

Eurocode 5: Design of timber structures —

**Part 1.1: General rules and rules for
buildings —**

**(together with United Kingdom
National Application Document)**

UDC 624.92.016.02:624.07

Cooperating organizations

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National foreword

This publication comprises the English language version of ENV 1995-1-1:1993 *Eurocode 5 — Design of timber structures — Part 1.1: General rules and rules for buildings*, as published by the European Committee for Standardization (CEN), plus the National Application Document (NAD) to be used with the ENV for the design of buildings to be constructed in the United Kingdom (UK).

ENV 1995-1-1:1993 results from a programme of work sponsored by the European Commission to make available a common set of rules for the design of building and civil engineering works.

An ENV is made available for provisional application, but does not have the status of a European Standard. The aim is to use the experience gained during the ENV period to modify the ENV so that it can be adopted as a European Standard.

The values for certain parameters in the ENV Eurocodes may be set by CEN members so as to meet the requirements of national regulations. These parameters are designated by (boxed values) in the ENV.

It should be noted that ENV 1995-1-1 design is based on partial factors and characteristic values for actions and material properties, in contrast to BS 5268 which uses permissible stress values.

During the ENV period reference should be made to the supporting documents listed in the National Application Document (NAD). The purpose of the NAD is to provide essential information, particularly in relation to safety, to enable the ENV to be used for buildings constructed in the UK. The NAD takes precedence over corresponding provisions in the ENV.

The Building Regulations 1991, Approved Document A 1992 (published December 1991), draws designers' attention to the potential use of ENV Eurocodes as an alternative approach to Building Regulation compliance. ENV 1995-1-1 has been thoroughly examined over a period of several years and is considered to offer such an alternative approach, when used in conjunction with this NAD.

Compliance with ENV 1995-1-1:1993 and the NAD does not of itself confer immunity from legal obligations.

Users of this document are invited to comment on its technical content, ease of use and any ambiguities or anomalies. These comments will be taken into account when preparing the UK national response to CEN to the question of whether the ENV can be converted to an EN.

Comments should be sent in writing to BSI, British Standards House, 389 Chiswick High Road, Chiswick, London W4 4AL, quoting the document reference, the relevant clause and, where possible, a proposed revision, within 2 years of the issue of this document.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to xxii, the ENV title page, pages 2 to 76, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

National Application Document for use in the UK with ENV 1995-1-1:1993

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Introduction

This National Application Document (NAD) has been prepared under the direction of the Technical Sector Board for Building and Civil Engineering. It has been developed from:

- a) a textual examination of ENV 1995-1-1:1993;
- b) a parametric calibration against UK practice, supporting standards and test data;
- c) trial calculations.

It should be noted that this NAD, in common with ENV 1995-1-1 and supporting CEN standards, uses a comma where a decimal point would be used in the UK.

1 Scope

This NAD provides information required to enable ENV 1995-1-1:1993 (EC5-1.1) to be used for the design of buildings and civil engineering structures to be constructed in the UK.

2 References

2.1 Normative references

This National Application Document incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and the cited publications are listed on page xxi. For dated references, only the edition cited applies: any subsequent amendments to or revisions of the cited publication apply to this British Standard only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the cited publication applies, together with any amendments.

2.2 Informative references

This National Application Document refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on page xxii, but reference should be made to the latest editions.

3 Partial safety factors, combination factors and other values

- a) The values for partial safety factors (γ) should be those given in Table 1 of this NAD.
- b) The values for combination factors (Ψ) should be those given in Table 2 of this NAD.
- c) The values for other boxed values should be those given in Table 3 of this NAD.

Table 1 — Partial safety factors (γ factors)

Reference in EC5-1.1	Definition	Symbol	Condition	Value	
				Boxed EC5	UK
2.3.3.1	Partial factors for variable actions	γ_A	Accidental	1,00	1,00
		$\gamma_{F,inf}$	Favourable	0,0	0,0
		γ_Q	Unfavourable	1,5	1,5
		γ_Q	Reduced favourable	0,0	0,0
		γ_Q	Reduced unfavourable	1,35	1,35
2.3.3.1	Partial factors for permanent actions	γ_{GA}	Accidental	1,0	1,0
		γ_G	Favourable	1,0	1,0
		γ_G	Unfavourable	1,35	1,35
		$\gamma_{G,inf}$	Favourable	0,9	0,9
		$\gamma_{G,sup}$	Unfavourable	1,1	1,1
		γ_G	Reduced favourable	1,0	1,0
		γ_G	Reduced unfavourable	1,2	1,2
2.3.3.2	Partial factors for materials	γ_M	Timber and wood based materials	1,3	1,3
		γ_M	Steel used in joints	1,1	1,1
		γ_M	Accidental	1,0	1,0
		γ_M	Serviceability	1,0	1,0

Table 2 — Combination factors (ψ factors)

Variable action	Building type	ψ_0	ψ_1	ψ_2
Imposed floor loads	Dwellings	0,5	0,4	0,2
	Other occupancy classes ^a	0,7	0,6	0,3
	Parking	0,7	0,7	0,6
Imposed ceiling loads	Dwellings	0,5	0,4	0,2
	Other occupancy classes ^a	0,7	0,2	0,0
Imposed roof loads	All occupancy classes ^a	0,7	0,2	0,0
Wind loads				

^a As listed and defined in Table 1 of BS 6399-1:1984.

Table 3 — Boxed values (other than γ values)

Reference in EC5-1.1	Definition	Value	
		Boxed EC5	UK ^a
4.3.1(2)	Deflections		
	General $u_{2,\text{inst}}$ For cantilever $u_{2,\text{inst}}$	$\leq l/300$ $\leq l/150$	$\leq l/300$ $\leq l/150$
4.3.1(3)	General $u_{2,\text{fin}}$	$\leq l/200$	$\leq l/200$
	For cantilever $u_{2,\text{fin}}$	$\leq l/100$	$\leq l/100$
	General $u_{\text{net,fin}}$	$\leq l/200$	$\leq l/200$
	For cantilever $u_{\text{net,fin}}$	$\leq l/100$	$\leq l/100$
4.4.2(2)	Vibrations From machinery multiplying factor	1	1
4.4.3(2)	Residential floors equation (4.4.3a) equation (4.4.3b)	1,5 100	1,5 100
^a Unlike EC5-1.1, this NAD requires 5-percentile stiffness moduli to be used to calculate deformations for solid timber members acting alone [see 6.4 a) of this NAD].			

4 Loading codes

The loading codes to be used are:

BS 648, *Schedule of weights of building materials*.

BS 6399, *Design loading for buildings*.

BS 6399-1, *Code of practice for dead and imposed loads*.

BS 6399-3, *Code of practice for imposed roof loads*.

CP 3, *Code of basic data for the design of buildings*.

CP 3:Chapter V, *Loading*.

CP 3:Chapter V-2, *Wind loads*.

In using these documents with EC5-1.1 the following modifications should be noted:

- Loads from separate sources or of different durations acting on a member or component should be considered as separate actions.
- The design loading on a particular member or component may include the relevant load combination factors described in 2.3.2.2 and 4.1 of EC5-1.1. Alternatively for the ultimate limit state the simplification of design load given in 2.3.3.1(5) of EC5-1.1 may be used. For deformations a simplification is given in 6.4 b) of this NAD.
- The reductions in total imposed floor load described in clause 5 of BS 6399-1:1984 should be disregarded.
- Snow loads arising from local drifting should be treated as an accidental loading condition with the local drift being the accidental action A_d in equation (2.3.2.2b) of EC5-1.1, and the duration of this accidental loading being short term.
- The wind loading should be taken as 90 % of the value obtained from CP 3:Chapter V-2.

5 Reference standards

The supporting standards to be used, including materials specifications and standards for construction, are listed in Table 4.

Table 4 — References in EC5 to other publications

Reference in EC5	Document referred to	Document title or subject area ^a	Status	UK document ^b
2.1P(2)	—	Requirements on accidental damage and structural integrity	—	Approved Document A of the Building Regulations [1]
2.2.2.2	ENV 1991	<i>Basis of design and actions on structures</i>	In preparation	BS 648 BS 6399 CP 3 (See clause 4 of this NAD)
2.3.1P(4)	—	Testing	—	Section 8 of BS 5268:1991 BS EN 380 BS EN 595 ^c BS 5268-6.1
2.4.2P(1)	EN 350-2 EN 335-1 EN 335-2 prEN 335-3 prEN 351-1 prEN 460	Durability of wood Hazard classes of wood and wood-based products against biological attack Preservative treatment Guide to durability requirements	prEN subject to CEN formal vote 1992 1992 prEN subject to CEN formal vote prEN subject to CEN formal vote Published 1994	BS EN 350-2 ^c BS EN 335-1 BS EN 335-2 BS EN 335-3 ^c BS EN 351-1 ^c BS EN 460
Table 2.4.3	ISO 2081 EN 10147	Metallic coatings	1986 1991	BS EN 10147
Table 3.1.7	prEN 312 prEN 300 prEN 622	Particleboards OSB Fibreboards	prEN subject to CEN formal vote prEN subject to CEN enquiry prEN subject to CEN enquiry	BS EN 312 ^c BS EN 300 ^c BS EN 622 ^c
3.2.1P(3)	prEN 518	Visual grading	prEN subject to CEN formal vote	BS EN 518 ^c
3.2.1P(4)	prEN 519	Machine grading	prEN subject to CEN formal vote	BS EN 519 ^c
3.2.2	prEN 338 prEN 384 prEN 408 prEN 1193	Strength classes of structural timber Characteristic values Test methods Test methods	prEN subject to CEN formal vote prEN subject to CEN formal vote prEN subject to CEN formal vote prEN subject to CEN enquiry	BS EN 338 ^c BS EN 384 ^c BS EN 408 ^c prEN 1193 ^c
3.2.3	prEN 336	Timber sizes and tolerances	prEN subject to CEN formal vote	BS EN 336 ^c
3.2.5	prEN 385	Finger joints	prEN subject to CEN formal vote	BS EN 385 ^c
3.3.1	prEN 386	Performance and production of glued laminated timber	prEN subject to CEN formal vote	BS EN 386 ^c
3.3.2	prEN 408 prEN 1193 prEN 1194	Test methods for glued laminated timber Test method Characteristic values	prEN subject to CEN formal vote prEN subject to CEN enquiry prEN subject to CEN enquiry	BS EN 408 ^c BS EN 1193 ^c BS EN 1194 ^c
3.3.3	prEN 390	Sizes of glued laminated timber	prEN subject to CEN formal vote	BS EN 390 ^c
3.3.5	prEN 387	Performance and production of large finger joints	prEN subject to CEN formal vote	BS EN 387 ^c
3.4.1	prEN 636-1 prEN 636-2 prEN 636-3 prEN 1058	Plywood Characteristic values of wood-based panels	prENs subject to CEN enquiry	BS EN 636-1 ^c BS EN 636-2 ^c BS EN 636-3 ^c BS EN 1058 ^c ITD/1 [2]

^a See 1.7 of EC5-1.1 for titles of European Standards, published and in preparation.

^b For titles of published UK documents see the list of references to this NAD.

^c British Standard in preparation.

Table 4 — References in EC5 to other publications

Reference in EC5	Document referred to	Document title or subject area ^a	Status	UK document ^b
3.4.2	prEN 312-4 prEN 312-5 prEN 312-6 prEN 312-7 prEN 300 prEN 1058	Particleboards OSB Characteristic values of wood-based panels	prENs subject to CEN enquiry	BS EN 312 ^c BS EN 300 ^c BS EN 1058 ^c ITD/2 [3]
3.4.3	prEN 622-3 prEN 622-5 prEN 1058	Fibreboards Characteristic values of wood-based panels	prENs subject to CEN enquiry	BS EN 622 ^c BS EN 1058 ^c ITD/2 [3]
3.5	EN 301	Adhesives	1992	BS EN 301
4.1(3)	EN 26891	Strength and deformation of joints made with mechanical fasteners	1991	BS EN 26891
Table 4.1	prEN 312-1 prEN 300 prEN 622-1	Particleboards OSB Fibreboards	prEN subject to CEN formal vote prEN subject to CEN enquiry prEN subject to CEN enquiry	BS EN 312 ^c BS EN 300 ^c BS EN 622-1 ^c
4.4.2	ISO 2631-2	Vibration	1989	
5.4.3	prEN 594	Design and testing of wall panels	prEN subject to CEN enquiry	BS 5268-6.1
6.1P(1)	EN 26891 EN 28970	Strength and deformation of joints made with mechanical fasteners Test methods	1991 1991	BS EN 26891 BS EN 28970
7.9.1 7.9.2(2)	prEN 1059	Trusses	prEN subject to CEN enquiry	—
D.2(3)	EN 26891	Strength and deformation of joints made with mechanical fasteners	1991	BS EN 26891
D.6.3 1) D.6.4	prEN 1075	Test methods for joints made from punched metal plates	No draft	

^a See 1.7 of EC5-1.1 for titles of European Standards, published and in preparation.
^b For titles of published UK documents see the list of references to this NAD.
^c British Standard in preparation.

6 Additional recommendations

6.1 Guidance on EC5-1.1

When designing to EC5-1.1, 6.2 to 6.7 should be followed.

6.2 Chapter 2. Basis of design

a) Clause 2.1P(2)

The design requirements for providing structural integrity by limiting the effects of accidental damage are given in sections 5 and 6 of Approved Document A 1992 of the Building Regulations 1991 [1].

a) Clause 2.3.1P(4)

ENV 1991-1 (EC1-1.1) *Basis of design* is currently being drafted to give guidance on the structural testing and evaluation procedures to be used when the design information in Eurocodes 2 to 9 is insufficient, or where economies may result from tests on prototypes.

Until the above document is available, prototype testing of assemblies should be carried out to the standards listed below, and the results assessed in accordance with the requirements of section 8 of BS 5268-3:1985 for trussed rafters and section 8 of BS 5268-2:1991 for other assemblies, modified as follows.

NOTE For the design and testing of timber frame wall panels, see 6.5 d) of this NAD.

Tests should be carried out to:

BS EN 380 for general structural components;

BS EN 595¹⁾ for trussed rafters.

The worst loading condition, referred to as the design load, should be determined without the use of the γ_F and Ψ factors.

For trussed rafters the acceptance load should be assessed in accordance with **39.3** of BS 5268-3:1985 except that the value for K_t should be taken from Table 5 of this NAD assuming loads are long term. For other assemblies the acceptance load should be determined by multiplying the design load by the relevant factor from Table 5. The material categories are those given in Table 3.1.7 of EC5-1.1, i.e.

Category 1 Solid and glued laminated timber and plywood

Category 2 Particleboards to BS EN 312-6^{ab} and BS EN 312-7^a
OSB to BS EN 300^a Grades 3 and 4

Category 3 Particleboards to BS EN 312-4^{ab} and BS EN 312-5^a
OSB to BS EN 300^a Grade 2
Fibreboards to BS EN 622-5^a (hardboard)

Category 4 Fibreboards to BS EN 622-3^a (medium boards and hardboards)

^a In preparation.

^b Not to be used in service class 2.

Where actions of different durations act in combination, the shortest duration of the actions may be used to determine a factor from Table 5, provided its induced stress is at least 50 % of the total.

For trussed rafters the permissible deflections should be assessed in accordance with **39.2** of BS 5268-3:1985. For other multiple member components the permissible deflections for a prototype test should be assessed as given in clause **62** of BS 5268-2:1991, but the following should be noted.

— The “specified amount of deflection in the design” should be calculated as the instantaneous deflection (u_{inst}) in EC5-1.1.

— The definitions of duration of load for determining K_{72} and K_{80} should be those given in BS 5268-2. For example, if a load is to simulate a snow load, then the factor (K_{72} or K_{80}) would be determined for medium term as given in BS 5268-2, and not short term as given in EC5-1.1.

Table 5 — Factors for testing

Service duration of actions or combination of actions	Number tested	Material category			
		1	2	3	4
Long term	1	2,50	3,50	3,84	4,28
	2	2,29	3,21	3,53	3,94
	3	2,16	3,02	3,32	3,70
	4	2,05	2,87	3,17	3,52
	5 or more	2,00	2,80	3,07	3,42
Medium term	1	2,18	2,50	2,66	2,58
	2	2,01	2,29	2,45	2,62
	3	1,89	2,16	2,30	2,47
	4	1,80	2,05	2,19	2,35
	5 or more	1,74	2,00	2,13	2,28
Short term	1		1,94	2,03	2,14
	2		1,79	1,88	1,96
	3		1,68	1,75	1,84
	4		1,60	1,68	1,77
	5 or more		1,56	1,62	1,71
Instantaneous	1			1,59	
	2			1,46	
	3			1,37	
	4			1,30	
	5 or more			1,27	

¹⁾ In preparation.

6.3 Chapter 3. Material properties

a) Clause 3.1.5

The examples in Table 6 indicate the appropriate service class.

Table 6 — Examples of appropriate service class

Service class	Environmental conditions
1	Timber in buildings with heating and protected from damp conditions. Examples are internal walls, internal floors (other than ground floors) and warm roofs.
2	Timber in covered buildings. Examples are ground floor structures where no free moisture is present, cold roofs, the inner leaf of cavity walls and external single leaf walls with external cladding.
3	Timber fully exposed to the weather. Examples are the exposed parts of open buildings and timber used in marine structures.

b) Clause 3.1.6P(2)

The load duration class is not only determined by the estimated duration of the characteristic load but also, to a lesser extent, by the duration of loads lower than the characteristic value. In view of the difficulty of assessing the appropriate duration without specialist knowledge, the examples given in EC5-1.1 should be used for design in the UK, with the following additional information.

- 1) Imposed roof loads, where access is limited to maintenance and repair, should be considered as short term actions.
- 2) Imposed roof loads, other than snow loads, where access is not limited should be considered as medium term actions.
- 3) Uniformly distributed imposed loads on ceilings should be considered as long term actions.

c) Table 3.1.7

Because it is difficult to dry timber more than 100 mm thick, unless it is specially dried, the stresses and moduli for service class 3 should normally be used for solid timber members more than 100 mm thick.

d) Clause 3.2.1

Visual and machine strength grading should be carried out under the control of a third party certification body, authorized by the UK Timber Grading Committee. Only structural timber carrying the mark of a certification body approved by the UK Timber Grading Committee should be used (see Annex A).

e) Clause 3.2.2

Characteristic values should be obtained from the strength classes given in BS EN 338²⁾.

If characteristic values are developed for use outside the strength class system, they should be assessed by the British Standards Institution working group, B525/5/WG1, and the grading quality control and certification should be assessed by the UK Timber Grading Committee.

Machine grading is carried out directly to the strength class boundaries and the timber is marked accordingly with a strength class number. Species which can be machine graded, and the strength classes to which they are assigned, are given in Table 7, Table 8 and Table 9.

Table 7, Table 8 and Table 9 also give the strength classes to which various visual grades and species are assigned.

f) Clause 3.2.2P(3)

No size adjustments to tension perpendicular to grain and shear stresses are applicable for solid timber.

g) Clause 3.2.3

Timber sizes normally available in the UK are given in the National annex to BS EN 336²⁾.

²⁾ In preparation.

Although BS 4978 does not permit cross-section sizes less than 35 mm × 60 mm to be stress graded, research now shows that sizes down to and including 35 mm × 45 mm may be graded to BS 4978 and are acceptable for use with the design rules of EC5-1.1 and the strength properties of BS EN 338³⁾.

In common with BS 5268-2, the S8 and S6 grades, specified in the ECE standard on sawn timber [6], are interchangeable with the SS and GS grades, respectively, of BS 4978.

NOTE The grading rules for the two standards mentioned above differ for square cross sections. The ECE rules for square sizes are equally acceptable in terms of strength properties and give higher yields.

h) Clause 3.2.5

For EC5-1.1, finger joints to any required specification should be manufactured and tested to BS EN 385³⁾ to determine their characteristic strength. This is different from the system used in BS 5268-2, where a table relates grades to efficiencies, and in BS 5291 which relates efficiencies to joint specification.

It is essential that finger joints in principal members, as defined in BS 5268-2, have third party quality assurance by a certification body approved by the National Accreditation Council of Certification Bodies (NACCB).

i) Clause 3.3.2(2)

The glulam strength class system given in prEN 1194 makes use of laminates from the solid timber strength class system given in BS EN 338³⁾.

j) Clause 3.3.2P(3)

The definition of width in tension is the largest cross-sectional dimension, which for glulam is usually the summation of the laminate thicknesses.

k) Clause 3.2.5

It is essential that large finger joints have third party quality assurance by a certification body approved by the NACCB.

l) Clause 3.4.1.2

Until the European standard with characteristic values for plywood is published, values converted from BS 5268-2 and given in ITD/1 [2] should be used.

m) Clause 3.4.2.2

Until the European standard with characteristic values for particleboards is published, values for structural grade chipboard should be obtained from ITD/2 [3].

n) Clause 3.4.3.2

Until the European standard with characteristic values for fibreboards is published, values converted from the grade values in BS 5268-2 may be used for tempered hardboard. These should be obtained from ITD/2 [3].

³⁾ In preparation.

Table 7 — BS 4978 and NLGA^a NGRDL^b joist and plank visual grades and species and CEN machine grades assigned to strength classes

Type	Species	Source	Grade ^c	Strength class
Imported softwoods	Redwood	Europe	SS GS Machine	C24 C16 C14 to C30
	Whitewood	Europe	SS GS Machine	C24 C16 C14 to C30
	Hem-fir, S-P-F and DF-L	Canada and USA	SS GS Machine JP Sel JP No. 1 JP No. 2	C24 C16 C14 to C30 C24 C16 C16
	Sitka spruce	Canada	SS GS JP Sel JP No. 1 JP No. 2	C18 C14 C18 C14 C14
	Southern pine	USA	SS GS Machine JP Sel JP No. 1 JP No. 2 JP No. 3 ^d	C24 C18 C14 to C30 C30 C22 C22 C16
	Western white woods	USA	SS GS JP Sel JP No. 1 JP No. 2	C18 C14 C18 C14 C14
	Western red cedar	Imported	SS GS	C18 C14
	Parana pine	Imported	SS GS	C24 C16
	Pitch pine	Caribbean	SS GS	C27 C18
	Radiata pine	New Zealand	Machine	C14 to C30
	Radiata pine	Chile	Machine	C14 to C30
	S. African pine	S. Africa	Machine	C14 to C30
	Zimbabwean pine	Zimbabwe	Machine	C14 to C30
British grown softwoods	Spruce	UK and Ireland	SS Machine	C18 C14 to C24
	Pine		SS GS Machine	C22 C14 C14 to C27
	Larch		SS GS Machine	C24 C16 C14 to C27
	Douglas fir		SS GS Machine	C18 C14 C14 to C24

^a National Lumber Grades Authority (Canada)

^b National Grading Rules for Dimension Lumber (USA).

^c Grades are from the following: SS and GS from BS 4978, JP from *Standard grading rules for Canadian lumber* [4] and *The national grading rules for dimensioned lumber* [5] and machine grades from EN 519 (in preparation).

^d Joist and plank grade No. 3 should not be used for tension members.

Table 8 — NLGA^a/NGRDL^b structural light framing, light framing and stud grades assigned to strength classes

Species	Source	Grade	Section size (mm)					
			38 × 89	38 × 38	38 × 63	63 × 63	63 × 89 89 × 89	38 × 114 38 × 140
DF-L	Canada and USA	SLF Sel	C24	C24	C24	C24	C24	
		SLF No. 1	C16	C18	C18			
		SLF No. 2	C16	C18	C18			
		SLF No. 3 ^c		C14	C14	C14		
		LF Const ^c	C14				C14	
		Stud ^c	C14	C14	C14	C14		
Hem-fir	Canada and USA	SLF Sel	C24	C24	C24	C24	C24	
		SLF No. 1	C16	C18	C18			
		SLF No. 2	C16	C18	C18			
		SLF No. 3 ^c		C14	C14	C14		
		LF Const ^c	C14				C14	
		Stud ^c	C14	C14	C14	C14		
S-P-F	Canada and USA	SLF Sel	C24	C24	C24	C24	C24	
		SLF No. 1	C16	C18	C18			
		SLF No. 2	C16	C18	C18			
		SLF No. 3 ^c		C14	C14	C14		
		LF Const ^c	C14				C14	
		Stud ^c	C14	C14	C14	C14		
Sitka spruce	Canada	SLF Sel	C16	C18	C18	C18	C18	
		SLF No. 1	C14	C14	C14			
		SLF No. 2	C14	C14	C14			
Western white woods	USA	SLF Sel	C16	C18	C18	C18	C18	
		SLF No. 1	C14	C14	C14			
		SLF No. 2	C14	C14	C14			
Southern pine	USA	SLF Sel	C27	C30	C27	C27	C24	
		SLF No. 1	C22	C24	C24			
		SLF No. 2	C22	C24	C24			
		SLF No. 3 ^c	C16	C18	C16	C18		
		LF Const ^c	C18	C16	C14	C14	C18	
		LF Std ^c	C14		C14	C14	C14	
		Stud ^c	C16	C18	C16	C16	C16	C16

^a National Lumber Grades Authority (Canada).^b National Grading Rules for Dimension Lumber (USA).^c Should not be used for tension members.

Table 9 — Hardwood grades and species assigned to strength classes

Type	Species	Source	Grade ^a	Strength class
Tropical hardwoods	Kapur	SE Asia	HS	D60
	Kempas	SE Asia	HS	D60
	Keruing	SE Asia	HS	D50
	Ekki	W Africa	HS	D60
	Balau	SE Asia	HS	D70
	Greenheart	SE Asia	HS	D70
	Iroko	Africa	HS	D40
	Jarrah	Australia	HS	D40
	Karri	Australia	HS	D50
	Merbau	SE Asia	HS	D60
	Opepe	Africa	HS	D50
	Teak	SE Asia and Africa	HS	D40
^a Grades are from BS 5756.				

6.4 Chapter 4. Serviceability limit states**a) Clause 4.1(3)**

Instantaneous deformation is defined as deformation without creep.

The instantaneous deformation of a solid timber member acting alone should be calculated using the appropriate 5-percentile modulus of elasticity ($E_{0,05}$) and/or 5-percentile shear modulus ($G_{0,05}$). Where two or more pieces of solid timber are joined together to act as a single member, the mean values of the elastic moduli may be used.

For solid timber, $G_{0,05} = 0,063E_{0,05}$

b) Clause 4.1(6)

Where a combination of actions with different load durations occurs on an element or structure, and when all such actions are uniformly distributed, the final deformation of that element or structure may be estimated directly under the combined action by using an effective k_{def} in equation (4.1b) of EC5-1.1 as follows: The design

$$k_{\text{def,ef}} = \frac{1}{Q_{\text{tot}}} \left[0.8 \Sigma G_{\text{kj}} + Q_{\text{k},1} k_{\text{def},1} + \sum_{i > 1} \psi_{1,i} Q_{\text{k},i} k_{\text{def},i} \right]$$

where

$k_{\text{def,ef}}$ is the effective deformation factor for the element or structure being considered under combined action Q_{tot} ;

Q_{tot} is the combined action calculated from equation (4.1a) of EC5-1.1;

$k_{\text{def},i}$ is the deformation factor from Table 4.1 of EC5-1.1 appropriate to the duration of action $Q_{\text{k},i}$.

c) Clause 4.2

The values of K_{ser} assume that holes in steel members have the minimum clearance compatible with the dowel-type fastener to be used (see 7.4 of EC5-1.1).

d) Clause 4.2(3) and 4.2(5)

The value of 1 mm incorporated in equations (4.2d) and (4.2e) of EC5-1.1 is to allow for the clearance hole for a bolt.

e) Clause 4.4.1

The requirements in this clause are not appropriate to normal UK floors, which are not fully supported on all four sides and do not have significant transverse stiffness. See 6.4 f) of this NAD.

f) *Clause 4.4.3*

For type 1 residential UK timber floors as defined in Table 1 of BS 6399-1, which are primarily supported on two sides only and do not have significant transverse stiffness, it is sufficient to check that the total instantaneous deflection of the floor joists under load does not exceed 14 mm or $l/333$, whichever is the lesser.

6.5 Chapter 5. Ultimate limit statesa) *Clause 5.2.2*

The value $\sigma_{m,crit}$ for rectangular sections can be obtained from the following equation:

$$\sigma_{m,crit} = \frac{0.75E_{0.05}b^2}{L_{ef}h}$$

The effective value of L_{ef} is governed by the degree of restraint against:

- 1) lateral deflection;
- 2) rotation in plan; and
- 3) twisting.

As a guide:

- where full restraint is provided against rotation in plan at both ends, $L_{ef} = 0,7L$;
- where partial restraint against rotation in plan is provided at both ends or full restraint at one end, $L_{ef} = 0,85L$;
- where partial restraint against twisting is provided at one or both ends, $L_{ef} = 1,2L$.

b) *Clause 5.4.1.4P(4)*

For the purposes of lateral stability calculations on rafter members, the effective length for out-of-plane buckling should be taken as the distance between discrete restraints, provided these are adequately fixed to the member and anchored to a braced part of the structure.

c) *Clause 5.4.1.5(1)*

Irrespective of any other design requirements, the maximum bay length of trussed rafter chord members, when measured on plan between node points, should be limited to the values given in Table 10 and the maximum overall length of internal members should be limited to the values given in Table 11.

Where necessary, intermediate values may be obtained by linear interpolation.

d) *Clause 5.4.3*

The design method for timber frame walls given in EC5-1.1 lacks sufficient information with regard to the determination of racking resistance (F_k) from the test method. This NAD will be revised in due course to give the required additional information. Until that revision is published, timber frame wall panels should be designed and tested to BS 5268-6.1.

6.6 Chapter 6. Jointsa) *Clause 6.1*

Glued joints should be designed taking into consideration the strength properties of the timber and/or wood-based members to be joined, which are assumed to be weaker than the glue.

Table 10 — Maximum bay length of rafters and ceiling ties

Depth of member (mm)	Maximum length (measured on plan between node points) (m)			
	35 mm thick		47 mm thick	
	Rafter	Ceiling tie	Rafter	Ceiling tie
72	1,8	2,4	3,0	3,0
97	2,3	3,0	3,3	4,0
122	2,5	3,4	3,6	4,7
147	2,8	3,7	3,8	5,0

Table 11 — Maximum length of internal members

Depth of member (mm)	Maximum length (measured on plan between node points) (m)	
	35 mm thick	47 mm thick
60	2,4	3,2
72	3,6	4,8
97	4,5	6,0

b) Clause 6.1(9)

The ultimate limit state slip modulus K_u is applicable in ultimate limit state calculations where the influence of fastener slip on strength needs to be considered. Joint slip should be calculated by dividing the applied design load per fastener (at either service load or ultimate load level) by the appropriate fastener slip modulus.

c) Clause 6.2.1

The formulae for R_d provide characteristic design resistances for the ultimate limit state. They should not be used to establish design resistances at the service load level.

Connected joints

Until prEN 912 (the specification for connectors for timber), a standard for characteristic load carrying capacities and slip moduli for connectors, and a design method for connected joints are available, characteristic load carrying capacities for connectors should be obtained by multiplying the basic loads tabulated in BS 5268-2 by a factor of 2,6 for toothed plate connectors and 2,9 for split-ring and shear plate connectors. The design capacity is given by the following equation:

$$R_d = k_{\text{mod}} \times \text{characteristic value for connector} / \gamma_M$$

where

k_{mod} and γ_M are the values for solid timber and glulam.

The standard spacings and distances given in BS 5268-2 should be used, unless the characteristic values are reduced by the appropriate factors for substandard spacings and distances, given in BS 5268-2.

6.6 Annex D. The design of trusses with punched metal plate fasteners**a) Clause D.2**

The value of u_{ser} can be taken as the average initial slip value published in the appropriate British Board of Agrément (BBA) certificate.

The value of K_{ser} can be obtained from the following equation:

$$K_{\text{ser}} = F_{a,00,k} \times A_{\text{ef}} / (2,5 \times u_{\text{ser}})$$

b) Clause D.6.2

Unless experience indicates that a larger tolerance is necessary, A_{ef} should allow for a minimum misalignment of 5 mm simultaneously in two directions parallel to the edges of the punched metal plate fastener. In addition, allowance should be made for any ineffective nails nearer than certain specified distances from the edges and ends of the timber and published in a British Standard, a BBA certificate or a certificate of assessment from an accredited body which provides equivalent levels of protection and performance.

c) Clause D.6.3

Characteristic anchorage capacities for metal plate fasteners should be obtained directly from a British Standard, a BBA certificate or a certificate of assessment from an accredited body which provides equivalent levels of protection and performance, by multiplying the permissible long-term loads per nail by a factor of 2,5 and dividing by the area per nail. Linear interpolation may be used to obtain intermediate characteristic capacities.

The design anchorage capacity is given by the following equation:

$$R_d = k_{\text{mod}} \times \text{characteristic anchorage capacity} / \gamma_M$$

where

k_{mod} and γ_{M} are the values for solid timber.

Characteristic tension, compression and shear capacities of metal plate fasteners should be obtained by multiplying the permissible forces published in the appropriate BBA certificate by 2,33. Linear interpolation may be used to obtain intermediate characteristic capacities.

The design tension, compression and shear capacities of metal plate fasteners are given by the following equation:

$$R_{\text{d}} = k_{\text{mod}} \times \text{characteristic capacity} / \gamma_{\text{M}}$$

where

$$k_{\text{mod}} = 1,0 \text{ and } \gamma_{\text{M}} = 1,1.$$

Annex A (informative)**Acceptable certification bodies for strength graded timber**

Certification bodies currently approved are listed in Table A.1 to Table A.5.

NOTE A leaflet containing an illustration of each grading stamp logo and the address of each certification body⁴⁾, can be obtained from the UK Timber Grading Committee, The Timber Trade Federation, Clareville House, 26/27 Oxendon Street, London SW1Y 4EL.

Table A.1 — Certification bodies approved to oversee the supply of visually strength graded timber to BS 4978

Certification body	UK representative	Certification body	UK representative
AFPA	COFI	QLMA	BPIB
CLA	BPIB	SPIB	WWPA/SFPA
CLMA	COFI	SGMCF	NTC
COFI	COFI	SSTCC	NTC
ILMA	COFI	TRADACERT	TRADACERT
MI	COFI	WCLIB	MI
MLB	BPIB	WWPA	WWPA
OLMA	BPIB		
PLIB (Can)	COFI		
PLIB (USA)	WWPA		

Table A.2 — Certification bodies operating under the Canadian Lumber Standards Accreditation Board (CLSAB) approved for the supply of visually strength graded timber to the NLGA grading rules

Certification body	UK representative	Certification body	UK representative
AFPA	COFI	MLB	BPIB
CLA	BPIB	NLPA	BPIB
CLMA	COFI	OLMA	BPIB
COFI	COFI	PLIB (Can)	COFI
ILMA	COFI	QLMA	BPIB
MI	COFI		

Table A.3 — Certification bodies operating under the American Lumber Standards Board of Review (ALS) approved for the supply of visually strength graded timber to the NGRDL grading rules

Certification body	UK representative	Certification body	UK representative
PLIB (USA)	WWPA	WCLIB	WWPA
SPIB	WWPA/SFPA	WWPA	WWPA
TPI	WWPA/SFPA		

⁴⁾ Addresses of the UK representative referred to in Table A.1 to Table A.5 are as follows:

Bureau de Promotion des Industries du Bois (BPIB), Unit 3, Blenheim Court, 7 Beaufort Park, Woodlands, Almondsbury, Bristol BS12 4NE Tel. 0454 616000 Fax 0454 616080

Council of Forest Industries of British Columbia (COFI), Tileman House, 131-133 Upper Richmond Road, London SW15 2TR Tel. 081 788 4446 Fax 081 789 01480

TRADA Certification Ltd. (TRADACERT), Stocking Lane, Hughenden Valley, High Wycombe, Bucks HP14 4NR Tel. 0494 565484 Fax 0494 565487

BSI Quality Assurance (BSIQA), PO Box 375, Milton Keynes, Bucks MK14 6LL Tel. 0908 220908 Fax 0908 220671

Nordic Timber Council UK (NTC), 17 Exchange Street, Retford, Notts DN22 6BL Tel. 0777 706616 Fax 0777 704695

Southern Pine Marketing Council (SFPA) and Western Wood Products Association (WWPA), 65 London Wall, London EC2M 5TU

Table A.4 — Certification bodies approved to oversee the supply of machine strength graded timber to BS EN 519^a (both machine control and output control systems)

Certification body	UK representative
BSIQA TRADACERT	BSIQA TRADACERT
^a In preparation.	

Table A.5 — Certification bodies approved to oversee the supply of machine strength graded timber to BS EN 519^a (output control system only)

Certification body	UK representative	Certification body	UK representative
AFPA CLMA COFI ILMA MI PLIB (Can)	COFI COFI COFI COFI COFI COFI	SPIB TPI WCLIB WWPA QLMA	WWPA/SFPA WWPA/SFPA WWPA WWPA BPIB
^a In preparation.			

List of references (see clause 2)

Normative references

BSI publications

BRITISH STANDARDS INSTITUTION, London

BS 648:1964, *Schedule of weights of building materials*.

BS 5268, *Structural use of timber*.

BS 5268-2:1991, *Code of practice for permissible stress design, materials and workmanship*.

BS 5268-3:1985, *Code of practice for trussed rafter roofs*.

BS 5268-6, *Code of practice for timber frame walls*.

BS 5268.6.1:1988, *Dwellings not exceeding three storeys*.

BS 6399, *Design loading for buildings*.

BS 6399-1:1984, *Code of practice for dead and imposed loads*.

BS 6399-3:1988, *Code of practice for imposed roof loads*.

CP 3, *Code of basic data for the design of buildings*.

CP 3:Chapter V, *Loading*.

CP 3:Chapter V-2:1972, *Wind loads*.

BS EN 301:1992, *Adhesives, phenolic and aminoplastic, for load-bearing timber structures: classification and performance requirements*.

BS EN 335, *Hazard classes of wood and wood-based products against biological attack*.

BS EN 335-1:1992, *Classification of hazard classes*.

BS EN 335-2:1992, *Guide to the application of hazard classes to solid wood*.

BS EN 380:1993, *Timber structures — Test methods — General principles for static load testing*.

BS EN 460:1994, *Durability of wood and wood-based products — Natural durability of wood — Guide to the durability requirements to be used in hazard classes*.

BS EN 10147:1992, *Specification for continuously hot-dip zinc coated structural steel sheet and strip — Technical delivery conditions*.

BS EN 26891:1991, *Timber structures — Joints made with mechanical fasteners — General principles for the determination of strength and deformation characteristics*.

BS EN 28970:1991, *Timber structures — Testing of joints made with mechanical fasteners — Requirements for wood density*.

ISO publications

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION (ISO), GENEVA. (All publications are available from BSI Sales.)

ISO 2081:1986, *Metallic coatings — Electroplated coatings of zinc on iron or steel*.

ISO 2631, *Evaluation of human exposure to whole-body vibration*.

ISO 2631-2:1989, *Part 2: Continuous and shock-induced vibrations in buildings (1 to 80 Hz)*.

Other references

[2] TIMBER RESEARCH AND DEVELOPMENT ASSOCIATION. *Plywood properties for use with Eurocode 5: Interim technical data sheet ITD/1*. London: TRADA, 1993⁵⁾.

[3] TIMBER RESEARCH AND DEVELOPMENT ASSOCIATION. *Structural chipboard and tempered hardboard properties for use with Eurocode 5: Interim technical data sheet ITD/2*. London: TRADA, 1993⁵⁾.

⁵⁾ Available from TRADA, Stocking Lane, Hughenden Valley, High Wycombe, Bucks HP14 4NR.

Informative references

BSI publications

BRITISH STANDARDS INSTITUTION, London

BS 4978:1988, *Specification for softwood grades for structural use*.

BS 5291:1984, *Specification for manufacture of finger joints of structural softwood*.

BS 5756:1980, *Specification for tropical hardwoods graded for structural use*.

Other references

[1] GREAT BRITAIN. The Building Regulations 1991, Approved Document A 1992 *Requirements on accidental damage and structural integrity*. London: HMSO.

[4] NATIONAL LUMBER GRADES AUTHORITY. *Standard grading rules for Canadian lumber*. Vancouver: NLGA, 1993⁶⁾⁷⁾.

[5] NATIONAL GRADING RULES FOR DIMENSIONED LUMBER. *The national grading rules for dimensioned lumber*. NGRDL, 1993⁸⁾.

[6] ECONOMIC COMMITTEE FOR EUROPE (ECE). *Sawn timber: Recommended standard for stress grading of coniferous sawn timber*. Geneva: ECE, 1982.

⁶⁾ The relevant section of the rules is Section 4 which is technically equivalent to the *National Grading Rules for Dimensioned Lumber* [5].

⁷⁾ Available from: Council of The Forest Industries of British Columbia, Tileman House, 131-133 Upper Richmond Road, London SW15 2TR.

⁸⁾ Available from: Southern Pine Marketing Council and Western Wood Products Association, 65 London Wall, London EC2M 5TU.

Eurocode 5 — Design of timber structures — Part 1.1: General rules and rules for buildings

Eurocode 5 — Calcul des structures en bois —
Partie 1.1: Règles générales et règles pour les
bâtiments

Eurocode 5 — Entwurf, Berechnung und
Bemessung von Holzbauwerken —
Teil 1.1: Allgemeine
Bemessungsregeln, Bemessungsregeln
für den Hochbau

This European Prestandard (ENV) was approved by CEN on 1992-11-20 as a prospective standard for provisional application. The period of validity of this ENV is limited initially to three years. After two years the members of CEN will be requested to submit their comments, particularly on the question whether the ENV can be converted into a European Standard (EN).

CEN members are required to announce the existence of this ENV in the same way as for an EN and to make the ENV available promptly at national level in an appropriate form. It is permissible to keep conflicting national standards in force (in parallel to the ENV) until the final decision about the possible conversion of the ENV into an EN is reached.

CEN members are the national standards bodies of Austria, Belgium, Denmark, Finland, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, Netherlands, Norway, Portugal, Spain, Sweden, Switzerland and United Kingdom.

CEN

European Committee for Standardization
Comité Européen de Normalisation
Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

Foreword

01 Objectives of the Eurocodes

The Eurocodes constitute a group of standards for the structural and geotechnical design of building and civil engineering works. They will cover execution and control to the extent that it is necessary to indicate the quality of the construction products and the standard of workmanship needed on and off-site to comply with the assumptions of the design rules. While the necessary set of harmonised technical specifications for products and methods for testing their performance is not available, the Eurocodes may cover some of these aspects.

The Eurocodes are intended to serve as reference documents for the following purposes:

- a) as a means to prove compliance of building and civil engineering works with the essential requirements of the Construction Products Directive;
- b) as a framework for drawing up harmonized technical specifications for construction products.

02 Background to the Eurocode programme

The Commission of the European Communities (CEC) initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which would initially serve as an alternative to the differing rules in force in the various Member States and, ultimately would replace them. These technical rules became known as the “Structural Eurocodes”.

In 1990, after consulting their respective Member States, CEC and EFTA Secretariat transferred the work on further development, issue and updating of Eurocodes to CEN.

In CEN, Technical Committee CEN/TC 250 has overall responsibility for the Structural Eurocodes.

03 Eurocode programme

Work is in hand on the following Eurocodes each consisting of a number of parts:

EC 1: Basis of design and actions on structures

EC 2: Design of concrete structures

EC 3: Design of steel structures

EC 4: Design of composite steel and concrete structures

EC 5: Design of timber structures

EC 7: Geotechnics

EC 8: Design of structures in seismic regions

EC 9: Design of aluminium structures (subject to Mandate)

For each Eurocode listed above, CEN/TC 250 has formed a Sub-committee.

This part of Eurocode EC5 which had been finalised and approved for publication under the direction of CEC, is being issued by CEN as European Prestandard (ENV). It is intended for experimental practical application in the design of building and civil engineering works covered by the scope of the Prestandard as given in Clause 1.1.2.

Feedback and comments on this prestandard should be sent to the Secretariat of Sub-Committee SC5 at the following address:

SIS

BST

Drottning Kristinas väg 73

S-11428 STOCKHOLM

04 National application documents

In view of the responsibilities of Members of states for the safety health and other matters covered by the essential requirements, certain safety elements in this ENV have been assigned indicative values. The authorities in each Member state are expected to assign definitive values to these safety elements.

Many of the supporting standards, including those giving values for actions to be taken into account and measures required for fire protection, will not be available by the time this Prestandard is issued. It is therefore anticipated that a National Application Document giving definitive values for safety elements, referencing compatible supporting standards and giving national guidance on the application of this Prestandard will be issued by each Member State or its Standard Organisation. This Prestandard should be used in conjunction with the National Application Document valid in the country where the building and civil engineering work is to be constructed.

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1 Introduction

1.1 Scope

1.1.1 Scope of Eurocode 5

P(1) Eurocode 5 applies to the design of timber structures — i.e., structures made of timber (solid timber, sawn, planed or in pole form, and glued laminated timber) or wood-based panels jointed together with adhesives or mechanical fasteners. It is subdivided into various separate parts, see 1.1.2 and 1.1.3.

P(2) Eurocode 5 is only concerned with the requirements for mechanical resistance, serviceability and durability of structures. Other requirements, e.g. concerning thermal or sound insulation, are not considered.

P(3) Execution⁹⁾ is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Execution and workmanship are covered in Chapter 7, and are to be considered as minimum requirements which may have to be further developed for particular types of buildings and methods of construction⁹⁾.

P(4) Eurocode 5 does not cover the special requirements of seismic design. Provisions related to such requirements are given in Eurocode 8 “*Design of Structures in Seismic Regions*”¹⁰⁾ which complements Eurocode 5.

P(5) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in Eurocode 5. They are provided in Eurocode 1 “*Basis of design and actions on structures*”¹⁰⁾.

1.1.2 Scope of part 1-1 of Eurocode 5

P(1) Part 1-1 of Eurocode 5 gives a general basis for the design of buildings and civil engineering works.

P(2) In addition, Part 1-1 gives detailed rules which are mainly applicable to ordinary structures. The applicability of these rules may be limited for practical reasons or due to simplifications; their use and any limits of applicability are explained in the text where necessary.

P(3) Chapters 1 and 2 are common to all Eurocodes, with the exception of some additional clauses which are required for timber structures.

P(4) This Part 1-1 does not cover:

- the design of bridges,
- resistance to fire,
- the design of structures subject to prolonged exposure to temperatures over 60 °C.
- particular aspects of special structures

1.1.3 Further parts of Eurocode 5

P(1) Further Parts of Eurocode 5 which, at present, are being prepared or are planned, include the following

Part 1-2 — Supplementary rules for structural fire design

Part 2 — Bridges (in preparation)

1.2 Distinction between principles and application rules

P(1) Depending on the character of the individual clauses, distinction is made in this Eurocode between Principles and Application Rules.

P(2) The Principles comprise:

- general statements and definitions for which there is no alternative, as well as
- requirements and analytical models for which no alternative is permitted unless specifically stated.

P(3) The Principles are preceded by the letter P.

⁹⁾ For the meaning of this term, see 1.4.1(2)

¹⁰⁾ At present at the draft stage

P(4) The Application Rules are generally recognised rules which follow the Principles and satisfy their requirements.

P(5) It is permissible to use alternative design rules which differ from the Application Rules given in this Eurocode, provided that it is shown that the alternative rules accord with the relevant Principles and are at least equivalent with regard to the mechanical resistance, serviceability and durability achieved for the structure with the present Eurocode.

1.3 Assumptions

P(1) The following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control are provided in factories, in plants, and on site.
- Construction is carried out by personnel having the appropriate skill and experience.
- The construction materials and products are used as specified in this Eurocode or in the relevant material or product specifications.
- The structure will be adequately maintained.
- The structure will be used in accordance with the design brief.

P(2) The design procedures are valid only when the requirements for execution and workmanship given in Chapter 7 are also complied with.

P(3) Numerical values identified by are given as indications. Other values may be specified by Member States.

1.4 Definitions

1.4.1 Terms common to all Eurocodes

P(1) Unless otherwise stated in the following, the terminology used in International Standard ISO 8930 applies.

P(2) The following terms are used in common for all Eurocodes with the following meanings:

— **Construction works:** Everything that is constructed or results from construction operations.¹¹⁾ This term covers both building and civil engineering works. It refers to the complete construction comprising both structural and non-structural elements.

— **Execution:** The activity of creating a building or civil engineering works. The term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

NOTE In English “construction” may be used in certain combinations of words, when there is no ambiguity (e.g. “during construction”).

— **Structure:** Organised combination of connected parts designed to provide some measure of rigidity.¹²⁾ This term refers to load carrying parts.

— **Type of building or civil engineering works:** Type of “construction works” designating its intended purpose, e.g. dwelling house, industrial building, road bridge.

NOTE “Type of construction works” is not used in English.

— **Form of structure:** Structural type designating the arrangement of structural elements, e.g. beam, triangulated structure, arch suspension bridge.

— **Construction material:** A material used in construction work, e.g. concrete, steel, timber, masonry.

— **Type of construction:** Indication of principal structural material, e.g. reinforced concrete construction, steel construction, timber construction, masonry construction.

— **Method of construction:** Manner in which the construction will be carried out, e.g. cast in place, prefabricated, cantilevered.

— **Structural system:** The load bearing elements of a building or civil engineering works and the way in which these elements are assumed to function, for the purpose of modelling.

¹¹⁾ This definition accords to the International Standard ISO 6707-1.

¹²⁾ The International Standard ISO 6707-1 gives the same definition, but, adds “or construction works having such an arrangement”. For Eurocodes this addition is not used, in order to avoid ambiguous translations.

1.4.2 Special terms used in part 1.1 of Eurocode 5

P(1) The following terms are used in this Part with the following meanings:

- **Balanced plywood:** a plywood in which the outer and inner plies are symmetrical about the centre plane with respect to thickness and species.
- **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.
- **Dowel:** circular cylindrical rod usually of steel (but may also be of other metal, plastics or wood) fitting tightly in prebored holes and used for transmitting loads perpendicular to the dowel axis.
- **Equilibrium moisture content:** the moisture content at which wood neither gains nor loses moisture to the surrounding air.
- **Moisture content:** the mass of water in wood expressed as a proportion of its oven-dry mass.
- **Target size:** size used to indicate the size desired (at a specified moisture content) and to which the deviations, which would ideally be zero, are related.

1.5 S.I. Units

P(1) S.I Units shall be used in accordance with ISO 1000.

(2) For calculations, the following units are recommended:

- loads : kN, kN/m, kN/m²
- unit mass : kg/m³
- unit weight : kN/m³
- stresses and strengths : N/mm² (– MN/m² or MPa)
- moments (bending ...) : kNm

1.6 Symbols used in part 1-1 of Eurocode 5

1.6.1 General

In general, the symbols used in Part 1 of Eurocode 5 are based on the schedule below and on derivatives of these as, for example,

$G_{d,sup}$ Upper design value of a permanent action

V_d Design shear force

$\sigma_{f,c}$ Flange compression stress

Such derivations together with any special symbols are defined in the text where they occur.

1.6.2 Symbols used in Chapter 2

MAIN SYMBOLS:

A	Accidental action
C	Fixed value in serviceability limit states
E	Effect of action
F	Action
G	Permanent action
Q	Variable action
R	Resistance
S	Force or moment
X	Material property
a	Geometrical data
	Δ_a deviations
γ	Partial coefficients

	γ_G for permanent actions
	γ_{GA} as γ_G for accidental situations
	γ_M for material properties
	γ_Q for variable actions
ψ	Coefficients defining representative values of variable actions
	ψ_0 for combination values
	ψ_1 for frequent values
	ψ_2 for quasi-permanent values

SUBSCRIPTS:

Subscripts are omitted when this will not cause confusion.

d	Design value
dst	Destabilizing
inf	Lower
k	Characteristic
mod	Modification
nom	Nominal
stb	Stabilizing
sup	Upper

1.6.3 Symbols used in Chapters 3–7 and Annexes

MAIN SYMBOLS:

A	Area
E	Modulus of elasticity
F	Action
G	Permanent action
I	Second moment of area
K	Slip modulus
L	Length
M	Bending moment
N	Axial force
Q	Variable action
R	Resistance
S	Internal forces and moments
V	Shear force
V	Volume
W	Section modulus
X	Value of a property of a material
a	Distance
b	Width
d	Diameter
e	Eccentricity
f	Strength (of a material)
h	Height (or depth of beam)
i	Radius of gyration
k	Coefficient; Factor (always with a subscript)
l or ℓ	Length; Span

m	Mass
r	Radius
s	Spacing
t	Thickness
u,v,w	Components of the displacement of a point
x,y,z	Coordinates
α	Angle; Ratio
β	Angle; Ratio
γ	Partial factor
λ	Slenderness ratio (l_{ef}/i)
ϕ	Rotational displacement
ρ	Mass density
σ	Normal stress
τ	Shear stress

SUBSCRIPTS

ap	apex
c	Compression
cr (or crit)	Critical
d	Design
def	Deformation
dis	Distribution
ef	Effective
ext	External
f	Flange
fin	Final
h	Embedding
ind	Indirect
inf	Inferior; Lower
inst	Instantaneous
in	Internal
k	Characteristic
l	Low; Lower
ls	Load sharing
m	Material; Bending
max	Maximum
min	Minimum
mod	Modification
nom	Nominal
q (or Q)	Variable action
ser	Serviceability
stb	stabilising
sup	Superior; Upper
t (or ten)	Tension
tor	Torsion
u	Ultimate

v	Shear
vol	Volume
w	Web
x,y,z	Coordinates
y	Yield
α	Angle between force (or stress) and grain direction
0,90	Relevant directions in relation to grain direction
05	Relevant percentage for a characteristic value

1.7 References

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

ISO-Standards

ISO 1000, *SI-units and recommendations for the use of their multiples and of certain other units.*

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).*

ISO 8930, *General principles on reliability for structures — list of equivalent terms.*

European Standards

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

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.*

EN 350-2, *Durability of wood and wood-based products — natural durability of wood — Part 2: Guide to natural durability and treatability of selected wood species of importance in Europe.*

EN 383, *Timber structures — Test methods. Determination of embedding strength and foundation values for dowel type fasteners.*

EN 409, *Timber structures — Test methods. Determination of the yield moment for dowel type fasteners — nails.*

EN 10147, *Continuously hot-dip zinc coated structural steel sheet and strip. Technical delivery conditions.*

EN 26891, *Timber structures. Joints made with mechanical fasteners. General principles for the determination of strength and deformation characteristics.*

EN 28970, *Timber structures. Testing of joints made with mechanical fasteners; requirements for wood density.*

Drafts of European Standards

prEN 300 *Particleboards. Oriented Strand boards (OSB).*

prEN 312-4, *Particleboards — Specifications — Part 4: Requirements for load-bearing boards for use in dry conditions.*

prEN 312-5, *Particleboards — Specifications — Part 5: Requirements for load-bearing boards for use in humid conditions.*

prEN 312-6, *Particleboards — Specifications — Part 6: Requirements for heavy duty load-bearing boards for use in dry conditions.*

prEN 312-7, *Particleboards — Specifications — Part 7: Requirements for heavy duty load-bearing boards for use in humid conditions.*

- prEN 335-3, *Durability of wood and wood-based products — definition of hazard classes of biological attack — Part 3: Application to wood based panels.*
- prEN 336, *Structural timber. Coniferous and poplar — timber sizes — permissible deviations.*
- prEN 338, *Structural timber. Strength classes.*
- prEN 351-1, *Durability of wood and wood-based products — preservative treated solid wood — Part 1: Classification of preservative penetration and retention.*
- prEN 384, *Structural timber. Determination of characteristic values of mechanical properties and density.*
- prEN 385, *Finger jointed structural timber. Performance requirements and minimum production requirements.*
- prEN 386, *Glued laminated timber. Performance requirements and minimum production requirements.*
- prEN 387, *Glued laminated timber — Production requirements for large finger joints. Performance requirements and minimum production requirements.*
- prEN 390, *Glued laminated timber. Sizes. Permissible deviations.*
- prEN 408, *Timber structures. Test methods. Solid timber and glued laminated timber. Determinations of some physical and mechanical properties.*
- prEN 460, *Durability of wood and wood-based products — natural durability of wood. Guide to the durability requirements for wood to be used in hazard classes.*
- prEN 518, *Structural timber — Grading. Requirements for visual strength grading standards.*
- prEN 519, *Structural timber — Grading. Requirements for machine strength graded timber and grading machines.*
- prEN 594, *Timber structures — Test methods. Racking strength and stiffness of timber framed wall panels.*
- prEN 622-3, *Fibreboards — Specifications — Part 3: Load bearing boards for use in dry conditions.*
- prEN 622-5, *Fibreboards — Specifications — Part 5: Load bearing boards for use in humid conditions.*
- prEN 636-1, *Plywood — Specifications — Part 1: Requirements for plywood for dry interior use.*
- prEN 636-2, *Plywood — Specifications — Part 2: Requirements for plywood for covered exterior use.*
- prEN 636-3, *Plywood — Specifications — Part 3: Requirements for plywood for non-covered exterior use.*
- prEN 912, *Timber fasteners. Specifications for connectors for timber.*
- prEN 1058, *Wood based panels. Determination of characteristic values of mechanical properties and densities.*
- prEN 1059, *Timber structures. Production requirements for fabricated trusses using punched metal plate fasteners.*
- prEN 1075, *Timber structures — Test methods. Joints made with punched metal plate fasteners.*
- prEN 1193, *Timber structures — Test methods. Structural and glued laminated timber. Determination of additional physical and mechanical properties.*
- prEN 1194, *Timber structures — Glued laminated timber. Strength classes and determination of characteristic values.*

2 Basis of design

2.1 Fundamental requirements

P(1) A structure shall be designed and constructed in such a way that:

- with acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life and costs, and
- with appropriate degrees of reliability, it will sustain all actions and influences likely to occur during execution and use and have adequate durability in relation to maintenance costs.

P(2) A structure shall also be designed in such way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(3) The potential damage should be limited or avoided by appropriate choice of one or more of the following:

- avoiding, eliminating or reducing the hazards which the structure may sustain
- selecting a structural form which has low sensitivity to the hazards considered
- selecting a structural form and design that can survive adequately the accidental removal of an individual element
- tying the structure together.

P(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by specifying control procedures for production, construction and use as relevant for the particular project.

2.2 Definitions and classifications

2.2.1 Limit states and design situations

2.2.1.1 *Limit states*

P(1) Limit states are states beyond which the structure no longer satisfies the design performance requirements.

Limit states are classified into:

- ultimate limit states
- serviceability limit states

P(2) Ultimate limit states are those associated with collapse, or with other forms of structural failure which may endanger the safety of people.

P(3) States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also classified and treated as ultimate limit states.

(4) Ultimate limit states which may require consideration include:

- loss of equilibrium of the structure or any part of it, considered as a rigid body.
- failure by excessive deformation, rupture, or loss of stability of the structure or any part of it, including supports and foundations.

P(5) Serviceability limit states correspond to states beyond which specified service criteria are no longer met.

(6) Serviceability limit states which may require consideration include:

- deformations or deflections which affect the appearance or effective use of structure (including the malfunction of machines or services) or cause damage to finishes or non-structural elements
- vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.

2.2.1.2 *Design situations*

P(1) Design situations are classified as:

- persistent situations corresponding to normal conditions of use of the structure
- transient situations, for example during construction or repair
- accidental situations.

2.2.2 Actions

2.2.2.1 *Definitions and principal classification*¹³⁾

P(1) An action (F) is:

- a force (load) applied to the structure (direct action), or
- an imposed deformation (indirect action); for example, temperature effects or settlement.

¹³⁾ Fuller definitions of the classification of actions will be found in Eurocode — 1.

P(2) Actions are classified:

i) by their variation in time

- permanent actions (G), e.g. self-weight of structures, fittings, ancillaries and fixed equipment
- variable actions (Q):
 - long-term actions, e.g. storage
 - medium-term actions, e.g. imposed loads
 - short-term actions, e.g. wind or snow
 - instantaneous actions
- accidental actions (A), e.g. explosions or impact from vehicles.

ii) by their spatial variation

- fixed actions, e.g. self-weight [but see 2.3.2.3(2) for structures very sensitive to variations in the self-weight]
- free actions, which result in different arrangements of actions, e.g. movable imposed loads, wind loads, snow loads.

2.2.2.2 Characteristic values of actions

P(1) Characteristic values F_k are specified:

- in ENV 1991 Eurocode 1 or other relevant loading codes, or
- by the client, or the designer in consultation with the client, provided that the minimum provisions specified in the relevant codes or by the relevant authority are observed.

P(2) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere a single characteristic value (G_k) is sufficient.

(3) The self-weight of the structure may, in most cases, be calculated on the basis of the target dimensions and mean unit masses.

P(4) For variable actions the characteristic value (Q_k) corresponds to either:

- the upper value with an intended probability of not being exceeded, or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
- the specified value.

P(5) For accidental actions the characteristic value A_k (when relevant) generally corresponds to a specified value.

2.2.2.3 Representative values of variable actions¹⁴⁾

P(1) The main representative value is the characteristic value Q_k .

P(2) Other representative values are expressed in terms of the characteristic value Q_k by means of a factor ψ_i . These values are defined as a:

- combination value : $\psi_0 Q_k$
- frequent value : $\psi_1 Q_k$
- quasi-permanent value : $\psi_2 Q_k$

P(3) The factors ψ_i are specified

- in ENV 1991 Eurocode 1 or other relevant loading codes, or
- by the client or the designer in conjunction with the client, provided minimum provisions specified in codes or by the authority are observed.

¹⁴⁾ Fuller definitions of the classifications of actions will be found in ENV 1991 Eurocode 1.

2.2.2.4 Design values of actions

P(1) The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F F_k$$

P(2) Specific examples are:

$$G_d = \gamma_G G_k \quad (2.2.2.4)$$

$$Q_d = \gamma_Q Q_k \text{ or } \gamma_Q \psi_i Q_k$$

$$A_d = \gamma_A A_k \text{ (if } A_d \text{ is not directly specified)}$$

where

γ_F , γ_G , γ_Q and γ_A are the partial safety factors for the action considered taking account of, for example, the possibility of unfavourable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effect of actions and in the assessment of the limit state considered.

P(3) With reference to 2.2.2.2(2) upper and lower design values of permanent actions are expressed as

$$G_{d,\text{sup}} = \gamma_{G,\text{sup}} G_{k,\text{sup}} \text{ or } \gamma_{G,\text{sup}} G_k$$

$$G_{d,\text{inf}} = \gamma_{G,\text{inf}} G_{k,\text{inf}} \text{ or } \gamma_{G,\text{inf}} G_k$$

2.2.2.5 Design values of the effects of actions

P(1) The effects of actions (E) are responses (for example, internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions (E_d) are determined from the design values of the actions, geometrical data and material properties when relevant:

$$E_d = E(F_d, a_d, \dots) \quad (2.2.2.5)$$

where a_d is defined in 2.2.4.

2.2.3 Material properties

2.2.3.1 Characteristic values

P(1) A material property is represented by a characteristic value X_k which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

P(2) In certain cases a nominal value is used as the characteristic value.

2.2.3.2 Design values

P(1) The design value X_d of a material property is defined as:

$$x_d = k_{\text{mod}} X_k / \gamma_M \quad (2.2.3.2a)$$

where symbols are defined as follows:

γ_M partial safety factor for the material property, given in 2.3.3.2.

k_{mod} modification factor taking into account the effect on the strength parameters of the duration of the load and the moisture content in the structure.

Values of k_{mod} are given in Chapter 3.

(2) Design values for the material properties, geometrical data and effects of actions, when relevant, should be used to determine the design resistance R_d from:

$$R_d = R(X_d, a_d, \dots) \quad (2.2.3.2b)$$

(3) The characteristic value R_k may be determined from tests.

2.2.4 Geometrical data

P(1) Geometrical data describing the structure are generally represented by their nominal values

$$a_d = a_{\text{nom}} \quad (2.2.4a)$$

P(2) In some cases the geometrical design values are defined by

$$a_d = a_{\text{nom}} + \Delta_a \quad (2.2.4b)$$

The values of Δ_a are given in the appropriate clauses.

2.2.5 Load arrangements and load cases¹⁵⁾

P(1) A load arrangement identifies the position, magnitude and direction of a free action.

P(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

2.3 Design requirements

2.3.1 General

P(1) It shall be verified that no relevant limit state is exceeded.

P(2) All relevant design situations and load cases shall be considered.

P(3) Possible deviations from the assumed directions or positions of actions shall be considered.

P(4) 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.

2.3.2 Ultimate limit states

2.3.2.1 Verification conditions

P(1) When considering a limit state of static equilibrium or of gross displacements or deformations of the structure, it shall be verified that

$$E_{d,dst} \leq E_{d,stab} \quad (2.3.2.1a)$$

where $E_{d,dst}$ and $E_{d,stab}$ are the design effects of destabilizing and stabilizing actions respectively.

P(2) When considering a limit state of rupture or excessive deformation of a section, member or connection it shall be verified that:

$$S_d \leq R_d \quad (2.3.2.1b)$$

where S_d is the design value of an internal force or moment (or of a respective vector of several internal forces or moments) and R_d is the corresponding design resistance.

P(3) When considering a limit state of transformation of the structure into a mechanism, it shall be verified that a mechanism does not occur unless actions exceed their design values — associating all structural properties with the respective design values.

P(4) When considering a limit state of stability induced by second-order effects it shall be verified that instability does not occur unless actions exceed their design values — associating all structural properties with the respective design values. In addition, sections shall be verified according to 2.3.2.1 P(2).

2.3.2.2 Combinations of actions

P(1) For each load case, design values E_d for the effects of actions shall be determined from combination rules involving design values of actions as identified by Table 2.3.2.2.

Table 2.3.2.2 — Design values of actions for use in the combination of actions

Design Situation	Permanent actions G_d	Variable actions		Accidental actions A_d
		one Q_d	all others	
Persistent and Transient	$\gamma_G G_k$	$\gamma_Q Q_k$	$\psi_0 \gamma_Q Q_k$	—
Accidental	$\gamma_{GA} G_k$	$\psi_1 Q_k$	$\psi_2 Q_k$	$\gamma_A A_k$ (if A_d not specified directly)

¹⁵⁾ Detailed rules on load arrangements and load cases are given in ENV 1991 Eurocode 1.

P(2) The design values of Table 2.3.2.2 shall be combined using the following rules (given in symbolic form¹⁶⁾

— Persistent and transient design situations (fundamental combinations):

$$\Sigma \gamma_{G,j} G_{k,j} + \gamma_{Q1} Q_{k,1} + \Sigma_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \quad (2.3.2.2a)$$

— Accidental design situations (if not specified differently elsewhere)

$$\Sigma \gamma_{GA,j} G_{k,j} + A_d + \psi_{1,1} Q_{k,1} + \Sigma_{i>1} \psi_{2,i} Q_{k,i} \quad (2.3.2.2b)$$

where symbols are defined as follows:

$G_{k,j}$	characteristic values of permanent actions
$Q_{k,1}$	characteristic value of one of the variable actions
$Q_{k,i}$	characteristic value of the other variable actions
A_d	design value (specified value) of the accidental action
$\gamma_{G,j}$	partial safety factors for permanent actions
$\gamma_{GA,j}$	as $\gamma_{G,j}$ but for accidental design situations
$\gamma_{Q,i}$	partial safety factors for variable actions
ψ_0, ψ_2, ψ_2	factors defined in 2.2.2.3

P(3) Combinations for accidental design situations either involve an explicit accidental action A or refer to a situation after an accidental event ($A = 0$). Unless specified otherwise, $\gamma_{GA} = 1$ should be used.

P(4) Simplified combinations for building structures are given in 2.3.3.1.

2.3.2.3 Design values of permanent actions

P(1) In the various combinations defined above, those permanent actions that increase the effect of the variable actions (i.e. produce unfavourable effects) shall be represented by their upper design values, those that decrease the effect of the variable actions (i.e. produce favourable effects) by their lower design values [see 2.2.2.4(3)].

P(2) Where the results of a verification will be very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and favourable parts of this action shall be considered as individual actions. This applies in particular to the verification of static equilibrium. In the aforementioned cases specific γ_G values need to be considered [see 2.3.3.1(3) for building structures].

P(3) In other cases, either the lower or upper design value (whichever gives the more unfavourable effect) shall be applied throughout the structure.

P(4) For continuous beams the same design value of the self-weight may be applied to all spans.

2.3.3 Partial safety factors for ultimate limit states

2.3.3.1 Partial safety factors for actions on building structures

P(1) Partial safety factors for the persistent and transient design situations are given in Table 2.3.3.1.

P(2) For accidental design situation to which expression (2.3.2.2b) applies, the partial safety factors for variable action are equal to unity.

¹⁶⁾ Detailed rules on combinations of actions are given in ENV 1991 Eurocode 1.

Table 2.3.3.1 — Partial safety factors for actions in building structures for persistent and transient design situations

	Permanent actions (γ_G)	Variable actions one with its characteristic value	(γ_Q) others with their combination value
<u>Normal partial coefficients</u>			
favourable effect ($\gamma_{F,inf}$)	1,0*	—**	—**
unfavourable effect	1,35 *	1,5	1,5
<u>Reduced partial coefficients</u>			
favourable effect	1,0	—**	—**
unfavourable effect	1,2	1,35	1,35
* cf. 2.3.3.1(3) below ** see ENV 1991 Eurocode 1; in normal cases on building structures $\gamma_{Q,inf} = 0$.			

(3) Where according to 2.3.2.3(2), favourable and unfavourable parts of a permanent action need to be considered as individual actions, the favourable part should be associated with $\gamma_{G,inf} = 0,9$ and the unfavourable part with $\gamma_{G,sup} = 1,1$.

(4) Reduced partial coefficients may be applied for one-storey buildings with moderate spans that are only occasionally occupied (storage buildings, sheds, green houses, and buildings and small silos for agricultural purposes), ordinary lighting masts, light partition walls, and sheeting.

For other structures normal coefficients should be applied.

(5) Adopting the γ values given in Table 2.3.3.1, the expression (2.3.2.2 a) may be replaced by:

— considering only the most unfavourable variable action

$$\Sigma \gamma_{G,j} G_{k,j} + 1,5 Q_{k,1} \quad (2.3.3.1a)$$

— considering all unfavourable variable actions

$$\Sigma \gamma_{G,j} G_{k,j} + 1,35 \sum_{i \geq 1} Q_{k,i} \quad (2.3.3.1b)$$

whichever gives the larger value.

2.3.3.2 Partial safety factors for materials

P(1) Partial safety factors for material properties (γ_m) are given in Table 2.3.3.2

Table 2.3.3.2 — Partial coefficients for material properties (γ_M)

<u>Ultimate limit states</u>	
— fundamental combinations:	
timber and wood-based materials	1,3
steel used in joints	1,1
— accidental combinations:	
	1,0
<u>Serviceability limit states</u>	
	1,0

2.3.4 Serviceability limit states

P(1) It shall be verified that

$E_d \leq C_d$ or $E_d \leq R_d$ (2.3.4)

where symbols are defined as follows:

- C_d nominal value or a function of certain design properties of materials related to the design effects of actions considered.
- E_d design effect of actions determined on the basis of the combination rules given in Chapter 4.

P(2) The partial safety factor for material properties (γ_M) is given in Table 2.3.3.2.

2.4 Durability

2.4.1 General

P(1) To ensure an adequately durable structure, the following interrelated factors shall be considered:

- the use of the structure
- the required performance criteria
- the expected environmental conditions
- the composition, properties and performance of the materials
- the shape of members and the structural detailing
- the quality of workmanship and level of control
- the particular protective measures
- the likely maintenance during the intended life.

P(2) The environmental conditions shall be estimated at the design stage to assess their significance in relation to durability and to enable adequate provisions to be made for protection of the materials.

2.4.2 Resistance to biological organisms

P(1) 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 and EN 335-2, and prEN 335-3), or be given a preservative treatment selected in accordance with prEN 351-1 and prEN 460.

2.4.3 Resistance to corrosion

P(1) 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 3.1.5) are given in Table 2.4.3.

Table 2.4.3 — Examples of minimum material or corrosion protection specifications for fasteners (related to ISO 2081)^a

Fastener	Service Class		
	1	2	3
Nails, dowels screws	None	None	Fe/Zn 25c ^b
Bolts	None	Fe/Zn 12c	Fe/Zn 25c ^b
Staples	Fe/Zn 12c	Fe/Zn 12c	Stainless steel
Punched metal plate fasteners and steel plates up to 3 mm thick	Fe/Zn 12c	Fe/Zn 12c	Stainless steel
Steel plates over 3 mm up to 5 mm in thickness	None	Fe/Zn 12c	Fe/Zn 25c ^b
Steel plates over 5 mm	None	None	Fe/Zn 25c ^b
^a If hot dip zinc coatings are used, then Fe/Zn 12c should be replaced by Z275, and Fe/Zn 25c should be replaced by Z350, both in accordance with EN 10147.			
^b For especially corrosive conditions consideration should be given to Fe/Zn 40, heavier hot dip coatings or stainless steel.			

3 Material properties

3.1 General

3.1.1 Strength and stiffness parameters

P(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 or wood-based materials or on well-established relations between the different properties.

P(2) It shall be shown that the dimensional stability and environmental behaviour are satisfactory for the intended purposes.

3.1.2 Characteristic values

P(1) Characteristic strength values are defined as the population 5-percentile values obtained from the results of tests with a duration of 300s using test pieces at an equilibrium moisture content resulting from a temperature of 20 °C and a relative humidity of 65 %.

P(2) Characteristic stiffness values are defined as either the population 5-percentile or the mean values obtained under the same test conditions as defined in P(1).

P(3) The characteristic density is defined as the population 5-percentile value with mass and volume corresponding to equilibrium moisture content at a temperature of 20 °C and a relative humidity of 65 %.

3.1.3 Stress-strain relations

P(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. For members subjected to combined bending and compression, however, a non linear relationship (elastic-plastic) may be used.

3.1.4 Calculation models

P(1) The structural behaviour shall generally be assessed by calculating the action effects with a linear material model (elastic behaviour). For lattice structures and other structures able to redistribute the loads, elastic-plastic methods may be used for calculating the resulting stresses in the members.

3.1.5 Service classes

P(1) Structures shall be assigned to one of the service classes given below¹⁷⁾:

P(2) Service class 1: is characterized 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 year¹⁸⁾.

P(3) Service class 2: is characterized 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 year¹⁹⁾.

P(4) Service class 3: climatic conditions leading to higher moisture contents than in service class 2²⁰⁾.

3.1.6 Load-duration classes

P(1) For strength and stiffness calculations actions shall be assigned to one of the load-duration classes given in Table 3.1.6.

P(2) The load-duration classes are characterized 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 and the rheological properties of the materials.

¹⁷⁾ The service class system is mainly aimed at assigning strength values and calculating deformations under defined environmental conditions.

¹⁸⁾ In service class 1 the average moisture content in most softwoods will not exceed 12 %.

¹⁹⁾ In service class 2 the average moisture content in most softwoods will not exceed 20 %.

²⁰⁾ Only in exceptional cases would covered structures be considered to belong to service class 3.

Table 3.1.6 — Load-duration classes

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	more than 10 years	self weight
Long-term	6 months — 10 years	storage
Medium-term	1 week — 6 months	imposed load
Short-term	less than one week	snow ^a and wind
Instantaneous		accidental load
^a In areas which have a heavy snow load for a prolonged period of time, part of the load should be regarded as medium-term		

3.1.7 Modification factors for service class and duration of load

- 1) The values of the modification factor k_{mod} given in Table 3.1.7 should be used.
- 2) If a load combination consists of actions belonging to different load-duration classes a value of k_{mod} should be chosen which corresponds to the action with the shortest duration, e.g. for a dead load and a short-term combination, a value of k_{mod} corresponding to the short-term load should be used.

Table 3.1.7 — Values of k_{mod}

Material/load-duration class	Service class		
	1	2	3
Solid and glued laminated timber			
Plywood.			
Permanent	0,60	0,60	0,50
Long-term	0,70	0,70	0,55
Medium-term	0,80	0,80	0,65
Short-term	0,90	0,90	0,70
Instantaneous	1,10	1,10	0,90
Particleboards to prEN 312-6 ^a and prEN 312-7			
OSB to prEN 300, Grades 3 and 4			
Permanent	0,40	0,30	—
Long-term	0,50	0,40	—
Medium-term	0,70	0,55	—
Short-term	0,90	0,70	—
Instantaneous	1,10	0,90	—
Particleboards to prEN 312-4 ^a and prEN 312-5			
OSB to prEN 300, Grade 2 ^a			
Fibreboards to prEN 622-5 (hardboard)			
Permanent	0,30	0,20	—
Long-term	0,45	0,30	—
Medium-term	0,65	0,45	—
Short-term	0,85	0,60	—
Instantaneous	1,10	0,80	—
Fibreboards to prEN 622-3 (medium boards and hardboards)			
Permanent	0,20	—	—
Long-term	0,40	—	—
Medium-term	0,60	—	—
Short-term	0,80	—	—
Instantaneous	1,10	—	—
^a Not to be used in service class 2			

3.2 Solid timber

3.2.1 Grading

P(1) Timber shall be strength graded in accordance with rules ensuring that the properties of the timber are satisfactory for use and especially that the strength and stiffness properties are reliable.

P(2) The grading rules shall be based on a visual assessment of the timber, on the non-destructive measurement of one or more properties, or on a combination of the two methods.

P(3) Visual grading standards shall fulfill the minimum requirements given in prEN 518.

(4) Requirements for machine graded timber and for grading machines are given in prEN 519.

3.2.2 Characteristic strength and stiffness values and densities

P(1) Characteristic strength and stiffness values and densities shall be derived according to the method given in prEN 384.

(2) Tests should be carried out in accordance with prEN 408 and prEN 1193.

P(3) The characteristic strength values shall be related to a depth in bending and width in tension of 150 mm, to a specimen size of 45 mm × 180 mm × 70 mm for the tensile strength perpendicular to the grain and to a uniformly stressed volume of 0,0005 m³ for shear strength.

(4) A strength class system is given in prEN 384.

(5) For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for $f_{m,k}$ and $f_{t,0,k}$ according to prEN 338 and prEN 384 may be increased by the factor k_h where:

$$k_h = \min. \begin{cases} (150/h)^{0,2} \\ 1,3 \end{cases} \quad (3.2.2)$$

with h in mm for depth in bending or width in tension.

3.2.3 Timber sizes

P(1) The effective cross-section and geometrical properties of a structural member shall be calculated from the target size, provided that the deviation of the cross-section from the target size²¹⁾ is within the limits of tolerance class 1 given in prEN 336.

P(2) Reductions in the cross-sectional area shall be taken into account, except for reductions caused by

- nails with a diameter of 6 mm or less, driven without predrilling
- symmetrically placed holes for bolts, dowels, screws and nails in columns
- holes in the compression area of members, if the holes are filled with a material of higher stiffness than the wood.

(3) 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.

3.2.4 Modification factors for service class and duration of load

(1) The values of the modification factor k_{mod} given in Table 3.1.7 should be used.

3.2.5 Finger joints

P(1) Finger joints shall comply with prEN 385.

3.3 Glued laminated timber

3.3.1 Performance requirements

P(1) Glued laminated timber shall comply with prEN 386.

3.3.2 Characteristic strength and stiffness values

P(1) Characteristic strength and stiffness values shall either be determined on the basis of tests carried out in accordance with prEN 408 and prEN 1193 or calculated on the basis of the properties of the laminates and their joints.

(2) A method of calculating characteristic values and a strength class system are given in prEN 1194.

²¹⁾ The target size relates to a timber moisture content of 20 %.

P(3) The characteristic strength values shall be related to a depth in bending and width in tension of 600 mm, to a volume of 0,01 m³ for the tensile strength perpendicular to the grain and to a uniformly stressed volume of 0,0005 m³ for shear strength.

(4) For depths in bending or widths in tension of glued laminated timber less than 600 mm the characteristic values for $f_{m,k}$ and $f_{t,0,k}$ given in prEN 1194 may be increased by the factor k_h , where

$$k_h = \min. \begin{cases} (600/h)^{0,2} \\ 1,15 \end{cases} \quad (3.3.2)$$

with h in mm for depth in bending or width in tension.

3.3.3 Sizes of glued laminated timber

P(1) The effective cross-section and geometrical properties of a glued laminated member shall be calculated from the target size, provided that the deviation of the cross-section from the target size²²⁾ is within the limits given in prEN 390.

P(2) Reductions in the cross-sectional area shall be taken into account, except for reductions caused by

- nails with a diameter of 6 mm or less, driven without predrilling
- symmetrically placed holes for bolts, 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.

(3) 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.

3.3.4 Modification factors for service class and duration of load

(1) The values of the modification factor k_{mod} given in Table 3.1.7 should be used.

3.3.5 Large finger joints

P(1) Large finger joints shall comply with prEN 387.

P(2) Large finger joints shall not be used for products to be installed in service class 3, where the direction of grain changes at the joint.

3.4 Wood-based materials

3.4.1 Plywood

3.4.1.1 Requirements

P(1) Plywood shall be produced so that it maintains its integrity and strength in the assigned service class throughout the expected life of the structure.

(2) Plywood which complies with prEN 636-3 may be installed in service classes 1, 2 or 3.

(3) Plywood which complies with prEN 636-2 should only be installed in service classes 1 or 2.

(4) Plywood which complies with prEN 636-1 should only be installed in service class 1.

(5) Plywood for structural purposes should be balanced.

3.4.1.2 Characteristic strength and stiffness values

P(1) The characteristic values given in the relevant European Standards shall be used; when no values are given in European Standards, characteristic strength and stiffness values shall be calculated according to the method given in prEN 1058.

3.4.1.3 Modification factors for service class and duration of load

(1) The values of the modification factor k_{mod} given in Table 3.1.7 should be used.

3.4.2 Particleboard

3.4.2.1 Requirements

P(1) Particleboard shall be produced so that it maintains its integrity and strength in the assigned service class throughout the expected life of the structure.

²²⁾ The target size relates to a timber moisture content of 12 %.

- (2) Particleboard which complies with prEN 312-5 or prEN 312-7 should only be installed in service classes 1 or 2.
- (3) Particleboard which complies with prEN 312-4 or prEN 312-6 should only be installed in service class 1.
- (4) Oriented strand board which complies with prEN 300 grades OSB 3 or 4 should only be installed in service classes 1 or 2.
- (5) Oriented strand board which complies with prEN 300, grade OSB 2 should only be installed in service class 1.

3.4.2.2 Characteristic strength and stiffness values

P(1) The characteristic values given in the relevant European Standards shall be used; when no values are given in European Standards, characteristic strength and stiffness values shall be calculated according to the method given in prEN 1058.

3.4.2.3 Modification factors for service class and duration of load

- (1) Values of the modification factor k_{mod} are given in Table 3.1.7.

3.4.3 Fibreboard

3.4.3.1 Requirements

P(1) Fibreboard shall be produced so that it maintains its integrity and strength in the assigned service class throughout the expected life of the structure.

- (2) Fibreboards which comply with prEN 622-5 should only be installed in service classes 1 or 2.
- (3) Fibreboards which comply with prEN 622-3 should only be installed in service class 1.

3.4.3.2 Characteristic strength and stiffness values

P(1) The characteristic values given in the relevant European Standards shall be used; when no values are given in European Standards, characteristic strength and stiffness values shall be calculated according to the method given in prEN 1058.

3.4.3.3 Modification factors for service class and duration of load

- (1) The values of the modification factor k_{mod} given in Table 3.1.7 should be used.

3.5 Adhesives

P(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.

4 Serviceability limit states

4.1 General requirements

P(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, partitions and finishes, and to the functional needs as well as any requirements of appearance.

- (2) Combinations of actions for serviceability limit states should be calculated from the expression

$$\Sigma G_{k,j} + Q_{k,1} + \Sigma_{i>1} \psi_{1,i} Q_{k,i} \quad (4.1a)$$

- (3) The instantaneous deformation, U_{inst} , under an action should be calculated using the mean value of the appropriate stiffness moduli, and the instantaneous slip modulus for the serviceability limit state K_{ser} , determined by testing according to the method for determining k_s ($= K_{\text{ser}}$) given in EN 26891.

(4) The final deformation, u_{fin} , under an action should be calculated as

$$u_{fin} = u_{inst} (1 + k_{def}) \quad (4.1b)$$

where k_{def} is a factor which takes into account the increase in deformation with time due to the combined effect of creep and moisture. The values of k_{def} given in Table 4.1 should be used.

(5) The final deformation of an assembly fabricated from members which have different creep properties should be calculated using modified stiffness moduli, which are determined by dividing the instantaneous values of the modulus for each member by the appropriate value of $(1 + k_{def})$.

(6) If a load combination consists of actions belonging to different load duration classes, the contribution of each action to the total deflection should be calculated separately using the appropriate k_{def} values.

Table 4.1 — Values of k_{def} for timber, wood-based materials and joints

Material/load-duration class	Service class		
	1	2	3
Solid timber ^a glued laminated timber			
Permanent	0,60	0,80	2,00
Long-term	0,50	0,50	1,50
Medium-term	0,25	0,25	0,75
Short-term	0,00	0,00	0,30
Plywood			
Permanent	0,80	1,00	2,50
Long-term	0,50	0,60	1,80
Medium-term	0,25	0,30	0,90
Short-term	0,00	0,00	0,40
Particleboard to prEN 312-6 ^b and prEN 312-7			
OSB to prEN 300 Grades 3 and 4			
Permanent	1,50	2,25	—
Long-term	1,00	1,50	—
Medium-term	0,50	0,75	—
Short-term	0,00	0,30	—
Particleboard to prEN 312-4 ^b and prEN 312-5			
OSB to prEN 300, Grade 2 ^b			
Fibreboards to prEN 622-5			
Permanent	2,25	3,00	—
Long-term	1,50	2,00	—
Medium-term	0,75	1,00	—
Short-term	0,00	0,40	—
Fibreboards to prEN 622-3			
Permanent	3,00	—	—
Long-term	2,00	—	—
Medium-term	1,00	—	—
Short-term	0,35	—	—

^a For solid timber which is installed at or near fibre saturation points, and which is likely to dry out under load, the value of k_{def} should be increased by 1,0.

^b Not to be used in service class 2.

4.2 Joint slip

(1) For joints made with dowel-type fasteners the instantaneous slip modulus K_{ser} per shear plane per fastener under service load should be taken from Table 4.2 with ρ_k in kg/m^3 and d in mm.

Table 4.2 — Values of K_{ser} for dowel-type fasteners in N/mm

Fastener type	Timber-to-timber Panel-to-timber Steel-to-timber
Dowels Screws Nails (with predrilling)	$\rho_k^{1,5} d / 20$
Nails (without predrilling)	$\rho_k^{1,5} d^{0,8} / 25$
Staples	$\rho_k^{1,5} d^{0,8} / 60$

(2) If the characteristic densities of the two jointed members are different ($\rho_{k,1}$ and $\rho_{k,2}$) then ρ_k in above formulae ρ_k should be taken as

$$\rho_k = \sqrt{\rho_{k,1} \rho_{k,2}} \quad (4.2a)$$

(3) The final joint slip (u_{fin}) should be taken as

$$u_{fin} = u_{inst} (1 + k_{def}) \quad (4.2b)$$

(4) The final deformation of a joint made from members with different creep properties ($k_{def,1}$, $k_{def,2}$), should be calculated as

$$u_{fin} = u_{inst} \sqrt{(1 + k_{def,1})(1 + k_{def,2})} \quad (4.1c)$$

(5) For bolted joints the instantaneous slip u_{inst} under service load should be taken as

$$u_{inst} = 1 \text{ mm} + F/K_{ser} \quad (4.2b)$$

with K_{ser} for dowels (see Table 4.2).

(6) The final joint slip for bolts (u_{fin}) is given by

$$u_{fin} = 1 \text{ mm} + u_{inst} (1 + k_{def}) \quad (4.2c)$$

where u_{inst} is the instantaneous dowel slip.

4.3 Limiting values of deflection

4.3.1 Beams

(1) The components of deflection are shown in Figure 4.3.1, where the symbols are defined as follows:

- u_0 precamber (if applied)
- u_1 deflection due to permanent loads
- u_2 deflection due to variable loads

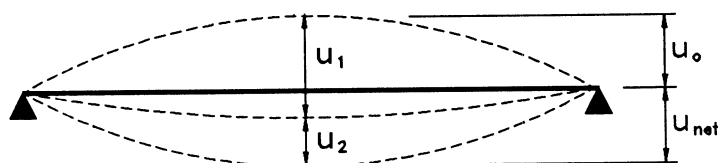


Figure 4.3.1 — Components of deflection

The net deflection below the straight line joining the supports, u_{net} , is given by

$$u_{\text{net}} = u_1 + u_2 - u_0 \quad (4.3.1)$$

(2) In cases where it is appropriate to limit the instantaneous deflections due to variable actions, the following values are recommended unless special conditions call for other requirements:

$$u_{2, \text{inst}} \leq \boxed{\ell/300} \quad (\text{cantilever } \boxed{\ell/150}) \quad (4.3.2)$$

where ℓ is the beam span or the length of a cantilever.

(3) In cases where it is appropriate to limit the final deflection, u_{fin} , the following values are recommended unless special conditions call for other requirements:

$$u_{2, \text{fin}} \leq \boxed{\ell/200} \quad (\text{cantilever } \boxed{\ell/100}) \quad (4.3.3)$$

$$u_{\text{net, fin}} \leq \boxed{\ell/200} \quad (\text{cantilever } \boxed{\ell/100}) \quad (4.3.4)$$

4.3.2 Trusses

(1) For trusses the limiting values of deflection for beams apply both to the complete span, and to the individual deflection of members between nodes.

4.4 Vibrations

4.4.1 General

P(1) It shall be ensured that the actions which are anticipated to occur often do not cause vibrations that can impair the function of the structure or cause unacceptable discomfort to the users.

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

(3) The mean values of the appropriate stiffness moduli should be used for the calculations.

(4) Unless other values are proved to be more appropriate, a modal damping ratio of $\zeta = 0,01$ (i.e. 1 %) should be assumed.

4.4.2 Vibrations from machinery

P(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) Acceptable levels for continuous floor vibration should be taken from Figure 5a in Appendix A of ISO 2631-2 (1989) with a multiplying factor of $\boxed{1}$.

4.4.3 Residential floors

(1) For residential floors with a fundamental frequency ≤ 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:

$$u/F \leq \boxed{1,5} \text{ mm/kN} \quad (4.4.3a)$$

and

$$v \leq \boxed{100}^{(f_1 \zeta^{-1})} \text{ m/(Ns}^2\text{)} \quad (4.4.3b)$$

where u is the maximum vertical deflection caused by a vertical concentrated static force F , and 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.

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

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

$$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{(EI)_\ell}{m}} \quad (4.4.3c)$$

where

m mass per unit area in kg/m^2

ℓ floor span in m

$(EI)_\ell$ equivalent plate bending stiffness of the floor about an axis perpendicular to beam direction in Nm^2/m

(5) The value of v may as an approximation be taken as

$$v = 4(0,4 + 0,6n_{40})/(mb l + 200) \text{ m/Ns}^2 \quad (4.4.3d)$$

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

$$n_{40} = \left\{ \left[\left(\frac{40}{f_1} \right)^2 - 1 \right] \left(\frac{b}{l} \right)^4 \frac{(EI)_\ell}{(EI)_b} \right\}^{0,25} \quad (4.4.3e)$$

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

5 Ultimate limit states

5.1 Basic rules

5.1.1 General

P(1) This section applies to members of solid timber or glued laminated timber.

5.1.2 Tension parallel to the grain

P(1) The following condition shall be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \quad (5.1.2)$$

5.1.3 Tension perpendicular to the grain

P(1) For a uniformly stressed volume V in m^3 the following condition shall be satisfied:

$$\sigma_{t,90,d} \leq f_{t,90,d} \quad \text{for solid timber} \quad (5.1.3a)$$

$$\sigma_{t,90,d} \leq f_{t,90,d}(V_0/V)^{0,2} \quad \text{for glued laminated timber} \quad (5.1.3b)$$

where V_0 is the reference volume of $0,01 \text{ m}^3$.

5.1.4 Compression parallel to the grain

P(1) The following condition shall be satisfied:

$$\sigma_{c,0,d} \leq f_{c,0,d} \quad (5.1.4)$$

P(2) A check shall also be made of the instability condition (see 5.2.1).

5.1.5 Compression at an angle to the grain

P(1) For compression perpendicular to the grain the following condition shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \quad (5.1.5a)$$

where $k_{c,90}$ (see Table 5.1.5) takes into account that the load can be increased if the loaded length, l in Figure 5.1.5a, is short.

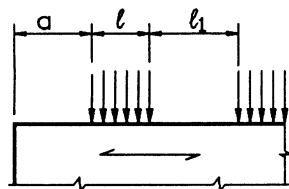


Figure 5.1.5a — Compression perpendicular to the grain

Table 5.1.5 — Values of $k_{c,90}$

	$l_1 \leq 150 \text{ mm}$	$l_1 > 150 \text{ mm}$	
		$a \geq 100 \text{ mm}$	$a < 100 \text{ mm}$
$l \geq 150 \text{ mm}$	1	1	1
$150 \text{ mm} > l \geq 15 \text{ mm}$	1	$1 + \frac{150 - l}{170}$	$1 + \frac{a(150 - l)}{17000}$
$15 \text{ mm} > l$	1	1,8	$1 + a/125$

(2) The compression stresses at an angle α to the grain, (see Figure 5.1.5b), should satisfy the following condition:

$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \quad (5.1.5b)$$

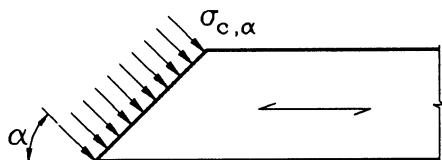


Figure 5.1.5b — Stresses at an angle to the grain

5.1.6 Bending

P(1) The following conditions shall be satisfied:

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.6a)$$

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.6b)$$

where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes as shown in Figure 5.1.6, and $f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths.

(2) The value of the factor k_m should be taken as follows:

- for rectangular sections; $k_m = 0,7$
- for other cross-sections; $k_m = 1,0$

P(3) A check shall also be made of the instability condition (see 5.2.2).

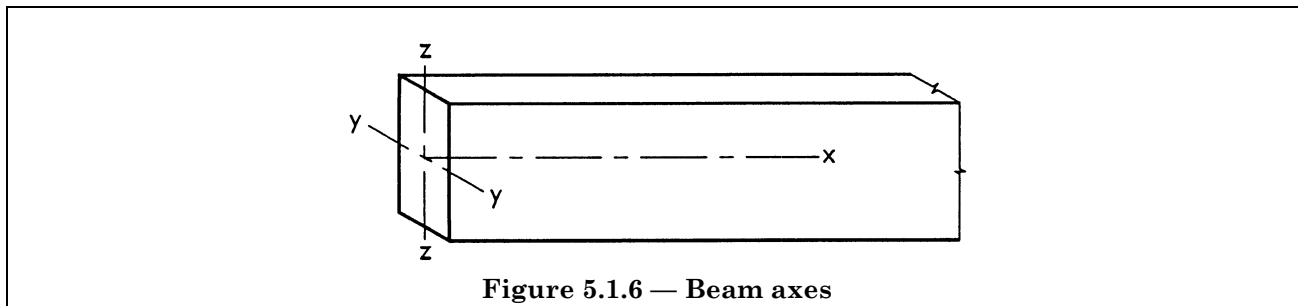


Figure 5.1.6 — Beam axes

5.1.7 Shear

5.1.7.1 General

P(1) The following condition shall be satisfied:

$$\tau_d \leq f_{v,d} \quad (5.1.7.1)$$

(2) At beam ends, the contribution to the total shear force of a point load F within a distance $2h$ of the support may be reduced according to the influence line shown in Figure 5.1.7.1.

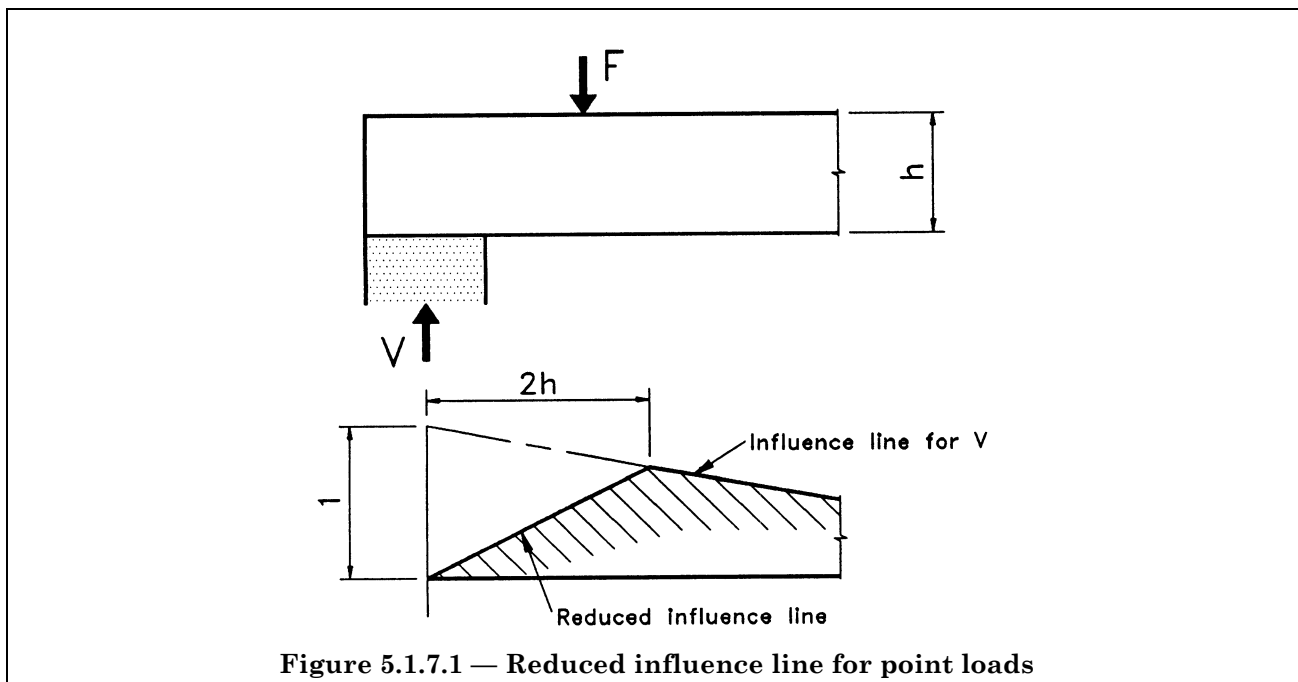


Figure 5.1.7.1 — Reduced influence line for point loads

5.1.7.2 End-notched beams

P(1) For beams notched at the ends, (see Figure 5.1.7.2), the shear stress shall be calculated using the effective (reduced) depth h_e .

P(2) For beams notched on the loaded side (see Figure 5.1.7.2a) the effect of stress concentrations at the re-entrant angle shall be taken into consideration.

(3) It should be verified that

$$\tau_d = 1,5V/bh_e \leq k_v f_{v,d} \quad (5.1.7.2a)$$

For beams notched at the unloaded side

$$k_v = 1 \quad (5.1.7.2b)$$

For beams notched at the loaded side

(5.1.7.2c)

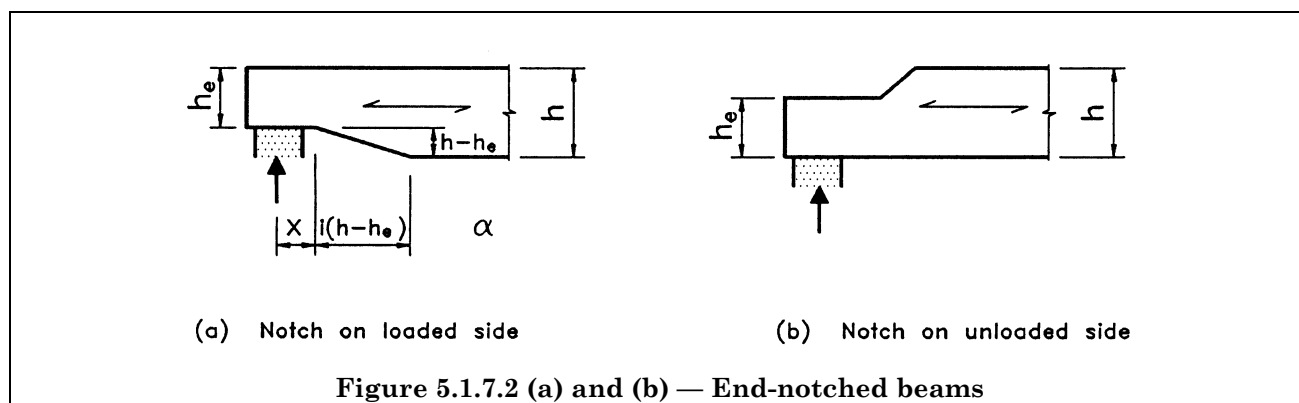
$$k_v = \min. \left\{ \begin{array}{l} 1 \\ \frac{k_n \left(1 + \frac{1,1 i^{1,5}}{\sqrt{h}} \right)}{\sqrt{h} \left(\sqrt{\alpha(1-\alpha)} + 0,8 \frac{x}{h} \sqrt{\frac{1}{\alpha} - \alpha^2} \right)} \end{array} \right. \quad (5.1.7.2d)$$

The value of the factor k_n should be taken as follows:

- for solid timber: $k_n = 5$
- for glued laminated timber: $k_n = 6.5$

The symbols are defined as follows:

- h beam depth in mm
- x distance from line of action to the corner
- a h_e/h
- i notch inclination [see Figure 5.1.7.2(a)]



5.1.8 Torsion

P(1) The torsional stresses shall satisfy the following condition:

$$\tau_{\text{tor},d} \leq f_{v,d} \quad (5.1.8)$$

5.1.9 Combined bending and axial tension

P(1) The following conditions shall be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.9a)$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.9b)$$

where $\sigma_{t,0,d}$ is the design tensile stress and $f_{t,0,d}$ is the design tensile strength.

(2) The values of k_m given in 5.1.6 apply.

5.1.10 Combined bending and axial compression

P(1) The following conditions shall be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.10a)$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \quad (5.1.10b)$$

where $\sigma_{c,0,d}$ is the design compressive stress and $f_{c,0,d}$ is the design compressive strength.

(2) The values of k_m given in 5.1.6 apply.

P(3) A check shall also be made of the instability condition (see 5.2.1)

5.2 Columns and beams

5.2.1 Columns

P(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) The relative slenderness ratios are defined by:

$$\lambda_{rel,y} = \sqrt{\frac{f_{c,0,k}}{\sigma_{c,crit,y}}} \quad (5.2.1a)$$

and

$$\lambda_{rel,z} = \sqrt{\frac{f_{c,0,k}}{\sigma_{c,crit,z}}} \quad (5.2.1b)$$

where

$$\sigma_{c,crit,y} = \frac{\pi^2 E_{0,05}}{\lambda_y^2} \quad (5.2.1c)$$

$$\sigma_{c,crit,z} = \frac{\pi^2 E_{0,05}}{\lambda_z^2} \quad (5.2.1d)$$

λ_y and $\lambda_{rel,y}$ correspond to bending about the y-axis (deflection in the z-direction).

λ_z and $\lambda_{rel,z}$ correspond to bending about the z-axis (deflection in the y-direction).

(3) For both $\lambda_{rel,y} \leq 0,5$ and $\lambda_{rel,z} \leq 0,5$ the stresses should satisfy the conditions in 5.1.10a and b.

(4) In all other cases the stresses should satisfy the following conditions:

$$\frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \quad (5.2.1e)$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \quad (5.2.1f)$$

with

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \quad (\text{similarly for } k_{c,z}) \quad (5.2.1g)$$

$$k_y = 0,5 (1 + \beta_c (\lambda_{rel,y} - 0,5) + \lambda_{rel,y}^2) \quad (\text{similarly for } k_{c,z}) \quad (5.2.1h)$$

The symbols are defined as follows:

σ_m bending stress due to any lateral loads

β_c a factor for members within the straightness limits defined in chapter 7:

— for solid timber: $\beta_c = 0,2$

— for glued laminated timber $\beta_c = 0,1$

k_m as given in 5.1.6

5.2.2 Beams

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

(2) The relative slenderness for bending is defined by

$$\lambda_{rel,m} = \sqrt{f_{m,k} / \sigma_{m,crit}} \quad (5.2.2a)$$

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

(3) The stresses should satisfy the following condition:

$$\sigma_{md} \leq k_{crit} f_{m,d} \quad (5.2.2b)$$

where k_{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 chapter 7, k_{crit} may be determined from (5.2.2c–e).

$$k_{crit} = \begin{cases} 1 & \text{for } \lambda_{rel,m} \leq 0,75 \\ 1,56 - 0,75\lambda_{rel,m} & \text{for } 0,75 < \lambda_{rel,m} \leq 1,4 \\ 1/\lambda_{rel,m}^2 & \text{for } 1,4 < \lambda_{rel,m} \end{cases} \quad (5.2.2c)$$

$$(5.2.2d)$$

$$(5.2.2e)$$

(5) The factor k_{crit} may also be put equal to 1 for a beam where lateral displacement of the compression side is prevented throughout its length and where torsional rotation is prevented at the supports.

5.2.3 Single tapered beams

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

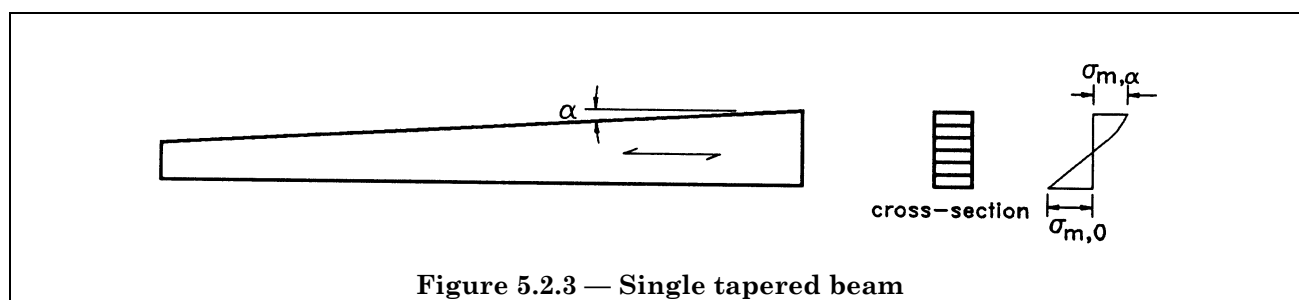


Figure 5.2.3 — Single tapered beam

(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

$$\sigma_{m,0,d} = (1 + 4 \tan^2 \alpha) \frac{6M_d}{bh^2} \quad (5.2.3a)$$

and as

$$\sigma_{m,\alpha,d} = (1 - 4 \tan^2 \alpha) \frac{6M_d}{bh^2} \quad (5.2.3b)$$

on the tapered side.

(3) In the outermost fibre at the tapered edge the stresses should satisfy the following condition:

$$\sigma_{m,\alpha,d} \leq f_{m,\alpha,d} \quad (5.2.3c)$$

where

$$f_{m,\alpha,d} = \frac{f_{m,d}}{\frac{f_{m,d}}{f_{t,90,d}} \sin^2 \alpha + \cos^2 \alpha} \quad (5.2.3d)$$

in the case of tensile stresses parallel to the tapered edge and

$$f_{m,\alpha,d} = \frac{f_{m,d}}{\frac{f_{m,d}}{f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha} \quad (5.2.3e)$$

in case of compressive stresses parallel to the tapered edge.

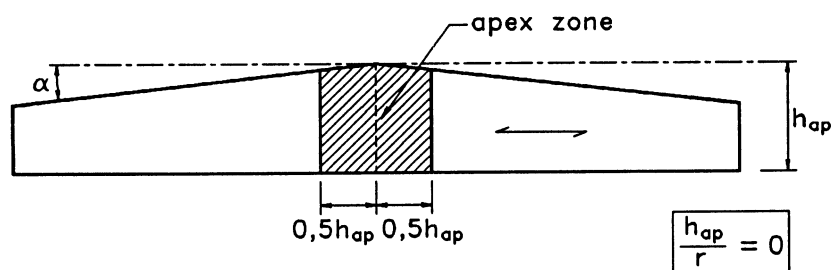
5.2.4 Double tapered, curved and pitched cambered beams

P(1) The requirements of section 5.2.3 apply to the lengths of the beam which have a single taper.

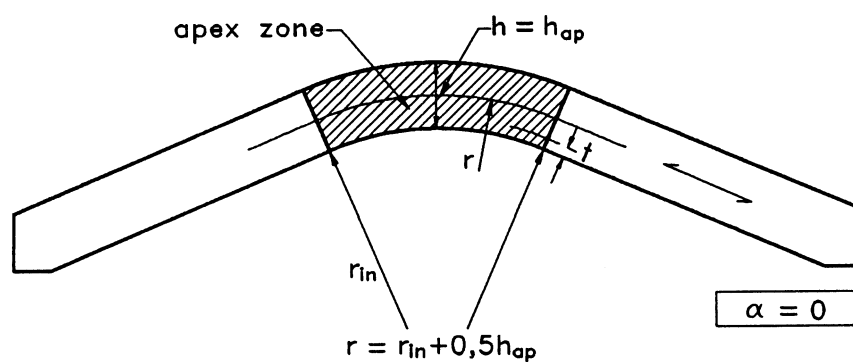
P(2) In the apex zone (see Figure 5.2.4), the bending stresses shall satisfy the following condition:

$$\sigma_{m,d} \leq k_r f_{m,d} \quad (5.2.4a)$$

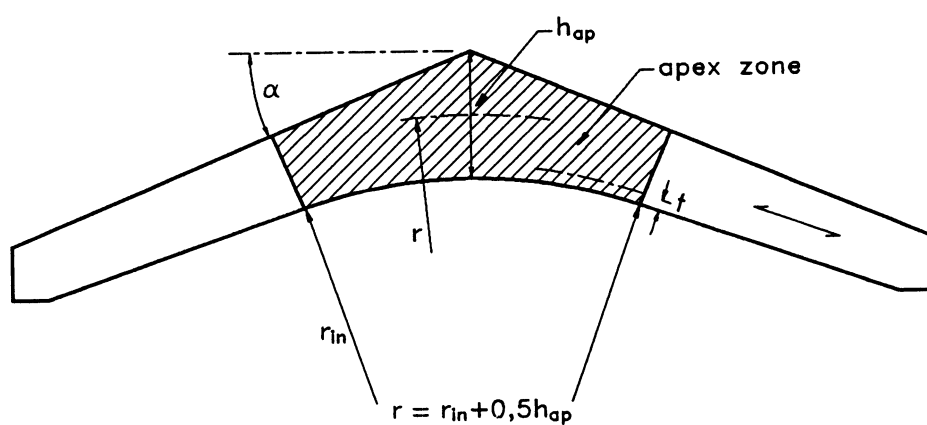
where the factor k_r takes into account the reduction in strength due to bending of the laminates during production.



a) Double tapered beam



b) Curved beam (constant depth)



c) Pitched cambered beam

Figure 5.2.4 — Double tapered a), curved b) and pitched cambered c) beams

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

$$\sigma_{m,d} = k_{\ell} \frac{6M_{ap,d}}{bh_{ap}^2} \quad (5.2.4b)$$

where $h_{ap,r}$ and α are defined in Figure 5.2.4, and

$$k_{\ell} = k_1 + k_2 \left(\frac{h_{ap}}{r} \right) + k_3 \left(\frac{h_{ap}}{r} \right)^2 + k_4 \left(\frac{h_{ap}}{r} \right)^3 \quad (5.2.4c)$$

where

$$k_1 = 1 + 1,4 \tan \alpha + 5,4 \tan^2 \alpha \quad (5.2.4d)$$

$$k_2 = 0,35 - 8 \tan \alpha \quad (5.2.4e)$$

$$k_3 = 0,6 + 8,3 \tan \alpha - 7,8 \tan^2 \alpha \quad (5.2.4f)$$

$$k_4 = 6 \tan^2 \alpha \quad (5.2.4g)$$

(4) For double tapered beams $k_r = 1$. For curved and pitched cambered beams k_r should be taken as

$$k_r = \begin{cases} 1 & \text{for } r_{in}/t \geq 240 \\ 0,76 + 0,001 r_{in}/t & \text{for } r_{in}/t < 240 \end{cases} \quad (5.2.4h)$$

where r_{in} is the radius of the inner beam face and t is the laminate thickness.

(5) In the apex zone the greatest tensile stress perpendicular to the grain should satisfy the following condition:

$$\sigma_{t,90,d} \leq k_{dis} (V_0/V)^{0,2} f_{t,90,d} \quad (5.2.4k)$$

where the symbols are defined as follows:

k_{dis} a factor which takes into account the effect of the stress distribution in the apex zone, with the following values:

— for double tapered and curved beams: $k_{dis} = 1,4$

— for pitched cambered beams: $k_{dis} = 1,7$.

V_0 reference volume of $0,01 \text{ m}^3$

V volume in m^3 of the apex zone (see Figure 5.2.4). As a maximum, V should be taken as $2V_b/3$, where V_b is the total volume of the beam

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

$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{bh_{ap}^2} \quad (5.2.4l)$$

where

$$k_p = k_5 + k_6 \left(\frac{h_{ap}}{r} \right) + k_7 \left(\frac{h_{ap}}{r} \right)^2 \quad (5.2.4m)$$

with

$$k_5 = 0,2 \tan \alpha \quad (5.2.4n)$$

$$k_6 = 0,25 - 1,5 \tan \alpha + 2,6 \tan^2 \alpha \quad (5.2.4o)$$

$$k_7 = 2,1 \tan \alpha - 4 \tan^2 \alpha \quad (5.2.4p)$$

5.3 Components

5.3.1 Glued thin-webbed beams

P(1) A linear variation of strain over the depth of the beam shall be assumed.

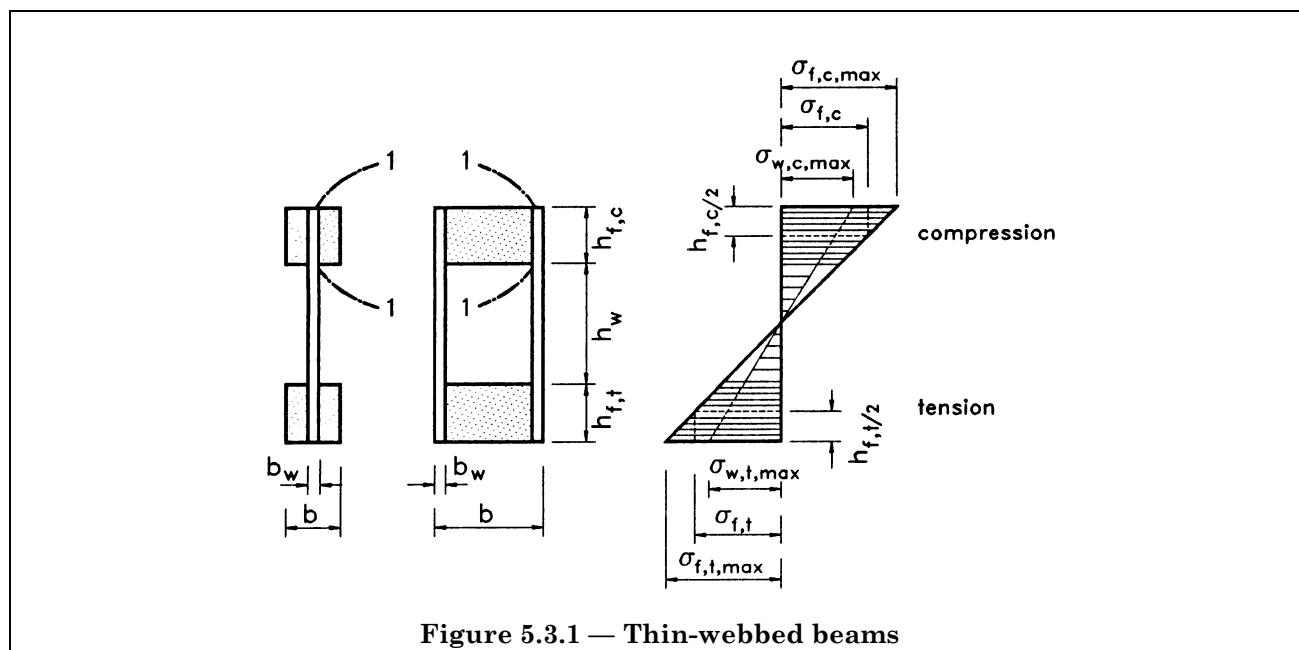


Figure 5.3.1 — Thin-webbed beams

(2) The axial stresses in the flanges should satisfy the following conditions:

$$\sigma_{f,c,max,d} \leq f_{m,d} \quad (5.3.1a)$$

$$\sigma_{f,t,max,d} \leq f_{m,d} \quad (5.3.1b)$$

$$\sigma_{f,c,d} \leq k_c f_{c,0,d} \quad (5.3.1c)$$

$$\sigma_{f,t,d} \leq f_{t,0,d} \quad (5.3.1d)$$

where the symbols are defined as follows:

$\sigma_{f,c,max,d}$ extreme fibre flange design compressive stress

$\sigma_{f,t,max,d}$ extreme fibre flange design tensile stress

$\sigma_{f,c,d}$ mean flange design compressive stress

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

k_c a factor which takes into account lateral instability.

(3) The factor k_c may be determined (conservatively, especially for box beams) according to 5.2.1 with $\lambda_z = \sqrt{12} l_c/b$, where l_c is the distance between the sections where lateral deflection of the compression flange is prevented, and b is given in Figure 5.3.1. If a special investigation is made into the lateral instability of the beam as a whole, $k_c = 1$ may be assumed.

(4) The axial stresses in the webs should satisfy the following conditions:

$$\sigma_{w,c,d} \leq f_{c,w,d} \quad (5.3.1e)$$

$$\sigma_{w,t,d} \leq f_{t,w,d} \quad (5.3.1f)$$

where $\sigma_{w,c,d}$ and $\sigma_{w,t,d}$ are the design compressive and design tensile stresses, and $f_{c,w,d}$ and $f_{t,w,d}$ the design compressive and design 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 should be verified that any splices have sufficient strength.

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

$$h_w \leq 70b_w \quad (5.3.1g)$$

and

$$V_d \leq \begin{cases} b_w h_w (1 + 0,5 (h_{f,t} + h_{f,c})/h_w) f_{v,0,d} & \text{for } h_w \leq 35b_w \\ 35b_w^2 (1 + 0,5 (h_{f,t} + h_{f,c})/h_w) f_{v,0,d} & \text{for } 35b_w \leq h_w \leq 70b_w \end{cases} \quad (5.3.1h)$$

$$(5.3.1j)$$

where symbols are defined as follows:

h_w	web depth
$h_{f,c}$	compression flange depth
$h_{f,t}$	tension flange depth
b_w	web width
$f_{v,0,d}$	design panel shear strength

(8) For sections 1-1 in Figure 5.3.1 it should be verified that

$$\tau_{\text{mean},d} \leq \begin{cases} f_{v,90,d} & \text{for } h_f \leq 4 b_w \\ f_{v,90,d} (4b_w/h_w)^{0.8} & \text{for } h_f > 4 b_w \end{cases} \quad (5.3.1k)$$

$$(5.3.1l)$$

where τ_{mean} is the shear stress at the sections, assuming a uniform distribution, $f_{v,90,d}$ is the design planar (rolling) shear strength of the web and h_f is either $h_{f,c}$ or $h_{f,t}$.

5.3.2 Glued thin-flanged beams

P(1) A linear variation of strain over the depth of the beam shall be assumed.

P(2) 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 5.3.2) with effective flange widths b_{ef} where

$$b_{\text{ef}} = b_{c,\text{ef}} + b_w \text{ (or } b_{t,\text{ef}} + b_w) \quad (5.3.2a)$$

or

$$b_{\text{ef}} = 0,5b_{c,\text{ef}} + b_w \text{ (or } 0,5b_{t,\text{ef}} + b_w) \quad (5.3.2b)$$

The values of $b_{c,\text{ef}}$ and $b_{t,\text{ef}}$ should not be greater than the maximum value calculated for shear lag. In addition the value of $b_{c,\text{ef}}$ should not be greater than the maximum value calculated for plate buckling.

(4) The maximum effective flange widths due to the effects of shear lag and plate buckling are given in Table 5.3.2, where l is the span of the beam.

Table 5.3.2 — Maximum effective flange widths due to the effect of shear lag and plate buckling

Flange material	Shear lag	Plate buckling
Plywood, with grain direction in the outer plies:		
— parallel to the webs	0,11	25 h_f
— perpendicular to the webs	0,11	20 h_f
Oriented strand board	0,15 l	25 h_f
Particleboard or fibreboard with random fibre orientation	0,21	30 h_f

(5) Unless a detailed buckling investigation is made, the free flange width should not be greater than twice the effective width due to plate buckling.

(6) For sections 1-1 in Figure 5.3.2 it should be verified that

$$\tau_{\text{mean},d} \leq f_{v,90,d} \quad (5.3.2c)$$

where $\tau_{\text{mean},d}$ is the design shear stress at the sections, assuming a uniform distribution, and $f_{v,90,d}$ is the design planar (rolling) shear strength of the flange.

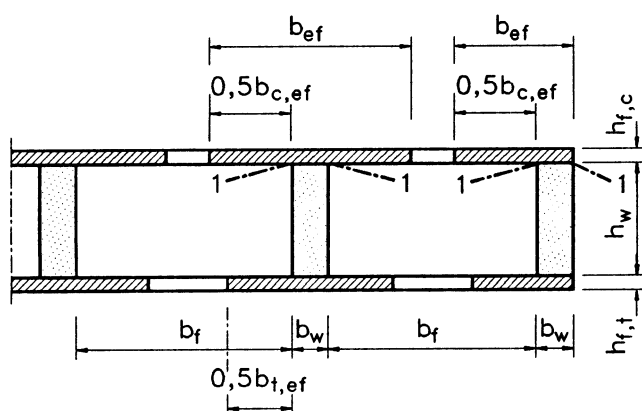


Figure 5.3.2 — Thin-flanged beam

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

$$\sigma_{f,c,d} \leq f_{f,c,d} \quad (5.3.2d)$$

$$\sigma_{f,t,d} \leq f_{f,t,d} \quad (5.3.2e)$$

where the symbols are defined as follows:

$\sigma_{f,c,d}$ ($f_{f,c,d}$) mean flange design compressive stress

$\sigma_{f,t,d}$ ($f_{f,t,d}$) mean flange design tensile stress

$f_{f,c,d}$ flange design compressive strength

$f_{f,t,d}$ flange design tensile strength

(8) It should be verified that any splices have sufficient strength.

5.3.3 Mechanically jointed beams

P(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) For dowel-type fasteners the instantaneous slip modulus K_u per shear plane for ultimate limit state design should be taken as

$$K_u = 2 K_{ser}/3 \quad (5.3.3a)$$

Values of K_{ser} are given in 4.2.

(4) If the spacing of the fasteners varies uniformly in the longitudinal direction according to the shear force between S_{min} and S_{max} ($\leq 4s_{min}$), an effective value s_{ef} may be used, where:

$$s_{ef} = 0,75 s_{min} + 0,25 s_{max} \quad (5.3.3b)$$

(5) The stresses should as a minimum be calculated at instantaneous and final deformation, using the appropriate values of k_{def} from Table 4.1.

(6) A method for the calculation of the load-carrying capacity of mechanically jointed beams is given in Annex B²³⁾.

5.3.4 Mechanically jointed and glued columns

P(1) The deformations due to slip in joints, to shear and bending in packs, gussets, shafts and flanges, and to the axial forces in the lattice shall be taken into account.

(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.

5.4 Assemblies

5.4.1 Trusses

5.4.1.1 General

P(1) Unless a more general model is used, trusses shall be represented for the purpose of analysis by beam elements set out along system lines and connected together at nodes (e.g. as shown in Figure 5.4.1.1).

P(2) The system lines for all members shall lie within the member profile, and for external members shall coincide with the member centre line.

(3) Fictitious beam elements may be used to model eccentric connections or supports. The orientation of fictitious beam elements should coincide as closely as possible with the direction of the force in the member.

(4) In the analysis the geometric non-linear behaviour of a member in compression (buckling instability) may be disregarded if it is taken into account in the strength verification of the individual member.

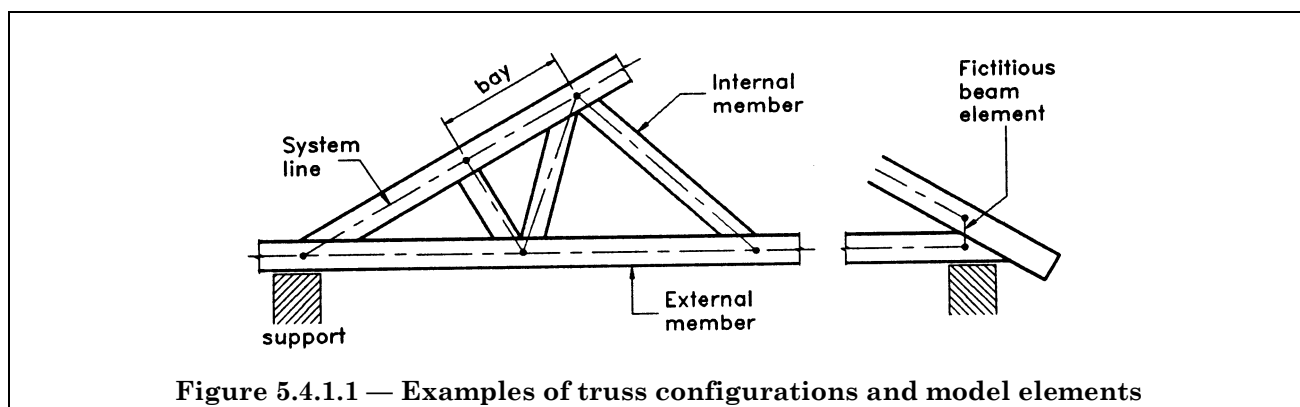


Figure 5.4.1.1 — Examples of truss configurations and model elements

5.4.1.2 General analysis

P(1) Trusses shall be analysed as framed structures, where 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.

P(2) 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.

(3) The analysis should be carried out using the appropriate values of member stiffness defined in chapter 3, and joint slip defined in 4.2 or Annex D. Fictitious beam elements should be assumed to be as stiff as the adjacent elements.

(4) If a geometric non-linear analysis is carried out, the member stiffness should be divided by the partial factor γ_m (given in Table 2.3.3.2).

(5) Joints may be generally assumed to be rotationally pinned.

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

(7) Joints may be assumed to be rotationally stiff, if their deformation would have no significant effect upon the distribution of member forces and moments.

²³⁾ The method described in this annex may be applied to composite members made from timber in combination with other materials.

5.4.1.3 Simplified analysis

(1) As an alternative to a general analysis, a simplified analysis is permitted for fully triangulated trusses which comply with the following conditions:

- there are no re-entrant angles in the external profile
- some part of the bearing width lies vertically below the support node (see Figure 5.4.1.1) or complies with D.4(2)
- the truss height exceeds 0,15 times the span and 10 times the maximum chord depth

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

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

5.4.1.4 Strength verification of members

(1) For elements in compression, the effective column length for in-plane strength verification should generally be taken as the distance between two adjacent points of contraflexure.

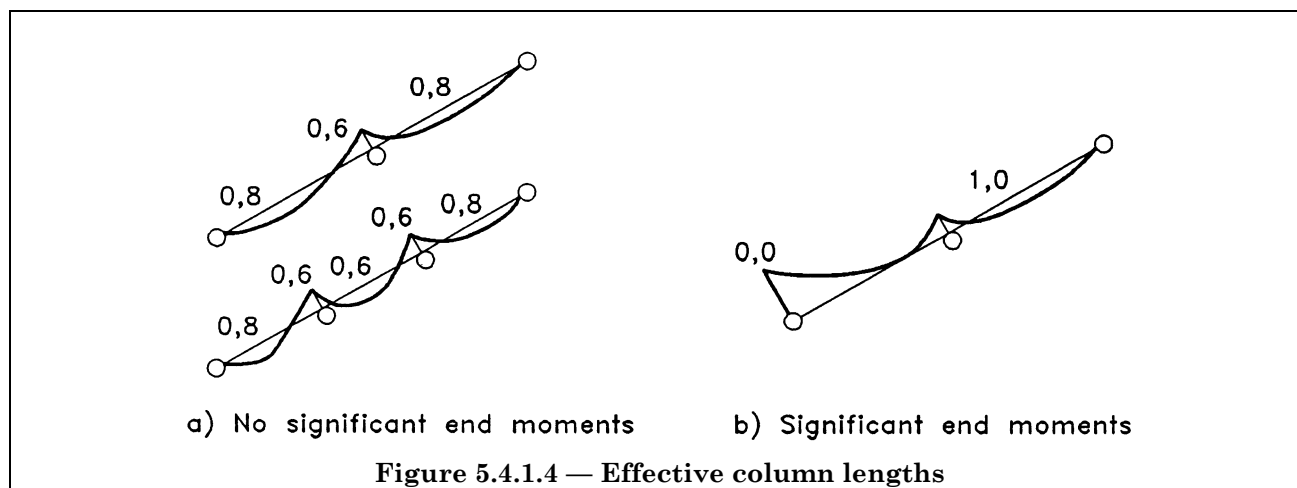
(2) For fully triangulated trusses, the effective column length for

- members which are only one bay long without especially rigid end connections, and
 - continuous members without lateral load
- should be taken as the bay length.

(3) When a simplified analysis has been carried out, the following effective column lengths may be assumed (see Figure 5.4.1.4).

- 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 times 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 (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 %.



P(4) A check shall also be made that the lateral (out-of-plane) stability of the members is adequate.

5.4.1.5 Trusses with punched metal plate fasteners

(1) Additional rules for trusses with punched metal plate fasteners are given in Annex D.

5.4.2 Roof and floor diaphragms

P(1) This section relates to the racking strength under wind action of horizontal 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 chapter 6.

(3) For diaphragms with a uniformly distributed load (see Figure 5.4.2) the following simplified analysis may be used provided:

- the span l lies between $2b$ and $6b$, where b is the width
- the critical ultimate design condition is failure in the fasteners (and not in the panels) and
- the panels are fixed in accordance with the detailing rules in chapter 7.

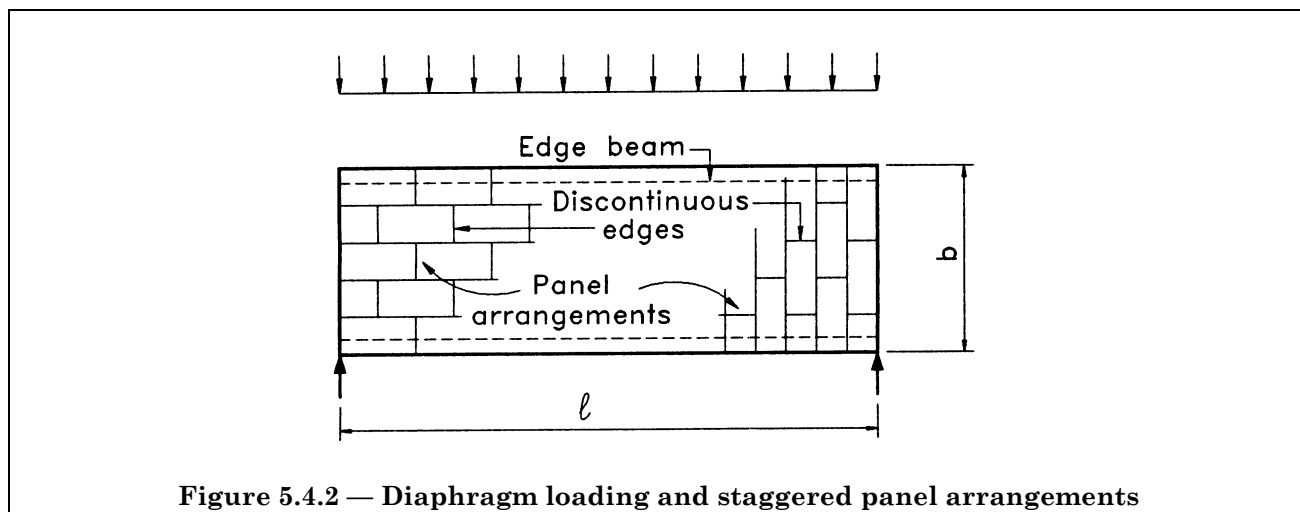


Figure 5.4.2 — Diaphragm loading and staggered panel arrangements

(4) Unless a more detailed analysis is made, the edge beams should be designed to resist the maximum bending moment in the diaphragm.

(5) The shear forces in the diaphragm may be assumed to be uniformly distributed over the width of the diaphragm.

(6) When the sheets are staggered, (see Figure 5.4.2), the nail spacings along the discontinuous panel edges may be increased by a factor of 1,5 (up to a maximum of 150 mm) without reduction of the load-carrying capacity.

5.4.3 Wall diaphragms

P(1) This section relates to the racking strength of cantilevered wall diaphragms. The diaphragms consist of framed panels made from sheets of board material fixed by mechanical fasteners to one or both sides of a timber frame.

(2) The load carrying capacity F_k (the racking resistance) under a force acting at the top of a cantilevered panel secured against uplift (by vertical actions or by anchoring) should be determined by:

- calculations, or
- testing of prototype structures in accordance with prEN 594.

(3) The following simplified analysis may be used for a wall panel which consist of sheets fixed to one side of a timber frame [see Figure 5.4.3(a)], provided that:

- there are no openings in excess of 200 mm square
- the fastener spacing is constant along the perimeter of every sheet
- $b \geq h/4$

(4) The design racking load carrying capacity $F_{v,d}$ should be calculated as

$$F_{v,d} = \Sigma F_{f,d} (b_i/b_1)^2 b_1/s \quad (5.4.3a)$$

where the symbols are defined as follows:

$F_{f,d}$ lateral design capacity per fastener

b_1 width of widest sheet

b_i width of other sheets (b_2, b_3, \dots)

s spacing of the fasteners

The design load-carrying capacity of fasteners along the edges of the panels may be increased by a factor of 1,2 over the corresponding values given in chapter 6.

(5) If there are sheets on both sides of the panel, of the same type and thickness, the load-carrying capacity may be taken as the sum of the calculated contributions. If the sheets or the fasteners are of different types only half the load-carrying capacity of the weaker side should be taken.

(6) The compression studs should be designed for a force

$$F_d = \begin{cases} 0,67F_{v,d} h/b & \text{for sheets on both sides} \\ 0,75F_{v,d} h/b & \text{for sheets on one side} \end{cases} \quad (5.4.3b)$$

$$(5.4.3c)$$

(7) The tensile studs should be directly anchored to the substrate, and designed for a force F_d , where

$$F_d = F_{v,d} h/b \quad (5.4.3d)$$

(8) If individual sheets within the diaphragm contain door or window openings, these sheets shall not be assumed to contribute to the overall racking strength. Each group of adjacent solid sheets should be anchored as an individual wall panel as shown in Figure 5.4.3c.

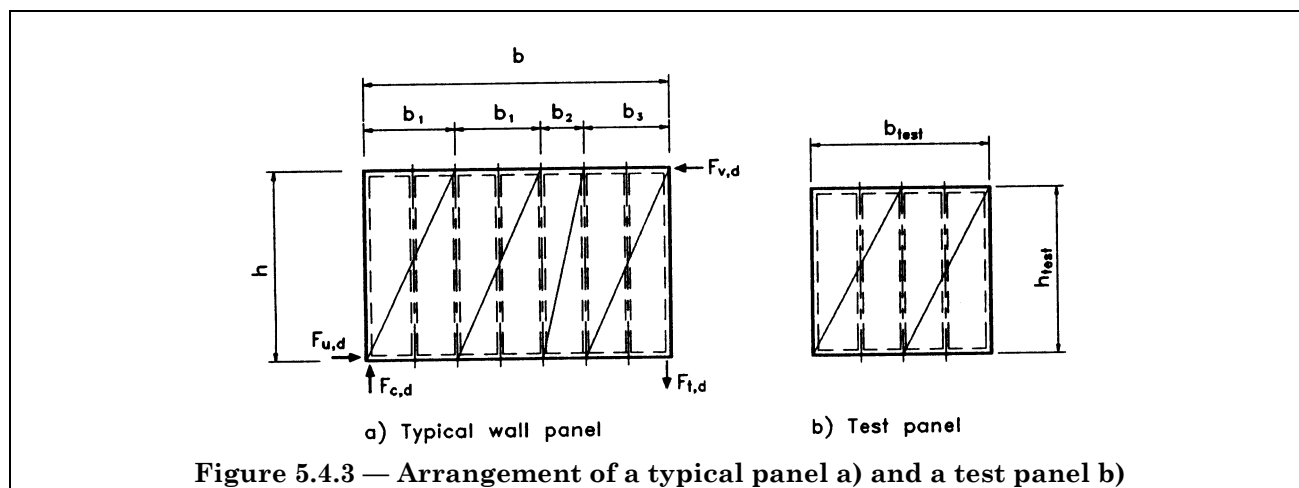


Figure 5.4.3 — Arrangement of a typical panel a) and a test panel b)

(9) If the characteristic strength of a test panel [see Figure 5.4.3 b)] has been determined, the strength of a panel of similar construction, but with a different height h and width b , is given by

$$F_k = k_b k_h F_{test,k} \quad (5.4.3e)$$

where

$$k_b = \begin{cases} b/b_{test} & \text{for } b_{test} \leq b \\ (b/b_{test})^2 & \text{for } 0,5b_{test} \leq b < b_{test} \\ 0 & \text{for } b \leq 0,5b_{test} \end{cases} \quad (5.4.3f)$$

$$(5.4.3g)$$

$$(5.4.3h)$$

and

$$k_h = \begin{cases} h_{test}/h & \text{for } h \geq h_{test} \\ 1 & \text{for } h < h_{test} \end{cases} \quad (5.4.3j)$$

$$(5.4.3k)$$

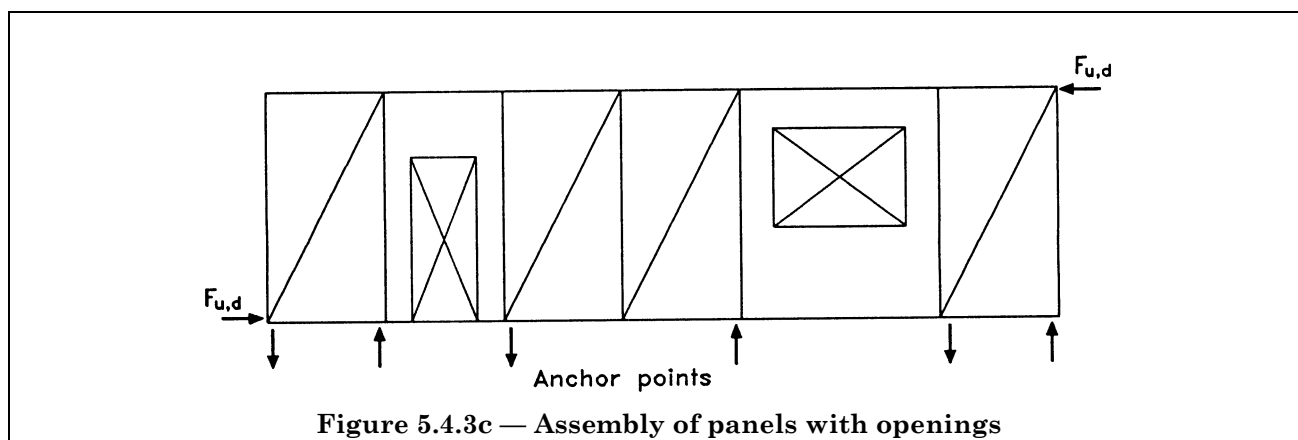


Figure 5.4.3c — Assembly of panels with openings

5.4.4 Plane frames

P(1) The stresses caused by geometrical and structural imperfections — i.e. deviations between the geometrical axis and the elastic centre of the cross-section due to e.g. material inhomogenities — and induced deflection shall be taken into account.

(2) This may be done 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 in approximate affinity to the relevant deformation figure, and found by applying an angle ϕ of inclination to the structure or relevant parts, together with an initial sinusoidal curvature between the nodes of the structure corresponding to a maximum eccentricity e .

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

$$\phi = 0,005 \quad \text{for } h \leq 5\text{m} \quad (5.4.4a)$$

$$\phi = 0,005 \sqrt{5/h} \quad \text{for } h > 5\text{m} \quad (5.4.4b)$$

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

— the value of e should as a minimum be taken as:

$$e = 0,003 \text{ l} \quad (5.4.4c)$$

— the deflection should be calculated using a value of E of:

$$E = E_{0,05} f_{m,d} / f_{m,k} \quad (5.4.4d)$$

Examples of assumed initial deflections are given in Figure 5.4.4.

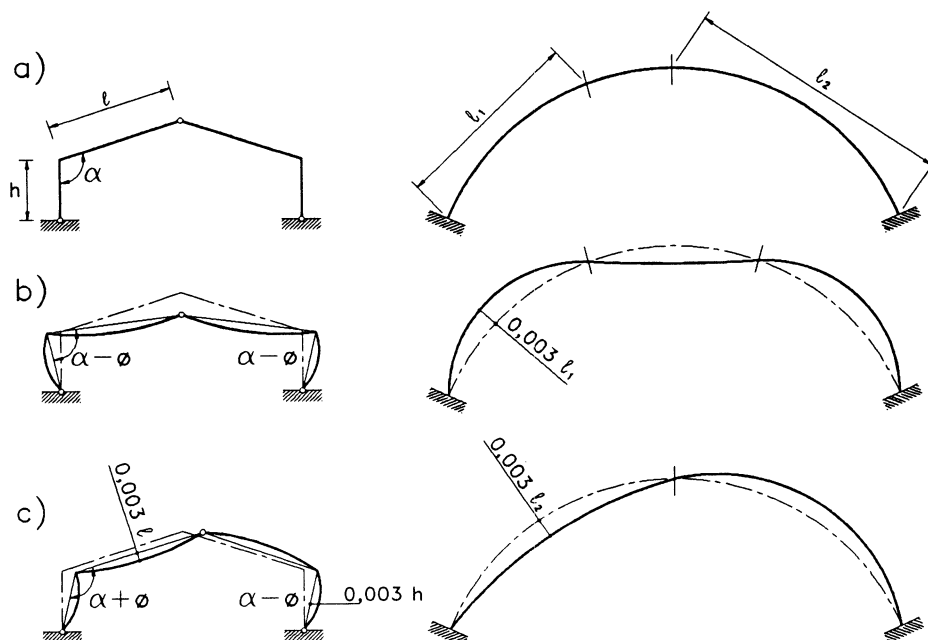


Figure 5.4.4 — Examples of assumed initial deflections for a frame a), corresponding to a symmetrical load b) and non-symmetrical load c)

5.4.5 Bracing

5.4.5.1 General

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

P(2) The stress caused by geometrical and structural imperfections, and by induced deflections (including the contribution from any joint slip) shall be taken into account.

P(3) The bracing forces shall be determined on the basis of the most unfavourable combination of structural imperfections and induced deflections.

5.4.5.2 Single members in compression

(1) For single elements in compression requiring lateral support at intervals (see Figure 5.4.5.2) the initial deviations from straightness between supports should be within $a/500$ for glued laminated members, and $a/300$ for other members.

(2) Each intermediate support should have a minimum spring stiffness C , given by

$$C = k_s \pi^2 EI / a^3 \quad (5.4.5.2a)$$

where

$$E = E_{0,05} f_{m,d} / f_{m,k} \quad (5.4.5.2b)$$

$$k_s = 2(1 + \cos \pi/m) \quad (5.4.5.2c)$$

and m is the number of bays each of length a .

(3) The design stabilising force F_d at each support should as a minimum be taken as:

$$F_d = N_d / 50 \quad \text{for solid timber;} \quad (5.4.5.2d)$$

$$F_d = N_d / 80 \quad \text{for glued laminated timber;} \quad (5.4.5.2e)$$

where N_d is the mean design compressive force in the element.

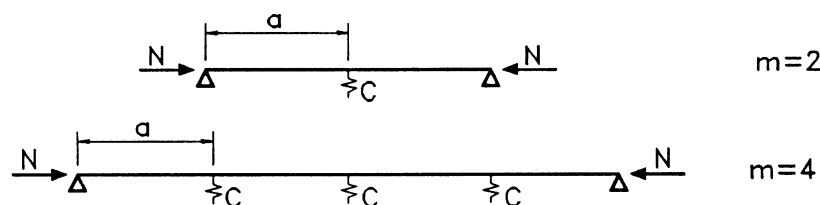


Figure 5.4.5.2 — Examples of single members in compression braced by lateral supports

(4) The design stabilising force F_d for the compression flange of a rectangular beam should be determined in accordance with 5.4.5.2(3), where

$$N_d = (1 - k_{crit}) M_d / h \quad (5.4.5.2f)$$

The value of k_{crit} should be determined from 5.2.2(4) for the unbraced beam. M_d is the maximum design moment in the beam of depth h .

5.4.5.3 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 5.4.5.3) a bracing system should be provided, which, in addition to the effects of a horizontal load should be capable of resisting a load per unit length q , where

$$q_d = k_1 \frac{n N_d}{30 \ell} \quad (5.4.5.3a)$$

and where

$$k_\ell = \min. \left\{ \begin{array}{l} 1 \\ \sqrt{15/\ell} \end{array} \right. \quad (5.4.5.3b)$$

$$(5.4.5.3c)$$

N_d is mean design axial compression force in the member, of overall length ℓ m.

(2) The horizontal deflection at midspan due to q_d acting alone should not exceed $\ell/700$.

(3) The horizontal deflection due to q_d and any other load should not exceed $\ell/500$.

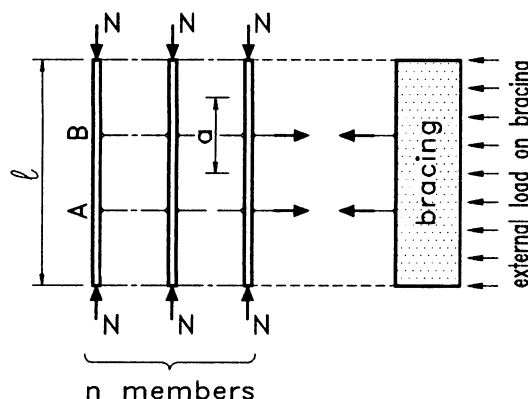


Figure 5.4.5.3 — Beam or truss system requiring lateral supports

5.4.6 Load sharing

(1) When an assembly of several equally spaced similar members is laterally connected by a continuous load-distribution system, the member design strengths may be multiplied by a load sharing factor k_{ls} .

- (2) Unless a more detailed analysis is made, a value of $k_{ls} = 1,1$ may be assumed for the assemblies and load-distribution systems described in Table 5.4.6 if the following requirements are fulfilled:
- the load-distribution system is designed to support the applied permanent and variable loads
 - each element of the load-distribution system is continuous over at least two spans, and any joints are staggered.

Table 5.4.6 — Description of assemblies and load-distribution systems

Assembly	Load-distribution system
Flat roof or floor joists (maximum span 6 m)	Boards or sheathing
Roof trusses (maximum span 12 m)	Tiling battens, purlins or sheathing
Rafters (maximum span 6 m)	Tiling battens or sheathing
Wall studs (maximum height 4 m)	Head and sole plates, sheathing at least one side.

6 Joints

6.1 General

- P(1) The characteristic load-carrying capacities and deformation characteristics of fasteners shall be determined on the basis of tests carried out in conformity with EN 26891, EN 28970, and the relevant European test standards unless design rules are given below. In cases where both compressive and tensile tests are described in the relevant standards, the tensile test shall be used.
- P(2) It shall be taken into account that the characteristic load-carrying capacity of a multiple-fastener joint will frequently be less than the sum of the individual fastener capacities.
- P(3) If the load at a joint is transferred by more than one type of fastener, account shall be taken of the effect of the different fastener properties²⁴⁾.
- P(4) It shall be taken into account that the characteristic load-carrying capacity of a joint will frequently be reduced if it is subject to reversal of load from long- and medium-term actions.
- (5) The effect on joint strength of long-term and medium-term actions alternating between tension F_t and compression F_c in the members should be taken into account by designing the joint for $F_{t,d} + 0,5F_{c,d}$ and $F_{c,d} + 0,5F_{t,d}$.
- P(6) The arrangement and sizes of the fasteners in a joint, and the fastener spacings, edge and end distances shall be chosen so that the expected strengths can be obtained.
- P(7) When the force in the joint acts at an angle to the grain the influence of the tension stresses perpendicular to the grain shall be taken into account.
- (8) Unless a more detailed calculation is made, for the arrangement shown in Figure 6.1 it should be shown that the following condition is satisfied:

$$V_d \leq 2 f_{v,d} b_e t / 3$$

(6.1a)

provided that $b_e > 0,5h$. The symbols are defined as follows:

- V_d

design shear force produced in the member of thickness t by the fasteners or connectors
($V_1 + V_2 = F \sin \alpha$)
- b_e

distance from the loaded edge to the furthest fastener or connector
- α

angle between force F and grain direction.

²⁴⁾ Glue and mechanical fasteners have very different stiffness properties and should not be assumed to act in unison.

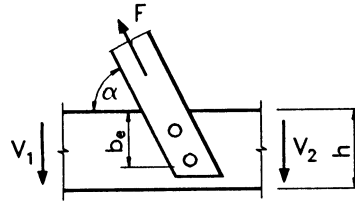


Figure 6.1 — Joint force acting at an angle to the grain

(9) For dowel-type fasteners the instantaneous slip modulus K_u per shear plane per fastener for ultimate limit state design should be taken as

$$K_u = 2K_{ser}/3 \quad (6.1b)$$

Values of K_{ser} are given in Table 4.2.

6.2 Lateral load-carrying capacity of dowel-type fasteners

6.2.1 Timber-to-timber and panel-to-timber joints

(1) The design load-carrying capacity per shear plane per fastener, for timber-to-timber and panel-to-timber joints made with fasteners covered in sections 6.3 to 6.7 should be taken as the smallest value found from the following formulae:

Design load-carrying capacities for fasteners in single shear:

$$R_d = \min. \left\{ \begin{array}{l} f_{h,1,d} t_1 d \quad (6.2.1a) \\ f_{h,1,d} t_2 d \beta \quad (6.2.1b) \\ \frac{f_{h,1,d} t_1 d}{1 + \beta} \left[\sqrt{\beta + 2\beta^2 \left[1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1} \right)^2 \right] + \beta^3 \left(\frac{t_2}{t_1} \right)^2} - \beta \left(1 + \frac{t_2}{t_1} \right) \right] \quad (6.2.1c) \\ 1,1 \frac{f_{h,1,d} t_1 d}{2 + \beta} \left[\sqrt{2\beta(1 + \beta) + \frac{4\beta(2 + \beta) M_{y,d}}{f_{h,1,d} d t_1^2}} - \beta \right] \quad (6.2.1d) \\ 1,1 \frac{f_{h,1,d} t_2 d}{1 + 2\beta} \left[\sqrt{2\beta^2(1 + \beta) + \frac{4\beta(1 + 2\beta) M_{y,d}}{f_{h,1,d} d t_2^2}} - \beta \right] \quad (6.2.1e) \\ 1,1 \sqrt{\frac{2\beta}{1 + \beta}} \sqrt{2M_{y,d} f_{h,1,d} d} \quad (6.2.1f) \end{array} \right.$$

Design load-carrying capacities for fasteners in double shear:

$$R_d = \min. \left\{ \begin{array}{l} f_{h,1,d} t_1 d \quad (6.2.1g) \\ 0.5 f_{h,1,d} t_2 d \beta \quad (6.2.1h) \\ 1,1 \frac{f_{h,1,d} t_1 d}{2 + \beta} \left[\sqrt{2 \beta (1 + \beta) + \frac{4 \beta (2 + \beta) M_{y,d}}{f_{h,1,d} d t_1^2}} - \beta \right] \quad (6.2.1j) \\ 1,1 \sqrt{\frac{2 \beta}{1 + \beta}} \sqrt{2 M_{y,d} f_{h,1,d} d} \quad (6.2.1k) \end{array} \right.$$

The various failure modes are illustrated in Figure 6.2.1. The symbols are defined as follows:

t_1 and t_2	timber or board thickness or penetration. (See also sections 6.3 to 6.7).
$f_{h,1,d}$ ($f_{h,2,d}$)	design embedding strength in t_1 (t_2)
β	$f_{h,2,d}/f_{h,1,d}$
d	fastener diameter
$M_{y,d}$	fastener design yield moment

(2) The design values of the embedding strengths, $f_{h,1,d}$ or $f_{h,2,d}$, respectively, should be calculated as:

$$f_{h,1,d} = \frac{k_{mod,1} f_{h,1,k}}{\gamma_M} \quad (6.2.1l)$$

$$f_{h,2,d} = \frac{k_{mod,2} f_{h,2,k}}{\gamma_M} \quad (6.2.1m)$$

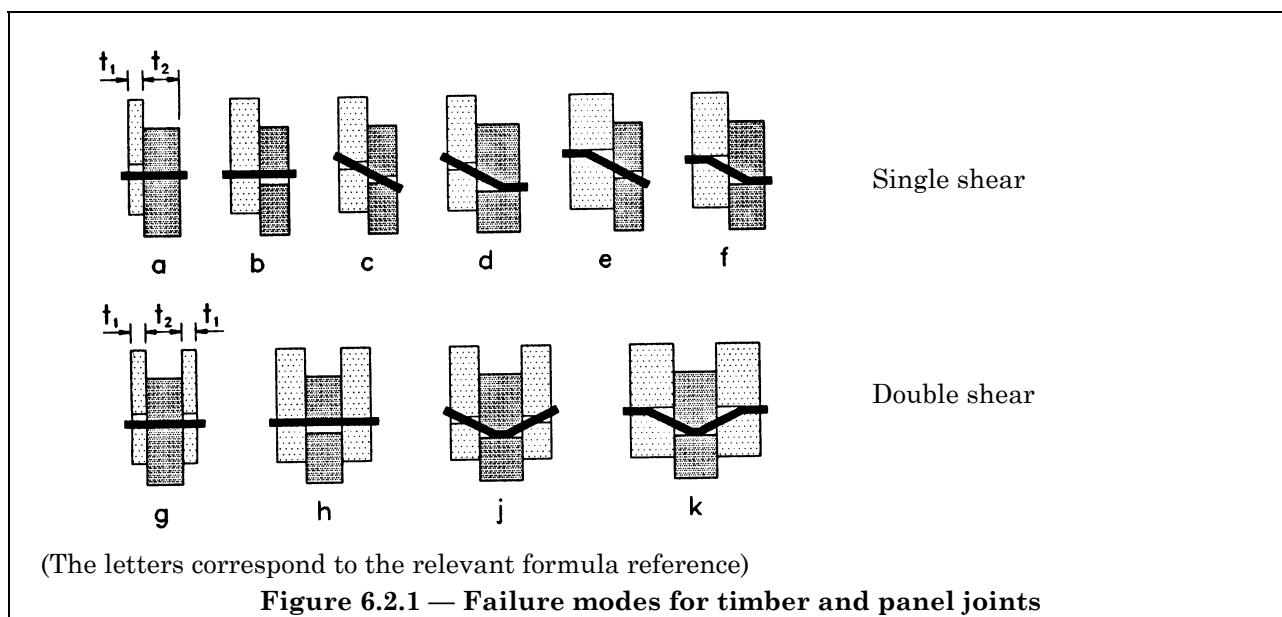
Values of the modification factor k_{mod} are given in Table 3.1.7, and the values of γ_M are given in Table 2.3.3.2.

(3) The design value of the fastener yield moment $M_{y,d}$ should be calculated as:

$$M_{y,d} = \frac{M_{y,k}}{\gamma_M} \quad (6.2.1n)$$

where γ_M is given in Table 2.3.3.2.

(4) The embedding strength f_h should be determined in accordance with prEN 383 and Annex A, unless specified in the following clauses.



(5) The yield moment M_y should be determined in accordance with prEN 409 and Annex A, unless specified in the following clauses.

6.2.2 Steel-to-timber joints

(1) The design load-carrying capacity per fastener for single shear steel-to-timber joints, for a thin steel plate (i.e. for $t \leq 0,5d$ where t is the thickness), should be taken as the smaller value found from the following formulae:

$$R_d = \min. \left\{ \begin{array}{l} 0,4 \ f_{h,1,d} t_1 d \\ 1,1 \ \sqrt{2M_{y,d} f_{h,1,d}} \end{array} \right. \quad (6.2.2a)$$

(6.2.2b)

For a thick steel plate (i.e. for $t \geq d$), the design load-carrying capacity should be taken as the smaller value found from the following formulae:

$$R_d = \min. \left\{ \begin{array}{l} 1,1 \ f_{h,1,d} t_1 d \left[\sqrt{2 + \frac{4M_{y,d}}{f_{h,1,d} d t_1^2}} - 1 \right] \\ 1,5 \ \sqrt{2M_{y,d} f_{h,1,d}} \end{array} \right. \quad (6.2.2c)$$

(6.2.2d)

For $0,5d < t < d$ linear interpolation is permitted.

The symbols are defined in 6.2.1(1), and the failure modes are illustrated in Figure 6.2.2a–d.

(2) The design load-carrying capacity per shear plane per fastener for double shear joints with the centre member of steel should be taken as the smallest value found from the following formulae:

$$R_d = \min. \begin{cases} 1,1 f_{h,1,d} t_1 d & (6.2.2e) \\ 1,1 f_{h,1,d} t_1 d \left[\sqrt{2 + \frac{4M_{y,d}}{f_{h,1,d} d t_1^2}} - 1 \right] & (6.2.2f) \\ 1,5 \sqrt{2M_{y,d} f_{h,1,d} d} & (6.2.2g) \end{cases}$$

where the symbols are defined in 6.2.1(1), and the failure modes are illustrated in Figure 6.2.2e–g.

(3) The design load-carrying capacity per shear plane per fastener for double shear joints with both outer members of thin steel should be taken as the smaller value found from the following formulae:

$$R_d = \min. \begin{cases} 0,5 f_{h,2,d} t_2 d & (6.2.2h) \\ 1,1 \sqrt{2M_{y,d} f_{h,2,d} d} & (6.2.2j) \end{cases}$$

(4) For thick steel plates (i.e. for $t \geq d$), the design load-carrying capacity should be taken as the smaller value found from the following formulae:

$$R_d = \min. \begin{cases} 0,5 f_{h,2,d} t_2 d & (6.2.2k) \\ 1,5 \sqrt{2M_{y,d} f_{h,2,d} d} & (6.2.2l) \end{cases}$$

For $0,5d < t < d$ linear interpolation is permitted.

The symbols are defined in 6.2.1(1), and the failure modes are illustrated in Figure 6.2.2h–l.

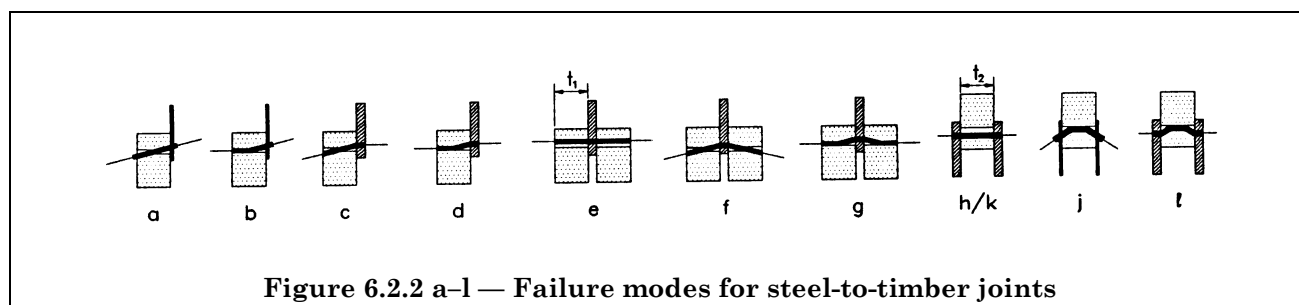


Figure 6.2.2 a–l — Failure modes for steel-to-timber joints

(5) A check should also be made on the strength of the steel plate.

6.2.3 Multiple shear joints

(1) In multiple shear joints the total load-carrying capacity should be determined by calculating the sum of the lowest load-carrying capacities for each shear plane, taking each shear plane as part of a series of three-member joints.

6.3 Nailed joints

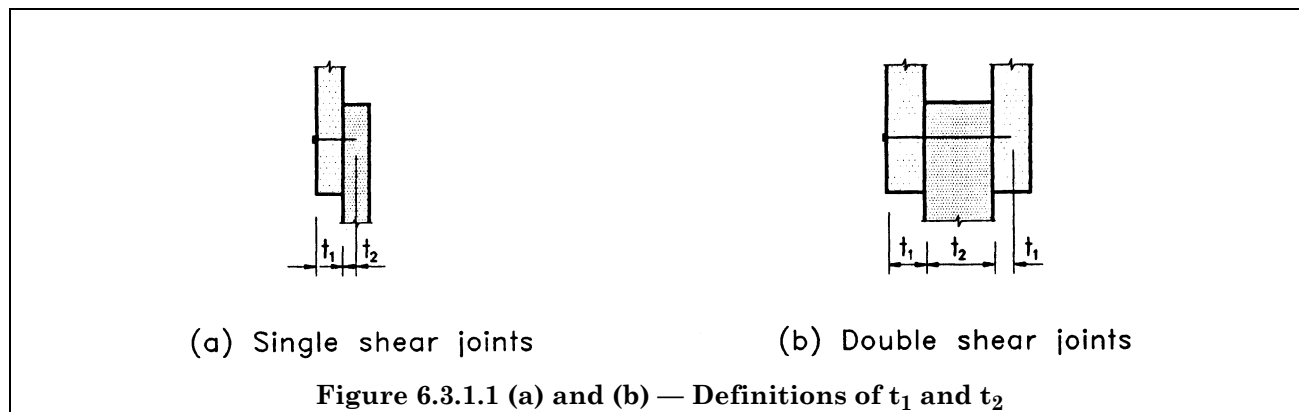
6.3.1 Laterally loaded nails

6.3.1.1 General

(1) The rules in 6.2 apply, with the symbols defined as follows:

- t_1 (for double shear joints) the lesser of the headside timber thickness and the pointside penetration (see Figure 6.3.1.1)
- t_2 pointside penetration for single shear joints and central thickness for double shear joints.

(2) For square nails d should be taken as the side dimension.



6.3.1.2 Nailed timber-to-timber joints

(1) The following characteristic embedding strength values should be used for nails up to 8 mm, for all angles to the grain:

— without predrilled holes $f_{h,k} = 0,082\rho_k d^{-0,3} \text{ N/mm}^2$ (6.3.1.2a)

— with predrilled holes $f_{h,k} = 0,082(1 - 0,01d) \rho_k \text{ N/mm}^2$ (6.3.1.2b)

with ρ_k in kg/m^3 and d in mm.

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

$$M_{y,k} = 180d^{2,6} \text{ Nmm} \quad (6.3.1.2c)$$

for round nails, and

$$M_{y,k} = 270d^{2,6} \text{ Nmm} \quad (6.3.1.2d)$$

for square nails, with d in mm.

(3) Holes should be pre-drilled for nails in timber with a characteristic density of 500 kg/m^3 or more.

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

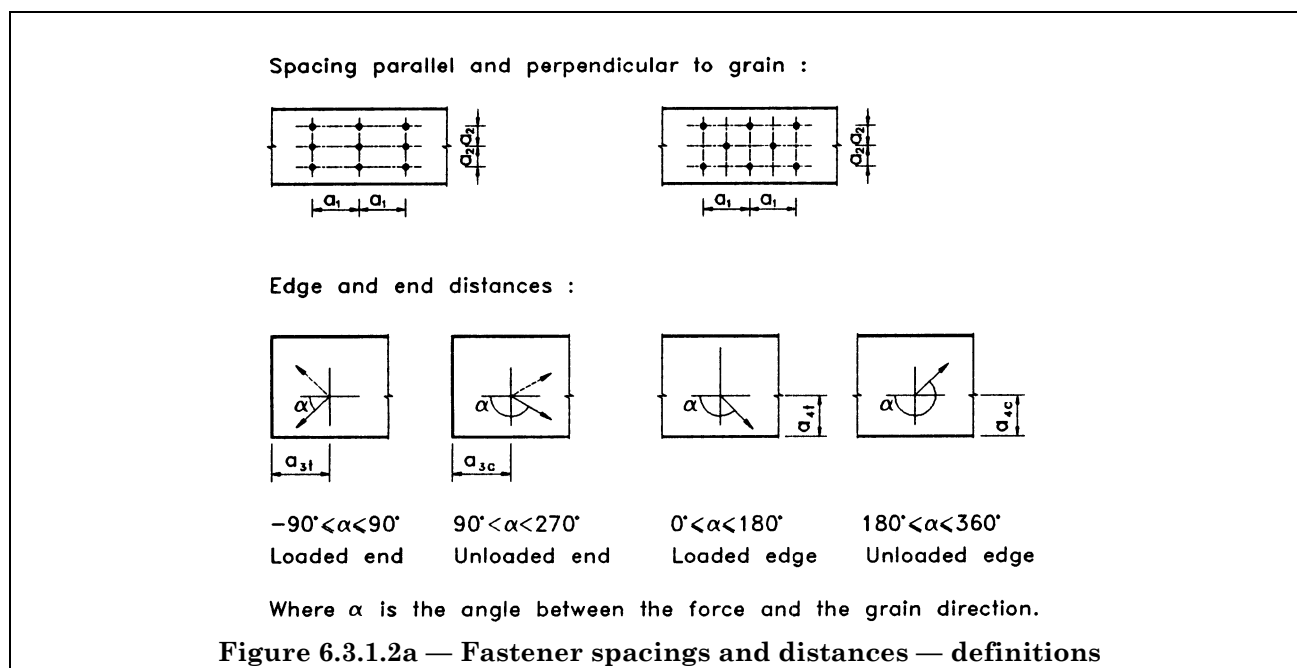
(5) For annular ringed shank and helically threaded nails the pointside penetration length should be at least $6d$.

(6) There should normally be at least two nails in a joint.

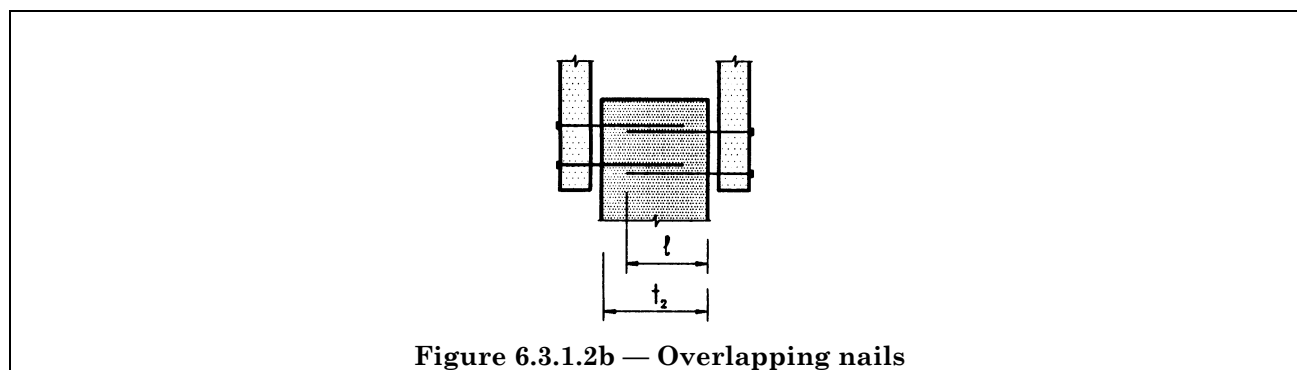
(7) Nails in end grain should normally not be considered capable of transmitting force. Where nails in end grain are used in secondary structures, e.g. for fascia boards nailed to rafters, the design value should be taken as $1/3$ of the value for normal nailing.

(8) Minimum spacings and distances are given in Table 6.3.1.2, with the definitions given in Figure 6.3.1.2a.

(9) For nails in predrilled holes the spacing a_1 may be reduced to a minimum of $4d$, if the embedding strength is reduced by the factor $\sqrt{a_1/4 + |3\cos\alpha|d}$.



(10) If $(t_2 - \ell)$ is greater than $4d$ (see Figure 6.3.1.2b) then nails without predrilled holes driven from two sides may overlap in the middle member.



(11) For nails without predrilled holes the timber members should have a minimum thickness of t , where

$$t = \max. \begin{cases} 7d \\ (13d - 30)\rho_k/400 \end{cases} \quad \begin{matrix} (6.3.1.2e) \\ (6.3.1.2f) \end{matrix}$$

with ρ_k in kg/m^3 and d in mm.

Table 6.3.1.2 — Minimum nail spacings and distances — values

Spacings and distances (see Figure 6.3.1.2a)	Without predrilled holes		Predrilled holes
	$\rho_k \leq 420 \text{ kg/m}^3$ $420 < \rho_k < 500 \text{ kg/m}^3$		
a ₁	d < 5 mm: (5 + 5 cos α)d		(4 + 3 cos α)d ^a
	(7 + 8 cos α)d		
	d ≥ 5 mm: (5 + 7 cos α)d		
a ₂	5d	5d	(3 + sin α)d
a _{3t} (loaded end)	(10 + 5cos α)d	(15 + 5cos α)d	(7 + 5cos α)d
a _{3c} (unloaded end)	10d	15d	7d
a _{4,t} (loaded edge)	(5 + 5sin α)d	(7 + 5sin α) d	(3 + 4sin α)d
a _{4,c} (unloaded edge)	5d	7d	3d
^a The minimum spacing a ₁ may be further reduced to 4d if the embedding strength f _{h,k} is reduced by the factor $\sqrt{a_1/(4 + 3 \cos \alpha)}$ d			

^a The minimum spacing a_1 may be further reduced to 4d if the embedding strength $f_{h,k}$ is reduced by the factor $\sqrt{a_1/(4 + 3 |\cos \alpha|)d}$

6.3.1.3 Nailed panel-to-timber joints

(1) The rules for timber-to-timber joints apply. Design values of the panel embedding strengths should be calculated as shown in 6.2.1(2).

(2) For plywood the following values of characteristic embedding strengths should be used:

$$f_{h,k} = 0.11 \rho_k d^{-0.3} \text{ N/mm}^2 \quad (6.3.1.3a)$$

with ρ_k in kg/m^3 and d in mm.

(3) For hardboard the following values of characteristic embedding strengths should be used:

$$f_{h,k} = 30 d^{-0.3} t^{0.6} \text{ N/mm}^2 \quad (6.3.1.3b)$$

with d and t in mm (t = panel thickness).

(4) The rules apply to ordinary nails with heads which have a diameter of at least 2d. For smaller heads the design load-carrying capacity should be reduced; for pins and oval headed nails, for example, the design load-carrying capacity in particleboards and fibreboards should be reduced by half.

(5) Minimum nail spacings for plywood in plywood-to-timber joints are those given in Table 6.3.1.2, multiplied by a factor of 0.85.

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

6.3.1.4 Nailed steel-to-timber joints

(1) The rules in 6.2.2 apply.

(2) Minimum nail spacings are those given in Table 6.3.1.2, multiplied by a factor of 0.7.

6.3.2 Axially loaded nails

P(1) Axially loaded smooth nails shall not be used for permanent and long-term load.

(2) The design withdrawal capacity of nails for nailing perpendicular to the grain [as in Figure 6.3.2(a)] and for slant nailing [as in Figure 6.3.2(b)] should be taken as the smallest of the values according to formula 6.3.2a (corresponding to withdrawal of the nail in the member receiving the point), and formulae 6.3.2b and c (corresponding to the head being pulled through). For smooth nails with a head diameter of at least 2d, formula 6.3.2b may be disregarded.

$$R_1 = \min \begin{cases} f_{1,d} l & \text{for all nails} & (6.3.2a) \\ f_{1,d} d h + f_{2,d} d^2 & \text{for smooth nails} & (6.3.2b) \\ f_{2,d} d^2 & \text{for annular ringed shank and threaded nails} & (6.3.2c) \end{cases}$$

The pointside penetration l should as a minimum be taken as 12d for smooth nails and as 8d for other nails.

(3) The parameters f_1 and f_2 depend, among other things, on the type of nail, timber species and grade (especially density) and should be determined by tests in accordance with the relevant European test standards unless specified in the following clause.

(4) The design values of the parameters f_1 and f_2 should be calculated as shown in 6.2.1(2).

(5) For smooth round nails the following characteristics values should be used:

$$f_{1,k} = (18 \times 10^{-6}) \rho_k^2 \text{ N/mm}^2 \quad (6.3.2d)$$

$$f_{2,k} = (300 \times 10^{-6}) \rho_k^2 \text{ N/mm}^2 \quad (6.3.2e)$$

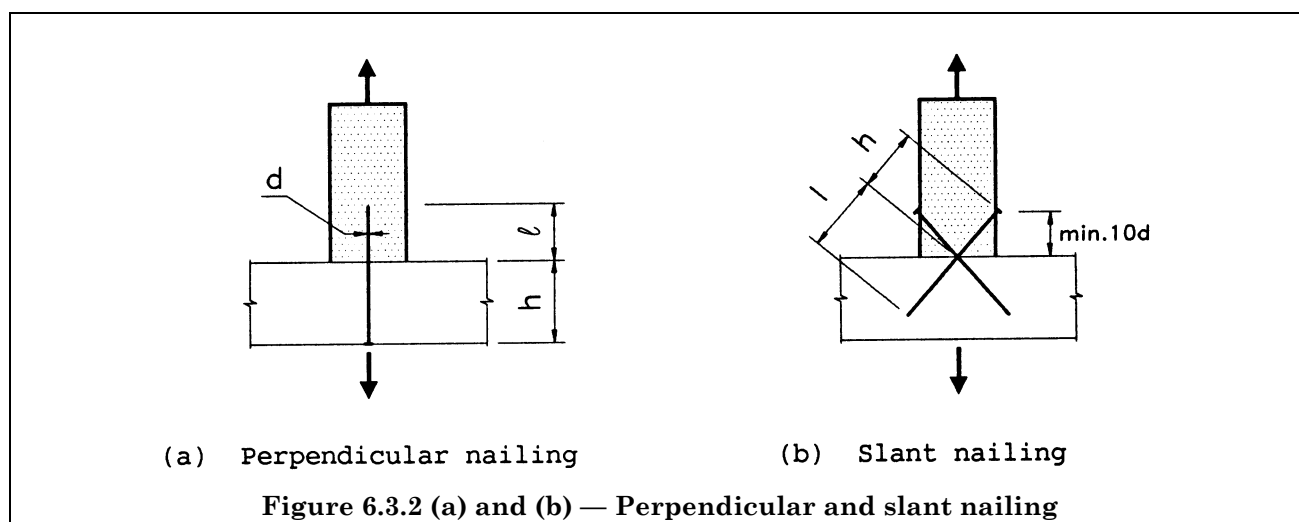
with ρ_k in kg/m^3 .

(6) For structural timber which is installed at or near fibre saturation point, and which is likely to dry out under load, the values of $f_{1,k}$ and $f_{2,k}$ should be multiplied by 2/3.

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

(8) For annular ring shanked and helically threaded nails only the threaded part should be considered capable of transmitting axial load.

(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 $10d$ [see Figure 6.3.2(b)].



6.3.3 Combined laterally and axially loaded nails

(1) For joints with a combination of axial load (F_{ax}) and lateral load (F_{la}) the following conditions should be satisfied:

for smooth nails

$$\frac{F_{ax,d}}{R_{ax,d}} + \frac{F_{la,d}}{R_{la,d}} \leq 1 \quad (6.3.3a)$$

for annular ringed shank and helically threaded nails

$$\left(\frac{F_{ax,d}}{R_{ax,d}} \right)^2 + \left(\frac{F_{la,d}}{R_{la,d}} \right)^2 \leq 1 \quad (6.3.3b)$$

where $R_{ax,d}$ and $R_{la,d}$ are the design load-carrying capacities of the joint loaded with axial load or lateral load alone.

6.4 Stapled joints

(1) The rules for nailed joints apply.

(2) The lateral design load-carrying capacity 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° .

(3) If the angle between the crown and the direction of the grain under the crown is equal to or less than 30° , then the lateral design load-carrying capacity should be multiplied by a factor of 0.7.

6.5 Bolted joints

6.5.1 Laterally loaded bolts

6.5.1.1 General

(1) The rules given in 6.2 apply.

6.5.1.2 Bolted timber-to-timber joints

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

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \quad (6.5.1.2a)$$

$$f_{h,0,k} = 0,082 (1 - 0,01d) \rho_k \text{ N/mm}^2 \quad (6.5.1.2b)$$

$$k_{90} = 1,35 + 0,015d \text{ for softwoods} \quad (6.5.1.2c)$$

$$k_{90} = 0,90 + 0,015d \text{ for hardwoods} \quad (6.5.1.2d)$$

with ρ_k in kg/m^3 and d in mm.

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

$$M_{y,k} = 0,8f_{u,k}d^3/6 \quad (6.5.1.2e)$$

where $f_{u,k}$ is the characteristic tensile strength.

(3) For more than 6 bolts in line with the load direction, the load carrying capacity of the extra bolts should be reduced by 1/3, i.e. for n bolts the effective number n_{ef} is

$$n_{\text{ef}} = 6 + 2(n - 6)/3 \quad (6.5.1.2f)$$

(4) Minimum spacings and distances are given in Table 6.5.1.2. The symbols are as defined in Figure 6.3.1.2a.

Table 6.5.1.2 — Minimum spacings and distances for bolts

a_1	Parallel to the grain	$(4 + 3 \cos \alpha)d^a$
a_2	Perpendicular to the grain	$4d$
$a_{3,t}$	$-90^\circ \leq \alpha \leq 90^\circ$	$7d$ (but not less than 80 mm)
$a_{3,c}$	$150^\circ \leq \alpha \leq 210^\circ$	$4d$
	$90^\circ \leq \alpha \leq 150^\circ$	$(1 + 6 \sin \alpha)d$ (but not less than $4d$)
	$210^\circ < \alpha < 270^\circ$	
$a_{4,t}$	$0^\circ \leq \alpha \leq 180^\circ$	$(2 + 2\sin \alpha)d$ (but not less than $3d$)
$a_{4,c}$	all other values of α	$3d$
^a The minimum spacing a_1 may be further reduced to $4d$ if the embedding strength $f_{h,0,k}$ is reduced by the factor $\sqrt{a_1/(4 + 3 \cos \alpha)d}$.		

6.5.1.3 Bolted panel-to-timber joints

(1) The rules for timber-to-timber joints apply. Design values of the panel embedding strengths should be calculated as shown in 6.2.1(2).

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

$$f_{h,k} = 0,11(1 - 0,01d) \rho_k \text{ N/mm}^2 \quad (6.5.1.3)$$

with ρ_k in kg/m³ and d in mm.

6.5.1.4 Bolted steel-to-timber joints

(1) The rules given in 6.2.2 and 6.5.1.1 apply.

6.5.2 Axially loaded bolts

P(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 $1,8f_{c,90,d}$.

6.6 Dowelled joints

(1) The rules for laterally loaded bolts apply, with the exception of 6.5.1.2(4).

(2) Minimum spacings and distances are given in Table 6.6a. The symbols are defined in Figure 6.3.1.2a

Table 6.6a — Minimum spacings and distances for dowels

a_1	Parallel to the grain	$(3 + 4 \cos \alpha)d^a$
a_2	Perpendicular to the grain	3d
$a_{3,t}$	$-90^\circ \leq \alpha \leq 90^\circ$	7d (but not less than 80 mm)
$a_{3,c}$	$150^\circ < \alpha < 210^\circ$	3d
	$90^\circ < \alpha < 150^\circ$	$a_{3,t} \sin \alpha $ (but not less than 3d)
	$210^\circ < \alpha < 270^\circ$	
$a_{4,t}$	$0^\circ \leq \alpha \leq 180^\circ$	$(2 + 2\sin \alpha)d$ (but not less than 3d)
$a_{4,c}$	all other values of α	3d

^a The minimum spacing may be further reduced to 4d if the embedding strength $f_{h,0,k}$ is reduced by the factor $\sqrt{a_1/(4 + 3|\cos \alpha|)}$.

6.7 Screwed joints

6.7.1 Laterally loaded screws

(1) For screws with a diameter less than 8 mm the rules in 6.3.1 apply.

For screws with a diameter equal to or greater than 8 mm the rules in 6.5.1 apply.

In the relevant formulae d should be taken as the diameter in mm of the screw measured on the smooth shank. To calculate the value of $M_{y,k}$ an effective diameter of $d_{ef} = 0,9d$ should be used, provided that the root diameter of the screw is not less than 0,7d.

If the length of the smooth shank in the pointside member is not less than 4d, the shank diameter may be used to calculate the value of $M_{y,k}$.

(2) It is assumed that:

- screws are turned into pre-drilled holes (see section 7.4)
- the length of the smooth shank is greater than or equal to the thickness of the member under the screw head.

(3) The penetration depth of the screw (i.e. the length in the member receiving the point), should be at least 4d.

6.7.2 Axially loaded screws

(1) The design withdrawal capacity of screws driven at right angles to the grain should be taken as:

$$R_d = f_{3,d}(l_{ef} - d) \text{ N} \quad (6.7.2a)$$

where the symbols are defined as follows:

- $f_{3,d}$ design withdrawal parameter
- l_{ef} threaded length in mm in the member receiving the screw

d diameter in mm measured on the smooth shank

The design withdrawal parameter $f_{3,d}$ should be calculated from the characteristic withdrawal parameter $f_{3,k}$ as shown in 6.2.1(2).

The characteristic value of $f_{3,k}$ should be taken as

$$f_{3,k} = (1,5 + 0,6d) \sqrt{\rho_k} \quad (6.7.2b)$$

with ρ_k in kg/m^3

The minimum distances and penetration length should be as given for laterally loaded screws.

6.7.3 Combined laterally and axially loaded screws

(1) The condition given in equation (6.3.3b) should be satisfied.

6.8 Joints made with punched metal plate fasteners

(1) For joints made with punched metal plate fasteners the rules given in Annex D apply.

7 Structural detailing and control

7.1 General

P(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.

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

7.2 Materials

P(1) The deviation from straightness measured midway between the supports shall for columns and beams where lateral instability can occur and members in frames be limited to 1/500 of the length for glued laminated members and to 1/300 of the length for structural timber²⁵⁾.

(2) Timber and wood-based components and structural elements should not be unnecessarily exposed to climatic conditions more severe than those to be encountered in the finished structure.

(3) Before 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.

7.3 Glued joints

(1) Where bond strength is a requirement for ultimate limit state design, the manufacturer of joints should be subject to a 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 a joint should be restricted for the necessary time.

7.4 Joints with mechanical fasteners

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

(2) 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.

²⁵⁾ 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.

- (3) Unless otherwise stated slant nailing should be carried out in conformity with Figure 6.3.2(b).
- (4) Bolt holes may have a diameter not more than 1 mm larger than the bolt.
- (5) Washers with a side length or a diameter of at least $3d$ and a thickness of at least $0,3d$ (d is the bolt diameter) should be used under the head and nut. Washers should have a full bearing area.
- (6) Bolts and 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 if this is necessary to ensure the load-carrying capacity or stiffness of the structure.
- (7) The minimum dowel diameter is 6 mm. The tolerances on the dowel diameter are $-0/+0,1$ mm and the pre-bored holes in the timber members should have a diameter not greater than the dowel.
- (8) The diameter of pre-drilled holes for nails should not exceed $0,8d$.
- (9) Screws with a diameter greater than 5 mm should be turned into holes which are pre-drilled, as follows:
 - the lead hole for the shank should have the same diameter as the shank and the same depth as the length of the unthreaded shank.
 - the lead hole for the threaded portion should have a diameter of about 70 per cent of the shank diameter.

7.5 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.

7.6 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.

7.7 Control

7.7.1 General

- (1) There should be a control plan comprising:
 - production and workmanship control off and on site
 - control after completion of the structure.

7.7.2 Production and workmanship control

- (1) This control should 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.

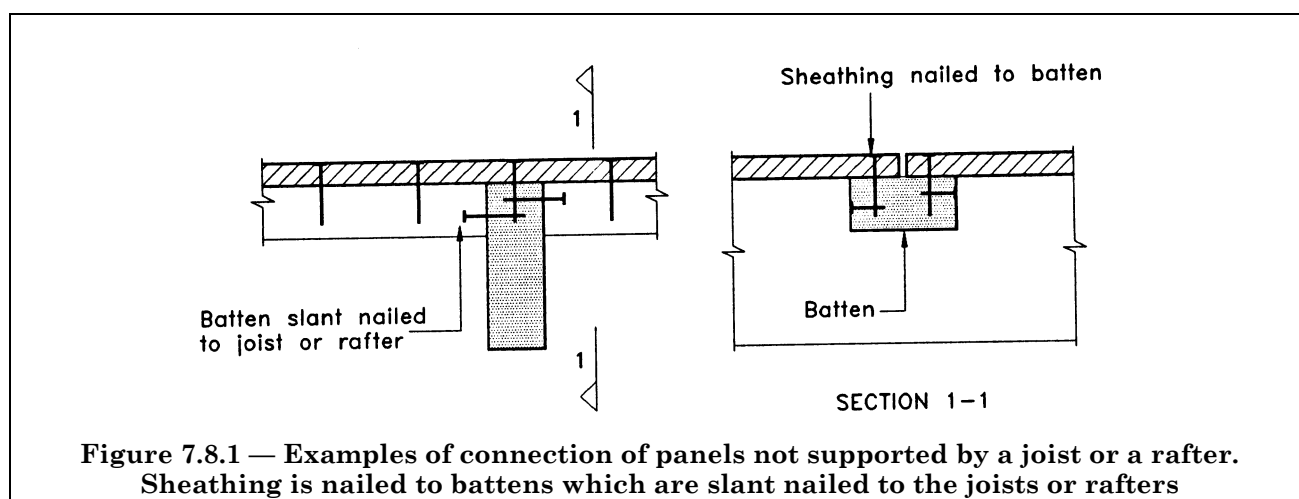
7.7.3 Controls after completion of the structure

- (1) A control programme should 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.
- (2) All the information required for the utilisation in service and the maintenance of a structure should be made available to the person or authority who undertakes responsibility for the finished structure.

7.8 Special rules for diaphragm structures

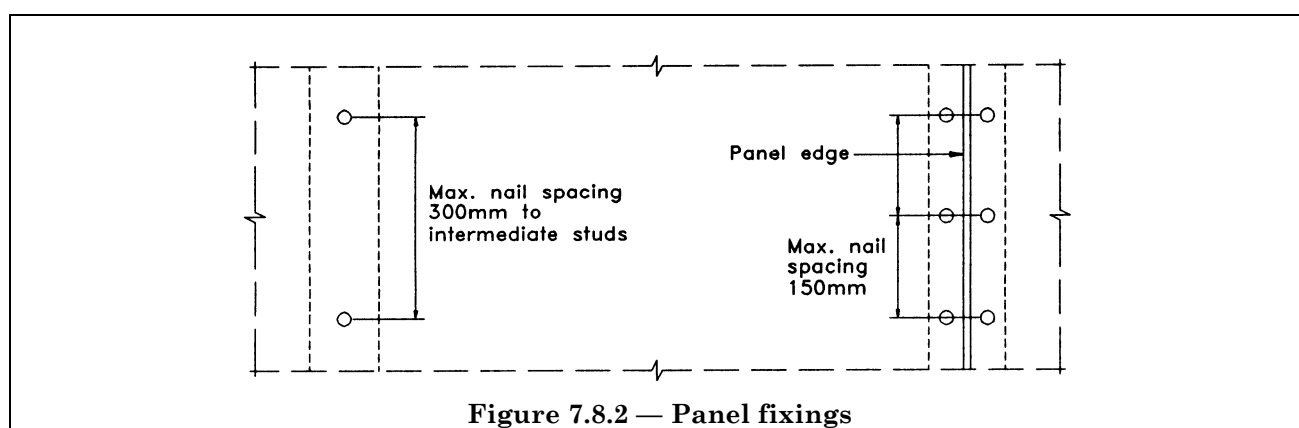
7.8.1 Floor and roof diaphragms

- (1) The simplified analysis given in 5.4.2 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 7.8.1. Annular ringed-shank or threaded nails, or screws, should be used, with a maximum spacing along the panel edges of 150 mm. Elsewhere the maximum spacing should be 300 mm.



7.8.2 Wall diaphragms

- (1) The maximum fastener spacing along the panel edges should be taken as 150 mm for nails and 200 mm for screws. Elsewhere the maximum spacing should be taken as 300 mm.



7.9 Special rules for trusses with punched metal plate fasteners

7.9.1 Fabrication

- (1) Trusses should be fabricated in accordance with prEN 1059.

7.9.2 Erection

- (1) Trusses should be checked for straightness and vertical alignment prior to fixing the permanent bracing.
- (2) When trussed rafters are fabricated, the members should be free from distortion within the limits given in prEN 1059. 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 trussed rafter may be considered satisfactory for use.
- (3) After erection, a maximum bow of 10 mm may be permitted in any trussed rafter member provided it is adequately secured in the completed roof to prevent the bow from increasing.
- (4) The maximum deviation from the true vertical alignment should not exceed $10 + 5 (H - 1)$ mm, with a maximum value of 25 mm, where H is the overall rise of the trussed rafter in m.

Annex A (informative)

Determination of 5-percentile characteristic values from test results and acceptance criteria for a sample

A.1 Scope

(1) This annex gives a method for calculating the 5-percentile characteristic value from the test results for a population, and a method to estimate whether the 5-percentile value for a sample drawn from the production is below a declared value.

(2) The method should not be used in cases covered by other European standards, or where other assumptions than those set out below can be shown to be more appropriate.

A.2 Determination of the 5-percentile characteristic value

A.2.1 Requirements

(1) The 5-percentile value shall be estimated as the lower endpoint in the one-sided 84,1 % confidence interval assuming a log-normal distribution. The coefficient of variation shall not be taken as less than 0,10.

(2) The sample size n shall not be less than 30.

A.2.2 Method

(1) Draw a sample of n test pieces from the population, and test the pieces in accordance with the appropriate standard for the property called x .

Determine the mean value $m\{x\}$ and the coefficient of variation $v\{x\}$. Estimate the characteristic value x_k as

$$x_k = k_1 m\{x\} \quad (\text{A2.2a})$$

where

$$k_1 = \exp \left[- (2,645 + 1/\sqrt{n}) v\{x\} + 0,15 \right] \quad (\text{A2.2b})$$

The value of $v\{x\}$ shall not be taken less than 0,10.

Values of k_1 are given in Table A.2.

NOTE The value determined by (A2.2a and A2.2b) is the highest value the producer may declare as the characteristic value. If the product is subject to a quality control procedure involving testing and evaluation as described in A.3 it may be advisable to declare a lower value to avoid an unreasonable rejection rate.

Table A.2 — Factor k_1

Coefficient of variation $v\{x\}$	Sample size n				
	30	40	50	100	∞
0,10	0,876	0,878	0,879	0,883	0,892
0,12	0,827	0,830	0,832	0,836	0,846
0,14	0,781	0,785	0,787	0,791	0,802
0,16	0,738	0,742	0,744	0,749	0,761
0,18	0,697	0,701	0,704	0,709	0,722
0,20	0,659	0,663	0,665	0,671	0,685
0,22	0,622	0,627	0,629	0,635	0,649
0,24	0,588	0,593	0,595	0,601	0,616
0,26	0,556	0,561	0,563	0,569	0,584
0,28	0,525	0,530	0,532	0,539	0,554
0,30	0,496	0,501	0,504	0,510	0,525

A.3 Acceptance criteria for a sample

A.3.1 Requirements

(1) The probability of accepting a sample with a 5-percentile value less than 95 % of the declared characteristic value f_k should be less than 15,9 % assuming a log-normal distribution. It is assumed that the value of the coefficient of variation is known, e.g., from a running production control. The coefficient of variation shall not be taken as less than 0,10.

A.3.2 Method

(1) Draw a sample of n test pieces from the batch, and test them in accordance with the appropriate standard for the property called x .

Calculate the mean value $m\{x\}$.

The sample shall be accepted if

$$m\{x\} \geq k_2 f_k$$

where

$$k_2 = \exp [(2,645 + 1/\sqrt{n}) v\{x\} - 0,1875]$$

Values of k_2 are given in Table A.3.

Table A.3 — Factor k_2

Coefficient of variation $v\{x\}$	Sample size n						
	3	5	10	20	50	100	∞
0,10	1,14	1,13	1,11	1,10	1,10	1,09	1,08
0,12	1,22	1,20	1,18	1,17	1,16	1,15	1,14
0,14	1,30	1,28	1,25	1,25	1,23	1,22	1,20
0,16	1,39	1,36	1,33	1,31	1,30	1,29	1,27
0,18	1,48	1,45	1,41	1,39	1,37	1,36	1,34
0,20	1,58	1,54	1,50	1,47	1,45	1,44	1,41
0,22	1,68	1,64	1,59	1,56	1,53	1,52	1,49
0,24	1,80	1,74	1,69	1,65	1,62	1,60	1,57
0,26	1,92	1,85	1,79	1,75	1,71	1,69	1,65
0,28	2,04	1,97	1,90	1,85	1,81	1,79	1,74
0,30	2,18	2,10	2,02	1,96	1,91	1,89	1,84

Annex B (informative)

Mechanically jointed beams

B.1 General

B.1.1 Cross sections

(1) The cross-sections shown in Figure B.1.1 are considered.

B.1.2 Structures and assumptions

(1) The design method is based on the theory of linear elasticity and the following assumptions:

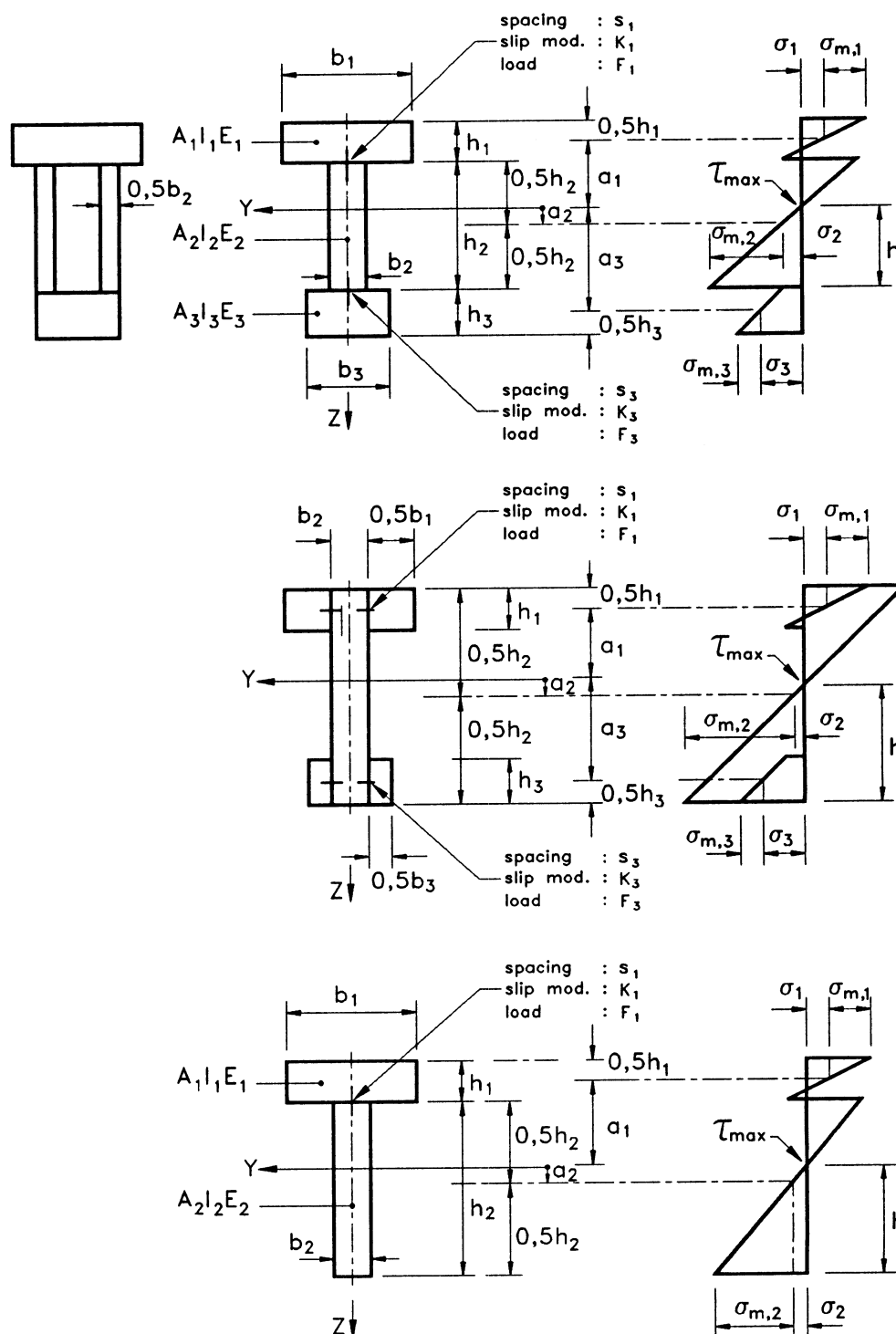
- the beams are simply supported with a span l . For continuous beams the formulae can be used with l equal to 0,8 times the relevant span: for cantilevered beams with l equal to twice the cantilever
- the individual parts (of wood, wood-based panels) are either full length or made with glued end joints
- the individual parts are connected to each other by mechanical fasteners with a slip modulus K
- the spacing s between the fasteners is constant or varies uniformly according to the shear force between s_{\min} and s_{\max} with $s_{\max} \leq 4 s_{\min}$
- the load is acting in the z -direction giving a moment $M = M(x)$ varying sinusoidally or parabolically and a shear force $V = V(x)$.

B.1.3 Spacings

(1) Where a flange consists of two parts jointed to a web or where a web consists of two parts (as in a box beam), the spacing S_i is determined by the sum of the fasteners per unit length in the two jointing planes.

B.1.4 Deflections resulting from bending moments

(1) Deflections are calculated by using an effective bending stiffness $(EI)_{\text{ef}}$ determined in accordance with B.2.



B.2 Effective bending stiffness

(1) The effective bending stiffness should be taken as

$$(EI)_{ef} = \sum_{i=1}^3 (E_i I_i + \gamma_i E_i A_i a_i^2) \quad (B2a)$$

with mean values of E, and where:

$$A_i = b_i h_i \quad (B2b)$$

$$I_i = b_i h_i^3 / 12 \quad (B2c)$$

$$\gamma_2 = 1 \quad (B2d)$$

$$\gamma_i = \left[1 + \pi^2 E_i A_i s_i / (K_i l^2) \right]^{-1} \text{ for } i=1 \text{ and } i=3 \quad (B2e)$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^3 \gamma_i E_i A_i} \quad (B2f)$$

For T-sections $h_3 = 0$

B.3 Normal stresses

(1) The normal stresses should be taken as:

$$\sigma_i = \gamma_i E_i a_i M / (EI)_{ef} \quad (B3a)$$

$$\sigma_{m,i} = 0,5 E_i h_i M / (EI)_{ef} \quad (B3b)$$

B.4 Maximum shear stress

(1) The maximum shear stresses occur where the normal stresses are zero. The maximum shear stress in part 2 of the cross-section should be taken as

$$\tau_{2,max} = (\gamma_3 E_3 A_3 a_3 + 0,5 E_2 b_2 h^2) V / (b_2 (EI)_{ef}) \quad (B4)$$

B.5 Fastener load

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

$$F_i = \gamma_i E_i A_i a_i s_i V / (EI)_{ef} \quad (B5)$$

with $i = 1$ and 3 , where $s_i = s_i(x)$ is the spacing of the fasteners as defined in **B.1.3**, 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 l
- the individual parts are full length
- the load is an axial force F_c acting in the geometric centre of gravity, (see however **C.2.4**).

C.1.2 Load carrying capacity

(1) For column deflection in the y-direction (see Figure C.3.1 and Figure C.4.1), the load-carrying capacity is equal to the sum of the load-carrying capacities of the individual members.

(2) For column deflection in the z-direction (see Figure C.3.1 and Figure C.4.1) it is required that:

$$\sigma_{c,0,d} \leq k_{c,f,c,0,d} \quad (C1.2a)$$

where

$$\sigma_{c,0,d} = F_{c,d}/A_{\text{tot}} \quad (\text{C1.2b})$$

A_{tot} is the total cross-sectional area

k_c is determined in accordance with clause 5.2.1 but with an effective slenderness ratio λ_{ef} determined in accordance with sections C.2 – C.4.

C.2 Mechanically jointed columns

C.2.1 Assumptions

(1) Built-up columns with the cross-sections shown in Annex B are considered. It is, however, assumed that

$$E_1 = E_2 = E_3 = E \quad (\text{C2.1})$$

where E_{mean} should be used.

C.2.2 Effective slenderness ratio

(1) The effective slenderness ratio should be taken as

$$\lambda_{\text{ef}} = 1 \sqrt{A_{\text{tot}}/I_{\text{ef}}} \quad (\text{C2.2a})$$

where

$$I_{\text{ef}} = (EI)_{\text{ef}}/E \quad (\text{C2.2b})$$

and $(EI)_{\text{ef}}$ is determined in accordance with Annex B.

C.2.3 Load on fasteners

(1) The load on a fastener should be determined in accordance with Annex B, (B.5), where:

$$V_d = \begin{cases} F_{c,d}/(120 k_c) & \text{for } \lambda_{\text{ef}} \leq 30 \\ F_{c,d}\lambda_{\text{ef}}/(3600 k_c) & \text{for } 30 < \lambda_{\text{ef}} \leq 60 \\ F_{c,d}/(60 k_c) & \text{for } 60 < \lambda_{\text{ef}} \end{cases} \quad (\text{C2.3a})$$

$$(\text{C2.3b})$$

$$(\text{C2.3c})$$

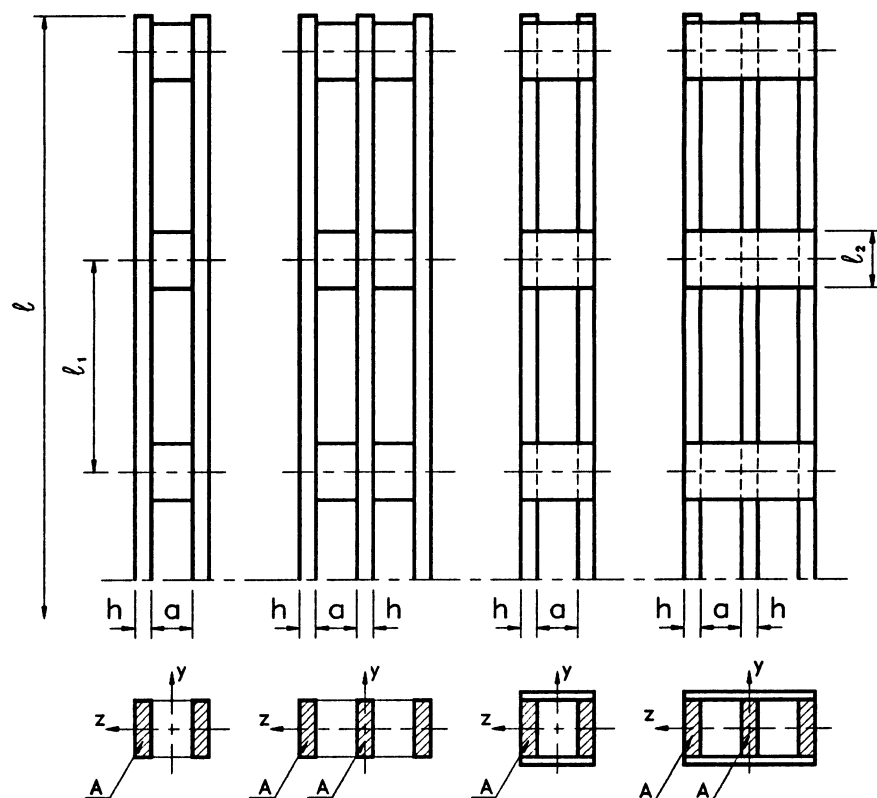
C.2.4 Combined loads

(1) In cases where small moments resulting from e.g. self weight are acting apart from axial load, 5.2.1(4) applies.

C.3 Spaced columns with packs or gussets

C.3.1 Assumptions

(1) Columns as shown in Figure C.3.1 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.



For columns with two shafts $A_{tot}=2A$ and $I_{tot}=b((2h+a)^3-a^3)/12$

For columns with three shafts $A_{tot}=3A$ and $I_{tot}=b((3h+2a)^3-(h+2a)^3+h^3)/12$

Figure C.3.1 — Spaced columns

(2) The following assumptions apply:

- the cross-section is composed of 2, 3 or 4 identical shafts
- the cross-sections are doubly symmetrical
- the number of free bays is at least 3, 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 3 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.3.3
- the pack length l_2 satisfies the condition: $l_2/a \geq 1,5$
- there are at least 4 nails or 2 bolts with connectors in each shear plane. For nailed joints there are at least 4 nails in a row at each end in the longitudinal direction of the column
- the length of the gussets satisfies the condition: $l_2/a \geq 2$
- the columns are subjected to concentric axial loads.

C.3.2 Axial load-carrying capacity

(1) For column deflection in the y -direction (see Figure C.3.1) the load-carrying capacity is equal to the sum of the load-carrying capacities of the individual members.

(2) For column deflection in the z-direction **C.1.2** applies with

$$\lambda_{\text{ef}} = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \quad (\text{C3.2a})$$

where

λ is the slenderness ratio for a solid column with the same length, the same area (A_{tot}) and the same second moment of area (I_{tot}), i.e.,

$$\lambda = 1 \sqrt{A_{\text{tot}} / I_{\text{tot}}} \quad (\text{C3.2b})$$

λ_1 is the slenderness ratio for the shafts. A minimum value of $\lambda_1 = 30$ should be used in (C3.2b).

$$\lambda_1 = \sqrt{12} l_1 / h \quad (\text{C3.2c})$$

n is the number of shafts

η is a factor given in Table C.3.2.

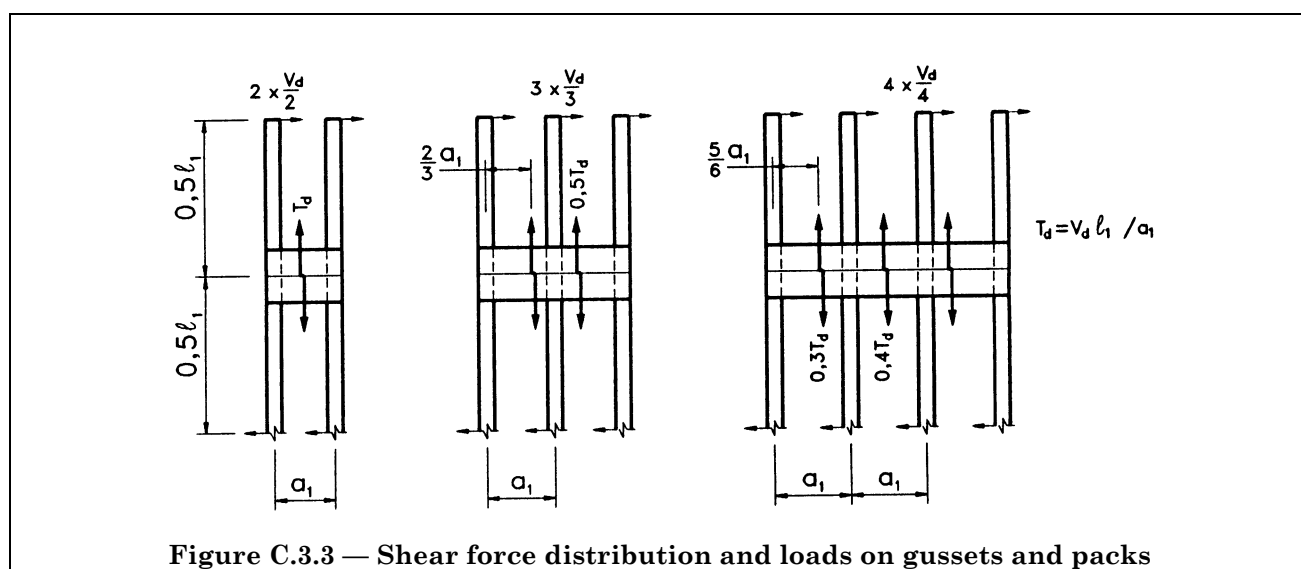
Table C.3.2 — The factor η

	packs			gussets	
	glued/nailed/bolted ^a			glued/nailed	
permanent/long-term loading	1	4	3,5	3	6
medium/short-term loading	1	3	2,5	2	4,5

^a with connectors

C.3.3 Load on fasteners gussets and packs

(1) The load on the fasteners gussets and packs should taken as shown in Figure C.3.3 with V_d according to section C.2.3.



C.4 Lattice columns with glued or nailed joints

C.4.1 Structures

(1) Lattice columns with N- or V-lattice and with glued or nailed joints are considered, see Figure C.4.1.

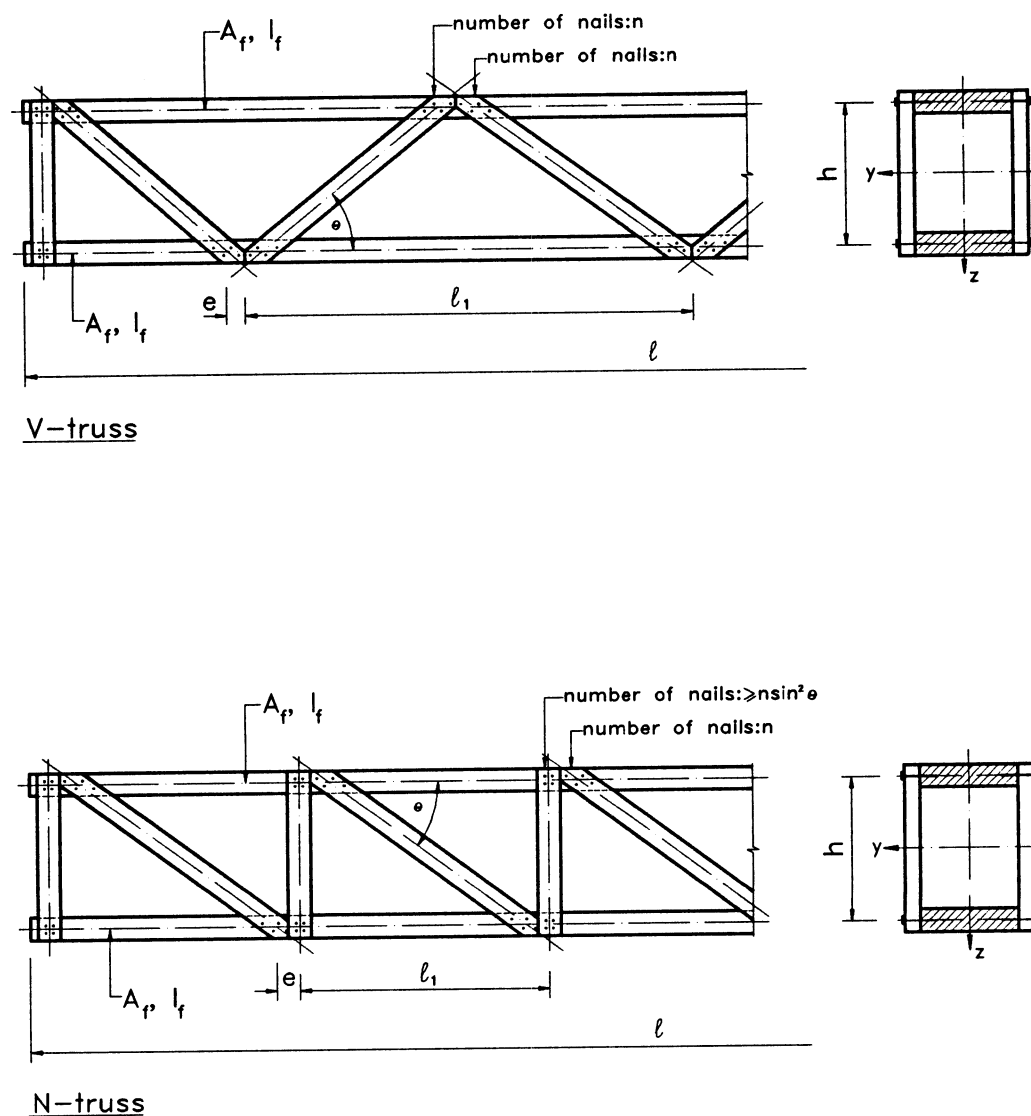


Figure C.4.1 — Lattice columns. The area of one flange is A_f and the second moment of area about its own axis of gravity is I_f

(2) The following assumptions apply:

- the structure is symmetrical about the y- and z-axes of the cross-section. The lattice of the two sides may be staggered by a length of $\ell_1/2$, where ℓ_1 is the node distance
- there are at least 3 bays
- in nailed structures there are at least 4 nails per shear plane in each diagonal at each nodal point
- each end is braced
- the slenderness ratio of the individual flange corresponding to the node length ℓ_1 is not greater than 60
- no local rupture occurs in the flanges corresponding to the column length ℓ_1
- the number of nails in the verticals (of an N-truss) is greater than $n \sin^2 \theta$, where n is the number of nails in the diagonals and θ is the inclination of the diagonals.

C.4.2 Load carrying capacity

(1) The load-carrying capacity corresponding to the deflection of the column in the y-direction is equal to the sum of the load-carrying capacities of the flanges for deflection.

(2) For column deflection in the z-direction **C.1.2** applies with:

$$\lambda_{ef} = \max \begin{cases} \lambda_{tot} \sqrt{1+\mu} \\ 1,05 \lambda_{tot} \end{cases} \quad (C4.2a)$$

(C4.2b)

where λ_{tot} is the slenderness ratio for a solid column with the same length, the same area and the same second moment of area, i.e.

$$\lambda_{tot} \approx \frac{2\ell}{h} \quad (C4.2c)$$

and μ takes the values given below.

(3) For glued V-trusses

$$\mu = 4 \frac{e^2 A_f}{I_f} \left(\frac{h}{\ell} \right)^2 \quad (C4.2d)$$

where e is defined in Figure C.4.1.

(4) 0 For glued N-trusses

$$\mu = \frac{e^2 A_f}{I_f} \left(\frac{h}{\ell} \right)^2 \quad (C4.2e)$$

where e is defined in Figure C.4.1.

(5) For nailed V-trusses

$$\mu = 25 \frac{h E A_f}{\ell^2 n K \sin 2\theta} \quad (C4.2f)$$

where n is the number of nails in a diagonal and K is the slip modulus of one nail. If a diagonal consists of two or more pieces, n is the sum of the nails (and not the number of nails per shear plane). E_{mean} should be used.

(6) For nailed N-trusses

$$\mu = 50 \frac{h E A_f}{\ell^2 n K \sin 2\theta} \quad (C4.2g)$$

where n is the number of nails in a diagonal and K is the slip modulus of one nail. If a diagonal consists of two or more pieces, n is the sum of the nails (and not the number of nails per shear plane). E_{mean} should be used.

C.4.3 Shear forces

C.2.3 applies.

Annex D (normative)

The design of trusses with punched metal plate fasteners

D.1 General

- (1) The requirements of 5.4.1.1 apply.
- (2) The method given in this annex may be applied to trusses with other fasteners of a similar form, such as nailed metal plates or plywood gussets.

D.2 Joints

- (1) Splice joints may be modelled as rotationally stiff if the actual rotation under load would have no significant effect upon member forces. This requirement is fulfilled by:
 - splice joints with a resistance which is at least equal to 1,5 times the combination of applied force and moment
 - splice joints with a resistance which corresponds at least to the combination of applied force and moment, provided that
 - the joint is 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 joints acted as pins.
- (2) The influence of slip in the joints should be modelled either as slip moduli, or as prescribed slip values which relate to the actual stress level in the joint.
- (3) Values of the instantaneous slip modulus K_{ser} , or the prescribed slip u_{ser} for the serviceability limit state should be determined by tests according to the method for determining k ($= K_{ser}$) given in EN 26891.
- (4) The instantaneous slip modulus for the ultimate limit state, K_u , is given by

$$K_u = 2K_{ser}/3 \quad (D2a)$$

- (5) The final slip modulus $K_{u,fin}$, is given by

$$K_{u,fin} = K_u/(1 + k_{def}) \quad (D2b)$$

- (6) The prescribed slip for the ultimate limit state, u_u , is given by

$$u_u = 2,0 u_{ser} \quad (D2c)$$

- (7) The final prescribed slip is given by

$$u_{u,fin} = u_u(1 + k_{def}) \quad (D.2d)$$

D.3 General analysis

- (1) The requirements of 5.4.1.2 apply.
- (2) For fully triangulated trusses where a small concentrated force (e.g. a man load) has a component perpendicular to the member of $< 1,5$ kN, and where $\sigma_{c,d} < 0,4 f_{c,d}$ and $\sigma_{t,d} < 0,4 f_{t,d}$ the requirements of 5.1.9 and 5.1.10 should be replaced by

$$\sigma_{m,d} \leq 0,75f_{m,d} \quad (D3)$$

D.4 Simplified analysis

- (1) The requirements of 5.4.1.3 apply.
- (2) The supports may be modelled as pinned if not less than half the width of the bearing is vertically below the eaves joint fastener, and the distance a_2 in Figure D.4 is not greater than $a_1/3$ or 100 mm, whichever is the greater.

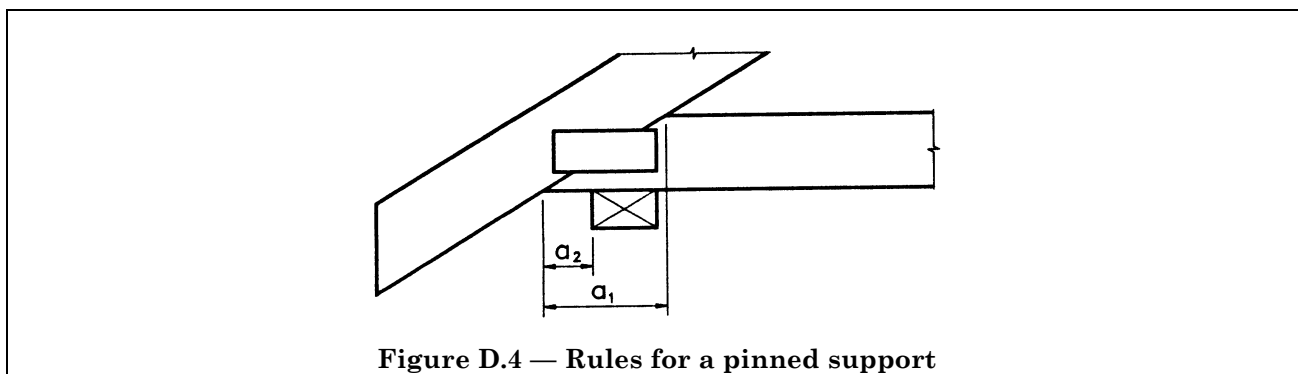


Figure D.4 — Rules for a pinned support

(3) For trusses which are loaded predominantly at the nodes, the sum of the combined bending and axial compression stress ratios given in equations 5.1.10a and b should be limited to 0,9.

D.5 Strength verification of members

(1) The requirements of Chapter 5 apply.

D.6 Punched metal plate fasteners

D.6.1 General

(1) The following rules apply only to plates with two orthogonal directions.

D.6.2 Plate geometry

(1) The geometry of the plate is given in Figure D.6.2. The symbols are defined as follows:

x-direction	main direction of plate
y-direction	perpendicular to the main direction
α	angle between the x-direction and the force F
β	angle between the grain direction and the force F
γ	angle between the x-direction and the joint line
A_{ef}	the effective area, that is, the 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
ℓ	length of the plate along the joint line

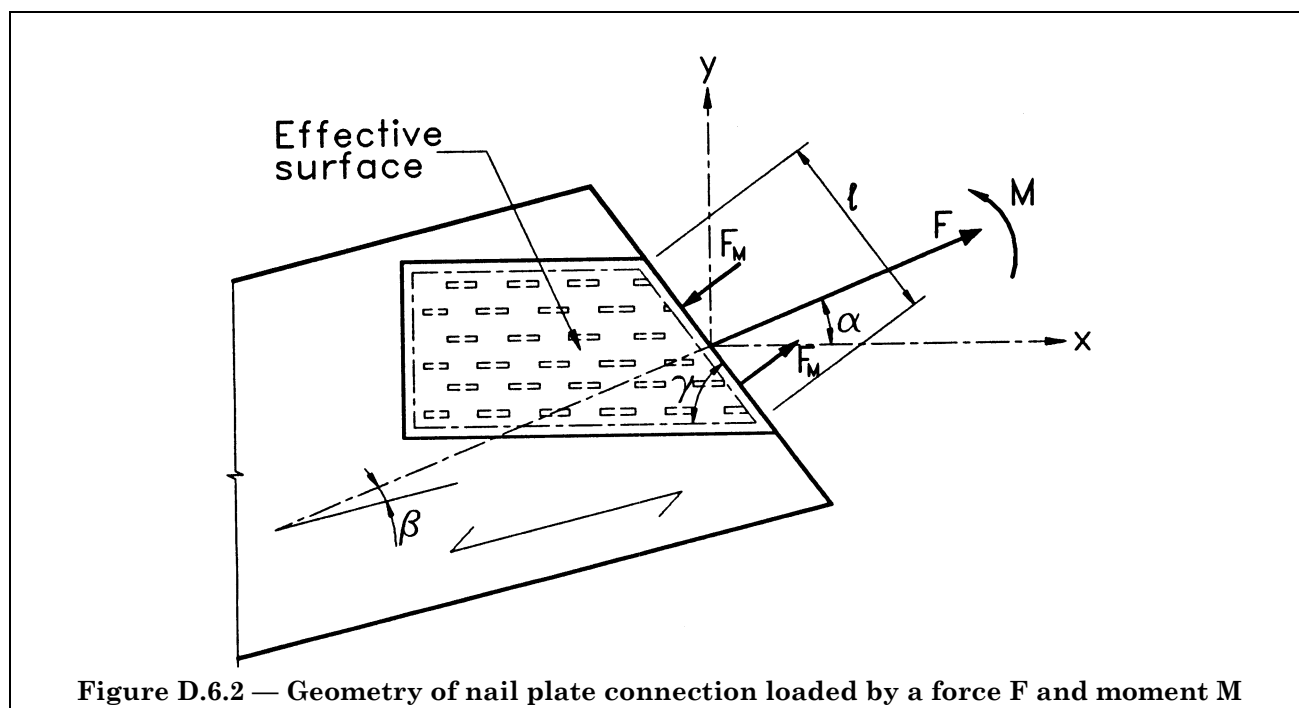


Figure D.6.2 — Geometry of nail plate connection loaded by a force F and moment M

D.6.3 Plate strength capacities

(1) The plate shall have approved characteristic values determined from the results of tests carried out in accordance with the methods described in prEN 1075 for the following properties:

$f_{a,0,0}$	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, α_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 and γ_m as 1,1.

D.6.4 Anchorage strengths

The design anchorage strength $f_{a,\alpha,\beta,d}$ should either be derived from tests or calculated from:

$$f_{a,\alpha,\beta,d} = \max \left\{ \begin{array}{l} f_{a,\alpha,0,d} - (f_{a,\alpha,0,d} - f_{a,90,90,d}) \beta/45^\circ \\ f_{a,0,0,d} - (f_{a,0,0,d} - f_{a,90,90,d}) \sin(\max(\alpha, \beta)), \end{array} \right. \quad \begin{array}{l} \text{(D6.4a)} \\ \text{(D6.4b)} \end{array}$$

when $\beta \leq 45^\circ$, or

$$f_{a,\alpha,\beta,d} = f_{a,0,0,d} - (f_{a,0,0,d} - f_{a,90,90,d}) \sin(\max(\alpha, \beta)), \quad \text{(D6.4c)}$$

when $45^\circ < \beta \leq 90^\circ$

The design anchorage strength in the grain direction is given by:

$$f_{a,\alpha,0,d} = \begin{cases} f_{a,0,0,d} + k_1 \alpha & \text{when } \alpha \leq \alpha_0 \\ f_{a,0,0,d} + k_1 \alpha_0 + k_2 (\alpha - \alpha_0) & \text{when } \alpha_0 < \alpha \leq 90^\circ \end{cases} \quad \begin{array}{l} \text{(D6.4d)} \\ \text{(D6.4e)} \end{array}$$

The constants k_1 , k_2 and α_0 should be determined by tests in accordance with prEN 1075 for the actual type of nail plate.

D.6.5 Joint strength verification

D.6.5.1 Plate anchorage capacity

(1) The anchorage stresses τ_F and τ_M are calculated from:

$$\tau_F = \frac{F_A}{A_{ef}} \quad (D6.5.1a)$$

$$\tau_M = \frac{M_A r_{max}}{I_p} \quad (D6.5.1b)$$

where the symbols are defined as follows:

F_A force acting on the plate at the centroid of the effective area

M_A moment acting on the plate

I_p polar moment of inertia of the effective area

r_{max} the distance from the centroid to the furthest point of the effective area.

(2) Contact pressure between timber members may be taken into account to reduce the value of F_A in compression provided that the gap between the members has an average value which is not greater than 1 mm, and a maximum value of 2 mm. In such cases the joint should be designed for a minimum compression force of $F_A/2$.

(3) The following conditions should be satisfied:

$$\tau_{F,d} \leq f_{a,\alpha,\beta,d} \quad (D6.5.1c)$$

$$\tau_{M,d} \leq 2f_{a,90,90,d} \quad (D6.5.1d)$$

$$\tau_{F,d} + \tau_{M,d} \leq 1,5f_{a,0,0,d} \quad (D6.5.1e)$$

D.6.5.2 Plate capacity

(1) For a connection with one straight joint the forces in the two main directions are determined from the following formulae. A positive value signifies a tension force, a negative value a compression force.

$$F_x = F \cos\alpha \pm 2F_M \sin\gamma \quad (D6.5.2a)$$

$$F_y = F \sin\alpha \pm 2F_M \cos\gamma \quad (D6.5.2b)$$

where the symbols are defined as follows:—

F is the force in the joint

F_M is the force from the moment M in the joint ($F_M = 2M/\ell$)

(2) The following condition should be satisfied:

$$\left(\frac{F_{x,d}}{R_{x,d}} \right)^2 + \left(\frac{F_{y,d}}{R_{y,d}} \right)^2 \leq 1 \quad (D.6.5.2c)$$

where $F_{x,d}$ and $F_{y,d}$ are the design values of the forces in the x- and y- directions, and $R_{x,d}$ and $R_{y,d}$ are the design values of the plate capacity in the x- and y- directions. The latter are determined as the maximum of the capacities at sections parallel with or perpendicular to the main axes.

$$R_{x,d} = \max \begin{cases} f_{ax,0,d} \ell \sin\gamma \\ f_{vx,0,d} \ell \cos\gamma \end{cases} : f_{ax,0,d} = \begin{cases} f_{t,0,d} & \text{if tension} \\ f_{c,0,d} & \text{if compression} \end{cases} \quad (D.6.5.2d)$$

$$R_{y,d} = \max \begin{cases} f_{ax,90,d} \ell \cos \gamma \\ f_{v,90,d} \ell \sin \gamma \end{cases} : f_{ax,90,d} = \begin{cases} f_{t,90,d} & \text{if tension} \\ f_{c,90,d} & \text{if compression} \end{cases} \quad (\text{D.6.5.2e})$$

(3) If the plate covers several joints, then the forces in each straight part of the joint line should be determined so that equilibrium is fulfilled and the condition in expressions (D.6.5.2c) is satisfied in each straight part.

(4) All critical sections should be considered.

D.6.5.3 *Minimum anchorage requirements*

(1) All joints should be capable of transferring a force $F_{r,d}$ acting in any direction. $F_{r,d}$ shall be assumed to be a short-term force, acting on timber in service class 2 with the value

$$F_{r,d} = 1,0 + 0,1L \text{ kN} \quad (\text{D.6.5.3})$$

where L is the length of the truss in metres.

(2) The minimum overlap of the punched metal plate and the timber should be at least equal to 40 mm or $h/3$, where h is the height of the timber member.

(3) Nail plates in chord splices should cover at least $\frac{2}{3}$ of the timber width.

National annex NA (informative)

Committees responsible

The preparation of the National Application Document for use in the UK with ENV 1995-1-1:1993 was entrusted by Technical Committee B/525, Building and civil engineering structures, to Subcommittee B/525/5, Structural use of timber, upon which the following bodies were represented.

British Woodworking Federation

Department of the Environment (Building Research Establishment)

Department of the Environment (Construction Directorate)

Health and Safety Executive

Institution of Civil Engineers

Institution of Structural Engineers

National House-building Council

Timber Research and Development Association

Timber Trade Federation

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