFNESS PARAMETERS

(1) 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) 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 $k_{\text{mod}}$ 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 $k_{\text{mod}}$ should be chosen which corresponds to the action with the shortest duration, e.g. for a dead load and a short-term combination, a value of $k_{\text{mod}}$ 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 $k_{\text{def}}$ given in Table 3.2 should be used.
Table 3.1 - Values of $k_{\text{mod}}$

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
<th>Load-duration class</th>
<th>Permanent Action</th>
<th>Long term Action</th>
<th>Medium term Action</th>
<th>Short term Action</th>
<th>Instant. Action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
<td>PrEN 14081</td>
<td>1</td>
<td>0,60</td>
<td>0,70</td>
<td>0,80</td>
<td>0,90</td>
<td>1,10</td>
<td></td>
</tr>
<tr>
<td>Glued Laminated timber LVL</td>
<td>PrEN 14080</td>
<td>2</td>
<td>0,60</td>
<td>0,70</td>
<td>0,80</td>
<td>0,90</td>
<td>1,10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>PrEN 124-aaa</td>
<td>3</td>
<td>0,50</td>
<td>0,55</td>
<td>0,65</td>
<td>0,70</td>
<td>0,90</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 636</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Part 1, Part 2, Part 3</td>
<td></td>
<td>1</td>
<td>0,60</td>
<td>0,70</td>
<td>0,80</td>
<td>0,90</td>
<td>1,10</td>
<td></td>
</tr>
<tr>
<td>Part 2, Part 3</td>
<td></td>
<td>2</td>
<td>0,60</td>
<td>0,70</td>
<td>0,80</td>
<td>0,90</td>
<td>1,10</td>
<td></td>
</tr>
<tr>
<td>Part 3</td>
<td></td>
<td>3</td>
<td>0,50</td>
<td>0,55</td>
<td>0,65</td>
<td>0,70</td>
<td>0,90</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>OSB/2</td>
<td></td>
<td>1</td>
<td>0,25</td>
<td>0,30</td>
<td>0,40</td>
<td>0,65</td>
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</tr>
<tr>
<td>OSB</td>
<td>EN 300</td>
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<td></td>
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</tr>
<tr>
<td>OSB/3, OSB/4</td>
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<td>0,40</td>
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<td>0,70</td>
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</tr>
<tr>
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<td></td>
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</tr>
<tr>
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<td>Part 4, Part 5</td>
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<tr>
<td>Plywood</td>
<td>EN 312</td>
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<td></td>
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<td></td>
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<td>0,50</td>
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</tr>
<tr>
<td>Plywood</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>HB.LA, HB.HLS</td>
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<td>0,30</td>
<td>0,40</td>
<td>0,65</td>
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</tr>
<tr>
<td>HB.HLS</td>
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<td>0,20</td>
<td>0,25</td>
<td>0,45</td>
<td>0,80</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 622-3</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>MBH.LA, MBH.HLS</td>
<td></td>
<td>1</td>
<td>0,25</td>
<td>0,30</td>
<td>0,40</td>
<td>0,65</td>
<td>1,10</td>
<td></td>
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<tr>
<td>MBH.HLS</td>
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<td>0,45</td>
<td>0,80</td>
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</tr>
<tr>
<td>Plywood</td>
<td>EN 622-5</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
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<td>0,40</td>
<td>0,60</td>
<td>1,10</td>
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<tr>
<td>MDF.HLS</td>
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<td>-</td>
<td>-</td>
<td>0,45</td>
<td>0,80</td>
<td></td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 622-4</td>
<td></td>
<td></td>
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<td></td>
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<tr>
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<td>-</td>
<td>0,80</td>
<td>1,10</td>
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<tr>
<td>Fibreboard, semi-hard</td>
<td></td>
<td>2</td>
<td>-</td>
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<td>-</td>
<td>0,60</td>
<td>0,80</td>
<td></td>
</tr>
</tbody>
</table>
Table 3.2 - Values of $k_{def}$ for timber, wood-based materials for quasi-permanent actions.

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
</tr>
</thead>
<tbody>
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<td></td>
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<tr>
<td>Solid timber</td>
<td>PrEN 14081</td>
<td>0.60</td>
</tr>
<tr>
<td>Glued Laminated timber</td>
<td>PrEN 14080</td>
<td>0.60</td>
</tr>
<tr>
<td>LVL</td>
<td>PrEN 124-aaa</td>
<td>0.60</td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 636</td>
<td>0.80</td>
</tr>
<tr>
<td>Part 1</td>
<td></td>
<td>0.80</td>
</tr>
<tr>
<td>Part 2</td>
<td></td>
<td>0.80</td>
</tr>
<tr>
<td>Part 3</td>
<td></td>
<td>0.80</td>
</tr>
<tr>
<td>OSB</td>
<td>EN 300</td>
<td>2.25</td>
</tr>
<tr>
<td>OSB/2</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>OSB/3, OSB/4</td>
<td></td>
<td>1.50</td>
</tr>
<tr>
<td>Particleboard</td>
<td>EN 312</td>
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<tr>
<td>Part 4</td>
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<td>2.25</td>
</tr>
<tr>
<td>Part 6</td>
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<td>1.50</td>
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<tr>
<td>Part 7</td>
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<td>1.50</td>
</tr>
<tr>
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<td>EN 622-2</td>
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</tr>
<tr>
<td>HB.LA</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>HB.HLS</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>Fibreboard, semi-hard</td>
<td>EN 622-3</td>
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</tr>
<tr>
<td>MBH.LA</td>
<td></td>
<td>3.00</td>
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<tr>
<td>MBH.HLS</td>
<td></td>
<td>3.00</td>
</tr>
<tr>
<td>Fibreboard, MDF</td>
<td>EN 622-5</td>
<td>2.25</td>
</tr>
<tr>
<td>MDF.LA</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>MDF.HLS</td>
<td></td>
<td>2.25</td>
</tr>
<tr>
<td>Fibreboard, softboard</td>
<td>EN 622-4</td>
<td>3.00</td>
</tr>
</tbody>
</table>

3.2 SOLID TIMBER

(1)P Rectangular timber members shall comply with prEN 14081-1. Timber members with round cross-section shall comply with EN TC 124-1.2

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

(3) For solid timber, 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 $f_{t,0,k}$ may be increased by the factor $k_h$, where:

$$k_h = \min \left\{ \frac{150}{h}^{0.2}, 1.3 \right\} \tag{3.1}$$

where:

$h$ is in mm.

(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 EN 385
3.3 GLUED LAMINATED TIMBER

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

(2) P The effect of member size on strength shall 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 $f_{mk}$ and $f_{t0,k}$ may be increased by the factor $k_h$, where

$$k_h = \min\left(\frac{600^{0.1}}{h}, 1, 1\right)$$  \hspace{1cm} (3.2)

where:

$h$ is 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 as described in 6.1.2(2) P

3.4 LAMINATED VENEER LUMBER (LVL)

(1) P LVL members shall comply with prEN 124-aaa.

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

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

$$k_h = \min\left(\frac{300^{0.1}}{h}, 1, 2\right)$$  \hspace{1cm} (3.3)

where:

$h$ is in mm.

(4) For LVL, the reference length in tension is 3000 mm. For lengths in tension not equal to 3000 mm the characteristic value for $f_{t0,k}$ should be multiplied by the factor $k_l$, where

$$k_l = \min\left(\frac{3000^{0.1}}{l}, 1, 1\right)$$  \hspace{1cm} (3.4)

where:

$l$ is in mm.

(5) P The size effect exponent $s$ for LVL shall be declared by the producers in accordance with prEN 124-aaa.
(6) 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.

### 3.5 WOOD-BASED PANELS

(1) Wood-based panels shall comply with prEN 13986.

(2) The use of wood-based panels shall be limited to specific service classes according to Table 3.3.

(3) The use of softboards according to EN 622-4 should be restricted to wind bracing.

#### Table 3.3 - Permitted use of wood-based panels complying with the requirements of their respective European standards

<table>
<thead>
<tr>
<th>Material, Standard</th>
<th>Service class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Plywood, EN 636</td>
<td>Permitted</td>
</tr>
<tr>
<td>Part 1</td>
<td>Permitted</td>
</tr>
<tr>
<td>Part 2</td>
<td>Permitted</td>
</tr>
<tr>
<td>Part 3</td>
<td>Permitted</td>
</tr>
<tr>
<td>OSB, EN 300</td>
<td>Permitted</td>
</tr>
<tr>
<td>OSB/2</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>OSB/3, OSB/4</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Particleboard, EN 312</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Part 4</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Part 5</td>
<td>PERMITTED</td>
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<tr>
<td>Part 6</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Part 7</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Fibreboard, hard, EN 622-2</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>HB.LA</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>HB.HLS</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Fibreboard, semi-hard, EN 622-3</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>MBH.LA</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>MBH.HLS</td>
<td>PERMITTED</td>
</tr>
<tr>
<td>Fibreboard, MDF, EN 622-5</td>
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</tr>
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<td>MDF.LA</td>
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<tr>
<td>MDF.HLS</td>
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</tr>
<tr>
<td>Fibreboard, softboard, EN 622-4</td>
<td>PERMITTED</td>
</tr>
</tbody>
</table>

### 3.6 ADHESIVES

(1) 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 EN 301 may be used in all service classes.

(3) Adhesives which comply with Type II specification as defined in EN 301 should only be used in service classes 1 or 2 and not under prolonged exposure to temperatures in excess of 50°C.

### 3.7 METAL FASTENERS

(1) Metal fasteners shall comply with prEN 124-5.3.
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 EN 350-2 for the particular hazard class (defined in EN 335-1 & EN 335-2 & EN 335-3), or be given a preservative treatment selected in accordance with EN 351-1 and EN 460.

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>
<thead>
<tr>
<th>Fastener</th>
<th>Service Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Nails and screws with $\varnothing \leq 4 \text{ mm}$</td>
<td>None</td>
</tr>
<tr>
<td>Bolts, dowels, nails and screws with $\varnothing &gt; 4 \text{ mm}$</td>
<td>None</td>
</tr>
<tr>
<td>Staples</td>
<td>Fe/Zn 12((c))</td>
</tr>
<tr>
<td>Punched metal plate fasteners and steel plates up to 3mm thick</td>
<td>Fe/Zn 12((c))</td>
</tr>
<tr>
<td>Steel plates over 3mm up to 5mm in thickness</td>
<td>None</td>
</tr>
<tr>
<td>Steel plates over 5mm</td>
<td>None</td>
</tr>
</tbody>
</table>

\(\text{\((b)\) NOTE : If hot dip zinc coating is used, Fe/Zn 12c should be replaced by Z275 and Fe/Zn 25c by Z350 in accordance with EN 10147.}\)

\(\text{\((b)\) NOTE : For especially corrosive conditions consideration should be given to Fe/Zn 40, heavier hot dip coatings or stainless steel.}\)
SECTION 5 BASIS OF STRUCTURAL ANALYSIS

5.1 GENERAL

(1) 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 shall generally 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) 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) 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) 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 6mm or less, driven without pre-drilling
- symmetrically placed holes for bolts, dowels, screws and nails in columns
- holes in the compression area of bending 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) 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, as defined in 5.1.

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

(3) 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) Structures shall be analysed using static models which consider in a realistic way the static 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) 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.

(2) 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) 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, instability of a member in compression may be disregarded if it is taken into account by the strength verification of the member.

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
(6) The frame analysis should be carried out using the appropriate values of member stiffness defined in SECTION 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 \(a_1\), and the distance \(a_2\) in Figure 5.2 is not greater than \(a_1/3\) or 100 mm, whichever is the greater
- the truss height is greater than 0,15 times the span and 10 times the maximum chord depth, see Figure 5.1

(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 element. The inner support moments should be used to calculate the span bending moments.

![Figure 5.2 - Geometry of support](image-url)
5.4.4 PLANE FRAMES AND ARCHES

(1) 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 \( \phi \) 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 \( \phi \) in radians should as a minimum be taken as

\[
\phi = \begin{cases} 
0.005 & \text{for } h \leq 5 \text{ m} \\
0.005 \sqrt{5/h} & \text{for } h \geq 5 \text{ m}
\end{cases}
\]  

(5.1)

where:

- the value of \( e \) should as a minimum be taken as:

\[ e = 0.0025 \ell \]  

(5.2)

Examples of assumed initial deviations in the geometry 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

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

Figure 6.1 - Member Axes

6.1.1 TENSION PARALLEL TO THE GRAIN

(1)P The following expression shall be satisfied:

\[ \sigma_{t,0,d} \leq f_{t,0,d} \]  

(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

6.1.2 TENSION PERPENDICULAR TO THE GRAIN

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

(2)P For a uniformly stressed volume, \( V \), the following expression shall be satisfied:

\[ \sigma_{t,90,d} \leq k_{vol} f_{t,90,d} \]  

(6.2)

where:

- \( \sigma_{t,90,d} \) is the design tensile stress perpendicular to the grain,
- \( f_{t,90,d} \) is the design tensile strength perpendicular to the grain

\[ k_{vol} = \begin{cases} 1,0 & \text{for solid timber} \\ \left( \frac{V_0}{V} \right)^{0.2} & \text{for glued laminated timber and LVL with all veneers parallel to the beam axis} \end{cases} \]  

(6.3)

where:

- \( V_0 \) is the reference volume of 0.01m³,
- \( V \) is the stressed volume in m³
6.1.3 COMPRESSION PARALLEL TO THE GRAIN

(1) The following expression shall be satisfied:

\[ \sigma_{c,0,d} \leq f_{c,0,d} \]  \hspace{1cm} (6.4)

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

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

6.1.4 COMPRESSION PERPENDICULAR TO THE GRAIN

(1) The following condition shall be satisfied:

\[ \sigma_{c,90,d} \leq k_{c,90}f_{c,90,d} \]  \hspace{1cm} (6.5)

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) Unless specified in the following clauses, \( k_{c,90} \) should be taken as 1.0.

(3) For a beam member which is able to accept in the ultimate limit state a compressive deformation of 10% of the member depth, the factor \( k_{c,90} \) should be in the range 1.2 \( \leq k_{c,90} \leq 4.0 \) and be calculated from the following expressions:

- At supports closer than \( h/3 \) from the end of the beam:
  \[ k_{c,90} = \left( 2.38 - \frac{\ell}{250} \right) \left( 1 + \frac{h}{12\ell} \right) \]  \hspace{1cm} (6.6)

- At internal supports:
  \[ k_{c,90} = \left( 2.38 - \frac{\ell}{250} \right) \left( 1 + \frac{h}{6\ell} \right) \]  \hspace{1cm} (6.7)

where:
- \( \ell \) is the contact length in mm.
- \( h \) is member depth in mm.

(4) For a member with a depth \( h \leq 2.5b \), where the reaction results from an applied compressive force occurring directly over a continuous or discrete support, the factor \( k_{c,90} \) should be in the range 1.2 \( \leq k_{c,90} \leq 4.0 \) and should be calculated from expression (6.8) provided the following conditions are fulfilled:

- the applied compressive force occurs over the full member width, \( b \)
- a compressive deformation of 10% of the depth, \( h \), may be accepted

\[ k_{c,90} = \left( 2.38 - \frac{\ell}{250} \right) \left( \frac{\ell_{ef}}{\ell} \right)^{0.5} \]  \hspace{1cm} (6.8)

where:
- \( \ell_{ef} \) is the effective length in mm according to Figure 6.2a or Figure 6.2b, but utilising a value for \( h \) of 40 mm for member depths less than 40 mm.
- \( \ell \) is the contact length in mm according to Figure 6.2a or Figure 6.2b.
The effective lengths should be determined from a stress distribution with a vertical inclination of 1:3 over the depth $h$, but no closer than a distance $a/2$ from any end, or a distance $\ell_1/4$ towards any neighbouring compressed area.

$$\ell_d = \ell + \frac{h}{3}$$

Figure 6.2a - Determination of effective lengths for a member with $h/b \leq 2.5$ on a continuous support

$$\ell_{ef} = 0.5(\ell + 2\frac{h}{3} + \ell_s) \quad \text{provided that} \quad a \geq h \quad \text{and} \quad \ell_1 \geq 2h$$

Figure 6.2b – Determination of effective lengths for a member with $h/b \leq 2.5$ on discrete supports

(5) For a member with a depth $h > 2.5b$, where the reaction results from an applied compressive force occurring directly over a continuous or discrete support, the factor $k_{c,90}$ should be calculated from (6.9) provided the following conditions are fulfilled:
- the applied compressive force occurs over the full member width, $b$;
- the contact length $\ell$ is less than the greater of $h$ or 100mm.

$$k_{c,90} = \frac{\ell_{ef}}{\ell} \quad (6.9)$$

where:
- $\ell_{ef}$ is the effective length in mm according to Figure 6.3a or Figure 6.3b. The effective length shall not exceed the contact length by more than $\ell$ on each side of the contact length;
- $\ell$ is the contact length in mm according to Figure 6.3a or Figure 6.3b.
Figure 6.3a – Determination of effective lengths for a member with \( h/b > 2.5 \) on a continuous support

Figure 6.3b – Determination of effective lengths for a member with \( h/b > 2.5 \) on discrete supports

(6) 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 support centreline and the effective length \( \ell_{ef} \) should be taken as equal to the contact length \( \ell \).

### 6.1.5 BENDING

(1) The following expressions shall be satisfied:

\[
\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

(6.10)

\[
k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

(6.11)

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

### 6.1.6 SHEAR

(1) For shear with a stress component parallel to the grain, see Figure 6.4 (a), as well as for shear with both stress components perpendicular to the grain, see Figure 6.4 (b), the following expression shall be satisfied:

$$
\tau_d \leq f_{\tau,d}
$$

where:
- $\tau_d$ is the design shear stress,
- $f_{\tau,d}$ is the design shear strength for the actual condition

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

![Figure 6.4 - (a) Member with a shear stress component parallel to the grain (b) Member with both stress components perpendicular to the grain (rolling shear)](image)

(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 $h_{ef}$ of the edge of the support may be disregarded (see Figure 6.5). For beams with a notch at the support this reduction in the shear force applies only when the notch is on the opposite side of the support.
6.1.7 TORSION

(1) The torsional stresses shall satisfy the following expression:

\[ \tau_{\text{tor,d}} \leq k_{\text{shape}} f_{v,d} \]  

(6.13)

where:

- \( \tau_{\text{tor,d}} \) is the design torsional stress,
- \( k_{\text{shape}} \) is a factor depending on the shape of the cross section:

\[
k_{\text{shape}} = \begin{cases} 
1,2 & \text{for a circular cross section} \\
\min \left( 1+0,15 \frac{h}{b}, 2,0 \right) & \text{for a rectangular cross section}
\end{cases}
\]

(6.14)

where:

- \( h \) is the largest cross-sectional dimension

6.2 DESIGN OF CROSS SECTIONS SUBJECTED TO COMBINED STRESSES

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

6.2.1 COMPRESSION STRESSES AT AN ANGLE TO THE GRAIN

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

(2) The compressive stresses at an angle \( \alpha \) to the grain, (see Figure 6.6), should satisfy the following expression:

\[
\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{f_{c,90,d} \sin^2 \alpha + \cos^2 \alpha}
\]

(6.15)

where:

- \( \sigma_{c,\alpha,d} \) is the compressive stress at an angle \( \alpha \) to the grain
6.2.2 COMBINED BENDING AND AXIAL TENSION

(1) The following expressions shall be satisfied:

\[
\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]  

(6.16)

\[
\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]  

(6.17)

(2) The values of \( k_m \) given in 6.1.5 apply.

6.2.3 COMBINED BENDING AND AXIAL COMPRESSION

(1) The following expressions shall be satisfied:

\[
\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]  

(6.18)

\[
\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]  

(6.19)

(2) The values of \( k_m \) given in 6.1.5 apply.

(3) A check should also be made of the instability condition (see 6.3)
6.2.4 COMBINED TENSION PERPENDICULAR TO THE GRAIN AND SHEAR

(1) The following expression shall be satisfied:

\[ \frac{\tau_{d}}{f_{\nu,d}} + \frac{\sigma_{c,90,d}}{k_{\text{vol}} f_{c,90,d}} \leq 1 \]  \hspace{1cm} (6.20)

where:

\( k_{\text{vol}} \) is defined in 6.1.2

6.3 STABILITY OF MEMBERS

6.3.1 GENERAL

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

(3) Column stability should be verified in accordance with 6.3.2.

(4) For members subjected to combined axial force and bending about the strong axis, the verification should be in accordance with 6.3.3.

6.3.2 MEMBERS SUBJECTED TO COMPRESSION AND BENDING (COLUMNS)

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

\[ \lambda_{\text{rel},y} = \frac{\lambda_{y}}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \]  \hspace{1cm} (6.21)

and

\[ \lambda_{\text{rel},z} = \frac{\lambda_{z}}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \]  \hspace{1cm} (6.22)

where:

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

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

\( E_{0.05} \) is the fifth percentile of the modulus of elasticity parallel to the grain

(2) Where both \( \lambda_{\text{rel},z} \leq 0.3 \) and \( \lambda_{\text{rel},y} \leq 0.3 \) the stresses should satisfy the expressions (6.18) and (6.19) in 6.2.3.

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

\[ \frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{k_{m} f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \]  \hspace{1cm} (6.23)
\[
\frac{\sigma_{c,0,d}}{k_{c,z}f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]  
(6.24)

where the symbols are defined as follows:

\[
k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \text{ similarly for } k_{c,z}
\]
(6.25)

\[
k_y = 0.5 \left( 1 + \beta_c \left( \lambda_{rel,y} - 0.3 \right) + \lambda_{rel,y}^2 \right) \text{ similarly for } k_z
\]
(6.26)

where:

\[
\beta_c = \begin{cases} 
0.2 & \text{for solid timber} \\
0.1 & \text{for glued laminated timber and LVL} 
\end{cases}
\]
(6.27)

\[
k_m \text{ as given in 6.1.5}
\]

### 6.3.3 LATERAL TORSIONAL BUCKLING OF BEAMS

(1) 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:

\[
\lambda_{rel,m} = \frac{f_{m,k}}{\sqrt{\sigma_{m,crit}}}
\]
(6.28)

where:

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

**NOTE 1:** The critical bending stress may be taken as:

\[
\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi \sqrt{E_{0.05} I_z G_{0.05} I_{tor}}}{\ell_{ef} W_y}
\]
(6.29)

where

\(E_{0.05}\) is the fifth percentile value of modulus of elasticity parallel to grain
\(G_{0.05}\) is the characteristic 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
\(\ell_{ef}\) is the span of the beam, depending on the support conditions and the load configuration, according to Table 6.1
\(W_y\) is the moment of resistance about the strong axis \(y\)

**NOTE 2:** For softwood with solid rectangular cross-section, \(\sigma_{m,crit}\) may be taken as:

\[
\sigma_{m,crit} = \frac{0.78 b^2}{h \ell_{ef}} E_{0.05}
\]
(6.30)
Table 6.1 - Effective length as a ratio of the span

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Loading type</th>
<th>$\ell_{ed}/\ell^{(a)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported</td>
<td>Constant moment</td>
<td>1,0</td>
</tr>
<tr>
<td></td>
<td>Constant distributed load</td>
<td>0,9</td>
</tr>
<tr>
<td></td>
<td>Concentrated force at the middle of the span</td>
<td>0,8</td>
</tr>
<tr>
<td>Cantilevered</td>
<td>Constant distributed load</td>
<td>0,5</td>
</tr>
<tr>
<td></td>
<td>Concentrated force at the free end</td>
<td>0,8</td>
</tr>
</tbody>
</table>

(a) NOTE: The ratio between the effective length $\ell_{ed}$ and the span $\ell$ is valid for a beam with torsional restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, $\ell_{ed}$ is to be increased by 2$h$ and may be decreased by 0,5$h$ for a load at the tension edge 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:

$$\sigma_{m,d} \leq k_{\text{crit}} f_{m,d}$$  \hspace{1cm} (6.31)

where:

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

(4) For beams with an initial lateral deviation from straightness within the limits defined in SECTION 10, $k_{\text{crit}}$ may be determined from expression (6.32)

$$k_{\text{crit}} = \begin{cases} 
1 & \text{for } \lambda_{\text{rel,m}} \leq 0,75 \\
1,56 - 0,75\lambda_{\text{rel,m}} & \text{for } 0,75 < \lambda_{\text{rel,m}} \leq 1,4 \\
1 & \lambda_{\text{rel,m}}^2 \end{cases}$$  \hspace{1cm} (6.32)

(5) The factor $k_{\text{crit}}$ may be taken as 1,0 for a beam where lateral displacement of the compressive side 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:

$$\left( \frac{\sigma_{m,y}}{k_{\text{crit}} f_{m,d}} \right)^2 + \frac{\sigma_c}{k_{c,z} f_{c,0,d}} \leq 1$$  \hspace{1cm} (6.33)

where:

$k_{c,z}$ is related to flexural buckling about the $z$ axis and defined in expression (6.25)

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. The relevant parts of 6.2 and 6.3 shall be verified.

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

\[
\sigma_N = \frac{N}{A}
\]  

where :

- \(\sigma_N\) 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 6.7 - Single tapered beam.

(2) Where the grain is parallel to one of the surfaces, and the angle of taper \(\alpha \leq 10^\circ\), the bending stress in the outermost fibre where the grain is parallel to the surface should be calculated as

\[
\sigma_{m,0,d} = (1 + 4 \tan^2 \alpha) \frac{6M_d}{bh^2}
\]  

where :

- \(M_d\) is the design moment
- \(\sigma_{m,0,d}\) is the design bending stress parallel to grain

and on the tapered side as

\[
\sigma_{m,\alpha,d} = (1 - 4 \tan^2 \alpha) \frac{6M_d}{bh^2}
\]  

where :

- \(\sigma_{m,\alpha,d}\) is the design bending stress at an angle \(\alpha\) to the grain

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

\[
\sigma_{m,\alpha,d} \leq f_{m,\alpha,d}
\]  

where :

- \(f_{m,\alpha,d}\) is the design bending strength at an angle \(\alpha\) to the grain

For tensile stresses parallel to the tapered edge :

For tensile stresses parallel to the tapered edge :
\[ f_{m.a,d} = \frac{f_{m.d}}{\sin^2 \alpha + \cos^2 \alpha} \]  

For compressive stresses parallel to the tapered edge:

\[ f_{m.a,d} = \frac{f_{m.d}}{\sin^2 \alpha + \cos^2 \alpha} \]  

### 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 6.8), the bending stresses should satisfy the following expression:

\[ \sigma_{m,d} \leq k_r f_{m,d} \]  

where:

- \( k_r \) takes into account the strength reduction due to bending of the laminates during production.

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

\[ \sigma_{m,d} = k_r \frac{6M_{ap,d}}{bh_{ap}^2} \]  

where:

- \( h_{ap}, r \) and \( \alpha \) are defined in Figure 6.8, and

\[ k_r = k_1 + k_2 \left( \frac{h_{ap}}{r} \right) + k_3 \left( \frac{h_{ap}}{r} \right)^2 + k_4 \left( \frac{h_{ap}}{r} \right)^3 \]  

where:

\[ k_1 = 1 + 1.4 \tan \alpha + 5.4 \tan^2 \alpha \]  

\[ k_2 = 0.35 - 8 \tan \alpha \]  

\[ k_3 = 0.6 + 8.3 \tan \alpha - 7.8 \tan^2 \alpha \]  

\[ k_4 = 6 \tan^2 \alpha \]  

(5) For double tapered beams \( k_r = 1.0 \). For curved and pitched cambered beams \( k_r \) should be taken as:
\( k_i = \begin{cases} 
1 & \text{for } \frac{r_n}{t} \geq 240 \\
0.76 + 0.001 \frac{r_n}{t} & \text{for } \frac{r_n}{t} \leq 240
\end{cases} \quad (6.47) 
\)

where:
the inner radius \( r_n \) and laminate thickness \( t \) are defined in Figure 6.8

(6) In the apex zone the greatest tensile stress perpendicular to the grain should satisfy the following expression:
\[
\sigma_{t,90,d} \leq k_{\text{dis}} k_{\text{vol}} f_{t,90,d}
\]
where:
\( k_{\text{vol}} \) is defined according to 6.1.2, using \( V \) as the volume in m\(^3\) of the apex zone (see Figure 6.8).
As a maximum, \( V \) should be taken as \( 2V_b/3 \), where \( V_b \) is the total volume of the beam.
\( k_{\text{dis}} \) is a factor which takes into account the effect of the stress distribution in the apex zone, with the following values:
\[
k_{\text{dis}} = \begin{cases} 
1.4 & \text{for double tapered and curved beams} \\
1.7 & \text{for pitched cambered beams}
\end{cases} \quad (6.49) 
\]

(7) The greatest tensile stress perpendicular to the grain due to the bending moment should be calculated as follows:
\[
\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{bh_{ap}^2} - 0.6 \frac{p_d}{b} 
\]
where:
\( p_d \) is the distributed load (compression) over the apex area acting at the top of the beam.
\( b \) is the width of the member.
\( M_{ap,d} \) is the design moment at apex resulting in tensile stresses parallel to the inner curved side.
\[
k_p = k_s + k_6 \left( \frac{h_{ap}}{r} \right) + k_7 \left( \frac{h_{ap}}{r} \right)^2 \quad (6.51) 
\]
where:
\( k_s = 0.2 \tan \alpha \quad (6.52) \)
\( k_6 = 0.25 - 1.5 \tan \alpha + 2.6 \tan^2 \alpha \quad (6.53) \)
\[ k_Y = 2.1 \tan \alpha - 4 \tan^2 \alpha \quad (6.54) \]

**Key**

(1) Apex Zone

**Figure 6.8** - 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. The effects of stress concentrations at the notch shall be taken into account in the strength 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 1:10, that is \( i \geq 10 \), see **Figure 6.9**(left),
   - bending with compressive stresses at the notch, see **Figure 6.9**(right)
Figure 6.9 - Bending at a notch. Left with tensile stresses at the notch, right with compressive stresses at the notch

6.5.2 BEAMS WITH A NOTCH AT THE SUPPORT

(1) For beams with rectangular cross sections and with the grain running essentially parallel, the shear stress at the notched support should be calculated using the effective (reduced) depth \( h_{ef} \) (see Figure 6.10).

(2) It should be verified that

\[ \tau_d = \frac{1.5V}{bh_{ef}} \leq k_v f_{v,d} \]  

(6.55)

where :

\( k_v \) is a reduction factor defined as follows :

For beams notched at the opposite side of the support (see Figure 6.10b)

\[ k_v = 1.0 \]  

(6.56)

For beams notched at the support side of the beam (see Figure 6.10a)

\[
\begin{align*}
k_v = \min \left( 1, \frac{k_s}{\sqrt{h}} \left( 1 + 1.1 i^{1.5} \right) \right) \\
\sqrt{h} \left( \sqrt{\alpha(1-\alpha)} + 0.8 \frac{x}{h} \frac{1}{\alpha} \left( \alpha - \alpha^2 \right) \right) \end{align*}
\]

(6.57)

where :

\( i \) is the notch inclination (see Figure 6.10a)

\( h \) is the beam depth in mm

\( x \) is the distance from line of action to the corner

\( \alpha = \frac{h_{ef}}{h} \)
\[ k_n = \begin{cases} 
4.5 & \text{for LVL} \\
5 & \text{for solid timber} \\
6.5 & \text{for glued laminated timber} 
\end{cases} \] (6.58)

6.6 MEMBERS WITH HOLES

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

(2) Axial stresses and bending moments in members with holes shall be calculated taking due account of the reduction in cross-section as described in 5.2.

(3) The following clauses are only applicable to the shear strength verification of beams in service classes 1 and 2, and where the considered loads do not result in tensile stresses perpendicular to the grain, as for example in beams with a curved shape. In this case, the following minimum and maximum distances should be fulfilled.

- \( \ell_z \geq h \)
- \( \ell_y \geq \max(h, 300 \text{ mm}) \)
- \( \ell_A \geq h/2 \)
- \( h_d \geq 0.25 h \)
- \( a \leq h \)
- \( h_d \leq 0.4 h \)
- \( r \geq 15 \text{ mm} \)

(4) Holes with a inner distance \( d \), see Figure 6.11, less than 50 mm and less than 0.1h may be disregarded.
(5) The strength verification for a shear force $V$ acting at a rectangular hole positioned on the centre axis of the beam should be carried out as for a notched beam according to 6.5.2 subjected to a shear force of $V/2$, see Figure 6.12. For a round hole the shear strength verification should be carried out as for a notch inclination of 1:1, see Figure 6.12.

![Figure 6.12](image-url)

Figure 6.12 - Holes at the centre of a beam should for the verification of shear strength be approximated as notched beams. To the left the holes, to the right the approximated notched beams.

6.7 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 and a partial coefficient for the material of $\gamma_M$ applies.

NOTE : For roof trusses with a maximum centre to centre distance of 1.2 m 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 6.13 should be used.
Key:
1 Nailed or screwed laminations
2 Laminations pre-stressed or glued together

Figure 6.13 - 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 $K_{s_{er}}$ per shear plane per fastener under service load should be taken from Table 7.1 with $\rho_m$ in kg/m³ and $d$ or $d_c$ in mm. $d_c$ is defined in EN 13271.

Table 7.1 - Values of $K_{s_{er}}$ ($=k_s$ in EN 26891) for dowel-type fasteners in N/mm

<table>
<thead>
<tr>
<th>Fastener type</th>
<th>Timber-to-timber</th>
<th>Panel-to-timber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowels</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bolts without clearance(^a)</td>
<td>$\rho_m^{1.5} \frac{d}{25}$</td>
<td></td>
</tr>
<tr>
<td>Screws</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nails (with pre-drilling)</td>
<td>$\rho_m^{1.5} \frac{d}{30}$</td>
<td></td>
</tr>
<tr>
<td>Nails (without pre-drilling)</td>
<td>$\rho_m^{1.5} \frac{d}{80}$</td>
<td></td>
</tr>
<tr>
<td>Staples</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear plate connectors (type A to EN 912)</td>
<td>$\rho_md_c/2$</td>
<td></td>
</tr>
<tr>
<td>Ring connectors (type B to EN 912)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Toothed connectors</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single sided connectors (types C1 to C9 &amp; C11 to EN 912)</td>
<td>$\rho_m d_c/5$</td>
<td></td>
</tr>
<tr>
<td>Double sided connectors (type C10 to EN 912)</td>
<td>$\rho_m d_c/3$</td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) NOTE : Clearance should be added separately to the deformation

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

$$\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}}$$  \hspace{1cm} (7.1)

(3) For steel to timber or concrete to timber connections, $K_{s_{er}}$ 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(6)) are shown in Figure 7.1, where the symbols are defined as follows:

- $u_0$ is the precamber (if applied)
- $u_{inst}$ is the instantaneous deflection (see 2.2.3)
- $u_{creep}$ is the creep deflection (see 2.2.3)
- $u_{fin}$ is the final deflection (see 2.2.3)
- $u_{net,fin}$ is the net final deflection (see 2.2.3)
(2) The net deflection below a straight line between the supports, $u_{\text{net,fin}}$, should be taken as:

$$u_{\text{net,fin}} = u_{\text{inst}} + u_{\text{creep}} - u_0 = u_{\text{fin}} - u_0$$  \hspace{1cm} (7.2)

(3) In cases where it is appropriate, it is recommended that limiting values of deflections are specified.

NOTE: The recommended range of limiting values of deflections for beams with span $\ell$ is given in Table 7.2 depending upon the level of deformation deemed to be acceptable. Information on Nationally the National choice may be found in the National Annex. For cantilevered beams, the values may be doubled.

### Table 7.2 – Examples of limiting values for deflections of beams on two supports

<table>
<thead>
<tr>
<th>$u_{\text{inst}}$</th>
<th>$u_{\text{net,fin}}$</th>
<th>$u_{\text{fin}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\ell / 300$ to $\ell / 500$</td>
<td>$\ell / 250$ to $\ell / 350$</td>
<td>$\ell / 150$ to $\ell / 300$</td>
</tr>
</tbody>
</table>

### 7.3 VIBRATIONS

#### 7.3.1 GENERAL

(1) 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) Mean values of the appropriate stiffness moduli should be used for the calculations.

(4) 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) 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 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 8 Hz ($f_1 \leq 8$ Hz) a special investigation should be made.

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

$$\frac{u}{F} \leq a \ \text{mm}/kN \quad (7.3)$$

and

$$v \leq b^{(f_1 \zeta^{-4})} \ \text{m/(Ns^2)} \quad (7.4)$$

where:
- $u$ is the maximum vertical deflection caused by a vertical concentrated static force $F$ applied at any point on the floor, taking account of load distribution, where $k_{det} = 0.0$
- $v$ 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.
- $\zeta$ is the modal damping ratio

NOTE: The recommended range of limiting values of $a$ and $b$ and the relationship between $a$ and $b$ is given in Figure 7.2. Information on the National choice may be found in the National Annex.

(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 $\ell$, the fundamental frequency $f_1$ may approximately be calculated as

$$f_1 = \frac{\pi}{2\ell^2} \sqrt{\frac{(EI)_l}{m}}$$  \hspace{1cm} (7.5)$$

where:

- $m$ is the mass per unit area in kg/m²
- $\ell$ is the floor span in m
- $(EI)_l$ is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction in Nm²/m

(5) The value $v$ may, as an approximation, be taken as:

$$v = \frac{4(0.4 + 0.6n_{40})}{mb\ell + 200} \frac{m}{Ns^2}$$  \hspace{1cm} (7.6)$$

where:

- $n_{40}$ is the number of first-order modes with natural frequencies below 40 Hz and $b$ is the floor width in m.

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

$$n_{40} = \left\{ \left( \frac{40}{f_1} \right)^2 - 1 \right\} \left( \frac{b^{0.25}}{\ell} \right) \left( \frac{(EI)_l}{(EI)_b} \right)$$  \hspace{1cm} (7.7)$$

where:

- $(EI)_b$ is the equivalent plate bending stiffness of the floor about an axis parallel to the beams
- Where $(EI)_b < (EI)_l$

NOTE: For floors having different boundary conditions, design rules may be taken from national annexes.
SECTION 8  CONNECTIONS WITH METAL FASTENERS

8.1 GENERAL

(1)P When a connection is made of a combination of 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.

(2)P Unless defined below, the characteristic load carrying capacity, and the stiffness of the connections shall be determined from tests according to EN 1380, EN 1381, EN 26891 and 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.

(3)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.

(4) The effect on connection strength of long term or medium term actions alternating between a tensile force \( F_t \) and a compressive force \( F_c \) should be taken into account by designing the connection for \( (F_{t,d} + 0.5F_{c,d}) \) and \( (F_{c,d} + 0.5F_{t,d}) \).

(5)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.

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

(7) The effective characteristic load-carrying capacity of a multiple fastener connection, \( R_{ef,k} \) should be taken as:

\[
R_{ef,k} = mn_d R_k
\]

(8.1)

where:

- \( R_{ef,k} \) is the effective characteristic load-carrying capacity of the connection
- \( m \) is the number of fastener rows in line with the force
- \( n_d \) is a factor for the effective number of fasteners in a line parallel to the grain
- \( R_k \) is the characteristic load-carrying capacity per fastener

(8)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, \( F \sin \alpha \) perpendicular to the grain shall be taken into account.

(9)P To take account of the possibility of splitting caused by the tension force component, \( F \sin \alpha \) perpendicular to the grain, the design splitting capacity, \( R_{90,d} \) of the (middle) timber member shall satisfy the following requirement:

\[
R_{90,d} \geq \max \left\{ \frac{V_1}{V_2} \right\}
\]

(8.2)

where:

- \( R_{90,d} \) is the design splitting capacity, calculated from the characteristic splitting capacity \( R_{90,k} \) according to 2.4.3
- \( V_1, V_2 \) are the design shear forces on either side of the connection. (see Figure 8.1).

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

\[ R_{90,k} = 14b \left( \frac{h_e}{1 - \frac{h_e}{h}} \right) \]  

(8.3)

where:

- \( R_{90,k} \) is the characteristic splitting capacity, in N
- \( h_e \) is the loaded edge distance to the most distant fastener, in mm
- \( h \) is the timber member height, in mm
- \( b \) is the member thickness, in mm

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

(12) 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) and (h/k) from Figure 8.3 with the other failure modes.

8.2 LATERAL LOAD-CARRYING CAPACITY OF METAL DOWEL-TYPE FASTENERS

8.2.1 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, at the specified minimum spacings should be the minimum value from the following expressions:

- For fasteners in single shear:
For fasteners in double shear:

\[ R_k = \min \begin{cases} 
  f_{h,1,k} t_1 d \\
  f_{h,2,k} t_2 d \\
  \frac{f_{h,1,k} t_1 d}{1 + \beta} \left[ \sqrt{\beta + 2\beta^2 \left[ 1 + \frac{t_2}{t_1} + \left( \frac{t_2}{t_1} \right)^2 \right]} + \beta^3 \left( \frac{t_2}{t_1} \right)^2 \right] - \beta \left( 1 + \frac{t_2}{t_1} \right) \\
  \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[ 2\beta (1 + \beta) + \frac{5\beta (2 + \beta) M_{y,k}}{f_{h,1,k} d t_1^2} \right] - \beta \\
  \frac{f_{h,1,k} t_2 d}{1 + 2\beta} \left[ 2\beta^2 (1 + \beta) + \frac{5\beta (2 + \beta) M_{y,k}}{f_{h,1,k} d t_2^2} \right] - \beta \\
  1.15 k_{cal} \frac{2\beta}{1 + \beta} \sqrt{2 M_{y,k} f_{h,1,k} d} 
\end{cases} \] (8.4) (8.5) (8.6)

with:

\[ \beta = \frac{f_{h,2,k}}{f_{h,1,k}} \]

where:

- \( R_k \) is the 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.
- \( M_{y,k} \) is the characteristic fastener yield moment.
- \( \beta \) is the ratio between the embedment strength of the members.
- \( k_{cal} \) is a factor to account for axial forces which develop in the fastener, see 0(1), 8.3.1.3(1), 8.5.1.1(2) and 8.6(2).

\textbf{NOTE 1:} The different modes of failure are illustrated in Figure 8.2.

\textbf{NOTE 2:} The subscript 1 corresponds to external members, the subscript 2 corresponds to the central member.

\textbf{NOTE 3:} Plasticity of joints can only be assured when relatively slender fasteners are used. In that case, failure modes (f) and (k) are governing.

\textbf{NOTE 4:} Simplified expressions can be found in D.1.1 and D.1.3 of Annex D (Informative).
(2) In absence of design rules given below, the characteristic embedment strength $f_{h,k}$ should be determined according to EN 383 and prEN 124-bbb.

(3) In absence of design rules given below, the characteristic yield moment $M_{y,k}$ should be determined according to EN 409 and prEN 124-bbb.

![Failure modes for timber and panel connections.](image)

Key
(1) Single shear
(2) Double shear

Figure 8.2 - Failure modes for timber and panel connections.
The letters correspond to the references of the expressions (8.4) and (8.5).

### 8.2.2 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 $0.5d$ are classified as thin plates and steel plates of thickness greater or equal to $d$ with the tolerance on hole diameters being less than $0.1d$ are classified as thick plates. The characteristic load-carrying capacity of connections with steel plate thickness intermediate between a thin and a thick plate should be calculated by linear interpolation between the limiting thin and thick plate values.

(2) 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 at the specified minimum spacings should be the minimum value from the following expressions:

- For a thin steel plate in single shear:

$$R_k = \min \left\{ \begin{array}{l} 0.4 f_{h,k} t_1 d \\ 1.15 k_{cal} \sqrt{2 M_{y,k} f_{h,k} d} \end{array} \right. \quad (8.7)$$

where:
- $R_k$ is the load carrying capacity per shear plane per fastener
- $f_{h,k}$ is the characteristic embedment strength in the timber member
- $t_1$ is the timber thickness or penetration
- $d$ is the fastener diameter
\( M_{yk} \) is the characteristic fastener yield moment
\( k_{cal} \) is a factor to account for axial forces which develop in the fastener, see 8.3.1.4(1), 8.5.1.1(2) and 8.6(2)

- For a thick steel plate in single shear:

\[
R_k = \min \left\{ \frac{f_{h,k} t_1 d}{2,3k_{cal} \sqrt{M_{yk} f_{h,k} d}} \right\}
\]

(8.8)

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

\[
R_k = \min \left\{ \frac{f_{h,1,k} t_1 d}{2,3k_{cal} \sqrt{M_{yk} f_{h,1,k} d}} \right\}
\]

(8.9)

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

\[
R_k = \min \left\{ \frac{0,5 f_{h,2,k} t_2 d}{1,15k_{cal} \sqrt{2M_{yk} f_{h,2,k} d}} \right\}
\]

(8.10)

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

\[
R_k = \min \left\{ \frac{0,5 f_{h,2,k} t_2 d}{2,3k_{cal} \sqrt{M_{yk} f_{h,2,k} d}} \right\}
\]

(8.11)

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

NOTE 2: Simplified conservative expressions can be found in D.2 of Annex D (Informative)
(4) It shall be taken into account that the load-carrying capacity of steel-to-timber connections near the ends of the timber member is affected by failure along the circumference of the fastener group (see Figure 8.4 and Figure 8.5).

(5) For connections with a force component parallel to grain and located near the end face of the timber member, the characteristic block shear capacity representing failure along the circumference of the fastener group, as shown in Figure 8.4 and Figure 8.5, should be taken as:

\[
R_{bs,k} = \max\begin{cases} 
1.5 A_{net,t} f_{t,0,k} \\
0.7 A_{net,v} f_{v,k} \\
0.9 A_{net,v} f_{v,k}
\end{cases} \quad \text{in the case of a loaded end}
\]

\[
\text{in the case of an unloaded end}
\]

with:

- for connections with bolts and dowels:

\[
A_{net,t} = L_{net,t} t
\]

\[
A_{net,v} = L_{net,v} t
\]

- for connections with nails, staples and screws:

\[
A_{net,t} = 2 L_{net,t} \sqrt{\frac{M_{y,k}}{d f_{h,k}}}
\]

\[
A_{net,v} = L_{net,v} \left( L_{net,t} + 2 \sqrt{\frac{M_{y,k}}{d f_{h,k}}} \right)
\]

where:

\(R_{bs,k}\) is the characteristic block shear capacity

\(A_{net,t}\) is the net cross-sectional area perpendicular to the grain

\(A_{net,v}\) is the net shear area parallel to the grain direction

\(L_{net,t}\) is the net width of the cross-section perpendicular to the grain, see Figure 8.4

\(L_{net,v}\) is the net length of the fracture area in shear, see Figure 8.4

\(t\) is the timber member thickness

\(M_{y,k}\) is the characteristic fastener yield moment

\(d\) 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 embedment strength of the timber member

NOTE: Examples of failure modes along the circumference of the fastener area are given in Figure 8.4 for bolts and dowels and in Figure 8.5 for nails, staples and screws.

![Figure 8.4 – Failure along the circumference of a connection with bolts and dowels](image-url)
8.3 NAILED CONNECTIONS

8.3.1 LATERALLY LOADED NAILS

8.3.1.1 General

(1) The rules given in 8.1 and 8.2 apply.

(2) The symbols for the thicknesses in single and double shear connections (see Figure 8.6) 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 point side penetration in a double shear connection

- \( t_2 \) is:
  - the point side penetration in a single shear connection
  - the central member thickness in a double shear connections.

(3) Timber should be pre-drilled when the characteristic density of the timber is greater or equal to 500 kg/m³ or when the diameter \( d \) of the nail exceeds 6mm.

(4) For square nails, the nail diameter \( d \) shall be taken as the side dimension.

(5) For common smooth steel wire nails with a minimum tensile strength of the wire from which the nails are produced of 600 N/mm², the following characteristic values for yield moment should be used:

\[
M_{y,k} = \frac{f_u}{600} 180d^{2.6} \text{ for round nails} \\
M_{y,k} = \frac{f_u}{600} 270d^{2.6} \text{ for square nails}
\]  

(8.15)

where:

- \( M_{y,k} \) is the characteristic value for the yield moment, in Nmm
- \( d \) is the nail diameter or the side length, in mm
- \( f_u \) is the tensile strength of the wire, in N/mm²

(6) When timber is predrilled, the spacing \( a_1 \) may be reduced to a minimum of \( 4d \), provided that the load carrying capacity is reduced by the factor:

\[
k_i = \frac{a_1}{(4 + 3|\cos \alpha|)d}
\]

(8.16)
where:

- $k_r$ is a reduction factor
- $a_1$ is the spacing, see Figure 8.7

Figure 8.6 - Definitions of $t_1$ and $t_2$

(a) single shear connection
(b) double shear connection

(7) For connections with a row of nails in grain direction subjected to a force component parallel to grain, the load-carrying capacity parallel to grain should be calculated from the effective number of fasteners in a row according to:

$$ n_{ef} = n^k_{ef} $$

(8.17)

where:

- $n_{ef}$ is the effective number of nails in a row,
- $n$ is the number of nails in a row.
- $k_{ef}$ is given in Table 8.1

Table 8.1 - Values of $k_{ef}$

<table>
<thead>
<tr>
<th>Spacing$^{a)}$</th>
<th>$k_{ef}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_1 \geq 14d$</td>
<td>1.0</td>
</tr>
<tr>
<td>$a_1 = 10d$</td>
<td>0.85</td>
</tr>
<tr>
<td>$a_1 = 7d$</td>
<td>0.7</td>
</tr>
</tbody>
</table>

$^{a)}$ For intermediate spacings, linear interpolation of $k_{ef}$ is permitted

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

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

8.3.1.2 Nailed timber-to-timber connections

(1) $k_{cal}$ should be:
- 1.3 for smooth nails of round, square or grooved cross-section
- 1.5 for threaded nails

NOTE: For the purpose of this clause, threaded nails are defined as nails having a withdrawal strength, $f_{ax,k} > 5.0$ N/mm² in a material density of 320 kg/m³ determined according to 8.3.2.

(2) The following characteristic embedment strength values should be used for nails with diameters up to 6mm, for all angles of load to the grain:
where:
\[ \rho_k \] is the characteristic timber density, in kg/m³
\[ d \] is the fastener diameter, in mm.

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

(4) For smooth nails the point side penetration length should be at least \( 8d \).

(5) For annular ringed shank and helically threaded nails the point side penetration length should be at least \( 6d \).

(6) Smooth nails in end grain should normally not be considered able to transmit lateral force. In secondary structures, e.g. for fascia boards nailed to rafters, the design value should be taken as 1/3 of the value for normal nailing.

(7) For annular ringed shank and helically threaded nails in end grain, the design value should be taken as 1/3 of the value for normal nailing, provided that:
- the nails are only laterally loaded,
- there are at least three nails per connection,
- the point side penetration is at least \( 10d \),
- the connection is not exposed to service class 3 conditions,
- the prescribed spacing values of Table 8.2 are satisfied.

(8) Minimum spacings and distances should be taken from Table 8.2, with symbols illustrated in Figure 8.7

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>without predrilled holes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \rho_k \leq 420 \text{ kg/m}^3 )</td>
</tr>
<tr>
<td>( a_1 ) (parallel to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( d &lt; 5\text{mm}: (5+5 \cos \alpha ) ( d ) )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( d \geq 5\text{mm}: (5+7 \cos \alpha ) ( d ) )</td>
</tr>
<tr>
<td>( a_2 ) (perpendicular to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 5d )</td>
</tr>
<tr>
<td>( a_3 ) (loaded end)</td>
<td>( -90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( (10+5\cos \alpha ) ( d ) )</td>
</tr>
<tr>
<td>( a_3 ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha \leq 270^\circ )</td>
<td>( 10d )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( 7d )</td>
</tr>
<tr>
<td>( a_4 ) (loaded edge)</td>
<td>( 0^\circ \leq \alpha \leq 180^\circ )</td>
<td>( (5+5\sin \alpha ) ( d ) )</td>
</tr>
<tr>
<td>( a_4 ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 5d )</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( 5d )</td>
</tr>
</tbody>
</table>
(1) Loaded end
(2) Unloaded end
(3) Loaded edge
(4) Unloaded edge

Key
(a) Spacings parallel and perpendicular to grain
(b) Edge and end distances
α is the angle between the force and the grain direction

(9) In a three member connection, nails may be driven without pre-drilling to overlap in the central member provided \((t - t_2)\) is greater than \(4d\) (see Figure 8.8).

(10) The minimum thickness of timber members without pre-drilled holes should be:

\[
t = \max \left\{ \frac{7d}{\rho_k}, \frac{(13d - 30)\rho_k}{400} \right\}
\]

where:
- \(t\) is the minimum thickness, in mm
- \(\rho_k\) is the characteristic timber density in kg/m³
is the nail diameter, in mm

(11) For single shear nailed timber-to-timber connections between timber members for which the point side nail penetration is at least $8d$, the design expressions may be simplified according to D.1.2 of Annex D(Informative).

### 8.3.1.3 Nailed panel-to-timber connections

(1) $k_{cal}$ should be:
- 1,3 for smooth nails,
- 1,5 for threaded nails

**NOTE:** For the purpose of this clause, threaded nails are defined as nails having a withdrawal strength, $f_{ax,k} > 5,0 \text{ N/mm}^2$ for a density of 320 kg/m$^3$ determined according to 8.3.2.

(2) Minimum nail spacings and distances for all nailed panel-to-timber connections other than plywood-to-timber connections should be taken as those given in Table 8.2, multiplied by a factor of 0,7.

(3) Minimum nail spacings in plywood-to-timber connections should be taken as those given in Table 8.2, multiplied by a factor of 0,85.

(4) The minimum edge and end distances in plywood members should be taken as $3d$ for an unloaded edge (or end) and $(3 + 4 \sin \alpha) d$ for a loaded edge (or end).

(5) The rules below apply to nails with heads which have a diameter of at least $2d$.

(6) For plywood the following values of characteristic embedment strength should be used:

\[
 f_{h,k} = 0,11 \rho_k d^{-0.3}
\]

where:
- $f_{h,k}$ is the characteristic embedment strength, in N/mm$^2$
- $\rho_k$ is the characteristic plywood density in kg/m$^3$
- $d$ is the nail diameter, in mm

(7) For hardboard in accordance with EN 622-2 the following values of characteristic embedment strength should be used:

\[
 f_{h,k} = 30 d^{-0.3} t^{0.6}
\]

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) $k_{cal}$ should be:
- 1,5 for smooth nails,
- 1,8 for threaded nails

provided that threaded nails have a withdrawal strength of $f_{ax,k} > 5.0 \text{ N/mm}^2$ for a density of 320 kg/m$^3$ determined according to 8.3.2.

(2) Minimum nail spacing and distances should be taken as those given in Table 8.2, multiplied by a factor of 0,7.
8.3.2 AXIALLY LOADED NAILS

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

(2) The characteristic withdrawal capacity of nails for nailing perpendicular to the grain (Figure 8.9 (a)) and for slant nailing (Figure 8.9 (b)), should be taken as the smaller of the values according to the following expressions:

For annular ringed shank or threaded nails:

\[
R_k = \begin{cases} 
  \frac{f_{\text{ax},k} d t_{\text{pen}}}{d_h^2} & \text{(a)} \\
  \frac{f_{\text{head},k} d_t^2}{d_h^2} & \text{(b)}
\end{cases}
\]  

(8.22)

NOTE: For the purpose of this clause, threaded nails are defined as nails having a withdrawal strength, \(f_{\text{ax},k} > 5.0 \text{ N/mm}^2\) for a density of 320 kg/m\(^3\) determined according to 8.3.2.

For smooth nails:

\[
R_k = \begin{cases} 
  \frac{f_{\text{ax},k} d t_{\text{pen}}}{d_h^2} & \text{(a)} \\
  \frac{f_{\text{ax},k} d t + f_{\text{head},k} d_t^2}{d_h^2} & \text{(b)}
\end{cases}
\]  

(8.23)

where:

- \(f_{\text{ax},k}\) and \(f_{\text{head},k}\) are determined by tests
- \(d\) is the fastener diameter according to 8.3.1.1
- \(t_{\text{pen}}\) is the point side penetration length or the length of the threaded part in the point side member
- \(t\) is the thickness of the head side member or the length of the threaded part in the head side member
- \(d_h\) is the fastener head diameter

NOTE: Expressions (8.22)(a) and (8.23)(a) correspond to withdrawal of the nail in the member receiving the point, and expressions (8.22)(b) and (8.23)(b) correspond to the head being pulled through).

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

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

(5) For smooth nails, the point side penetration \(t_{\text{pen}}\) should be at least 8\(d\). For nails with a point side penetration smaller than 12\(d\) the withdrawal capacity should be multiplied by \(t_{\text{pen}}/4d - 2\). For threaded nails, the point side penetration should be at least 6\(d\). For nails with a point side penetration smaller than 8\(d\) the withdrawal capacity should be multiplied by \(t_{\text{pen}}/2d - 3\).

(6) The parameters \(f_{\text{ax}}\) and \(f_{\text{head}}\) depend, amongst other things, on the type of nail, the timber species and grade (especially density) and should be determined by tests in accordance with EN1382 and EN1383 and prEN 124-bbb unless specified in the following clause.

(7) Characteristic values for the withdrawal and pull through parameters, \(f_{\text{ax},k}\) and \(f_{\text{head},k}\) for smooth nails should be taken as:

\[
f_{\text{ax},k} = 20 \times 10^{-6} \rho_k^2 
\]  

(8.24)

\[
f_{\text{head},k} = \min \left\{ \left( \frac{d_h}{d} - 1 \right) \times 250 \times 10^{-6} \rho_k^2, \frac{375 \times 10^{-6}}{375} \rho_k^2 \right\} 
\]  

(8.25)

where:
\[ \rho_k \text{ is the characteristic timber density in kg/m}^3 \]
\[ d_h \text{ is the head diameter in mm} \]

(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 and distances for axially loaded nails should be the same as for laterally loaded nails. For slant nailing the distance to the loaded edge should be at least 10\(d\) (see Figure 8.9(b)).

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

\[ \begin{array}{c}
\text{Figure 8.9 - Perpendicular and slant nailing} \\
(a) \text{ perpendicular nailing} \quad (b) \text{ slant nailing}
\end{array} \]

8.3.3 COMBINED LATERALLY AND AXIALLY LOADED NAILS

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

for smooth nails:

\[
\frac{F_{ax,d}}{R_{ax,d}} + \frac{F_{la,d}}{R_{la,d}} \leq 1
\]

for annular ringed shank and helically threaded nails:

\[
\left( \frac{F_{ax,d}}{R_{ax,d}} \right)^2 + \left( \frac{F_{la,d}}{R_{la,d}} \right)^2 \leq 1
\]

where:

\( R_{ax,d} \) and \( R_{la,d} \) 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 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) Minimum dimensions for staples, with reference to Figure 8.10 are:
   - the minimum width \( b \) of the staple crown should be taken as \( b \geq 6d \)
   - the minimum point side penetration length \( t_2 \) should be at least \( 14d \)

(4) 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\(^\circ\), see Figure 8.11. If the angle between the crown and the direction of the grain under the crown is equal to or less than 30\(^\circ\), then the lateral design load-carrying capacity should be multiplied by a factor of 0.7.

(5) For staples with a minimum tensile strength of the wire from which the staples are produced of 800 N/mm\(^2\), the following characteristic yield moment per leg should be used:

\[
M_{y,k} = 240 d^{2.6} \text{ Nmm}
\]  

where:
\( d \) is the staple leg diameter, in mm

(6) A minimum of two staples should be used to form a connection.

Figure 8.10 - Staple dimensions

(7) Minimum spacings and distances should be taken from Table 8.3, with symbols illustrated in Figure 8.11.

Figure 8.11 - Definition of spacings for staples
Table 8.3 - Minimum spacings and distances for staples

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_1$ (parallel to grain) for $\theta \geq 30^\circ$</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$(10 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td>for $\theta &lt; 30^\circ$</td>
<td></td>
<td>$(15 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td>$a_2$ (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$15 d$</td>
</tr>
<tr>
<td>$a_{3t}$ (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$(15 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td>$a_{3c}$ (unloaded end)</td>
<td>$90^\circ \leq \alpha \leq 270^\circ$</td>
<td>$15 d$</td>
</tr>
<tr>
<td>$a_{4t}$ (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>$(15 + 5 \sin \alpha) d$</td>
</tr>
<tr>
<td>$a_{4c}$ (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$10 d$</td>
</tr>
</tbody>
</table>

(8) For connections with a row of staples in the grain direction subjected to a force component parallel to grain, the load carrying capacity parallel to grain should be calculated from the effective number of fasteners in a row according to 8.3.1.1(7)

8.5 BOLTED CONNECTIONS

8.5.1 LATERALLY LOADED BOLTS

8.5.1.1 General

(1) The rules given in 8.2 apply.

(2) $k_{cal}$ should be 1.2

(3) For connections with a row of bolts in the grain direction subjected to a force component parallel to grain, the load carrying capacity parallel to grain should be calculated from the effective number of fasteners in a row according to:

\[
n_{ef} = \min \left\{ n, 0.9 \frac{a_1}{4 \sqrt{13d}} \right\}
\]

(8.29)

where:

- $n_{ef}$ is the effective number of fasteners in a row
- $n$ is the number of fasteners in a row
- $a_1$ is the spacing distance in the grain direction
- $d$ is the fastener diameter

(4) Provided that there are not more than four bolts per row parallel to the grain direction, the spacing $a_1$ may be reduced to a minimum of $4d$, if the load carrying capacity is reduced by the factor:

\[
k_i = \frac{a_1}{(4 + 3 \cos \alpha)d}
\]

(8.30)

(5) Minimum spacings and distances should be taken from Table 8.4, with symbols illustrated in Figure 8.7
### Table 8.4 - Minimum spacings and distances for bolts

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a_1$, (parallel to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$(4 + 3 \cos \alpha) d$</td>
</tr>
<tr>
<td>$a_2$, (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$4d$</td>
</tr>
<tr>
<td>$a_3$, (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>max (7d; 80 mm)</td>
</tr>
<tr>
<td>$a_3$, (unloaded end)</td>
<td>$90^\circ \leq \alpha &lt; 150^\circ$</td>
<td>max([1 + ($6 \sin \alpha$) $d$; 4d])</td>
</tr>
<tr>
<td></td>
<td>$150^\circ \leq \alpha &lt; 210^\circ$</td>
<td>$4d$</td>
</tr>
<tr>
<td></td>
<td>$210^\circ \leq \alpha \leq 270^\circ$</td>
<td>max([1 + ($6 \sin \alpha$) $d$; 4d])</td>
</tr>
<tr>
<td>$a_4$, (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>max([2 + ($2 \sin \alpha$) $d$; 3d])</td>
</tr>
<tr>
<td>$a_4$, (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$3d$</td>
</tr>
</tbody>
</table>

(6) For round steel bolts the following characteristic value for the yield moment should be used:

$$ M_{y,k} = 0.3 f_{u,k} d^{2.6} $$.  

where:

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

(7) Requirements for bolt diameter and washer dimensions are given in 10.4.3

### 8.5.1.2 Bolted timber-to-timber connections

(1) For bolts up to 30mm diameter the following characteristic embedment strength values should be used, at an angle $\alpha$ to the grain:

$$ f_{h.o,k} = \frac{f_{h.o,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} $$  

where:

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

$$ f_{h.o,k} = 0.082 (1 - 0.01d) \rho_k \text{ N/mm}^2 $$

where:

- $\rho_k$ is the characteristic timber density in kg/m³
- $d$ is the bolt diameter, in mm

### 8.5.1.3 Bolted panel-to-timber connections

(1) For plywood the following embedment strength value should be used at all angles to the face grain:

$$ f_{h.o,k} = 0.11 (1 - 0.01d) \rho_k \text{ N/mm}^2 $$

where:

- $\rho_k$ is the characteristic plywood density in kg/m³
- $d$ is the bolt diameter, in mm
8.5.1.4 Bolted steel-to-timber connections

(1) The rules given in 8.2.2 apply.

8.5.2 AXIALLY LOADED BOLTS

(1) A check shall be made of the adequacy of the bolt tensile strength and washer thickness.

(2) The design compressive stress under the washer should not exceed \(3.0f_{c,90,d}\).

8.6 DOWELLED CONNECTIONS

(1) The rules given in 8.5.1.1 (2), 8.5.1.1(6), 8.5.1.2, 8.5.1.3 and 8.5.1.4 apply.

(2) \(k_{\text{cal}}\) should be 1.0

(3) The spacing \(a_1\) may be reduced to a minimum of \(4d\), if the load carrying capacity is reduced by the factor:

\[
k_i = \frac{a_1}{(3 + 4|\cos\alpha|)d}
\]

(8.36)

(4) Minimum spacings and distances should be taken from Table 8.5, with symbols illustrated in Figure 8.7.

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1) (parallel to grain)</td>
<td>(0^\circ \leq \alpha \leq 360^\circ)</td>
<td>((3 + 4</td>
</tr>
<tr>
<td>(a_2) (perpendicular to grain)</td>
<td>(0^\circ \leq \alpha \leq 360^\circ)</td>
<td>(3d)</td>
</tr>
<tr>
<td>(a_{3,t}) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ)</td>
<td>(\max(7d; 80\text{ mm}))</td>
</tr>
<tr>
<td>(a_{3,c}) (unloaded end)</td>
<td>(90^\circ \leq \alpha &lt; 150^\circ)</td>
<td>(\max(a_{3,t}; \sin\alpha d; 3d))</td>
</tr>
<tr>
<td></td>
<td>(150^\circ \leq \alpha &lt; 210^\circ)</td>
<td>(3d)</td>
</tr>
<tr>
<td></td>
<td>(210^\circ \leq \alpha \leq 270^\circ)</td>
<td>(\max(a_{3,t}; \sin\alpha d; 3d))</td>
</tr>
<tr>
<td>(a_{4,t}) (loaded edge)</td>
<td>(0^\circ \leq \alpha \leq 180^\circ)</td>
<td>(\max(2 + 2\sin\alpha d; 3d))</td>
</tr>
<tr>
<td>(a_{4,c}) (unloaded edge)</td>
<td>(180^\circ \leq \alpha \leq 360^\circ)</td>
<td>(3d)</td>
</tr>
</tbody>
</table>

(5) Dowels installed closer to the loaded end grain than the prescribed minimum, \(a_{3,t}\) shall be disregarded in the calculation of the load carrying capacity of the connection.

(6) Requirements for structural detailing and control of dowelled joints are given in 10.4.4.

8.7 SCREWED CONNECTIONS

8.7.1 LATERALLY LOADED SCREWS

(1) 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) On condition that a tight contact between the connecting members is assured, the following apply:

(3) For smooth shank screws, where the diameter of the threaded part is equal to the smooth shank, the rules for the load carrying capacity in 8.2 apply, provided that:
   - The effective diameter \( d_{ef} \) is taken as the smooth shank diameter,
   - The pointside penetration of the smooth shank is not less than 4\( d \)

(4) For all other cases, the rules of 8.2 apply provided that the effective diameter \( d_{ef} \) is taken as 1,1 times the inner thread diameter.

(5) For smooth shank screws with a diameter \( d > 6 \text{mm} \), the rules in 8.5.1 apply. For other screws, the rules of 8.3.1 apply.

(6) For all screws, the rules for \( k_{cal} \) applicable for threaded nails apply.

(7) 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 by the:
   - the withdrawal capacity of the screwed-in 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 strength of the head against being pulled through.
   - 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)

(2) Minimum spacings and distances should be taken from Table 8.6.

<table>
<thead>
<tr>
<th>Screws driven</th>
<th>Minimum spacing</th>
<th>Minimum edge distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>At right angle to the grain</td>
<td>4( d )</td>
<td>4( d )</td>
</tr>
<tr>
<td>In end grain face</td>
<td>4( d )</td>
<td>2.5( d )</td>
</tr>
</tbody>
</table>

(3) The minimum point side penetration length of the threaded part should be 6\( d \); however, the effective penetration length, \( l_{ef} \) used in calculations should be taken equal to the point side length minus one screw diameter.

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

\[
R_{ax,\alpha,k} = n_{ef} \pi d \ l_{ef} \ f_{v,ax,\alpha,k}
\]

(8.37)

where:
- \( R_{ax,\alpha,k} \) is the characteristic withdrawal strength of the connection,
- \( n_{ef} \) is the effective number of screws,
- \( d \) is the outer diameter measured on the threaded part,
- \( l_{ef} \) is the point side penetration length of the threaded part minus one screw diameter.
- \( f_{v,ax,\alpha,k} \) is the characteristic withdrawal strength at an angle \( \alpha \) to the grain,

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

\[
f_{v,ax,\alpha,k} = \frac{f_{v,ax,90,k}}{\sin^2 \alpha + 1.5 \cos^2 \alpha}
\]

(8.38)

with:
\[ f_{v,\text{ax},90,k} = 3.6 \times 10^{-6} (\pi d l_{ef})^{0.2} \rho_k^{1.5} \]  \tag{8.39}

where:
- \( f_{v,\text{ax},90,k} \) is the characteristic withdrawal strength at an angle \( \alpha \) to the grain
- \( \rho_k \) 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 a limited possibility for stress redistribution.

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

\[ n_{ef} = n^{0.9} \]  \tag{8.40}

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 (8.27) should be satisfied.

### 8.8 CONNECTIONS MADE WITH PUNCHED METAL PLATE FASTENERS

#### 8.8.1 GENERAL

(1) Connections made with punched metal plate fasteners shall comprise punched metal plate fasteners of the same 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 8.12** and defined as follows:

- \( x \)-direction: main direction of plate
- \( y \)-direction: perpendicular to the main plate direction
- \( \alpha \): angle between the \( x \)-direction and the force
- \( \beta \): angle between the grain-direction and the force
- \( \gamma \): 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 those parts of the surface which are outside some specified dimension from the edges and ends
- \( \ell \): length of the plate measured along the connection line

#### 8.8.3 PLATE STRENGTH PROPERTIES

(1) The plate shall have characteristic values for the following properties, determined from tests carried out in accordance with EN 1075:
- \( f_{a,0,0} \) the anchorage capacity per unit area for \( \alpha = 0^\circ \) and \( \beta = 0^\circ \)
- \( f_{a,90,90} \) the anchorage capacity per unit area for \( \alpha = 90^\circ \) and \( \beta = 90^\circ \)
- \( f_{t,0} \) the tension capacity per unit width of the plate in the \( x \)-direction (\( \alpha = 0^\circ \))
- \( f_{c,0} \) the compression capacity per unit width of the plate in the \( x \)-direction (\( \alpha = 0^\circ \))
- \( f_{v,0} \) the shear capacity per unit width of the plate in the \( x \)-direction (\( \alpha = 0^\circ \))
- \( f_{t,90} \) the tension capacity per unit width of the plate in the \( y \)-direction (\( \alpha = 90^\circ \))
- \( f_{c,90} \) the compression capacity per unit width of the plate in the \( y \)-direction (\( \alpha = 90^\circ \))
- \( f_{v,90} \) the shear capacity per unit width of the plate in the \( y \)-direction (\( \alpha = 90^\circ \))
- \( k_1, k_2, \alpha_0 \) constants

(2) 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.

**NOTE:** The value for the partial factor for the plate material may be taken from **Table 2.2**

**Figure 8.12 - Geometry of punched metal plate connection loaded by a force \( F \) and moment \( M \).**

### 8.8.4 PLATE ANCHORAGE STRENGTHS

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

\[
f_{a,\alpha,\beta,\kappa} = \max \left\{ \begin{array}{ll}
\frac{f_{a,0,0,\kappa} - (f_{a,0,0,\kappa} - f_{a,90,90,\kappa}) \beta}{45^\circ} & \text{when } \beta \leq 45^\circ, \\
\frac{f_{a,0,0,\kappa} - (f_{a,0,0,\kappa} - f_{a,90,90,\kappa}) \sin \left( \max (\alpha, \beta) \right) }{ } & \text{when } 45^\circ < \beta \leq 90^\circ \\
\end{array} \right.
\]

(8.41)

when \( \beta \leq 45^\circ \), or

\[
f_{a,\alpha,\beta,\kappa} = f_{a,0,0,\kappa} - (f_{a,0,0,\kappa} - f_{a,90,90,\kappa}) \sin \left( \max (\alpha, \beta) \right)
\]

(8.42)

when \( 45^\circ < \beta \leq 90^\circ \)

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

\[
f_{a,\alpha,0,\kappa} = \begin{cases} 
 f_{a,0,0,\kappa} + k_1 \alpha & \text{when } \alpha \leq \alpha_0 \\
 f_{a,0,0,\kappa} + k_1 \alpha_0 + k_2 (\alpha - \alpha_0) & \text{when } \alpha_0 < \alpha \leq 90^\circ 
\end{cases}
\]

(8.43)
The constants $k_1$, $k_2$ and $\alpha_0$ should be determined by tests in accordance with EN 1075 for the actual type of plate.

### 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 $F$ and the design anchorage stress $\tau_{M,d}$ imposed from a moment $M$, should be taken as:

$$
\tau_{F,d} = \frac{F_{A,d}}{A_{ef}}
$$

$$
\tau_{M,d} = \frac{M_{A,d}}{W_p}
$$

with:

$$
W_p = \int_{A_{ef}} r \, dA
$$

where:

- $F_{A,d}$ 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)
- $M_{A,d}$ is the design moment acting on a single plate on the centroid of the effective area
- $r$ is the distance from the centre of gravity of the nail plate to the segmental nail plate area $dA$
- $A_{ef}$ is the effective plate area

**NOTE:** $W_p$ may be conservatively approximated from:

$$
W_p = \frac{A_{ef} d}{4}
$$

with:

$$
d = \sqrt{\left(\frac{A_{ef}}{h_{ef}}\right)^2 + h_{ef}^2}
$$

where:

- $h_{ef}$ is the maximum height of the effective anchorage area perpendicular to the longest side.

(2) Contact pressure between timber members may be taken into account to reduce the value of $F$ 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 $F_{A,min,d}/2$.

(3) Contact pressure between the timber members in chord splices in compression may be taken into account by designing for a design force, $F_{A,d}$, and a design moment $M_{A,d}$ according to the following expressions:

$$
F_{A,d} = \sqrt{\left(\frac{F_d}{4} - \frac{3 M_d}{4 h}\right)^2 + \left(\frac{V_d}{2}\right)^2}
$$

$$
M_{A,d} = \frac{M_d}{4}
$$

where:

- $F_d$ is the design normal force of the chord (compression or zero),
- $V_d$ is the design shear force of the chord,
- $M_d$ is the design moment of the chord,
\( h \) is the height of the chord.

(4) The following expression should be satisfied:

\[
\left( \frac{\tau_{F, d}}{f_{a, \theta, d}} \right)^2 + \left( \frac{\tau_{M, d}}{f_{a, 0,0, d}} \right)^2 \leq 1
\]  

(8.51)

8.8.5.2 Plate capacity

(1) For each joint interface, the forces in the two main directions should be taken as:

\[
F_{x, d} = F_d \cos \alpha \pm 2F_{M, d} \sin \gamma
\]

\[
F_{y, d} = F_d \sin \alpha \pm 2F_{M, d} \cos \gamma
\]  

(8.52)

(8.53)

where:

- \( F_d \) is the design force in a single plate (i.e. half of the total force in the timber member)
- \( F_{M, d} \) is the design force from the moment \( M_d \) on a single plate (\( F_{M, d} = 2M_d/\ell \)).

(2) The following expression should be satisfied:

\[
\left( \frac{F_{x, d}}{R_{x, d}} \right)^2 + \left( \frac{F_{y, d}}{R_{y, d}} \right)^2 \leq 1
\]  

(8.54)

where:

- \( F_{x, d} \) and \( F_{y, d} \) are the design forces acting in the \( x \) and \( y \) direction,
- \( R_{x, d} \) and \( R_{y, d} \) 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

\[
R_{x, k} = \max \left\{ f_{n, n, k} \ell \sin(\gamma - \gamma_0 \sin(2\gamma)) \right\}
\]  

where \( f_{n, n, k} = \begin{cases} f_{t, 0, k} & \text{if } F_{x, d} > 0 \\ f_{t, 0, k} & \text{if } F_{x, d} \leq 0 \end{cases} \)  

(8.55)

\[
R_{y, k} = \max \left\{ f_{n, n, 90, k} \ell \cos \gamma \right\}
\]  

where \( f_{n, 90, k} = \begin{cases} f_{t, 90, k} & \text{if } F_{y, d} > 0 \\ f_{c, 90, k} & \text{if } F_{y, d} \leq 0 \end{cases} \)  

(8.56)

where:

- \( \gamma_0 \) and \( k_v \) are constants determined from shear tests in accordance with EN 1075 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 (8.54) is satisfied in each straight part of the connection line. All critical sections should be taken into account.
8.9 CONNECTIONS MADE WITH RING OR SHEAR PLATE CONNECTORS

(1) For connections made with ring connectors of type A (timber to timber) or shear plate connectors of type B (steel to timber) according to EN 912, and with a diameter not bigger than 200 mm, the characteristic load-carrying capacity for a force along the grain, \( R_{0,k} \), per connector and per shear plane should be taken as:

\[
R_{0,k} = \min \left\{ k_1 k_2 k_3 k_4 (35 d_c^{1.5}) , \frac{k_1 k_3 h_e}{35} (31.5 d_c) \right\}
\]

where :
- \( d_c \) is the connector diameter, in mm
- \( h_e \) is the embedment depth, in mm
- \( k_i \) are modification factors, with \( i = 1 \) to 4

NOTE: The first term of expression (8.57) represents shear failure and the lower embedment failure

(2) The factor \( k_1 \), which depends on the member thicknesses \( t_1 \) and \( t_2 \) and embedment depth \( h_e \) of the connection, should be taken as:

\[
k_1 = \min \left\{ 1, \frac{t_1}{3h_e}, \frac{t_2}{5h_e} \right\}
\]

where:
- \( t_1 \geq 2.25 h_e \) and \( t_2 \geq 3.75 h_e \)

(3) The factor \( k_2 \), which depends on the end distance \( a_{3,t} \) of the connection and applies only for connections loaded in tension (i.e. \(-30^\circ \leq \alpha \leq 30^\circ\)), should be taken as:

\[
k_2 = \min \left\{ k_u a_{3,t} , \frac{1.25}{2d_c} \right\}
\]

where:
- \( a_{3,t} \geq 1.5d_c \)
- \( k_u = \begin{cases} 1.25 & \text{for connections with one connector per shear plane} \\ 1.0 & \text{for connections with more than one connector per shear plane} \end{cases} \)

(4) For other connections, the factor \( k_2 \) should be taken equal to 1.0.

(5) The factor \( k_3 \), which depends on the timber density, should be taken as:

\[
k_3 = \min \left\{ 1.75 \frac{\rho_k}{350} \right\}
\]

where:
- \( \rho_k \) 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:

\[
k_4 = \begin{cases} 
1,0 & \text{for timber to timber connections} \\
1,1 & \text{for steel to timber connections}
\end{cases}
\]  

(8.63)

(7) For connections with one connector per shear plane loaded in compression \((150^\circ \leq \alpha \leq 210^\circ)\), the first condition in expression (8.57) may be disregarded.

(8) For a force at an angle \( \alpha \) to the grain, the characteristic load-carrying capacity, \( R_{\alpha,k} \), per connector per shear plane should be calculated using the following expression:

\[
R_{\alpha,k} = \frac{R_{0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha}
\]  

(8.64)

with:

\[
k_{90} = 1,3 + 0,001 d_c
\]  

(8.65)

where:

- \( R_{0,k} \) is the characteristic load-carrying capacity of the connector for a force parallel to grain according to 8.9(1),
- \( d_c \) is the connector diameter, in mm

(9) When a row of connectors positioned parallel to the grain are loaded by a force parallel to the grain, the effective number of connectors should be taken as:

\[
n_{ef} = 2 + \left( 1 - \frac{n}{20} \right) (n - 2)
\]  

(8.66)

where:

- \( n_{ef} \) is the effective number of connectors
- \( n \) is the number of connectors in a line parallel to grain

NOTE: Connectors are considered to be positioned parallel to the grain when \( k_{a2} a_2 < 0,5 \) \( k_{a1} a_1 \), see 8.9(11)

(10) Minimum spacings and distances should be taken from Table 8.7, with the symbols illustrated in Figure 8.7

Table 8.7 – Minimum spacings and distances for ring and shear plate connectors.

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_1 ) (parallel to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (1,2 + 0,8 \left</td>
</tr>
<tr>
<td>( a_2 ) (perpendicular to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 3 d_c )</td>
</tr>
<tr>
<td>( a_3,t ) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( 1,5 d_c )</td>
</tr>
<tr>
<td>( a_3,c ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( (0,4 + 1,6 \left</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( 1,2 d_c )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha \leq 270^\circ )</td>
<td>( (0,4 + 1,6 \left</td>
</tr>
<tr>
<td>( a_4,t ) (loaded edge)</td>
<td>( 0^\circ \leq \alpha \leq 180^\circ )</td>
<td>( (0,4 + 0,2 \left</td>
</tr>
<tr>
<td>( a_4,c ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 0,6 d_c )</td>
</tr>
</tbody>
</table>

(11) Where the connectors are staggered (see Figure 8.13), the respective minimum spacing distances parallel and perpendicular to the grain direction may be changed provided the following expression is fulfilled:
\[(k_{a1})^2 + (k_{a2})^2 \geq 1 \quad \text{with} \quad \begin{cases} 0 \leq k_{a1} \leq 1 \\ 0 \leq k_{a2} \leq 1 \end{cases} \quad \text{(8.67)}\]

where:
- \(k_{a1}\) is a reduction factor for the minimum distance parallel to the grain
- \(k_{a2}\) is a reduction factor for the minimum distance perpendicular to the grain

**Figure 8.13 - Reduced distances for connectors**

(12) The distance parallel to grain, \(k_{a1}a_1\), may further be reduced by up to 50%, provided that the load carrying capacity is reduced proportionately by up to 40%.

### 8.10 CONNECTIONS MADE WITH TOOTHED-PLATE CONNECTORS

(1) The characteristic load-carrying capacity of joints made using toothed plate connections 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 \(R_k\) per toothed-plate for connectors of type C according to EN 912 (single-sided: type C1 to C9, double sided: type C10 and C11) should be calculated according to:

\[
R_k = \begin{cases} 
18\ k_i k_2 k_3 d_c^{1.5} & \text{for types C1 to C9} \\
25\ k_1 k_2 k_3 d_c^{1.5} & \text{for types C10 to C11}
\end{cases} \quad \text{(8.68)}
\]

where:
- \(R_k\) is the characteristic load-carrying capacity per toothed-plate
- \(k_i\) are modification factors, with \(i = 1\) to \(3\)
- \(d_c\) is:
  - the toothed-plate diameter for types C1, C2, C6, C7, C10 and C11, in mm
  - the toothed-plate side length for types C5, C8 and C9, in mm
  - the square root of the product of both side lengths for types C3 and C4, in mm

(3) The factor \(k_1\), which depends on the timber thickness, should be taken as:

\[
k_1 = \min \left( \frac{t_1}{3h_e}, \frac{t_2}{5h_e} \right) \quad \text{(8.69)}
\]

where:
- \(t_1\) is the side member thickness, with a minimum value \(t_1 \geq 1.1\ h_e\)
- \(t_2\) is the middle member thickness, with a minimum value \(t_2 \geq 1.9\ h_e\)
- \(h_e\) is the tooth penetration depth, in mm
(4) The factor \( k_2 \), which depends on the end distance, \( a_{3,t} \), should be taken as:

- For types C1 to C9:
  \[
  k_2 = \min \left\{ \frac{1}{a_{3,t}} \right\} \text{ with } a_{3,t} = \max \left\{ \frac{1,1 \, d_c}{7 \, d}, \frac{1,11}{80 \, \text{mm}} \right\}
  \tag{8.70}
  \]
  where:
  \( d \) is the bolt diameter, in mm.

- For types C10 and C11:
  \[
  k_2 = \min \left\{ \frac{1}{2,0 \, d_c} \right\} \text{ with } a_{3,t} = \max \left\{ \frac{1,5 \, d_c}{7 \, d}, \frac{1,51}{80 \, \text{mm}} \right\}
  \tag{8.71}
  \]
  where:
  \( d \) is the bolt diameter, in mm.

(5) The factor \( k_3 \), which depends on the timber density, should be taken as:

\[
  k_3 = \min \left\{ \frac{1,5}{\rho_k} \right\}
  \tag{8.72}
  \]

where:
\( \rho_k \) is the characteristic density of the timber, in kg/m\(^3\).

(6) For toothed plate connector types C1 to C9, minimum spacings and distances should be taken from Table 8.8, with the symbols illustrated in Figure 8.7

**Table 8.8 – Minimum spacings and distances for toothed plate connector types C1 to C9.**

<table>
<thead>
<tr>
<th>Spacings and distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_1 ) (parallel to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (1,2 + 0,3 \sin \alpha) , d_c )</td>
</tr>
<tr>
<td>( a_2 ) (perpendicular to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 1,2 , d_c )</td>
</tr>
<tr>
<td>( a_{3,t} ) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( 2,0 , d_c )</td>
</tr>
<tr>
<td>( a_{3,c} ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( (0,4 + 1,6 \sin \alpha) , d_c )</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( 1,2 , d_c )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha \leq 270^\circ )</td>
<td>( (0,4 + 1,6 \sin \alpha) , d_c )</td>
</tr>
<tr>
<td>( a_{4,t} ) (loaded edge)</td>
<td>( 0^\circ \leq \alpha \leq 180^\circ )</td>
<td>( (0,6 + 0,2 \sin \alpha) , d_c )</td>
</tr>
<tr>
<td>( a_{4,c} ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 0,6 , d_c )</td>
</tr>
</tbody>
</table>

(7) For toothed plate types C10 and C11, minimum spacings and distances should be taken from Table 8.9, with the symbols illustrated in Figure 8.7
Table 8.9 - Minimum spacings and distances for toothed plate connector types C10 and C11.

<table>
<thead>
<tr>
<th>Spacings and distances</th>
<th>Angle</th>
<th>minimum distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>(see Figure 8.7)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a_1) (parallel to grain)</td>
<td>(0^\circ \leq \alpha \leq 360^\circ)</td>
<td>((1.2 + 0.8 \cos \alpha) d_c)</td>
</tr>
<tr>
<td>(a_2) (perpendicular to grain)</td>
<td>(0^\circ \leq \alpha \leq 360^\circ)</td>
<td>(1.2 d_c)</td>
</tr>
<tr>
<td>(a_{3,t}) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ)</td>
<td>(1.5 d_c)</td>
</tr>
<tr>
<td>(a_{3,c}) (unloaded end)</td>
<td>(90^\circ \leq \alpha &lt; 150^\circ) (\lor) (150^\circ &lt; \alpha \leq 210^\circ) (\lor) (210^\circ &lt; \alpha \leq 270^\circ)</td>
<td>((0.9 + 0.6 \sin \alpha) d_c) (\lor) (1.2 d_c) (\lor) ((0.9 + 0.6 \sin \alpha) d_c)</td>
</tr>
<tr>
<td>(a_{4,t}) (loaded edge)</td>
<td>(0^\circ \leq \alpha \leq 180^\circ)</td>
<td>((0.6 + 0.2 \sin \alpha) d_c)</td>
</tr>
<tr>
<td>(a_{4,c}) (unloaded edge)</td>
<td>(180^\circ \leq \alpha \leq 360^\circ)</td>
<td>(0.6 d_c)</td>
</tr>
</tbody>
</table>

(8) Where connectors of types C1, C2, C6 and C7 with circular shape are staggered (see Figure 8.13), the respective minimum spacing distances parallel and perpendicular to the grain direction may be changed provided the following expression is fulfilled:

\[
(k_{a_1})^2 + (k_{a_2})^2 \geq 1 \quad \text{with} \quad \begin{cases} 0 \leq k_{a_1} \leq 1 \\ 0 \leq k_{a_2} \leq 1 \end{cases}
\]

(8.73)

where:

- \(k_{a_1}\) is a reduction factor for the minimum distance parallel to the grain
- \(k_{a_2}\) is a reduction factor for the minimum distance perpendicular to the grain

(9) Requirements for bolts used for toothed connectors should be taken from 10.4.3
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:

\[ \sigma_{f,c,\text{max},d} \leq f_{c,m,d} \]  \hspace{1cm} (9.1)

\[ \sigma_{f,t,\text{max},d} \leq f_{c,m,d} \]  \hspace{1cm} (9.2)

\[ \sigma_{f,c,d} \leq k_c f_{c,0,d} \]  \hspace{1cm} (9.3)

\[ \sigma_{f,t,d} \leq f_{t,0,d} \]  \hspace{1cm} (9.4)

where:

- \( \sigma_{f,c,\text{max},d} \) is the extreme fibre flange design compressive stress
- \( \sigma_{f,t,\text{max},d} \) 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

$$\lambda_c = \sqrt{12 \left( \frac{\ell_c}{b} \right)},$$

where $\ell_c$ is the distance between the sections where lateral deflection of the compressive flange is prevented, and $b$ is given in Figure 9.1. If a special investigation is made into 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:

$$\sigma_{w,c,d} \leq f_{c,w,d}$$

$$\sigma_{w,t,d} \leq f_{t,w,d}$$

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 tensile strength and compressive strength of the webs should be taken as the in-plane design tensile or compressive strength.

(6) It shall be verified that any glued splices have sufficient strength.

(7) Unless a detailed buckling analysis is made it should be verified that:

$$h_w \leq 70b_w$$

and

$$V_d \leq \left\{ \begin{array}{ll}
b_w h_w \left( 1 + \frac{0.5(h_{f,t} + h_{f,c})}{h_w} \right) f_{v,0,d} & \text{for } h_w \leq 35b_w \\
35 b_w^2 \left( 1 + \frac{0.5(h_{f,t} + h_{f,c})}{h_w} \right) f_{v,0,d} & \text{for } 35b_w \leq h_w \leq 70b_w \end{array} \right.$$
9.1.2 GLUED THIN-FLANGED BEAMS

(1) In the following, a linear variation of strain over the depth of the beam is assumed.

(2) In the strength verification of glued thin-flanged beams, account shall be taken of the non-uniform 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:

\[
 b_{ef} = b_{c,ef} + b_{w} \quad \text{or} \quad b_{ef} = 0.5b_{c,ef} + b_{w}
\]

(9.11)

(9.12)

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 $\ell$ is the span of the beam.

![Table 9.1 - Maximum effective flange widths due to the effects of shear lag and plate buckling](image)

(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:

\[
 \tau_{\text{mean,d}} \leq \begin{cases} 
 f_{v,90,d} & \text{for } b_w \leq 8h_l \\
 f_{v,90,d} \left( \frac{8h_l}{b_w} \right)^{0.8} & \text{for } b_w > 8h_l
\end{cases}
\]

(9.13)

where:

- $\tau_{\text{mean,d}}$ 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.
Figure 9.2 - Thin-flanged beam

For section 1-1 of a U-shaped cross-section, the same expressions should be verified, but with $8h_t$ substituted by $4h_t$.

(7) The axial stresses in the flanges, based on the relevant effective flange width, should satisfy the following expressions:

$$\sigma_{f,c,d} \leq f_{f,c,d}$$  \hspace{1cm} (9.14)

$$\sigma_{f,t,d} \leq f_{f,t,d}$$  \hspace{1cm} (9.15)

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) It shall be verified that any glued splices have sufficient strength.

(9) The axial stresses in the wood based webs should satisfy the expressions (9.5) to (9.6) defined in 9.1.1

9.1.3 MECHANICALLY JOINTED BEAMS

(1) 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) In order to determine the fastener spacing in the longitudinal direction, where the shear force varies between $s_{\text{min}}$ and $s_{\text{max}}$ ($\leq 4s_{\text{min}}$), an effective shear force $s_{\text{ef}}$ may be used as follows:

$$s_{\text{ef}} = 0.75 s_{\text{min}} + 0.25 s_{\text{max}}$$  \hspace{1cm} (9.16)

(4) 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) 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.

(2) 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 (informative).

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 (6.18) and (6.19) 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 has been carried out, the following effective column lengths may be assumed (see Figure 9.3).

- for continuous members with a lateral load but without significant end moments
  - 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 a lateral load and with significant end moments
  - 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 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. A check shall be made that the lateral (out-of-plane) stability of the truss members is adequate.

7. 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:

$$F_{r,d} = 1.0 + 0.1L \text{ (kN)}$$

where:

$L$ is the overall length of the truss, in m

9.2.2 TRUSSES WITH PUNCHED METAL PLATE FASTENERS

1. The requirements of 5.4.1 and 9.2.2 apply.

2. For fully triangulated trusses where a small concentrated force (e.g. a man load) has a component perpendicular to the member of $< 1.5 \text{kN}$, and where $\sigma_{c,d} < 0.4 f_{c,d}$ and $\sigma_{t,d} < 0.4 f_{t,d}$, then the requirements of 6.2.2 and 6.2.3 may be replaced by:

$$\sigma_{m,d} \leq 0.75 f_{m,d}$$

3. The minimum overlap of the punched metal plate on any timber member should be at least equal to 40mm or one third of the height of the timber member, whichever is the greater.

4. 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 $\ell$ lies between $2b$ and $6b$, 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 150mm) without reduction of the load-carrying capacity.

![Figure 9.4 - Diaphragm loading and staggered panel arrangements](image)

**Key**
- (1) Edge beam
- (2) Discontinuous edges
- (3) Panel arrangements

9.2.4 **WALL DIAPHRAGMS**

9.2.4.1 **General**

(1) Wall diaphragms shall be designed to resist both horizontal and vertical actions imposed upon them.

(2) The wall shall be adequately restrained to avoid overturning and sliding.

(3) Wall diaphragms deemed to provide resistance to racking shall be stiffened in plane by board materials, diagonal bracing or moment connections.

(4) The racking resistance of a wall shall be determined either by test according to EN 594 or calculations, employing appropriate analytical methods or design models.
(5) The design of wall diaphragms shall take account of both the material construction and geometric make-up of the wall under consideration.

(6) The response of wall diaphragms to actions shall be assessed to ensure the construction remains within appropriate serviceability limits.

(7) For wall diaphragms with a tie-down at their end, a method of calculation is given in 9.2.4.2.

NOTE: For wall diaphragms having different tie-down conditions, design rules may be taken from National Annexes.

### 9.2.4.2 Simplified analysis of wall diaphragms

(1) The design load carrying capacity \( R_{v,d} \) (the racking resistance) under a force \( F_k \) 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 and
- the width of each sheet is at least \( h/4 \)

(2) For a wall made up of several wall panels, the design racking load carrying capacity of a wall should be calculated from

\[
R_{v,d} = \sum R_{v,d} \tag{9.19}
\]

where:

- \( R_{v,d} \) 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).

(3) The design racking load carrying capacity of each wall panel, \( R_{v,d} \) according to Figure 9.5 should be calculated from

\[
R_{v,d} = \frac{R_{f,d} b c_i}{s} \tag{9.20}
\]

where:

- \( R_{f,d} \) is the lateral design capacity of an individual fastener
- \( b_i \) is the wall panel width
- \( s \) is the fastener spacing

and

\[
c_i = \begin{cases} 
1 & \text{for } b_i \geq b_0 \\
\frac{b_i}{b_0} & \text{for } b_i < b_0 
\end{cases} \tag{9.21}
\]

where:

\( b_0 = h/2 \)

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 considered to be unloaded.
(4) Unless it can be shown otherwise, a wall panel which contains a door or window opening should not be considered to contribute to the total racking load carrying capacity.

(5) 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.

(6) The external forces $F_{ic,d}$ and $F_{it,d}$ according to Figure 9.5 should be determined from

$$F_{ic,d} = F_{it,d} = \frac{F_{iv,d} h}{b_1}$$

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.

(7) 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.
(8) Shear buckling of the sheet may be disregarded, provided that
\[ \frac{b_{\text{net}}}{t} \leq 100. \]
where:
- \( b_{\text{net}} \) is the clear spacing between studs
- \( t \) is the thickness of the sheet

(9) 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.

(10) Where each panel consists of a prefabricated wall element, the transfer of shear forces between the separate wall elements should be verified.

(11) In contact areas between vertical studs and horizontal timber members, compression stresses perpendicular to grain should be verified in the timber members.

### 9.2.5 BRACING

#### 9.2.5.1 General

(1) Structures which are not otherwise adequately stiff shall be braced to prevent instability or excessive deflection.

(2) 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) 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 9.7), 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 \( C \)

\[
C = k_s \frac{N_d}{a}
\]

where:
- \( N_d \) is the mean design compressive force in the element
- \( a \) is the bay length (see Figure 9.7)

\[
k_s = 2 \left[ 1 + \cos \left( \frac{\pi}{m} \right) \right]
\]

where:
- \( m \) is the number of bays each of length \( a \).

(3) The design stabilizing force \( F_d \) at each support should, as a minimum, be taken as:
(4) The design stabilizing force $F_d$ for the compressive flange of a rectangular beam should be determined in accordance with 9.2.5.2(3)

where:

$$N_d = \left(1 - k_{\text{crit}}\right) \frac{M_d}{h}$$  \hspace{1cm} (9.26)

The value of $k_{\text{crit}}$ should be determined from 6.3.3(4) for the unbraced beam, and $M_d$ is the maximum design moment in 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 9.8) a bracing system should be provided, which, in addition to the effects of an external horizontal load (e.g. wind), should be capable of resisting an internal stability load per unit length $q$, as follows:

$$q_d = k_i \frac{n N_d}{50 \ell}$$  \hspace{1cm} (9.27)

where:

$$k_i = \min\left\{1, \frac{15}{\sqrt{\ell}}\right\}$$  \hspace{1cm} (9.28)

$N_d$ is the mean design compressive force in the member,

$\ell$ is the overall span of the stabilizing system, in m.
(2) The horizontal deflection of the bracing system due to force $q_d$ acting alone, should not exceed $\ell/700$.

(3) The horizontal deflection of the bracing system due to force $q_d$ and any other external load (e.g. wind), should not exceed $\ell/500$.

Figure 9.8 - Beam or truss system requiring lateral supports

Key
(1) n members
(2) Bracing
(3) External load on bracing
SECTION 10 STRUCTURAL DETAILING AND CONTROL

10.1 GENERAL

(1) Timber structures shall be so constructed that they conform with the principles of the design. Materials for the structures shall be applied, used or fixed in such a way as to perform adequately the functions for which they are designed.

(2) Workmanship in fabrication, preparation and installation of materials shall conform to accepted good practice.

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 or LVL members and to 1/300 times the length of structural timber5.

(2) Timber and wood-based components and structural elements should not be unnecessarily exposed to climatic conditions more severe than those to be 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 the subject of quality control, to ensure that the reliability and quality of the joint is in accordance with the technical specification.

(2) The adhesive manufacturers' 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 JOINTS WITH MECHANICAL FASTENERS

10.4.1 GENERAL

(1) Wane, splits, knots or other defects shall be limited in the region of the joint such that the load-carrying capacity of the joint is not reduced.

---

5 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.
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 conformity with Figure 8.9(b).

(3) The diameter of pre-drilled holes should not exceed 0.8d.

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.

(2) Washers with a side length or a diameter of at least 3d and a thickness of at least 0.3d (where d is the bolt diameter) 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 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:

<table>
<thead>
<tr>
<th>Type of connector EN 912</th>
<th>$d_c$ minimum</th>
<th>$d$ maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>A – A5</td>
<td>$\le 130$</td>
<td>12</td>
</tr>
<tr>
<td>A1, A4, A5</td>
<td>$&gt; 130$</td>
<td>0.1$d_c$</td>
</tr>
<tr>
<td>B</td>
<td>$d - 1$</td>
<td>$d$</td>
</tr>
</tbody>
</table>

where:

- $d_c$ is the connector diameter, in mm
- $d$ is the bolt diameter, in mm

10.4.4 DOVELS

(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 less than 6 mm, predrilling is not required. For screws with a diameter $d \geq 6$ mm, predrilling 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 unthreaded part.
- 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/m³, the predrilling diameter should be determined by tests.

10.5 ASSEMBLY

(1) The structure should be assembled in such a way that over-stressing 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 and erection should be avoided. If the structure is loaded or supported in a different manner than in the finished building the temporary condition should be considered as a relevant load case, including any possible dynamic components. In the case of e.g. framed arches, portal frames, etc., special care should be taken to avoid distortion in hoisting from the horizontal to the vertical position.

10.7 CONTROL

(1) There should be a control plan comprising

- 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 preboring
  - spacings and distances to end and edge
  - 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. Annular ringed-shank or threaded nails, or screws should be used, with a maximum spacing along the edges of 150mm. Elsewhere the maximum spacing should be 300mm.

Key
(1) Batten slant nailed to joist or rafter
(2) Batten
(3) Sheathing nailed to batten

Figure 10.1 - Examples of connection of panels not supported by a joist or a rafter. Sheathing are nailed to battens which are slant nailed to the joists or rafters.

10.8.2 WALL DIAPHRAGMS

(1) The simplified method of analysis given in 9.2.4.2 assumes that panel fixings have a maximum fastener spacing along the edges of 150 mm for nails, and 200mm for screws. Elsewhere the maximum spacing should be 300 mm.

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

(1) Trusses should be fabricated in accordance with EN TC 124-1.3.

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 EN TC 124-1.3. 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) After erection, a maximum bow of 10mm may be permitted in any truss member provided that it is adequately secured in the completed roof to prevent the bow from increasing.

(4) After erection, the maximum deviation of a truss from true vertical alignment should not exceed \(10 + 5(H-1)\) mm, with a maximum value of 25mm, where \(H\) is the overall rise of the truss in m.
ANNEX A (INFORMATIVE): CONNECTIONS WITH GLUED-IN STEEL RODS

A.1 GENERAL

(1) The use of glued-in rods should be limited to structural parts assigned to service classes 1 and 2.

(2) It should be verified that the properties of the adhesive and its bond to steel and wood are reliable during the lifetime of the structure within the temperature and moisture ranges envisaged.

(3) Rods should be threaded or deformed bars.

   NOTE: The durability of adhesion of smooth steel surfaces is not well-known and the adhesion can be adversely affected by corrosion for example.

(4) It should be taken into account that the load-carrying capacity of a group of glued-in rods acting together is normally less than the sum of the resistances of individual glued-in rods.

   NOTE: This effect is normally caused by brittle failure modes.

(5) For rods inserted perpendicular to grain, the risk of splitting of the timber due to changes of moisture content should be taken into account.

A.2 AXIALLY LOADED RODS

A.2.1 General

(1) The load-carrying capacity of connections made with glued-in axially loaded rods should be verified for the following failure modes:

   - failure of the steel rod
   - failure of the adhesive and its bond to steel and wood
   - failure of the timber adjacent to the glue-line
   - failure of the timber member (e.g. pull-out failure of a whole timber block with several glued-in rods).

(2) The load-carrying capacity should generally be limited by the strength of the rod, not by the load transfer capacity of the adhesive and its bond to the rod or the wood or by the strength of the timber.

   NOTE: This requirement is aimed at preventing brittle failure.

(3) The expressions given are based either on the outer diameter $d$ of the rod; or when strength of the adhesive is not critical, on an equivalent diameter $d_{eq}$, equal to the smaller of the hole diameter or $1.25d$.

   NOTE: For threaded rods, the outer diameter is equal to the nominal diameter; for most deformed reinforcing bars used as rods, the outer diameter is approximately 10 % greater than the nominal diameter.

(4) Minimum spacings and distances should be taken according to Figure A.1.
(5) The minimum anchorage length $\ell_{a,\text{min}}$ should be taken as

$$\ell_{a,\text{min}} = \max\left\{ \frac{0.4 d^2}{8 d}, \frac{2.5 d}{4 d}, \frac{2.5 d}{2.5 d} \right\}$$

(A.1)

where:

- $\ell_{a,\text{min}}$ is the minimum anchorage length in mm, see Figure A.1
- $d$ is the outer diameter of the rod in mm.

NOTE: By using the anchorage length $\ell_a = \ell_{a,\text{min}}$ according to expression (A.1), adhesion failure becomes decisive.

### A.2.2 Failure modes

#### A.2.2.1 Failure of an individual rod

(1) The axial resistance in tension of the steel rod should be determined with respect to the yield strength of the steel. In compression, the possibility of buckling should be taken into account for design compressive stresses greater than 300 N/mm².

(2) The characteristic axial resistance corresponding to shear in the wood should be taken as

$$R_{\text{ax},k} = \pi \, d_{\text{eq}} \, \ell \, f_v, k$$

(A.2)
(3) The characteristic withdrawal strength at an angle \( \alpha \) to the grain, where the ratio \( f_v/d \leq 18 \), should be taken as:

\[
f_{v,a,k} = \frac{f_{v,90,k}}{\sin^2 \alpha + 1.5 \cos^2 \alpha}
\]

where:

\[
f_{v,90,k} = 1.2 \times 10^{-3} \rho_k^{1.5} d^{-0.2}
\]

where:

\[ \rho_k \] is the characteristic density, in kg/m³

(4) The shear strength of the adhesive and its bond to steel and wood should be verified by tests.

(5) For service class 2 the values of \( k_{mod} \) should be reduced by 20%.

NOTE: For ultimate limit state considerations the same modification factors \( k_{mod} \) are normally used for solid and glued laminated timber and their connections, for both service classes 1 and 2. Pull-out tests on glued-in bolts in softwood with the load direction parallel to grain, have shown a noticeable decrease in strength at high moisture content.

A.2.2.2 Failure in the timber member

(1) The effective timber failure area, \( A_{ef} \), of a rod, see Figure A.2, when subjected to an axial force parallel to the grain should be taken as the smaller of:

- an effective width, \( b_{ef} \), of 3\( d \), measured from the centre, on each side of the rod or
- the area derived from the actual geometry.

(2) In a group of rods, the characteristic resistance parallel to the grain of one rod, \( R_{ax,k} \), should be taken as:

\[
R_{ax,k} = f_{t,0,k} A_{ef}
\]

where:

\[ R_{ax,k} \] is the characteristic load-carrying capacity of one rod

\[ f_{t,0,k} \] is the characteristic tensile strength of the wood

\[ A_{ef} \] is the effective timber failure area.

Figure A.2 - Effective areas for anchorage forces parallel to grain.

A.3 LATERALLY LOADED RODS

(1) The provisions of 8.6 for laterally loaded dowels apply.
(2) For laterally loaded glued-in rods inserted parallel to the grain, the embedding strength should be taken as 10% of the embedding strength perpendicular to grain.

(3) For glued-in rods inserted at an angle $\alpha$ to the grain, linear interpolation between the embedding strengths at 0° and 90° to the grain should be applied.

### A.4 COMBINED LATERALLY AND AXIALLY LOADED RODS

(1) For combined laterally and axially loaded glued-in rods, the following expression should be satisfied:

$$\left( \frac{F_{ax,d}}{R_{ax,d}} \right)^2 + \left( \frac{F_{la,d}}{R_{la,d}} \right)^2 \leq 1$$

(A.6)

where:

$R_{ax,d}$ and $R_{la,d}$ are the design load-carrying capacities of the glued-in rod with axial load $F_{ax,d}$ or lateral load $F_{la,d}$ alone.

### A.5 SERVICEABILITY LIMIT STATES

(1) For joints made with glued-in rods the slip modulus $K_{ser}$ per shear plane per fastener at design load should be taken from Table A.1 with $\rho_m$ in kg/m³ and $d$ in mm.

<table>
<thead>
<tr>
<th>Glued-in rods</th>
<th>Axially loaded</th>
<th>Laterally loaded</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inserted parallel to grain</td>
<td>$\rho_m^{1.5} d^{1.8}/40$</td>
<td>$\rho_m^{1.5} d/12$</td>
</tr>
<tr>
<td>Inserted perpendicular to grain</td>
<td>$\rho_m^{1.5} d^{1.8}/40$</td>
<td>$\rho_m^{1.5} d/25$</td>
</tr>
</tbody>
</table>

(2) For glued-in rods inserted at an angle $\alpha$ to grain, linear interpolation should be applied between the $K_{ser}$ values for axially loaded and laterally loaded rods.

### A.6 STRUCTURAL DETAILING

(1) The surfaces of the holes should be clean cut.

(2) Where a number of rods in a group are to be tightened, the tightening should be uniform across the group.

(3) It should be ensured that the hole is completely filled with adhesive.

(4) At the time of gluing rods perpendicular to the grain, the moisture content of the timber should not be more than 3% higher than the minimum moisture content expected in service, when averaged across the section of the piece. For rods glued parallel to grain, this moisture content deviation should be limited to 6%.
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 $\ell$. For continuous beams the expressions may be used with $\ell$ equal to 0.8 of the relevant span and for cantilevered beams with $\ell$ 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 $s_{\text{min}}$ and $s_{\text{max}}$, with $s_{\text{max}} \leq 4 \times s_{\text{min}}$
- 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 $(EI)_e$ determined in accordance with B.2.
Key
(1) spacing : s₁  slip modulus : K₁  load : F₁
(2) spacing : s₃  slip modulus : K₃  load : F₃

Figure B.1 - Cross-section (left) and distribution of bending stresses (right). All measurements are positive except for a₂ which is taken as positive as shown.
B.2 EFFECTIVE BENDING STIFFNESS

(1) The effective bending stiffness should be taken as:

\[(EI)_{ef} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i \gamma^2)\]  \hspace{1cm} (B.1)

using mean values of \(E\) and where:

\[A_i = b_i h_i\]  \hspace{1cm} (B.2)

\[I_i = \frac{b_i h_i^3}{12}\]  \hspace{1cm} (B.3)

\[\gamma_2 = 1\]  \hspace{1cm} (B.4)

\[\gamma_i = \left[1 + \pi^2 E_i A_i \gamma^2 (K_i d_i^2)\right]^{-1}\] for \(i = 1\) and \(i = 3\) \hspace{1cm} (B.5)

\[\gamma_i = \frac{\gamma_i E_i A_i (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^{3} \gamma_i E_i A_i}\] \hspace{1cm} (B.6)

where

symbols are defined in Figure B.1

\[K_i = K_{si,i}\] for the serviceability limit states calculations

\[K_i = K_{ui,i}\] for the limit states calculations

For T-sections \(h_3 = 0\)

B.3 NORMAL STRESSES

(1) The normal stresses should be taken as:

\[\sigma_i = \frac{\gamma E_i a_i M}{(EI)_{ef}}\] \hspace{1cm} (B.7)

\[\sigma_{m,i} = \frac{0.5 E_i b_i M}{(EI)_{ef}}\] \hspace{1cm} (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:

\[\tau_{2,max} = \frac{\gamma_3 E_3 a_3 + 0.5 E_2 b_2 h_3}{b_2 (EI)_{ef}} V\] \hspace{1cm} (B.9)
B.5 FASTENER LOAD

(1) The load on a fastener should be taken as:

\[ F_i = \frac{\gamma E_i A_i a_i s_i}{(EI)_{et}} V \]  \hspace{1cm} (B.10)

where:

\[ i = 1 \text{ and } 3, \]

and where:

\[ s_i = s_i(x) \] is the spacing of the fasteners as defined in B.1.3(1) and \( V = V(x) \)
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 \( \ell \)
- 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:

\[
\sigma_{c,0,d} \leq k_c f_{c,0,d}
\]

where:

\[
\sigma_{c,0,d} = \frac{F_{c,d}}{A_{tot}}
\]

\( A_{tot} \) is the total cross-sectional area
\( k_c \) is determined in accordance with 6.3.2 but with an effective slenderness ratio \( \lambda_{ef} \) 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 :

\[
\lambda_{ef} = \ell \sqrt{\frac{A_{tot}}{I_{ef}}}
\]

where :

\[
I_{ef} = \frac{(EI)_{ef}}{E_{mean}}
\]

and \((EI)_{ef}\) 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
C.2.3 COMBINED LOADS

(1) In cases where small moments resulting from e.g. self weight are acting apart from axial load, B.4 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 with shafts spaced with 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 \( \ell_2 \) satisfies the condition: \( \ell_2/a \geq 1.5 \)
- 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: \( \ell_2/a \geq 2 \)
- the columns are subjected to concentric axial loads.

(4) For columns with two shafts \( A_{tot} \) and \( I_{tot} \) should be calculated as

\[
A_{tot} = 2A
\]

\[
I_{tot} = \frac{b\left[(2h + a)^3 - a^3\right]}{12}
\]

(5) For columns with three shafts \( A_{tot} \) and \( I_{tot} \) should be calculated as

\[
A_{tot} = 3A
\]

\[
I_{tot} = \frac{b\left[(3h + 2a)^3 - (h + 2a)^3 + h^3\right]}{12}
\]
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

\[
\lambda_{\text{ef}} = \sqrt{\lambda^2 + \frac{n}{2} \lambda_i^2}
\]  

where:

\( \lambda \) is the slenderness ratio for a solid column with the same length, the same area \( A_{\text{tot}} \) and the same second moment of area \( I_{\text{tot}} \), i.e.,
\[ \lambda = \ell \sqrt{\frac{A_{tot}}{I_{tot}}} \]  

(C.11)

\( \lambda_i \) is the slenderness ratio for the shafts and has to be set into expression (C.11) with a minimum value of at least 30.

\( n \) is the number of shafts

\[ \lambda_i = \sqrt{\frac{\ell_i}{h}} \]

\( \eta \) is a factor given in Table C.1

<table>
<thead>
<tr>
<th></th>
<th>Packs</th>
<th>Gussets</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Glued</td>
<td>Nailed</td>
</tr>
<tr>
<td>Permanent/long-term loading</td>
<td>1 4 3,5 3</td>
<td>6</td>
</tr>
<tr>
<td>Medium/short-term loading</td>
<td>1 3 2,5 2</td>
<td>4,5</td>
</tr>
</tbody>
</table>

\(^a\) 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:

\[ T_d = \frac{V_d l_t}{a_t} \]  

(C.12)
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 $\ell/2$, where $\ell$ 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 $\ell_1$ is not greater than 60
- no local buckling occurs in the flanges corresponding to the column length $\ell_1$
- 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 $\theta$ is the inclination of the diagonals.
Figure C.3 - Lattice columns. The area of one flange is \( A_f \) and the second moment of area about its own centre of gravity is \( I_f \).

(a) V-truss  (b) N-truss

**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

\[
\lambda_{\text{ef}} = \max \left\{ \lambda_{\text{tot}} \sqrt{1 + \mu}, 1.05 \lambda_{\text{tot}} \right\}
\]  

(C.13)
where:

\[ \lambda_{\text{tot}} \]

is the slenderness ratio for a solid column with the same length, the same area and the same second moment of area, i.e.

\[ \lambda_{\text{tot}} \approx \frac{2\ell}{h} \]  \hfill (C.14)

and \( \mu \) takes the values given in clauses (3) to (6)

(3) For a glued V-truss:

\[ \mu = 4 \frac{e^2 A_f}{I_f} \left( \frac{h}{\ell} \right)^2 \]  \hfill (C.15)

where:

- \( e \) is the eccentricity of the joints (see Figure C.3)
- \( A_f \) is the area of the flange (see Figure C.3)

(4) For a glued N-truss:

\[ \mu = e^2 A_f \left( \frac{h}{\ell} \right)^2 \]  \hfill (C.16)

(5) For a nailed V-truss:

\[ \mu = \frac{25}{\ell^2 n} \frac{h E_{\text{mean}} A_f}{K_u \sin 2\theta} \]  \hfill (C.17)

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.

(6) For a nailed N-truss:

\[ \mu = \frac{50}{\ell^2 n} \frac{h E_{\text{mean}} A_f}{K_u \sin 2\theta} \]  \hfill (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).
- \( K_u \) 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): SIMPLIFIED DESIGN EXPRESSIONS FOR DOWEL TYPE FASTENERS

D.1  Timber-to-timber and panel-to-timber connections

D.1.1  General

(1) Simplified general conservative expressions for timber-to-timber or panel-to-timber connections are given below:

- Characteristic load-carrying capacities for fasteners in single shear:

\[
R_k = \min \left\{ \frac{1}{1 + \frac{1 + \beta}{\beta}} f_{h,1,k} t_1 d \right\} \tag{a}
\]

\[
= \min \left\{ \frac{1}{1 + \sqrt{(1 + \beta)}} f_{h,2,k} t_2 d \right\} \tag{b}
\]

\[
= 1.15 k_{cal} \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,k} f_{h,1,k} d} \tag{c}
\]

- Characteristic load-carrying capacities for fasteners in double shear:

\[
R_k = \min \left\{ \frac{1}{1 + \frac{1 + \beta}{\beta}} f_{h,1,k} t_1 d \right\} \tag{a}
\]

\[
= \min \left\{ 0.5 f_{h,2,k} t_2 d \right\} \tag{b}
\]

\[
= 1.15 k_{cal} \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,k} f_{h,1,k} d} \tag{c}
\]

(2) \( k_{cal} \) should be taken from SECTION 8

D.1.2  Nails

(1) For nailed timber-to-timber connections, the characteristic load carrying capacity may be found from:

\[
R_k = 1.15 k_{cal} \sqrt{2M_{y,k} f_{h,1,k} d} \tag{D.3}
\]

provided that :

- The headside member thickness is larger than 7\( d \)
- The pointside penetration depth is at least :
  - 12\( d \) for smooth nails
  - 8\( d \) for helically threaded nails
D.1.3 Special case of connections with only one timber strength class

(1) For this special case, simplified expressions may be found below:

- Characteristic load-carrying capacities for fasteners in single shear:
  \[
  R_k = \min \begin{cases} 
    0.4 \ f_{h,1,k} \ t_1 \ d & \text{(a)} \\
    0.4 \ f_{h,2,k} \ t_2 \ d & \text{(b)} \\
    1.15 k_{cal} \sqrt{2M_{y,k} f_{h,1,k}} & \text{(c)}
  \end{cases}
  \]

- Characteristic load-carrying capacities for fasteners in double shear:
  \[
  R_k = \min \begin{cases} 
    0.4 \ f_{h,1,k} \ t_1 \ d & \text{(a)} \\
    0.5 \ f_{h,2,k} \ t_2 \ d & \text{(b)} \\
    1.15 k_{cal} \sqrt{2M_{y,k} f_{h,1,k}} & \text{(c)}
  \end{cases}
  \]

(2) \( k_{cal} \) should be taken from SECTION 8

D.2 Steel to timber connections

(1) The characteristic load-carrying capacity per fastener for double shear steel-to-timber connections should, for inner thick (i.e. for \( t \geq d \)) or thin steel plates (i.e. for \( t \leq 0.5d \)) or for outer thick steel plates, be taken as the smaller value found from the following expressions:

  \[
  R_k = \min \begin{cases} 
    0.5 \ f_{h,k} \ t \ d & \text{(a)} \\
    2.3 \frac{k_{cal} \sqrt{M_{y,k} f_{h,k}}}{d} & \text{(b)}
  \end{cases}
  \]

  - For outer thin steel plates the smaller value found from the following expressions apply:

  \[
  R_k = \min \begin{cases} 
    0.5 \ f_{h,k} \ t \ d & \text{(a)} \\
    1.15 k_{cal} \sqrt{2M_{y,k} f_{h,k}} & \text{(b)}
  \end{cases}
  \]

  - For all other cases:

  \[
  R_k = \min \begin{cases} 
    0.4 \ f_{h,k} \ t \ d & \text{(a)} \\
    k_{cal} \sqrt{2M_{y,k} f_{h,k}} & \text{(b)}
  \end{cases}
  \]

For intermediate steel plate thicknesses \( 0.5d \leq t \leq d \), linear interpolation may be used.

(2) \( k_{cal} \) should be taken from SECTION 8