CONSEIL INTERNATIONAL DU BÂTIMENT ★ INTERNATIONAL COUNCIL FOR BUILDING RESEARCH STUDIES AND DOCUMENTATION

WORKING GROUP W18
TIMBER STRUCTURES

CIB
STRUCTURAL TIMBER DESIGN CODE

Fifth edition, August 1980

FOREWORD

A first draft of the CIB-Structural Timber Design Code was discussed at CIB-W18 meetings in June 1976, February 1977, and October 1977. Since those meetings took place three further drafts have been prepared and discussed at the regular meetings of CIB-W18.

This present version of the Code was prepared by a Code Drafting Sub-committee consisting of W18 members: L. G. Booth, W. T. Curry, J. Ehlbeck, J. Kuipers, H. J. Larsen, J. G. Sunley, and J. R. Tory, on the basis of comments from both the meetings of CIB-W18 and ISO/TC 165 - Timber Structures to whose members an earlier draft has also been circulated.

The Code contains rules peculiar to the design of timber and wood-based structures, and recommendations which define their validity. It is equally applicable to both the deterministic and the partial factor methods of design, provided material properties are derived from characteristic values and suitable safety factors for strength and stiffness are available for the design calculations. It does not contain safety factors, partial factors or loads since it is recognized that these are the responsibility of national public authorities.

To adequately cover the material properties of the extremely varied worldwide range of timber species a system of strength classes is defined in the Code. It is not intended that this system should preclude the use of properties for individual species and grades; these may be included in annexes to the Code.

It is the intention of CIB-W18 that their Code should be presented to ISO/TC 165 - Timber Structures as the basis for a Draft International Standard. It may also be submitted to the European Economic Community as a European Structural Timber Code.

RELATED DOCUMENTS

The Code makes reference to several documents produced by a sub-committee RILEM-3TT/CIB-W18 concerned with the testing of wood-based materials and related components. These are:

Testing Methods for Timber in Structural Sizes.

Testing Methods for Plywood in Structural Grades for Use in Load-bearing Structures.

Testing Methods for Joints with Mechanical Fasteners in Load-bearing Timber Structures.

Testing Methods for Joints with Mechanical Fasteners in Load-bearing Timber Structures.
Annex A1: Punched Metal Plate Fasteners.

Other documents relating to the sampling of test specimens and the analysis of test data to produce characteristic valurs will be prepared by CIB-W18.

BACKGROUND PAPERS

Technical papers prepared for and discussed at meetings of CIB-W18 form the background for the CIB-Structural Timber Design Code. A complete list of these technical papers is given in Annex 01 while notes on the derivation and interpretation of some particular clauses in the Code are contained in Annex 02.

ANNEXES

Annexes to the Code, containing background information, individual species and grade stresses and more specialized design procedures are published under separate cover.

LIST OF CONTENTS

1. GENERAL

1.1 Scope

The primary purpose of this code is to provide an agreed background for the international bodies and national committees responsible for formulating timber codes, to ensure a reasonable and consistent quality of timber structures.

The code relates to the structural use of timber and wood based materials and is intended for use in the design, execution and appraisal of structural elements made from timber or wood products and of structures containing such elements.

It is based on the principles of structural mechanics, engineering design, and experimental data, interpreted statistically as far as possible.

Deviations from the requirements of this code and the use of materials and methods of design or construction of wood structures not covered by this code are permitted when the validity is substantiated by analytical and engineering principles or reliable test data, or both.

1.2 Conditions for the validity of this document

Safety and serviceability are not simply functions of design, but depend also on the care and skill of all personnel involved in the construction process, and on the proper use and maintenance of the structure. Essential requirements are therefore that

- projects are carried out by qualified engineers,
- the construction is carried out by personnel having both the required skill and experience,
- the required supervision is always available,
- the structure, by design or the use of suitable materials or by impregnation, is protected against attack by fungi, insects, shipworm, gribble, etc., and its integrity is ensured by correct maintenance,
- the actual conditions of use of the structure during its life do not depart significantly from those specified during the design stage.

1.3 Units

The units used are generally in accordance with the »International System of Units, SI» and »Rules for the Use of the International System of Units» established by ISO and prepared by ISO/TC98/SC2.

Exceptions are the units for time,temperature and plane angle. In accordance with common and well established practice the Celsius scale is used rather than the Kelvin scale for thermodynamic temperature; degrees are used rather than radians as the non-dimensional units of plane angle; and hours, days, weeks, months and years are accepted as derived units of time.

The following basic units and derived units are used for structural timber design calculations:

Table 1.3 Units for structural timber design

Physical quantity	Unit	Abbreviation (and derivation)
Length	Metre	m
Mass	Kilogram	kg
Temperature	Degree Celsius	°C
Time	Second	s
Plane angle	Degree	° ($1° = \frac{\pi}{180}$ radian)
Force	Newton	N ($1\,N = 1\,kgm/s^2$)
Stress, pressure Elastic moduli	Pascal	Pa ($1\,Pa = 1N/m^2$, $1MPa = 1N/mm^2$)

Only multiples of $10^{\pm 3}$; e.g. MN, kN, N are used.

1.4 Notation

The notation used is in accordance with International Standard ISO 3898.

In addition the notation given in document CIB-W18-1 »Symbols for Use in Structural Timber Design» is used.

The following general terms and symbols are used. Symbols which are not explained here are defined when used. Attention is drawn to the special notation used in Annex 7A - B - C.

Main symbols

A	Area
E	Modulus of elasticity
F	Force
G	Shear modulus
I	Second moment of area (moment of inertia)
M	Moment, unless otherwise stated bending moment
N	Axial force
V	Shear force Volume
a	Distance
b	Width
d	Diameter Side measurement for square nails
e	Eccentricity
f	Strength
h	Depth of beam
i	Radius of gyration
k	Factor, always with a subscript
ℓ	Span Length
r	Radius
t	Thickness
x y z	Coordinates
α	Angle
β	Factor
γ	Partial coefficient (load factor, material factor)
η	Factor
κ	Factor
λ	Slenderness ratio
ν	Poisson's ratio
ρ	Relative density

Main symbols (continued)

σ	Normal stress
τ	Shear stress
φ	Ratio

Subscripts

buck	Buckling
c	Compression
col	Column
con	Connector
crit	Critical
curv	Curvature
d	Design
E	Euler
e	Effective
f	Flange Load (on γ)
i	Inner
inst	Instability
k	Characteristic
m	Bending Material property (on γ)
o	Outer
t	Tension
tang	Tangency
tor	Torsion
v	Shear
w	Web
x	Related to the x-direction
y	Related to the y-direction Yield

Numbers 1, 2, . . . are used. The following have a special meaning:

0	In the fibre direction, parallel to grain
90	Perpendicular to the fibre direction, perpendicular to grain.

Subscripts are omitted whenever possible without confusion.

As an example (5.1.1.1 a): $\sigma_t \leqslant k_{size,0} \cdot f_{t,0}$ is to be read as $\sigma_{t,d} \leqslant k_{size,0} \cdot f_{t,0,d}$.

2. BASIC ASSUMPTIONS

2.1 Characteristic values and mean values

2.1.1 Characteristic values

The characteristic strength and stiffness values given in this code for timber and wood-based materials are defined as the population lower 5-percentile values directly applicable to a load duration of 3 to 5 mins. at a temperature of 20 ± 2°C and a relative humidity of 0.65 ± 0.05. The characteristic values should be estimated with a confidence level of 0.75.

The characteristic strength values are related also to a volume of 0.02 m^3 for the tensile strength perpendicular to grain.

The characteristic relative density for a species or species group is defined as the lower 5-percentile value with mass at moisture content ω = 0 and volume at a temperature of 20 ± 2°C and relative humidity of 0.65 ± 0.05.

2.1.2 Mean values

For some elastic properties the mean values are also given in this code and are defined at the same temperature and humidity conditions as the characteristic values.

2.2 Moisture classes

Structures dependent on moisture content shall be assigned to one of the moisture classes given below:

Moisture class 1
The moisture class is characterized by a moisture content in the materials corresponding to a temperature of 20 ± 2°C and the relative humidity of the surrounding air only exceeding 0.65 for limited periods and never exceeding 0.80.

Moisture class 2
The moisture class is characterized by a moisture content in the materials corresponding to a temperature of 20 ± 2°C and a relative humidity of the surrounding air only exceeding 0.80 for limited periods.

Moisture class 3
All other climatic conditions.

Based on the moisture properties of ordinary softwoods Figure 2.2 shows the moisture class dependent on temperature and relative humidity.

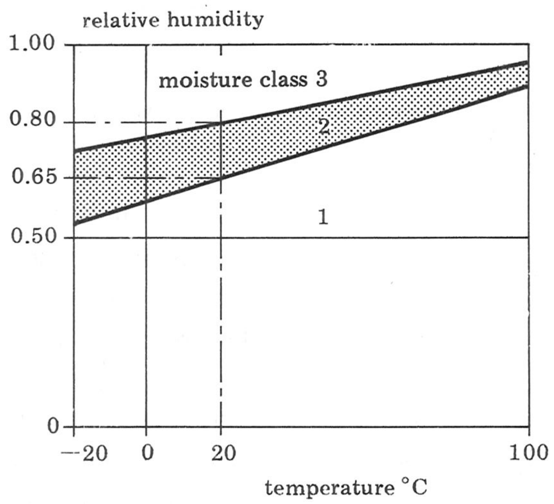


Figure 2.2

2.3 Load-duration classes

For strength and stiffness calculations actions shall be assigned to one of the load-duration classes given in table 2.3.

The load-duration classes are characterized by the effect of a constant load acting for a certain period of time. For variable action the appropriate class is 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 or structures.

Table 2.3. Load-duration classes

Load-duration class	Duration	
Long-term	$> 10^5$ hours	(> 10 years)
Medium-term	10^3 - 10^5 hours	(6 weeks - 10 years)
Short-term	10 - 10^3 hours	(10 hours - 6 weeks)
Very short-term	< 10 hours	
Instantaneous	< 3 seconds	

3. BASIC DESIGN RULES

3.0 General

Structures should be designed in such a way that there is a prescribed safety against the limit states described below being reached.

Furthermore, they should be designed in such a way that when exposed to fire they have adequate load-carrying capacity and integrity for a certain amount of time, out of regard for evacuation, limitation of flame spread and protection of firemen.

The main structure should normally be designed in such a way that when exposed to an accidental load it should not subsequently be damaged to an extent disproportionate to the extent and probability of the original incident.

The design may be based on calculations, on testing, or on a combination thereof.

3.1 Limit states

A structure, or part of a structure, is considered to have become unfit for its intended purpose when it reaches a particular state, called a limit state, in which one of the criteria relating to its load-bearing capacity or its conditions of service is infringed.

Limit states are classified as ultimate limit states and serviceability limit states.

3.1.1 Ultimate limit states

Ultimate limit states correspond to the maximum load-carrying capacity or to complete unserviceability.

Ultimate limit states may for example correspond to

- loss of static equilibrium of the structure, or part of the structure, considered as a rigid body (overturning)
- rupture of critical sections of the structure due to exceeding the material strength (in some cases dependent on the load history) or by deformations,
- loss of stability (due to among other things buckling),
- unlimited slip of the whole structure or mutually between parts of is.

3.1.2 Serviceability limit states

Serviceability limit states are related to the criteria governing normal use.

Serviceability limit states may for example correspond to

- deformations which affect the efficient use of a structure or the appearance of structural or non-structural elements,
- excessive vibrations producing discomfort or affecting non-structural elements or equipment (especially if resonance occurs),
- local damage (including cracking) which reduces the durability of a structure or affects the efficiency or appearance of structural or non-structural elements,
- local buckling of thin plates (for example in thin webs or flanges) without rupture,
- excessive impressions due to stresses perpendicular to the grain and not affecting the ultimate strength.

3.2 Verification of design

3.2.0 General principles

In the verification of the design

- actions are expressed by design values F_d according to 3.2.1.,
- strength parameters are expressed by design values f_d according to 3.2.2. Other relevant properties (e.g. modulus of elasticity in connection with instability design) are treated in a similar way,
- geometrical parameters are expressed by design values a_d according to 3.2.3.

If the general condition for the actual limit state not being reached is expressed as

$$\theta(F, f, a, \mu, C) > 0 \qquad (3.2.0a)$$

the design criterion will be

$$\theta(F_d, f_d, a_d, \mu_d, C) > 0 \qquad (3.2.0b)$$

where

- F represents actions,
- f represents material properties,
- a represents geometrical parameters,
- μ are quantities covering the uncertainties of the calculation model,
- C are constants including preselected design constraints,
- $\theta(\,)$ represents the limit state function, and
- d denotes design value.

3.2.1 Design values of actions

The actions, partial coefficients and load combinations to be taken into account should be prescribed by the relevant public authority.

It is assumed that actions will be classified according to ISO . . . (DP 6116, under preparation) and that the following values will be given

- characteristic value, F_k
- combination value, $\psi_0 F_k$

It is also assumed that the necessary information will be provided to assign the actions to the load-duration classes given in section 2.3.

The design value of an action should be obtained from the characteristic value or combination value by multiplication by a partial coefficient γ_f

$$F_d = \gamma_f F_k \qquad (3.2.1a)$$

or

$$F_d = \gamma_f \psi_0 F_k \qquad (3.2.1b)$$

The design load combination should be given as

$$\sum_{i=1}^{n} \gamma_{f,i} F_{k,i} + \sum_{j=1}^{m} \gamma_{f,j} \psi_{0,j} F_{k,j} \qquad (3.2.1c)$$

3.2.2 Design values of strength parameters

The design values should be obtained from the characteristic values, modified according to climate class and load-duration class, by division by a partial coefficient γ_m

$$f_d = f_k/\gamma_m \qquad (3.2.2a)$$

$$E_d = E_k/\gamma_m \qquad (3.2.2b)$$

For serviceability limit states γ_m = 1.0 and deflections are calculated with the mean values of the elastic properties.

For the ultimate limit states γ_m is prescribed by the relevant public authority.

3.2.3 Design values of geometrical parameters

In general the geometrical parameters may be assumed to be those specified in the design. Where deviation from the specification may have a significant effect on the structural behaviour or the resistance of the structure, the design values a_d should be obtained from the characteristic value a as

$$a_d = a + \Delta a \qquad (3.2.3a)$$

or

$$a_d = a - \Delta a \qquad (3.2.3b)$$

where Δa takes account of the importance of variations in a and the given tolerance limits for a.

3.3 Design methods

3.3.1 Design by calculation

For the ultimate limit states elastic and plastic theories may be applied according to the response of the structure, structural member, or joints to the actions. The characteristic values in chapter 5 are, however, derived from the test loads by the theory of linear elasticity and this theory should therefore also be used in the design of individual members.

> For the strength values and design methods given in this code to be applicable individual members and cross-sections must be designed in accordance with the theory of elasticity. However, the stress resultants, in for example lattice structures, need not be calculated under the assumption of elastic behaviour.

For the serviceability limit states elastic methods of analysis will usually be appropriate.

In the calculation of distribution of forces in statically indeterminate structures consideration should be given to slip in joints etc.

3.3.2 Design by testing

The testing of structures should be in accordance with (test standard in preparation by RILEM - CIB TBS - 57).

4. REQUIREMENTS FOR MATERIALS

4.0 General

Strength and stiffness properties should be based on tests for the actions to which the material may be subjected in the structure.

This requirement does not prevent properties in some cases being based on comparisons with similar species or on well-established relations between the different properties.

It must be shown that the dimensional stability, environmental behaviour etc. are satisfactory for the purposes of construction and eventual end-use.

4.1 Solid structural timber

4.1.0 General

Structural timber, i.e. timber where the strength and stiffness are of importance, should be graded in accordance with rules ensuring that the strength, stiffness and other properties of the timber are satisfactory.

The strength grading rules may 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.

Strength and stiffness parameters should be determined by standardized short-term tests in accordance with RILEM/CIB-3TT-3: Timber structures - Timber in structural sizes - Determination of some physical and mechanical properties. The test specimens should contain a grade-determining defect - preferably knots - in the zone of maximum force or bending moment.

4.1.1 Standard strength classes

In this code the following standard strength classes are used for solid sawn and round timber: SC6, SC8, SC10, SC12, SC15, SC19, SC24, SC30, SC38, SC48, SC60, and SC75.

For the standard strength classes the strength and stiffness values given in table 4.1.1 are assumed.

A given grade can be assigned to one of the standard strength classes if the characteristic bending strength f_m is not less than the value given in table 4.1.1, and if the characteristic compression strength $f_{c,0}$, shear strength f_v and the mean modulus of elasticity $E_{0,mean}$ are not less than 95 per cent of the tabulated values.

The specification of standard strength classes does not prevent the use of other strength and stiffness values for individual species and grades.

Annex 4.1 contains a survey of which national grades can be assumed to satisfy the requirements of the different standard grades. (In preparation).

Annex 4.2 contains strength and stiffness values for a number of the most important structural species and grades e.g. for European redwood/whitewood graded according to UN/ECE Recommended standard for stress grading of coniferous sawn timber (Supplement 2 to Volume XXX of the Timber Bulletin for Europe, 1977).

4.2 Finger jointed structural timber

4.2.0 General

Finger jointed structural timber should be manufactured in accordance with rules and controls which are no less stringent than those of UN/ECE Recommended standard for finger jointing in structural coniferous sawn timber (Supplement 3 to Volume XXX of the Timber Bulletin for Europe, 1977).

Strength and stiffness parameters should be determined according to section 4.1.0.

4.2.1 Standard strength classes

Finger jointed structural timber can be assigned to one of the standard strength classes stated in 4.1.1 in accordance with the criteria for solid structural timber.

European redwood/whitewood finger jointed according to the UN/ECE Recommended standard category A can be assumed to satisfy the requirements of SC24, and category B the requirements of SC19.

Table 4.1.1 Characteristic values and mean elastic moduli, in MPa *Provisional*

		SC6	SC8	SC10	SC12	SC15	SC19
Characteristic values							
Bending	f_m	6.0	7.5	9.5	12	15	19
Tension parallel to grain	$f_{t,0}$	3.6	4.5	5.7	7.2	9.0	11.5
Tension perpendicular to grain	$f_{t,90}$	0.22	0.26	0.30	0.38	0.45	0.55
Compression parallel to grain	$f_{c,0}$	5.7	7.0	9.0	11.5	14	18
Compression perpendicular to grain	$f_{c,90}$	1.8	2.2	2.9	3.6	4.5	5.7
Shear parallel to grain *	f_v	0.9	1.05	1.2	1.5	1.8	2.2
Modulus of elasticity	E_0	3900	4100	4400	4800	5250	5850
Mean values							
Modulus of elasticity, parallel	$E_{0,mean}$	5200	5500	5900	6400	7000	7800
Modulus of elasticity, perpendicular	$E_{90,mean}$	210	230	250	270	290	330
Shear modulus	G_{mean}	320	340	370	400	440	490

		SC24	SC30	SC38	SC48	SC60	SC75
Characteristic values							
Bending	f_m	24	30	38	48	60	75
Tension parallel to grain	$f_{t,0}$	15	18	23	29	36	45
Tension perpendicular to grain	$f_{t,90}$	0.68	0.82	1.0	1.3	1.6	1.9
Compression parallel to grain	$f_{c,0}$	23	29	36	45	57	70
Compression perpendicular to grain	$f_{c,90}$	7.2	9.0	11.5	14.5	18.0	22.5
Shear parallel to grain *	f_v	2.7	3.3	4.1	5.1	6.3	7.8
Modulus of elasticity	E_0	6600	7500	8600	10100	12000	14250
Mean values							
Modulus of elasticity, parallel	$E_{0,mean}$	8800	10000	11500	13500	16000	19000
Modulus of elasticity, perpendicular	$E_{90,mean}$	370	420	480	560	670	790
Shear modulus	G_{mean}	550	630	720	840	1000	1190

* Rolling shear strength may be assumed to be $f_v/2$.

4.3 Glued laminated timber

4.3.0 General

Glued laminated timber (glulam) should be manufactured in accordance with rules and controls which do not require less of the production than those of (CIB-glulam standard in preparation).

Strength and stiffness parameters should be determined in accordance with section 4.1.0, combined with appropriate methods for determining the strength and stiffness of the glulam from the properties of the laminae.

4.3.1 Standard glulam strength classes

In this code the following standard glulam strength classes are used: SCL30, SCL38, SCL48.

For the standard strength classes the strength and stiffness values given in table 4.3.1 are assumed.

Glulam may be assigned to one of the standard glulam strength classes if the characteristic bending strength, f_m, is not less than the value given in table 4.3.1, and if the characteristic compressive strength, $f_{c,0}$, shear strength, f_v, and the mean modulus of elasticity, $E_{0,mean}$, are not less than 95 per cent of the tabulated values.

Table 4.3.1 Characteristic values and mean elastic moduli, in MPa *Provisional*

		SCL30	SCL38	SCL48
Characteristic values (for strength calculations)				
Bending	f_m	30	38	48
Tension parallel to grain	$f_{t,0}$	20	26	32
Tension perpendicular to grain	$f_{t,90}$	0.70	0.80	1.0
Compression parallel to grain	$f_{c,0}$	29	36	45
Compression perpendicular to grain	$f_{c,90}$	7.2	9.0	11.5
Shear parallel to grain*	f_v	2.7	3.3	4.1
Modulus of elasticity	E_0	8500	9600	11200
Mean values (for deformation calculation)				
Modulus of elasticity parallel to grain	$E_{0,mean}$	10500	12000	14000
Modulus of elasticity perpendicular to grain	$E_{90,mean}$	450	500	600
Shear modulus	G_{mean}	650	750	900

* Rolling shear strength may be assumed to be $f_v/2$.

4.4 Wood-based sheet materials

Testing must be carried out in accordance with the following standards:

For plywood:	RILEM/CIB, 3TT-2: Timber structures, Plywood, Determination of some physical and mechanical properties.
For particle board and fibre board:	RILEM/CIB-TBS57 (test methods in preparation)

Standards on sampling and the analysis and interpretation of test data are in the course of preparation.

4.5 Glue

The glue should produce joints of such strength and durability that the integrity of the glue-line is maintained throughout the life of the structure.

4.6 Mechanical fasteners

Refer to chapter 6.

4.7 Steel parts

Nails, screws, and other steel parts should have the minimum protection against corrosion given in table 4.7. The protection is described in relation to ISO 2081, Electroplated Coatings on Zinc on Iron or Steel, but other protection systems may be used. The requirements for protection against corrosion may be relaxed where surface corrosion will not significantly reduce the load-carrying capacity.

Table 4.7 Minimum protection against corrosion

Moisture class	Nails with $d > 2.8$ mm, screws and bolts	Nails with $d \leqslant 2.8$ mm and other steel parts
1	None	None[1] - Fe/Zn 5c
2	Fe/Zn 12c	Fe/Zn 12c
3	Fe/Zn 25c[2]	Fe/Zn 25c[2]

[1] In permanently heated buildings without artificial humidifying.

[2] Under severe conditions: Fe/Zn 40c or Hot dip zinc coatings.

The consideration for the finish of the structures may call for stricter rules for corrosion protection, especially in moisture class 2. Certain woods, e.g. oak, and some treatments may have a corroding effect, and other protection could be specified.

5. DESIGN OF BASIC MEMBERS

5.1 Solid timber members

5.1.0 General

This section applies to prismatic, cylindrical and slightly conical members (i.e. timber logs and poles).

Design values of strength and stiffness, determined in accordance with section 3.2 (see also 1.4) should be used in the design equations given in this chapter, unless specifically stated otherwise.

Characteristic values for the standard strength classes defined in section 4.1.1 are given in table 4.1.1. For the load-duration classes and moisture classes defined in sections 2.2 and 2.3 the factors in table 5.1.0 should be applied.

The table is based on the behaviour of clear wood and may be conservative to some structural grades, cf. Annex 4.2. Deformations 2 - 3 times those calculated with the values given are to be expected if green timber is allowed to dry under design load.

Table 5.1.0 Modification factors to characteristic and mean values

	Characteristic values		Mean values		
Moisture classes	1 and 2	3	1	2	3
Long-term	0.55 (0.35)	0.45 (0.30)	0.7	0.6	0.4
Medium-term	0.6 (0.4)	0.5 (0.35)	1	0.8	0.7
Short-term	0.7 (0.6)	0.6 (0.5)	1	0.8	0.7
Very short-term	0.9 (0.85)	0.75 (0.70)	1	0.8	0.7
Instantaneous	1.1 (1.1)	0.95 (0.95)			

Where a load case consists of loads belonging to different load-duration classes the factors corresponding to the shortest load-duration may be used.

Values in parentheses apply to tension perpendicular to grain.

The effective cross-section and geometrical properties of a structural member should be calculated from the minimum cross-section acceptable for the given nominal size or from the actual cross-section. Nominal dimensions may be used in calculations when the actual dimensions at a moisture content of 0.20 are not less than the nominal dimensions reduced by 1 mm for dimensions of 100 mm or less; 2 mm for dimensions between 100 mm and 200 mm and 1 per cent for larger dimensions.

Reductions in cross-sectional area due to notching etc. shall be taken into account. No reductions are necessary for nails with a diameter of 5 mm or less and without predrilling.

5.1.1 Tension

The stresses should satisfy the following conditions for tension parallel to the grain direction:

$$\sigma_t \leqslant f_{t,0} \tag{5.1.1a}$$

and for tension perpendicular to the grain

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.1.1b}$$

where, for a volume of V uniformly loaded in tension perpendicular to the grain

$$k_{size,90} = \begin{cases} 1 & \text{for} \quad V \leq 0.02\ m^3 \\ \left(\frac{0.02}{V}\right)^{0.2} & \text{for} \quad V > 0.02\ m^3 \end{cases} \qquad (5.1.1c)$$

5.1.2 Compression without column effect

The stresses at an angle θ to the grain should satisfy the following condition:

$$\sigma_c \leq f_{c,0} - (f_{c,0} - f_{c,90})\sin\alpha \qquad (5.1.2a)$$

see fig. 5.1.2a.

This condition ensures that the compressive stress directly under the load is acceptable, but not that an element in compression can carry the load in question, refer to section 5.1.7.

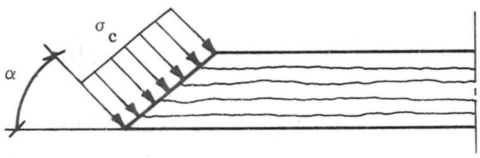
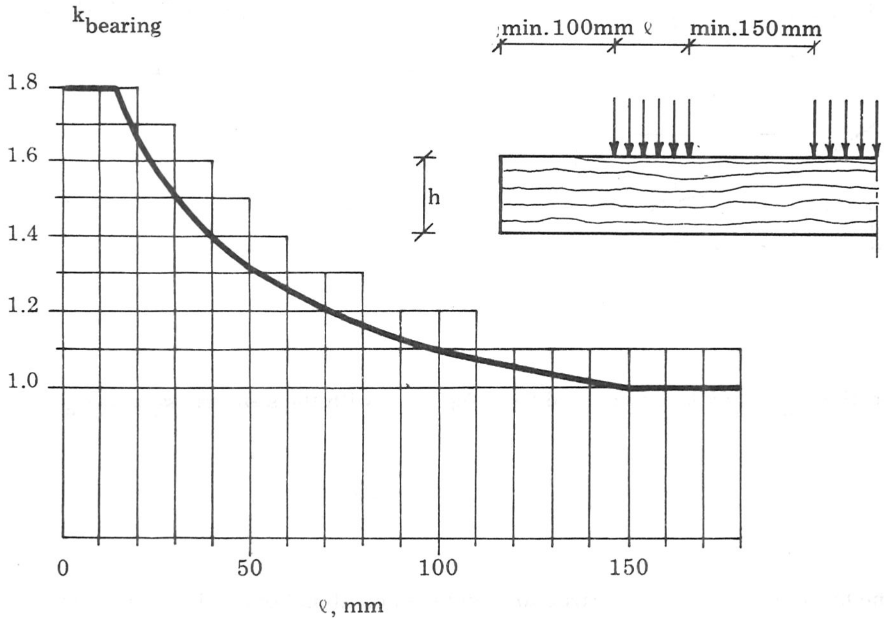


Fig. 5.1.2a

For bearings on the side grain ($\alpha = 90°$) formula (5.1.2a) may be replaced by

$$\sigma_c \leq k_{bearing} f_{c,90} \qquad (5.1.2b)$$

For bearings located at least 100 mm from the end and 150 mm from other loads, $k_{bearing}$ may be taken from fig. 5.1.2b. In other cases $k_{bearing} = 1$.



$k_{bearing} = \sqrt[4]{150/\ell}$

$1 \leq k_{bearing} \leq 1.8$

Fig. 5.1.2b.

An estimate of the influence of the deformations resulting from compression perpendicular to the grain should be made.

The strain perpendicular to the grain can be estimated as $\sigma_c/(k_{bearing}E_{90,mean})$.

5.1.3 Bending

The effective span of flexural members shall be taken as the distance between the centres of areas of bearing. With members extending further than is necessary over bearings the span may be measured between the centres of bearings of a length which would be adequate according to this code; attention should be paid to the eccentricity of the load where advantage is taken of this provision.

The bending stresses should satisfy the following condition

$$\sigma_m \leqslant k_{inst} f_m \tag{5.1.3a}$$

k_{inst} is a factor ($\leqslant 1$) taking into account the reduced strength due to failure by lateral instability (lateral buckling). k_{inst} is determined so that the total bending stresses, taking into account the effect of initial curvature, eccentricities and the deformations developed, do not exceed f_m.

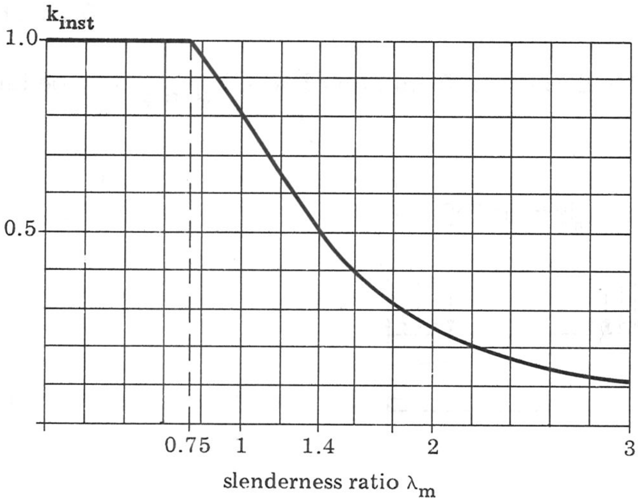
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

$$\lambda_m = \sqrt{f_m/\sigma_{m,crit}} \leqslant 0.75 \tag{5.1.3b}$$

In (5.1.3b) λ_m is the slenderness ratio for bending, and $\sigma_{m,crit}$ is the critical bending stress calculated according to the classical theory of stability.

k_{inst} may also be put equal to 1 for beams where lateral displacement of the compression side is prevented throughout its length and where torsion is prevented at the supports.

k_{inst} may be determined from fig. 5.1.3 if the initial lateral deviation from straightness measured at midspan is less than $\ell/200$.



The curve corresponds to

$\lambda_m < 0.75$; $k_{inst} = 1$

$0.75 < \lambda_m < 1.4$; $k_{inst} = 1.56 - 0.75\,\lambda_m$

$1.4 < \lambda_m$; $k_{inst} = 1/\lambda_m^2$

Fig. 5.1.3

For a beam with rectangular cross-section k_{inst} may be determined from fig. 5.1.3 with the slenderness ratio λ_m determined from

$$\lambda_m = \sqrt{\frac{\ell_e h}{\pi b^2}\,\frac{f_m}{E_0}\sqrt{\frac{E_{0,mean}}{G_{mean}}}} \tag{5.1.3c}$$

where ℓ_e is the effective length of the beam. For a number of structures and load combinations ℓ_e is given in table 5.1.3 in relation to the free beam length ℓ.

The free length is determined as follows:

a) When lateral support to prevent rotation is provided at points of bearing and no other support to prevent rotation or lateral displacement is provided throughout the length of a beam, the unsupported length shall be the distance between points of bearing, or the length of a cantilever.

b) When beams are provided with lateral support to prevent both rotation and lateral displacement at intermediate points as well as at the ends, the unsupported length may be the distance between such points of intermediate lateral support. If lateral displacement is not prevented at points of intermediate support, the unsupported length shall be the distance between points of bearing.

Table 5.1.3 Relative effective beam length ℓ_e/ℓ

Type of beam and load	ℓ_e/ℓ
Simply supported, uniform load or equal end moment	1.00
Simply supported, concentrated load at centre	0.85
Cantilever, uniform load	0.60
Cantilever, concentrated end load	0.85
Cantilever, end moment	1.00

The values apply to loads acting in the gravity axis. For downwards acting loads ℓ_e is increased by 2 h for loads applied to the top and reduced by 0.5 h for loads applied to the bottom.

5.1.4 Shear

The shear stresses should satisfy the following condition

$$\tau \leqslant f_v \tag{5.1.4a}$$

For beams supported in the bottom and loaded on the top, loads placed nearer than the beam depth from the support may be disregarded in the calculation of the shear force.

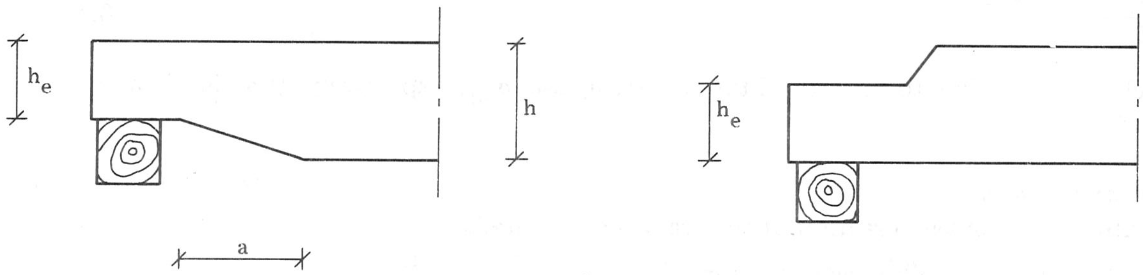


Fig. 5.1.4

For beams notched at the ends, see fig. 5.1.4, the shear stresses should be calculated on the effective depth h_e, and for notches in the bottom the condition (5.1.4a) should, for $a < 3(h - h_e)$, be replaced by

$$\tau \leqslant \left(\frac{h_e}{h} + \frac{a}{3h}\right) f_v \tag{5.1.4b}$$

Notches with $h_e < 0.5$ h are not allowed.

Notches or abrupt changes of section that will produce tension perpendicular to grain stresses at the notch should be avoided. Stress concentrations produced are likely to cause splitting at the notch at low tension values and no satisfactory means are available for determining this tension stress. A gradual change of section will reduce these stress concentrations.

5.1.5 Torsion

The torsional stresses should satisfy the following condition

$$\tau_{tor} \leqslant k_{tor} f_v \qquad (5.1.5)$$

where k_{tor} is usually taken as 1.0.

5.1.6 Combined stresses

5.1.6.0 General

At present no general theory of rupture exists, only empirical or semi-empirical expressions for the most important practical cases, some of which are given below.

5.1.6.1 Plane stress

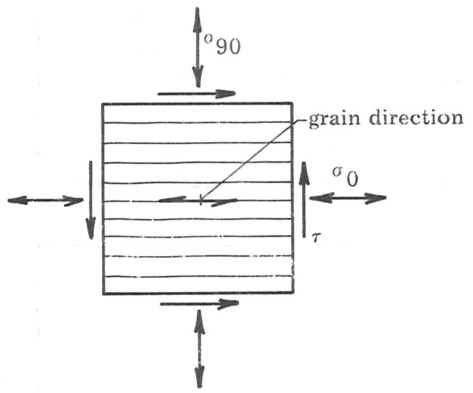


Fig. 5.1.6.1

The stresses shown in fig. 5.1.6.1 should - unless otherwise stated (see e.g. 5.1.2a) - satisfy the following condition

$$\left(\frac{\sigma_0}{f_0}\right)^2 + \left(\frac{\sigma_{90}}{f_{90}}\right)^2 + \left(\frac{\tau}{f_v}\right)^2 \leqslant 1 \qquad (5.1.6.1)$$

f_0 and f_{90} are chosen according to the sign of the stresses σ_0 and σ_{90}, respectively. If σ_0 is a bending stress then $f_0 = f_m$.

5.1.6.2 Tension and bending

Only the case with tension in the direction of the grain is considered.

The stresses should satisfy the following condition

$$\frac{\sigma_t}{f_{t,0}} + \frac{\sigma_m}{f_m} \leqslant 1 \qquad (5.1.6.2a)$$

and in the parts of the cross-section, if any, where $\sigma_t + \sigma_m \leqslant 0$, furthermore

$$|\sigma_m| - \sigma_t \leqslant f_m \qquad (5.1.6.2b)$$

5.1.6.3 Compression and bending without column effect

Only the case with compression in the direction of the grain is considered.

The stresses in the parts of the cross-section, where $\sigma_m + \sigma_c \leqslant 0$ should satisfy the following condition

$$\frac{|\sigma_c|}{f_{c,0}} + \frac{|\sigma_m|}{f_m} \leqslant 1 \qquad (5.1.6.3a)$$

and in the parts of the cross-section, if any, where $\sigma_c + \sigma_m \geqslant 0$

$$\sigma_m + \sigma_c \leqslant f_m \qquad (5.1.6.3b)$$

The condition only ensures that the stresses directly under the load are acceptable, but not that e.g. a laterally loaded column can carry the load in question. Reference is made to section 5.1.7.

5.1.6.4 Torsion and shear

The stress τ from shear and τ_{tor} from torsion should satisfy the following condition

$$\left(\frac{\tau}{f_v}\right)^2 + \frac{\tau_{tor}}{k_{tor} f_v} \leqslant 1 \qquad (5.1.6.4)$$

5.1.7 Columns

For columns it must be verified that the conditions in section 5.1.6 for compression and bending are satisfied, when apart from bending stresses from lateral load, if any, the bending stresses from initial curvature and stresses caused by the deflections are taken into consideration.

These conditions may be assumed to be satisfied if the stresses satisfy the following condition

$$\frac{|\sigma_c|}{k_{col} f_{c,0}} + \frac{|\sigma_m|}{f_m} \; \frac{1}{1 - \dfrac{k_{col}}{k_E} \dfrac{|\sigma_c|}{f_{c,0}}} \leqslant 1 \qquad (5.1.7a)$$

where σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_{col} and k_E are factors depending on the slenderness ratio $\lambda = \ell/i$, the material parameters and the initial curvature.

The initial curvature is assumed to correspond to a maximum eccentricity of the axial force of

$$e = \eta \, r_{core} \lambda \qquad (5.1.7b)$$

where r_{core} is the core radius.

$$k_E = \frac{\sigma_E}{f_{c,0}} = \frac{\pi^2 E_0}{f_{c,0} \lambda^2} \qquad (5.1.7c)$$

$$k_{col} = 0.5\left[\left(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E\right) - \sqrt{(1 + (1 + \eta\lambda \frac{f_{c,0}}{f_m})k_E)^2 - 4k_E}\,\right] \qquad (5.1.7d)$$

σ_E is the Euler stress.

For the purpose of calculating the slenderness ratio of compression members, the values of the length ℓ_c should be calculated for the worst conditions of loading to which a compression member is subjected, paying regard to the induced forces at the supports or along the length of the compression member and to slip in the connections at the supports. The length should be judged to be the distance between two adjacent points of zero bending moment, these being two points between which the deflected member would be in single curvature.

The slenderness ratio should not exceed 170, or for secondary members, 200.

5.2 Glued laminated members

5.2.0 General and straight members

Section 5.1 for solid timber applies except that formula (5.1.4b) should be replaced by

$$\tau \leqslant \left[1 - 2.8 \frac{h - h_e}{h} \left(1 - \frac{a}{14(h - h_e)}\right)\right] f_v \qquad (5.2.0)$$

and notches with $h_e < 0.75h$ are not allowed.

5.2.1 Tapered beams

This section applies to single-tapered beams (fig. 5.2.1a) and double-tapered beams (fig. 5.2.1b) with rectangular cross-sections. For double-tapered beams the shear forces are assumed to be small near the apex.

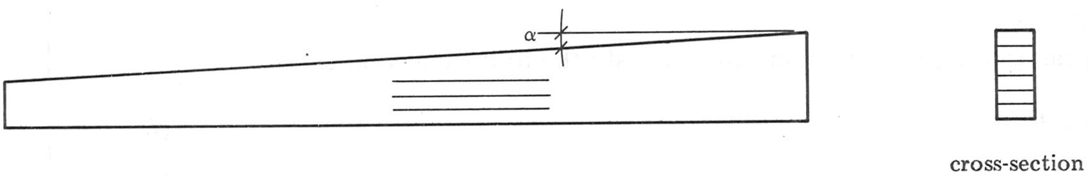


Fig. 5.2.1a. Single-tapered beam.

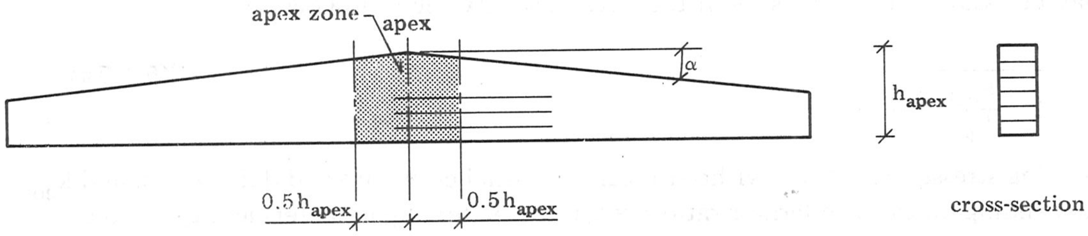


Fig. 5.2.1b. Double-tapered beam.

The influence of the taper should be taken into account in determining the stresses. For single-tapered beams and for outside the apex zone of double-tapered beams the bending stresses in the outermost fibres should satisfy condition (5.1.6.1).

In the apex zone of double-tapered beams the bending stresses should satisfy the following condition

$$\sigma_m \leqslant f_m \tag{5.2.1a}$$

In the apex zone of double-tapered beams the tensile stresses perpendicular to the grain should satisfy the following condition

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.2.1b}$$

where

$$k_{size,90} = \begin{cases} \dfrac{0.9}{V^{0.2}} & \text{for uniformly distributed load} \\ \\ \dfrac{0.6}{V^{0.2}} & \text{for other loading} \end{cases} \tag{5.2.1c}$$

V is the volume in m³ of the apex zone (corresponding to the shaded area in fig. 5.2.1b).

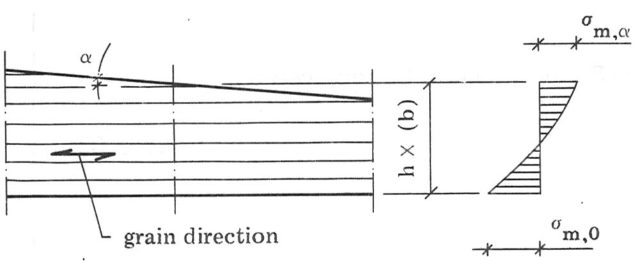


Fig. 5.2.1c

For a tapered beam with a rectangular cross-section with the grain direction parallel to one of the surfaces and $\alpha \leqslant 20°$ the bending stresses in the outermost fibres may be calculated as

$$\sigma_{m,0} = (1 + 3.7 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.2.1d)$$

$$\sigma_{m,\alpha} = (1 - 4.4 \tan^2 \alpha) \frac{6M}{bh^2} \qquad (5.2.1e)$$

The stresses should satisfy the following conditions

$$\sigma_{m,0} \leqslant f_m \qquad (5.2.1f)$$

$$\sigma_{m,\alpha} \leqslant \frac{f_m}{\sqrt{1 + (\frac{f_m}{f_v} \tan\alpha)^2 + (\frac{f_m}{f_{90}} \tan^2 \alpha)^2}} \qquad (5.2.1g)$$

where $f_{90} = f_{t,90}$ or $f_{90} = f_{c,90}$ depending on the sign of the stresses.

In the apex zone of double-tapered beams the greatest bending stresses can be calculated as

$$\sigma_m = (1 + 1.4\tan\alpha + 5.4\tan^2 \alpha) \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.1h)$$

and the greatest tensile stresses perpendicular to the grain as

$$\sigma_t = 0.2\tan\alpha \frac{6M_{apex}}{bh^2_{apex}} \qquad (5.2.1i)$$

5.2.2 Curved beams

This section applies to curved beams with constant, rectangular cross-section, see fig. 5.2.2.

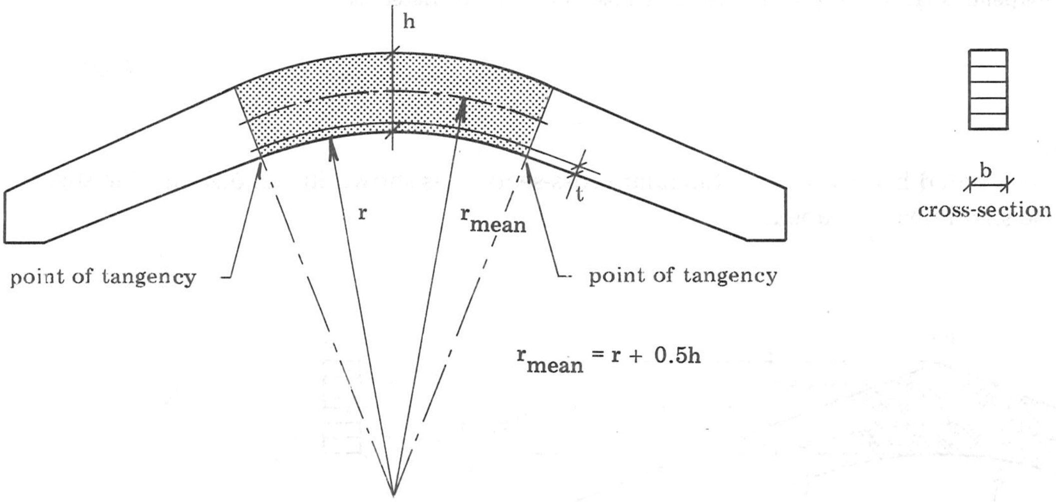


Fig. 5.2.2

The ratio between the radius of curvature, r, and the laminae thickness, t, should be greater than 125. For $r/t < 240$ the reduction of the strength in bending, tension and compression parallel to the grain due to the bending of the laminae should be taken into account.

This can be done by multiplying f_m, $f_{c,0}$, and $f_{t,0}$ by the factor k_{curv}, where

$$k_{curv} = 0.76 + 0.001 \frac{r}{t} \qquad (5.2.2a)$$

In sharply curved beams (i.e. the ratio between minimum mean-radius of curvature, r_m, and depth, h, less than 15) the influence of the curvature on the distribution of axial stresses from bending moments should be taken into consideration.

The bending stresses in the innermost fibre may be calculated as

$$\sigma_{m,i} = k_i \frac{6M}{bh^2} \tag{5.2.2b}$$

where

$$k_i = \begin{cases} 1 + 0.5h/r_{mean} & \text{for } r_{mean}/h \leqslant 10 \\ 1.15 - 0.01 r_{mean}/h & \text{for } 10 < (r_{mean}/h) < 15 \end{cases} \tag{5.2.2c}$$

The bending stresses in the outermost fibre may be calculated by the usual expression

$$\sigma_{m,o} = \frac{6M}{bh^2} \tag{5.2.2d}$$

When the bending moments tend to reduce the curvature (increase the radius) the tensile stresses perpendicular to the grain should satisfy the condition

$$\sigma_t \leqslant k_{size,90} f_{t,90} \tag{5.2.2e}$$

where

$$k_{size,90} = \begin{cases} \dfrac{0.6}{V^{0.2}} & \text{for uniformly distributed load} \\ \dfrac{0.5}{V^{0.2}} & \text{for other loading} \end{cases} \tag{5.2.2f}$$

V is the volume in m^3 of the curved part of the beam (corresponding to the shaded area in fig. 5.2.2).

The tensile stresses perpendicular to the grain in the curved part may be calculated as

$$\sigma_t = \frac{1.5\,M}{r_{mean}\,bh} \tag{5.2.2g}$$

5.2.3 Cambered beams

This section applies to cambered beams with rectangular cross-section as shown in fig. 5.2.3a. The shear forces are assumed to be small near the apex.

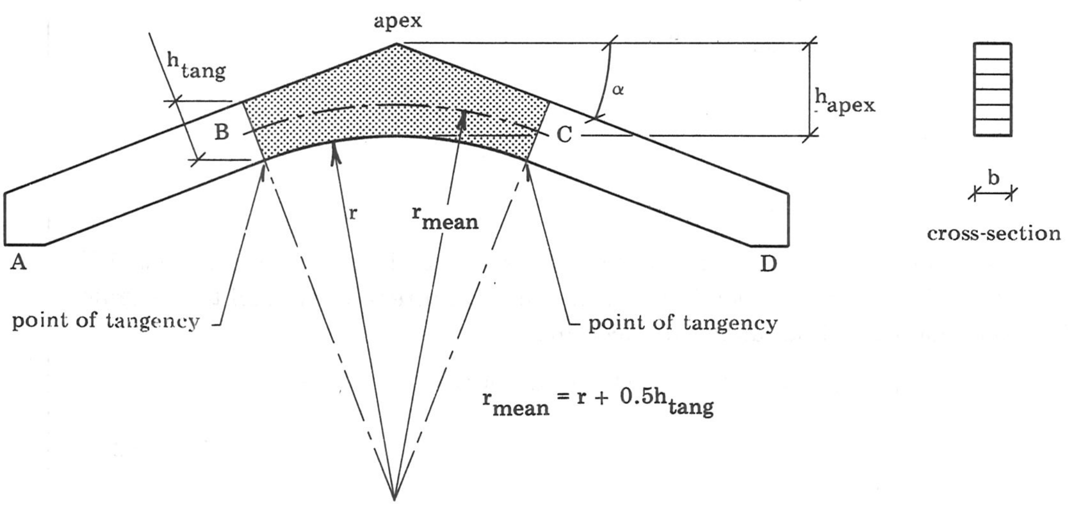


Fig. 5.2.3a. Cambered beam.

For the straight parts (A-B and C-D) section 5.2.1 applies.

In the apex zone (C-D) the conditions (5.2.1a) and (5.2.1b) apply with

$$k_{size} = \begin{cases} \dfrac{0.9}{V^{0.2}} & \text{for uniformly distributed load} \\[2ex] \dfrac{0.6}{V^{0.2}} & \text{for other loading} \end{cases} \qquad (5.2.3a)$$

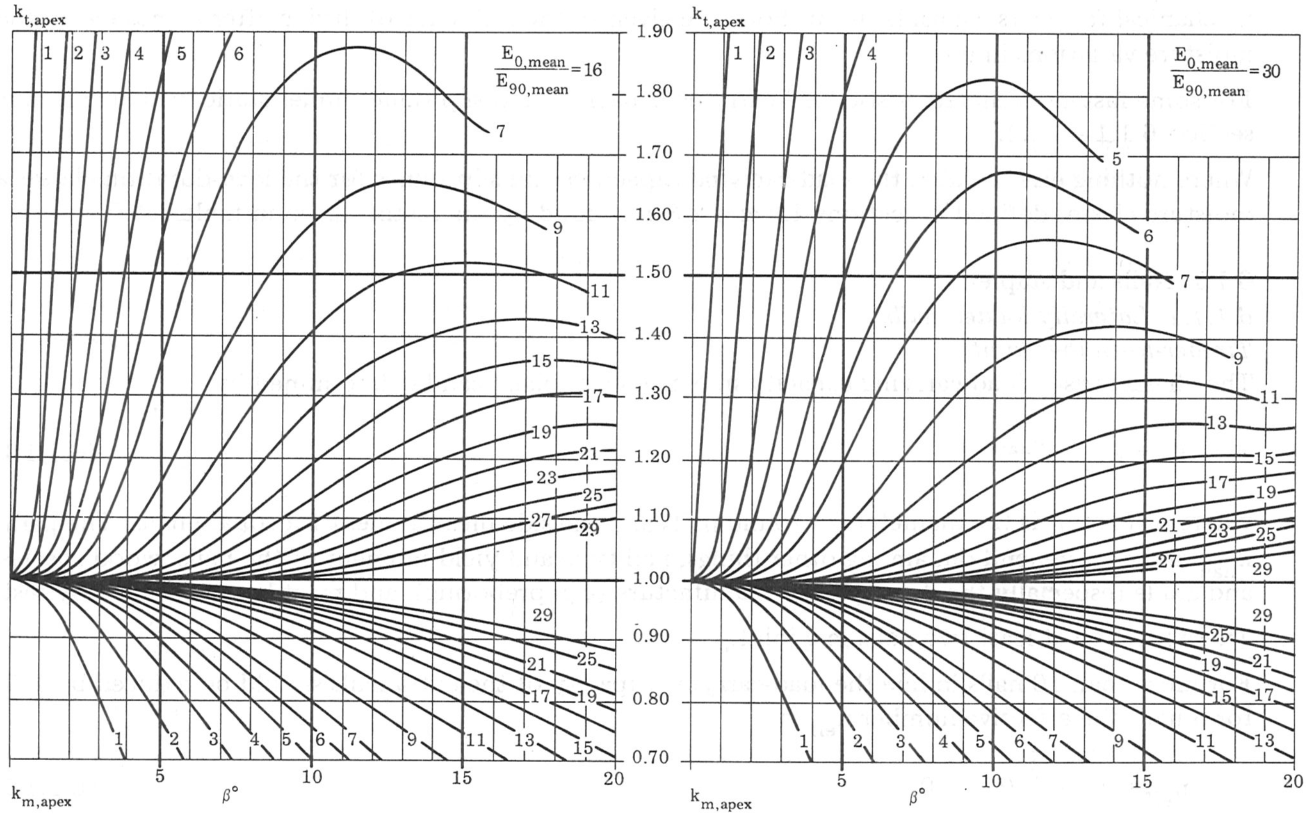
V is the volume in m^3 between the points of tangency (corresponding to the shaded area in fig. 5.2.3a).

The bending stresses and the tensile stresses perpendicular to the grain in the apex zone may be calculated as

$$\sigma_{m,apex} = k_{m,apex}\, \sigma_{m,curv} \qquad (5.2.3b)$$

$$\sigma_{t,apex} = k_{t,apex}\, \sigma_{t,curv} \qquad (5.2.3c)$$

where $\sigma_{m,curv}$ and $\sigma_{t,curv}$ are the corresponding stresses in a curved beam (section 5.2.2) where $h = h_{tang}$. The factors $k_{m,apex}$ and $k_{t,apex}$ are given in figure 5.2.3b.



Number on the curves denotes $\frac{100 h_{tang}}{r_{mean}}$

Fig. 5.2.3b. $k_{m,apex}$ and $k_{t,apex}$.

6. JOINTS

6.0 General

When the joint is non-symmetric, or where the load is eccentric, consideration should be given to these factors by the determination of the behaviour of the fasteners as well as by the design of the members.

It should be taken into account that the load-carrying capacity of a multiple-fastener joint will frequently be less than the sum of the individual fastener capacities and that some types of fluctuating load may cause a reduction, especially when the stresses alternate between tension and compression.

If the load on a joint is carried by more than one type of fastener due consideration should be given to the differences in stiffness.

> Where the load is carried by nails in combination with bolts, screws or connectors, the usual design load of one of the fasteners should be reduced by 1/3.
>
> Glue and mechanical fasteners have very different stiffness properties and thus they can never be assumed to act in unison.

The arrangement of timber joints and the size of the fasteners, spacings and distances to the ends or edges of the timber should be chosen so that the expected strengths can be obtained.

6.1 Joints with mechanical fasteners

6.1.0 General

The characteristic load-carrying capacity should be based on tests carried out in conformity with RILEM/ CIB 3TT-1: Timber structures - Joints - Determination of strength and deformation characteristics of mechanical fasteners. Consideration should be given to the influence of drying after manufacture and to moisture variations in use.

For some fasteners characteristic load-carrying capacities and slip values under static load are given in section 6.1.1 - 6.1.4.

Where nothing else is stated the load-carrying capacities and slip values for the load-duration classes and moisture classes defined in sections 2.2 and 2.3 are found by the factors given in table 5.1.0.

6.1.1 Nails and staples

6.1.1.1 Laterally loaded nails

Timber-to-timber joints

The characteristic load-carrying capacity in N per shear plane can be determined by

$$F_k = k_{nail}\, d^{\alpha_{nail}} \qquad (6.1.1.1a)$$

where d (in mm) is the diameter for round nails and the side measurement for square nails. The parameters k_{nail} and α_{nail} depend on, among other things, nail type and yield moment of the nails, wood species and grade (especially the density), the manufacture (e.g. preboring), and must be determined by tests.

There must be at least two nails in a joint.

For more than 10 nails in line the load-carrying capacity of the extra nails should be reduced by 1/3, i.e. for n nails the effective number n_{eff} is

$$n_{eff} = 10 + \frac{2}{3}(n - 10) \qquad (6.1.1.1b)$$

Nails in end grain should normally be considered incapable of transmitting force.

The slip u for a load $F \leqslant F_k/3$ may be taken as

$$u = 0.5d(F/F_k)^{1.5} \qquad (6.1.1.1c)$$

> For round nails with a characteristic tensile strength of at least $40(20 - d)$MPa the following values can be used for Nordic softwood at least corresponding to SC19 and other woods with corresponding properties

$$\alpha_{nail} = 1.7$$

$$k_{nail} = 200\sqrt{\rho} \qquad (6.1.1.1d)$$

where ρ is the relative density defined in section 2.1. No preboring is assumed.

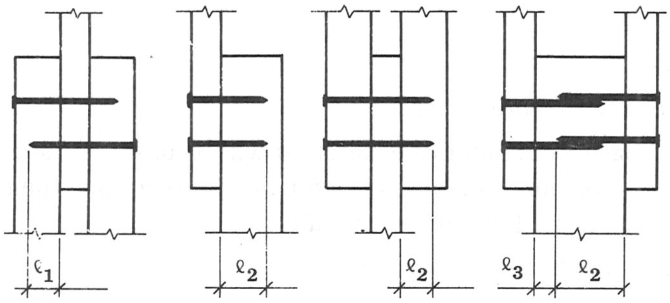


Fig. 6.1.1.1a

The values assume that the nails are driven in perpendicular to the grain, that the thinnest member has a thickness of not less than 7d for d $\leqslant$ 5 mm and 13d - 30 mm for d > 5 mm and that the penetration depths (including the point) satisfy the following conditions (cf. fig. 6.1.1.1a)

Nails in double shear	
(driven in alternately from either side)	$\ell_1 \geqslant 8d$
Other cases	
smooth nails	$\ell_2 \geqslant 12d$
annularly and spirally grooved nails	$\ell_2 \geqslant 8d$

For smaller thicknesses and lengths the load-carrying capacity is reduced in proportion to the length. For smooth nails it is required that the nail length in any timber member is at least 5d and that the penetration length ℓ_2 is at least 6d. For annular nails the penetration length should at least be 4d.

If ℓ_3 is greater than 3d (cf. fig. 6.1.1.1a) nails from the two sides are allowed to overlap in the middle member.

Minimum distances for timber-to-timber joints are given in fig. 6.1.1.1b. The nails should be staggered in the best possible way, for example as shown in fig. 6.1.1.1b, one nail thickness in relation to the system lines.

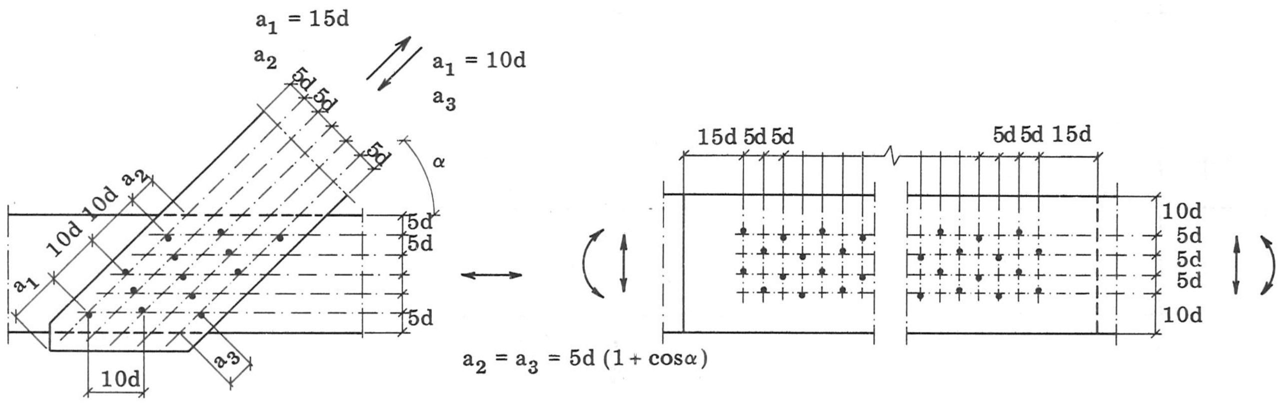


Fig. 6.1.1.1b. Minimum distances for timber-to-timber joints.

Steel-to-timber joints

The recommendations for timber apply, but the load-carrying capacities for timber-to-timber joints may be multiplied by 1.25.

Adequate strength of the steel plates is assumed. Internal nail spacing should be 0.7 times those given for timber-to-timber joints. Staggering is not required.

Board materials-to-timber joints
The recommendations for timber apply, but a board with thickness t can be assumed to correspond to a softwood timber member of quality SC19 with the thickness

2.5t for plywood of birch, beech, and similar hardwood

2.0t for plywood with plies of alternating hardwood and fir or pine (e.g. combi-plywood)

1.5t for plywood of fir, pine, and similar softwood

2.5t for hard or oil-tempered structural fibre board

1.0t for structural particle board and semihard structural fibre board.

It is assumed that ordinary nails with heads which have a diameter of at least 2d will be used. For smaller heads the load-carrying capacity should be reduced. For pins and oval headed nails, for example, the load-carrying capacity in particle boards and fibre boards should be reduced by half.

Internal nail spacing should be 0.8 times those given for timber-to-timber joints.

6.1.1.2 Axially loaded nails
The characteristic withdrawal resistance of nails for nailing perpendicular to the grain as in fig. 6.1.1.2a and for slant nailing as in fig. 6.1.1.2b is the smallest of the values according to formula (6.1.1.2a), corresponding to withdrawal of the nail in the member receiving the point, and formulas (6.1.1.2b-c) corresponding to the head being pulled through. For smooth nails with heads with a diameter of at least 2d (6.1.1.2b) can be disregarded.

$$F = \min \begin{cases} f_{axial}\, d & (6.1.1.2a) \\ f_{axial}\, dh + f_{head}\, d^2 \quad \text{for smooth nails} & (6.1.1.2b) \\ f_{head}\, d^2 \quad \text{for annularly and spirally grooved nails} & (6.1.1.2c) \end{cases}$$

$\ell \geqslant 4d$ is assumed.

The parameters f_{axial} and f_{head} depend on, among other things, type of nail, timber species and grade (especially density) and must be determined by tests.

For spirally or annularly threaded nails only the threaded part is considered capable of transmitting force.

Nails in end grain should normally be considered incapable of transmitting force.

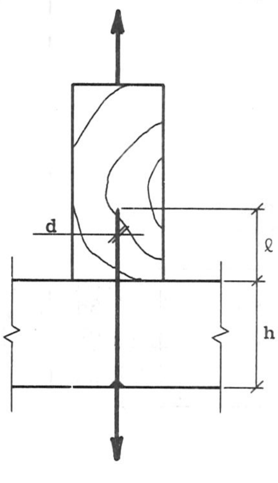


Fig. 6.1.1.2a

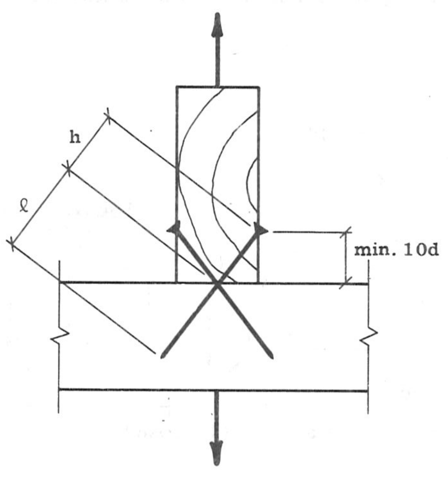


Fig. 6.1.1.2b

The distances for laterally loaded nails should be complied with and the distance to the loaded edge by slant nailing should be at least 10d, see fig. 6.1.1.2b.

Normally the values of f given in table 6.1.1.2 can be assumed. For structural timber from strength class SC19 a characteristic relative density of $\rho = 0.36$ has been assumed.

Table 6.1.1.2

	f_{axial} in MPa		f_{head} in MPa	
	general	SC19	general	SC19
Ordinary nails, round	$12.5\ \rho^2$	1.6	$45\ \rho^2$	5.8
Ordinary nails, square	$15\ \rho^2$	1.9	$45\ \rho^2$	5.8
Spirally threaded nails	- - -	to be determined by tests		- - -
Annularly threaded nails	- - -	to be determined by tests		- - -

6.1.1.3 Staples

The general rules for nailed joints apply with the two staple legs acting as two nails with the same diameter, provided the angle between the crown and the direction of the grain of the timber under the crown is greater than 30°. If the angle between the crown and the direction of the grain of the timber under the crown is equal to or less than 30° then the load-carrying capacity should by multiplied by 0.7.

6.1.2 Bolts and dowels

6.1.2.1 Bolts

Timber-to-timber joints

The characteristic load-carrying capacity in N per shear plane for bolts with a yield strength f_y of at least 240 MPa (corresponding to ISO grade 4.6) is the smallest value found from the formulae (6.1.2.1a) - (6.1.2.1d).

$$F = \min \begin{cases} 18\rho(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2.1a) \\ 35\rho\, k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2.1b) \\ 70\rho\, k_1 t_1 d & (6.1.2.1c) \\ 75 d^2 \sqrt{\rho}\sqrt{(k_1 + k_2)/2}\sqrt{f_y/240} & (6.1.2.1d) \end{cases}$$

where

- t is timber thickness in mm
- d is the diameter in mm
- k is a factor taking into consideration the influence of the angle, v, between force and the direction of the grain

$$k = \frac{k_{90}}{k_{90}\cos^2 v + \sin^2 v} \qquad (6.1.2.1e)$$

$$k_{90} = 0.45 + 8d^{-1.5} \quad \text{(d in mm)} \qquad (6.1.2.1f)$$

In three-member joints subscript 1 denotes the side member and subscript 2 denoted the middle member. In two-member joints the subscripts are chosen so that $k_1 t_1 \leqslant k_2 t_2$.

For more than 4 bolts in line the load-carrying capacity of the extra bolts should be reduced by 1/3, i.e. for n bolts the effective number n_{eff} is

$$n_{eff} = 4 + \frac{2}{3}(n - 4) \qquad (6.1.2.1g)$$

When the force acts at an angle to the grain it should further be shown that

$$V \leqslant \frac{2}{3} f_v b_e t \qquad (6.1.2.1h)$$

where V is the shear force produced by the bolts or dowels, t is the thickness of the member, and b_e is the distance from the loaded edge to the farthest point of the bolt, see fig. 6.1.2.1a.

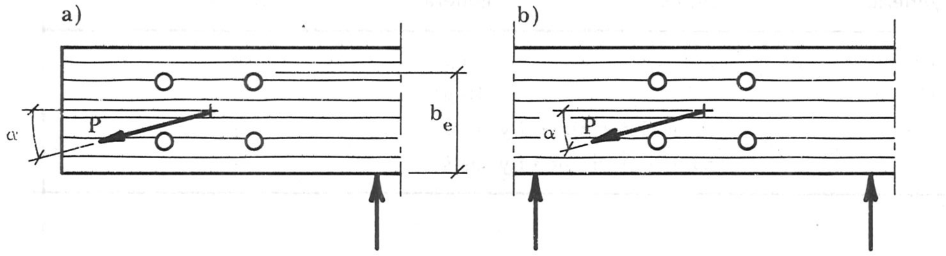


Fig. 6.1.2.1a. In case a) $V = P \sin\alpha$. In case b) $V = \frac{1}{2} P \sin\alpha$.

In multiple shear the load-carrying capacity may be found by considering the structure as a number of three-member joints.

The slip at one third of the characteristic load is approximately 0.1 d + 1 mm.

Steel-to-timber joints

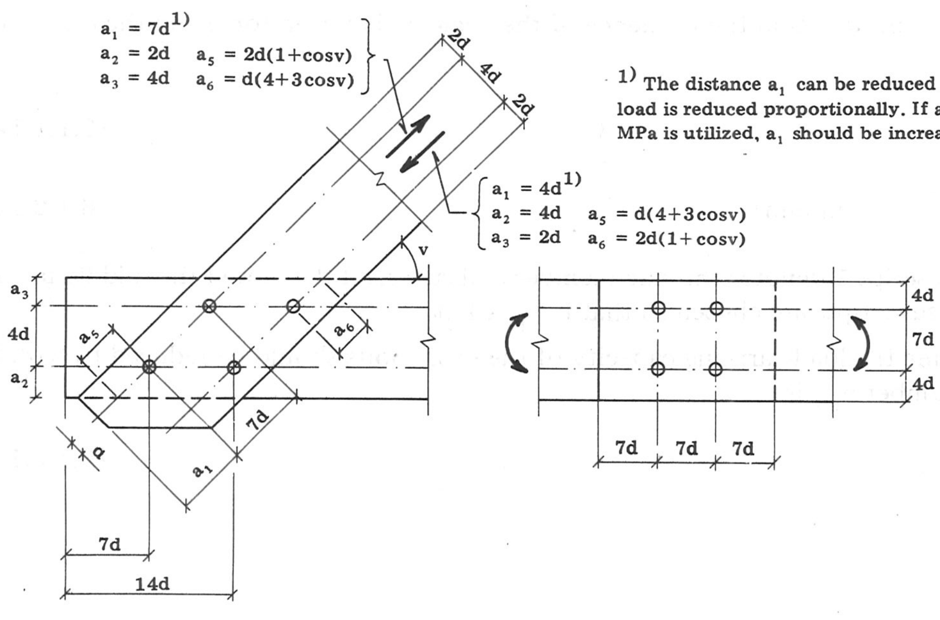
Where the side members are steel plates the loads calculated from the above forumlae may be used with t_1 equal to t_2 equal to the thickness of the wood member.

Where the middle member is a steel plate formula (6.1.2.1b) is omitted and the values of formula (6.1.2.1d) should be multiplied by 1.4.

For structural timber from strength class SC19 (i.e. $\rho = 0.36$) the following is found by inserting into (6.1.2.1a) - (6.1.2.1d)

$$F = \min \begin{cases} 6.5(k_1 t_1 + k_2 t_2)d & \text{(only for two-member joints)} \quad (6.1.2.1i) \\ 12.5 k_2 t_2 d & \text{(only for three-member joints)} \quad (6.1.2.1j) \\ 25 k_1 t_1 d & (6.1.2.1k) \\ 45 d^2 \sqrt{(k_1 + k_2)/2}\ \sqrt{f_y/240} & \end{cases}$$

Minimum distances are given in fig. 6.1.2.1b.



1) The distance a_1 can be reduced to a minimum of 4d provided the load is reduced proportionally. If a yield strength f_y greater than 240 MPa is utilized, a_1 should be increased by the factor $\sqrt{f_y/240}$.

Fig. 6.1.2.1b.

6.2.2.2 Dowels
A dowel is a smooth steel rod, i.e. a bolt without a head.

The rules for bolted joints apply, but the load-carrying capacities for bolted joints should be multiplied by 1.25.

Refer to chapter 8 for requirements for predrilling, tolerances, etc.

The slip at one third of the characteristic load is approximately 0.1 d.

6.1.3 Wood and lag screws
6.1.3.1 Laterally loaded screws
Timber-to-timber joints
The characteristic load-carrying capacity in N of screws with a yield strength f_y of at least 240 MPa screwed at right angles to the grain is the smaller of the values from the formulae (6.1.3.1 a) - (6.1.3.1 b)

$$F = \min \begin{cases} 70 \rho k_1 t d & (6.1.3.1\,a) \\ 75 d^2 \sqrt{\rho}\, \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.3.1\,b) \end{cases}$$

where

- t is the thickness in mm of the timber
- d is the diameter in mm of the screw, measured on the smooth shank
- k_1, k_2 are factors, obtained from table 6.1.1.2, taking into consideration the influence of the angle between force and direction of the grain in the member under the screw head (k_1) and the member receiving the point (k_2).

Furthermore, it should be verified that the condition (6.1.2.1 h) is satisfied.

The slip at one third of the characteristic load is approximately 0.1 d.

It is assumed that

- screws are driven into pre-bored holes, see section 8.3,
- the length of the smooth shank is greater than or equal to the thickness of the member under the screw head,
- the penetration depth of the screw, i.e. the length in the member receiving the point, is at least 8d.

If the penetration depth is less than 8d the load-carrying capacity is reduced proportionally. However, the penetration depth should be at least 4d.

Screws in end grain should normally be considered incapable of transmitting force.

For structural timber from strength class SC19 (i.e. $\rho = 0.36$) the following is found by inserting into (6.1.3.1 a) - (6.1.3.1 b)

$$F = \min \begin{cases} 25 k_1 t d & (6.1.3.1\,c) \\ 45 d^2 \sqrt{(k_1 + k_2)/2}\, \sqrt{f_y/240} & (6.1.3.1\,d) \end{cases}$$

Minimum distances are the same as for bolts (refer to section 6.1.2.1).

Steel-to-timber joints
The characteristic load-carrying capacity in N is given by

$$1.25 \cdot 75 d^2 \sqrt{\rho}\, \sqrt{(1 + k_2)/2}\, \sqrt{f_y/240} \qquad (6.1.3.1\,e)$$

and furthermore, what is stated for timber-to-timber joints applies. Adequate strength of the steel plates is assumed.

6.1.3.2 Withdrawal loads of screws

The characteristic withdrawal capacity in N of screws driven at right angles to the grain is

$$F = f_{screw}(\ell_t - d)d \qquad (6.1.3.2a)$$

where

- d is the diameter in mm measured on the smooth shank,
- ℓ_t is the threaded length in mm in the member receiving the screw,
- f_{screw} is a parameter dependent on, among other things, the shape of the screw, the timber species and the grade.

It is assumed that the strength of the screw is adequate.

For screws according to ISO 0000 the following can be assumed for structural timber from strength class SC19, $f_{screw} = 12 + 30/d$, i.e.

$$F = (30 + 12d)(\ell_t - d) \qquad (6.1.3.2b)$$

It is assumed that the minimum distances and penetration lengths given for laterally loaded screws are complied with.

6.1.4 Connectors

The characteristic load-carrying capacity and deformation characteristics of joints with connectors should in general be determined by testing.

The testing should give consideration, among other things, to the influence of

- the angle between force direction and the direction of the grain,
- the diameter of the bolts or screws,
- the dimensions of the members,
- the spacings and distances to the ends and the edges,
- the manufacturing conditions.

When a load is applied at an angle to the direction of the grain it should be shown that the condition (6.1.2.1h) is satisfied. In this case b_e is the distance from the loaded edge to the farthest edge of the connectors, see fig. 6.1.4.

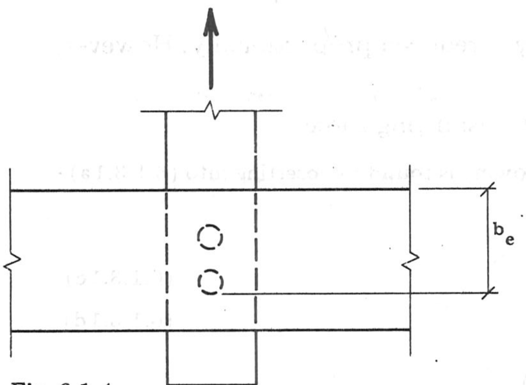


Fig. 6.1.4

If a connector is to be used with several bolt diameters the investigation should comprise at least maximum and minimum bolt diameters and it can be assumed that the load-carrying capacity of the joint F_{joint} is

$$F_{joint} = F_{bolt} + F_{conn}$$

where F_{bolt} is the load-carrying capacity of the bolt (or screw) calculated as stated in 6.1.2 (or 6.1.3), and F_{conn} is the contribution from the connector.

6.1.5 Nail plates

The load-carrying capacity should be derived from tests carried out in accordance with Annex A1 Punched Metal Plate Fasteners to test standard RILEM/CIB-3TT.

6.2 Glued joints

For continuous glued joints connecting laminae (e.g. in glued laminated timber and between flanges and webs in beams or columns) the glued joint may be assumed to have the same shear strength and tension perpendicular to the grain strength as the weakest of the jointed materials.

For other glued joints consideration should be given to the reduction in strength caused by non-uniform distribution of stresses over the glued area, including concentration of stresses at edges etc.

For lap joints or gusset joints a characteristic shear strength of $(1.5 - 0.75 \sin\alpha)$MPa, where α is the angle between the force and grain direction, may be assumed for structural timber from strength class SC19. The force should not exceed $(75 - 37.5 \sin\alpha)$kN, which corresponds to an area of $0.05\ m^2$.

7. DESIGN OF COMPONENTS AND SPECIAL STRUCTURES

7.1 Glued components

7.1.1 Thin-webbed beams

The stresses in thin-webbed beams may be calculated assuming a linear variation of strain over the depth. In principle the stresses must satisfy the conditions given in section 5.

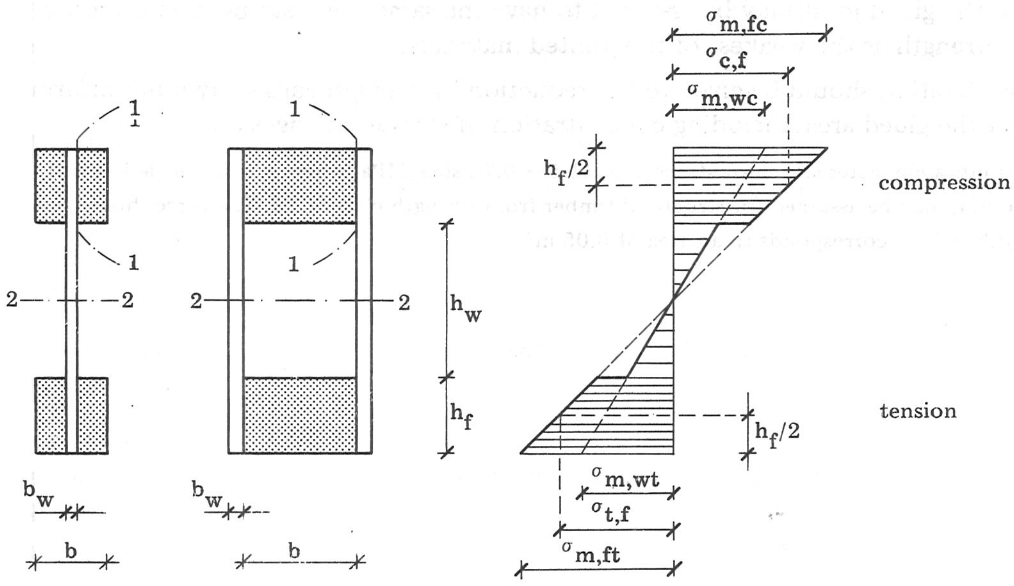


Fig. 7.1.1a

For a beam with flanges of timber or glulam the stresses in the flanges should satisfy the following conditions, cf. fig. 7.1.1a

$$|\sigma_{m,fc}| \leqslant f_m \qquad (7.1.1a)$$

$$|\sigma_{c,f}| \leqslant k_{col} f_{c,0} \qquad (7.1.1b)$$

$$\sigma_{t,f} \leqslant f_{t,0} \qquad (7.1.1c)$$

$$\sigma_{m,ft} \leqslant f_m \qquad (7.1.1d)$$

k_{col} is determined according to section (5.1.1.9) with $\lambda = \sqrt{12}\,\ell_c/b$, where ℓ_c is the distance between the sections where lateral deflection of the compression flange is prevented, and b is given in fig. 7.1.1a. If a special investigation into the lateral instability of the beam is made as a whole, $k_{col} = 1$ may be assumed.

For box beams an investigation of the lateral instability may be omitted if (7.1.1a) is replaced by

$$|\sigma_{m,fc}| \leqslant k_{inst} f_m \qquad (7.1.1e)$$

where k_{inst} is determined according to section 5.1.1.3.

The shear stresses may be assumed to be uniformly distributed over the width of the sections 1-1 and 2-2 shown in fig. 7.1.1a.

It must be shown that the webs do not buckle or that the forces can otherwise be resisted.

A buckling investigation is not necessary if the webs are made from structural plywood and the free depth, h_w, of the webs is less than $2h_{max}$, where h_{max} is given in table 7.1.1 and the shear force V satisfies the following conditions

$$V \leqslant \begin{cases} f_v b_w (h_w + h_f) & \text{for } h_w \leqslant h_{max} \\ f_v b_w h_{max} (1 + \frac{h_f}{h_w}) & \text{for } h_{max} \leqslant h_w \leqslant 2h_{max} \end{cases} \tag{7.1.1f}$$

It is assumed that the web is stiffened at the supports and under concentrated loads. The stiffeners should be fastened to the web and fit tightly between the top and bottom flanges. The cross-sections of the stiffeners should be chosen so that the whole force can be transmitted from the flanges to the web stiffeners.

Table 7.1.1

Web	h_{max}
Plywood with $\varphi < 0.5$	$(20 + 50\varphi)b_w$
Plywood with $\varphi \geqslant 0.5$	$45\ b_w$

φ is the ratio between the bending stiffness of a strip of unit width cut perpendicularly to the beam axis, and the bending stiffness of a corresponding strip cut parallel to the longitudinal direction of the beam

In cases where a special investigation must be undertaken it should be carried out in accordance with the linear elastic theory for perfect plates simply supported along the flanges and web stiffeners.

For the case shown in fig. 7.1.1b these assumptions lead to the following condition

$$\frac{\sigma}{\sigma_{crit}} + \left(\frac{\tau}{\tau_{crit}}\right)^2 \leqslant 1 \tag{7.1.1g}$$

where σ_{crit} is the critical stress if only the axial stresses were acting and τ_{crit} the critical stress if only the shear stresses were acting.

σ_{crit} should be determined as

$$\sigma_{crit} = k_{buck,\sigma} \frac{\pi^2 \sqrt{(EI)_x (EI)_y}}{ta^2} \tag{7.1.1h}$$

where $k_{buck,\sigma}$ for some cases is given in fig. 7.1.1c and fig. 7.1.1d.

τ_{crit} should be determined as

$$\tau_{crit} = k_{buck,\tau} \frac{\pi^2 \sqrt[4]{(EI)_x^3 (EI)_y}}{ta^2} \tag{7.1.1i}$$

where $k_{buck,\tau}$ for pure shear is given in fig. 7.1.1e.

The following notation is used

$(EI)_x$ is the bending stiffness of the panel per unit width in bending about the X-axis. For a homogeneous orthotropic panel with the main directions X and Y, $(EI)_x = \frac{1}{12} Et^3/(1 - \nu_{xy}\nu_{yx})$, where ν_{xy} and ν_{yx} are Poisson's ratios. For wood-based panels $\nu_{xy}\nu_{yx} \approx 0$ can be assumed.

$(EI)_y$ as $(EI)_x$, but in bending about the Y-axis.

$(GI)_{tor}$ is the torsional stiffness per unit width of the panel. For a homogeneous orthotropic panel, $(GI)_{tor} = Gt^3/3 + [\nu_{xy}(EI)_x + \nu_{yx}(EI)_y] \approx Gt^3/3$.

β_1 $= \frac{\ell}{a} \sqrt[4]{(EI)_x/(EI)_y}$. For an isotropic panel, $\beta_1 = \ell/a$.

β_2 $= 0.5(GI)_{tor}/\sqrt{(EI)_x(EI)_y}$. For an isotropic panel, $\beta_2 = 2G/E$.

a, ℓ, t see fig. 7.1.1b.

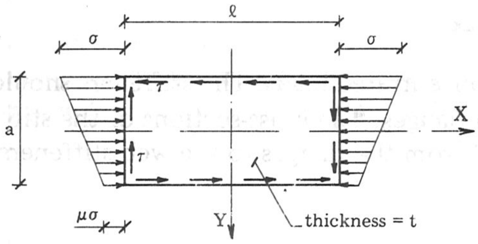


Fig. 7.1.1 b

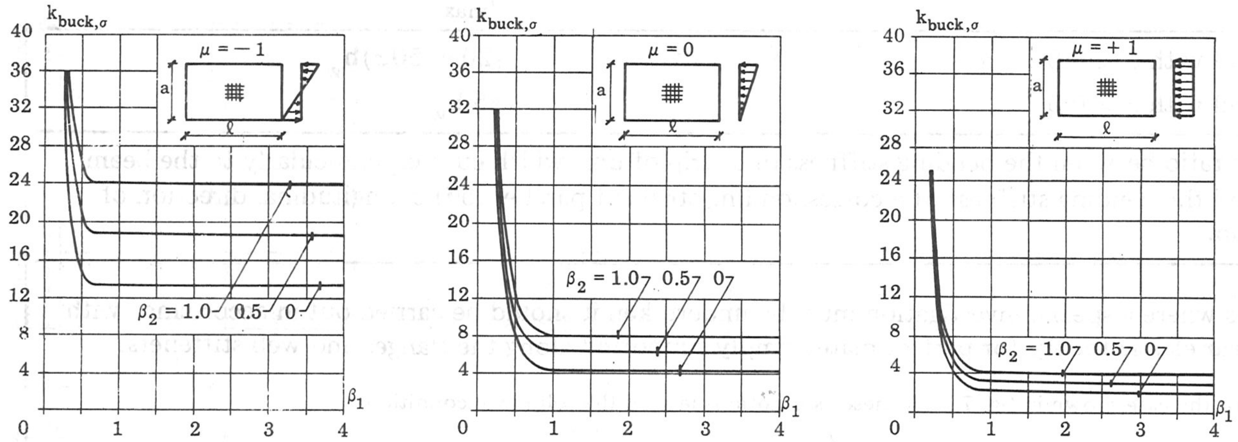


Fig. 7.1.1 c

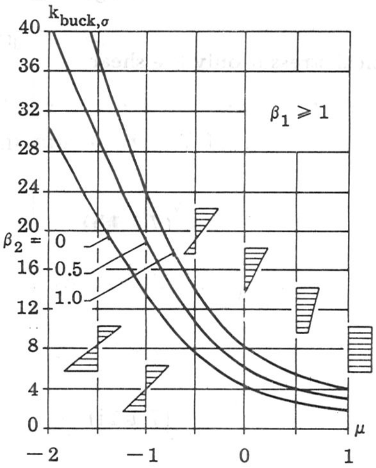


Fig. 7.1.1 d

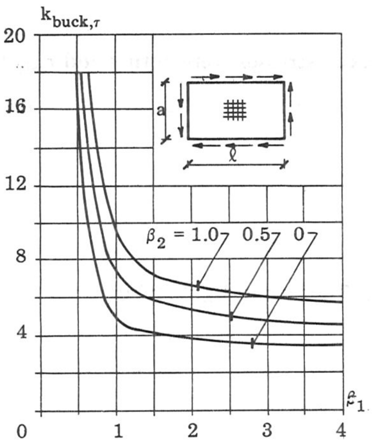


Fig. 7.1.1 e

In calculations of deflection the contributions from the shearing stresses in the webs should be taken into account.

7.1.2 Thin-flanged beams (stiffened plates)

The stresses may be calculated assuming a linear variation of strain over the depth and the stresses must in principle satisfy the conditions given in section 5.

The influence of the stresses being non-uniformly distributed over the flange width should be taken into consideration. Unless otherwise proved the calculations should be based on a number of I-beams (taking the load on a width of $b_f + b_w$) or U-beams (taking the load on a width of $0.5\ b_f + b_w$) with an effective flange width, b_e, see fig. 7.1.2, where

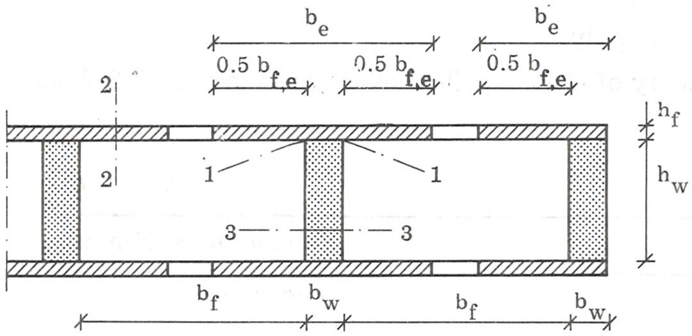


Fig. 7.1.2

$$b_e = b_{f,e} + b_w \qquad (7.1.2a)$$

or

$$b_e = 0.5\, b_{f,e} + b_w \qquad (7.1.2b)$$

respectively.

The free effective width, $b_{f,e}$ ($\leqslant b_f$), is given in table 7.1.2.

Unless an investigation into the buckling instability of the compression flange is made, $b_{f,e}$ should not be put higher than b_{max}, also given in table 7.1.2.

Table 7.1.2

Flange	$b_{f,e}/\ell$	b_{max}
Plywood with fibre direction in extreme plies		
parallel to the web	0.1	20 h_f
perpendicular to the web	0.1	25 h_f
Particle or fibre board with random fibre orientation	0.2	

ℓ is the span, however, for continuous beams ℓ is the distance between the points with zero moment

The buckling investigation of the compression flange should be made in accordance with section 7.1.1.

The buckling instability of webs made of plywood or other sheet materials should be investigated in accordance with section 7.1.1, unless $h_w \leqslant 0.5\, h_{max}$, where h_{max} is given in table 7.1.1.

7.1.3 I- and box columns, spaced columns, lattice columns

The relevant parts of section 5.1.1.9, 7.1.1 and 7.1.2 apply to I- and box columns.

The requirements for solid columns (section 5.1.1.7) apply to spaced columns and lattice columns, but furthermore, the deformation due to shear and bending in packs, battens, shafts and flanges and to the extension of the lattice should be taken into consideration.

Design methods for spaced columns are given in Annex 7B and for lattice columns in Annex 7C.

7.2 Mechanically jointed components

If the cross-section of a structural member is composed of several parts connected by mechanical fasteners consideration should be given to the influence of the slip occurring in the fasteners.

In addition the recommendations of sections 5 and 7.1 apply.

Calculations may be carried out according to the theory of elasticity. The values given in table 7.2 should be used for slip modulus.

Table 7.2 *Provisional*

Fastener	Slip modulus (N/mm)
Round nails with $d < 5$ mm	$0.02\ E_0 d^*$
Round nails with $d > 5$ mm	$0.1\ E_0{}^*$
Bolts with toothed connectors	$1.3\ E_0$

E_0 is the modulus of elasticity of the timber in N/mm^2. d is the diameter in mm for round nails or the side measurement for square nails.

* For square nails 15% higher values are allowed.

For beams a design method for a number of cross-sections is given in Annex 7A and for columns in Annex 7A-B-C.

7.3 Trusses

Trusses may be analysed as frame structures where the influence of initial curvature of the elements, eccentricities, deformations of elements, slip and rotation of the joints, strength and stiffness variation in members and joints, and stress redistribution are taken into consideration in the determination of the resultant stresses.

As an alternative a simplified calculation after the guidelines given in Annex 7D is permitted.

8. CONSTRUCTION

8.0 General

The recommendations given in this chapter are necessary conditions for the applicability of the design rules given in this code.

Timber structures shall be so constructed that they conform with the principles and practical considerations of the design.

Materials for the structures shall be applied, used or fixed so as to adequately perform the functions for which they are designed.

Workmanship in fabrication, preparation and installation of materials shall conform in all respects to accepted good practice.

8.1 Materials

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. In particular they should not be subject to prolonged exposure to the weather or to conditions conducive to fungal or insect attactk.

Timber which is damaged, crushed or otherwise misused should not be used for structural work.

Before construction timber should be seasoned as near as practicable to the moisture content appropriate to its climatic condition in the completed structure.

The limitations on bow in most national stress grading rules are inadequate for the selection of material for columns and beams where lateral instability may occur. Particular attention should therefore be paid to the straightness of columns (e.g. limiting bow to approximately 1/300 of the length), and to beams where lateral instability may occur (e.g. limiting bow to approximately 1/200 of the length). It may also be necessary to introduce more stringent limits on some other members, e.g. twist for torsional members.

8.2 Machining

The size, shape and finish of all timber and other materials shall conform with the detailed design drawings and specifications for the structure.

The cutting of timber after preservative treatment should be avoided. However, when it is unavoidable, and untreated timber is exposed, a liberal application of preservative should be made to the exposed surfaces.

8.3 Joints

Fasteners shall be placed in conformity with the drawings. The minimum distances given in section 6.1.1 - 6.1.4 should be complied with.

Wane, splits, knots or other defects are not allowed in joints to such a degree that the load-carrying capacity of the joints is reduced.

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.

Unless otherwise stated slant nailing should be carried out in conformity with fig. 6.1.1.2b.

Bolt holes may have a diameter not more than 1 mm larger than the bolt. Washers with a side length of at least 3d or a diameter of at least 4d and a thickness of at least 0.3d (d is the bolt diameter) should be used under the head and nut.

Bolts shall be tightened so that the members fit closely, and they shall be tightened up if necessary when the timber has reached its equilibrium moisture content.

The minimum dowel diameter is 8 mm. The tolerances on the dowel diameter are — 0/+ 0.1 mm and the pre-bored holes in the timber members should have the same diameter as the dowel.

A bolt or screw shall be placed through the centre of each connector. Connectors should fit tightly in the grooves.

When using toothed plates the teeth should be completely pressed into the timber. Impressing should normally be carried out with special press tools or special clamping bolts with washers large and stiff enough to protect the timber from damage. If the bolt is used for impressing it should be carefully checked that the bolt has not been damaged in tightening. The washer should in this case have at least the same side length as the connector and the thickness should at least be 0.1 times the side length.

Lag screw holes shall be pre-drilled and treated as follows:

a. The lead hole for the shank shall have the same diameter as the shank and the same depth as the length of the unthreaded shank.

b. The lead hole for the threaded portion shall have a diameter determined by the relative density of the species and by the length and diameter of the screw.

c. Soap, or other non-corrosive lubricant (but not ordinary petroleum) may be used to facilitate insertion of the screw.

d. Screws are to be inserted by turning with a suitable wrench, not by driving with a hammer.

8.4 Assembly

Assembly should be in such a way that unintentional stresses do not occur. Members which are warped, split or badly fitting at the joints should be replaced.

8.5 Transportation and erection

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 it must be proved that this is permissible and it must be taken into consideration that such loads may have dynamic effects. 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.

9. FIRE RESISTANCE

9.0 General

The recommendations in this chapter give methods of assessing the performance of timber members in fire.

Charring may be assumed to occur at a steady rate and the timber beneath the charred layer may be assumed to retain its original strength. These assumptions make it possible to predict the performance of timber components in fire.

9.1 Rates of charring

9.1.1 Flame retardants

Timber treatments, including impregnation to retard the surface spread of flame, should normally not be assumed to affect the charring rate.

9.1.2 Solid members

Calculation of the residual section of solid members should be based on the values given in table 9.1. These values should be modified in the case of fully exposed columns and tension members as set out in 9.2.1. respectively.

Table 9.1 Notional rate of charring for the calculation of residual section for periods between 15 min. and 90 min.

Species	Charring rate, mm/min.
Western red cedar	0.83
Other softwoods	0.67
Oak, utile, keruing (gurjun), teak, greenheart, jarrah	0.50

Notional charring rates for particular species and for longer periods of time may be established.

9.1.3 Glued laminated members

The charring rates given in 9.1.2 may be applied to members laminated with the following thermosetting phenolic and aminoplastic synthetic resin adhesives: resorcinol-formaldehyde, urea-formaldehyde, and urea-melamine-formaldehyde.

When other adhesives are to be used their performance in fire should be verified by tests.

9.1.4 Finger joints

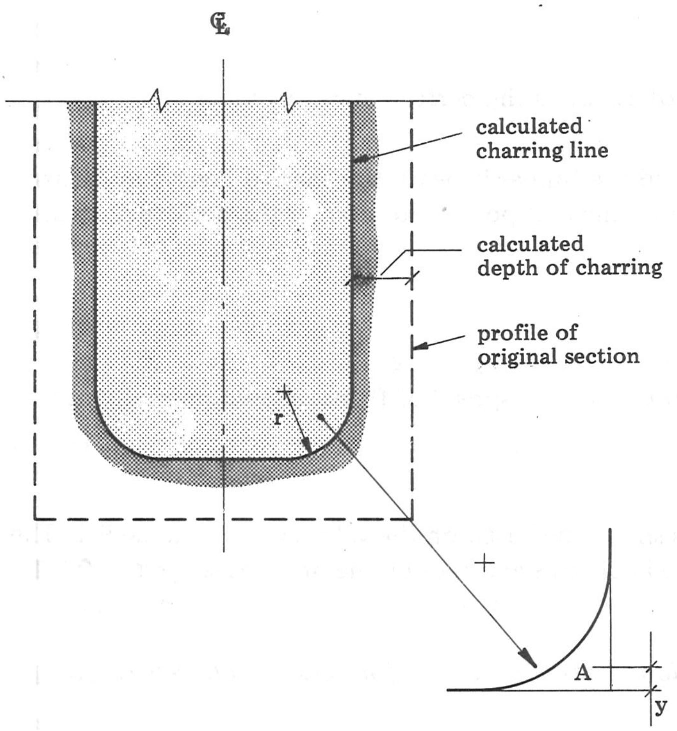
Finger joints manufactured using the adhesives given in 9.1.3 may be considered to char at the rates given in table 9.1.2.

9.1.5 Sections and joints with metal fasteners

The charring rates in 9.1.2 may only be applied to the section as a whole if metal fasteners on which the structural performance of the built-up member depends are fully protected from the effects of fire (see 9.2.2). Where such protection is not given, local structural weaknesses may occur and the member can only be assessed for fire performance by applying the residual section calculation assuming charring on all faces of each component of the built-up member, or by conducting a fire resistance test.

9.1.6 Increased rate of charring on exposed arrises

Arrises will become progressively rounded during fire exposure. The radius of this rounding will be equal to the depth of the charring and the centre will lie equidistant from the two aspect faces at a distance of twice the charring depth, see fig. 9.1.



The radius of arris rounding, r, equals the calculated depth of charring.

The area of the section lost due to rounding will be

$$A = 0.215\ r^2$$

and the centre of gravity of this area will lie at a distance from either side of

$$y = 0.223\ r$$

Fig. 9.1 Radius of arris rounding.

For periods of exposure not exceeding 30 min., where the least dimension of the residual section is not less than 50 mm, rounding is insignificant and may be disregarded.

9.2 Design requirements

9.2.1 Tension members, compression members and beams

9.2.1.0 General

It should be shown, either by fire test or by calculation, that the residual section of a member will support the normal service loads for a time prescribed by the relevant public authority.

The strength and deflection of a member should be calculated in accordance with the relevant parts of clause 5.1.1 using the residual section, the characteristic stress and the mean or minimum modulus of elasticity, as appropriate. When the initial minimum dimension of a section is less than 70 mm the characteristic strength should be multiplied by 0.9.

The residual section should be determined by subtracting from the appropriate faces the notional amount of charring assumed to occur by table 9.1 during the required period of fire exposure; making allowance where necessary for the rounding on the exposed arrises. For tension and compression members the additional provisions of clauses 9.2.1.1 and 9.2.1.2 also apply.

9.2.1.1 Tension members

To determine the residual section of a tension member the rates of charring given in table 9.1 should be multiplied by 1.25.

9.2.1.2 Compression members

A column that is exposed to the fire on all faces (including a column which abuts on or forms part of a wall that does not have fire resistance, as in figures 9.2b and 9.2d) should be assumed to char equally on all faces during the whole period of fire exposures. To determine the residual section of such columns, the rates of charring given in table 9.1 should be multiplied by 1.25.

Where a column abuts on or forms part of a wall which provides fire resistance from either side not less than that of the column, charring on all faces is unlikely. Calculations should therefore be based on charring of the column occurring on the side of the wall on which the column has the greater surface exposure, using the rates of charring given in table 9.1 (see fig. 9.2a).

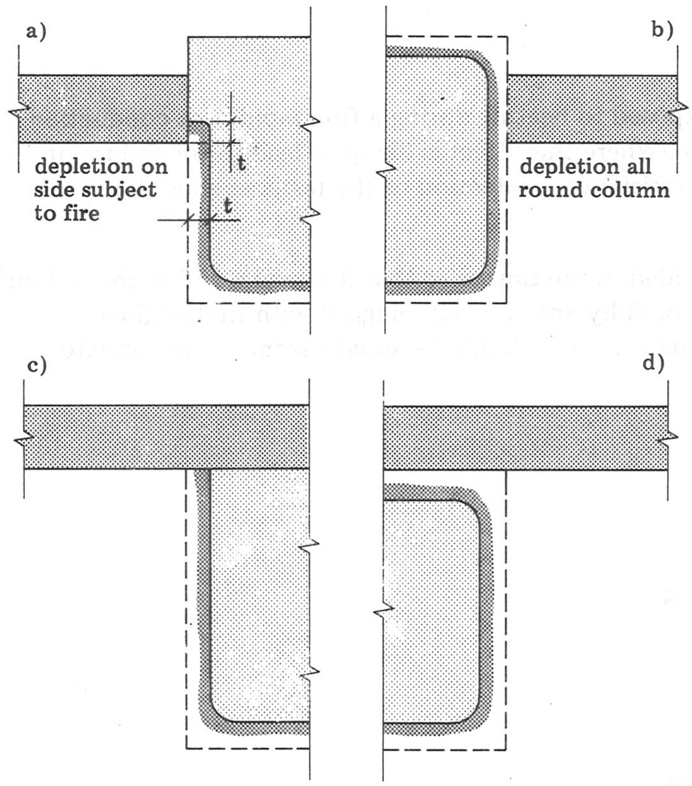


Fig. 9.2. a) and c) Wall having fire resistance not less than the column.
b) and d) Wall having less fire resistance than the column.

Where a column abuts on or forms part of a wall which is required to provide fire resistance from one side only (such as in an external wall) and which has fire resistance not less than the column, only charring on the faces of the column which can be exposed to fire need be considered and the rates of charring given in table 9.1 should be used. In establishing the vulnerable column faces, due regard should be given to the protection afforded by the walling materials.

In the last two cases care should be taken to ensure that the junctions between the wall and the column will be adequate as a barrier to fire so that the integrity of the construction is unimpaired.

No angular restraint at the ends (as distinct from positional restraint) should be assumed in determining the effective length of residual column sections unless consideration of the residual joint (as indicated in 9.2.2) shows that a degree of restraint would be provided.

The maximum slenderness ratio based on the residual section should not exceed 200 and the stress modification factor for long-term loading for the slenderness ratio of the residual column should be derived from clause 5.1.7.

9.2.2 Joints

9.2.2.1 General

The charring rates given in table 9.1 may be applied provided that in all cases the faces of the abutting pieces of timber are held in close contact and that special attention is paid to the placement or protection of metal fasteners and components (see 9.2.2.2).

The methods of calculation given in 9.2.1 are directly applicable to the performance of individual flexural, tension and/or compression members. Junctions between members may be particularly vulnerable to the effects of fire and require special consideration. Where a compressive force is transferred by direct timber-to-timber bearing, the loss in strength of the joint is unlikely to be significant where members have been designed in accordance with the recommendations of this code.

However, where a structure is designed to have joints that transfer moments or forces from one member to another, special account should be taken of the behaviour of such joints. An assessment should be made of the residual timber after the specified period, with particular attention to the effects of any metal connectors and the probability of rounding at abutting arrises (as indicated in 9.1.6). In redundant structures, charring may alter the relative stiffness of various parts of the structure and result in a redistribution of forces; and account should be taken of complete or partial yielding of the joints as this may change the structural action. The structure with redistributed forces should be assessed for fire resistance as detailed in 9.2.1.

9.2.2.2 Metal fasteners

Where any part of a nail, screw or bolt becomes exposed to heating during a fire, rapid heat conduction will lead to localised charring and loss of anchorage. Where this effect is likely to lead to the failure of a structural member which is required to have fire resistance, protection of the fastener should be provided by any one of the following methods.

a) Ensuring that every part of the fastener is embedded in the timber so that it remains within the residual section, as shown in fig. 9.2e. Any holes should be fully and securely plugged with timber glued in position. Advice on the use of alternative plugging materials should be sought from an appropriate authority.

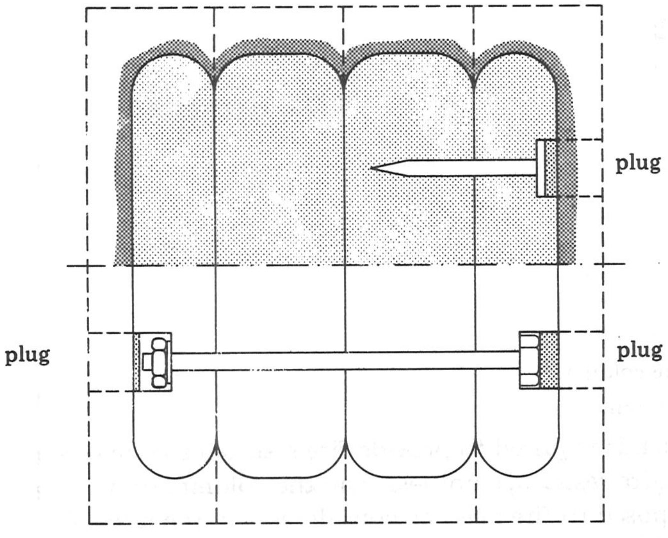


Fig. 9.2e. Sections and joints with metal fasteners. All arrises of each piece are assumed to be depleted. All metal to be within residual section.

b) Covering the exposed part of the fastener with a suitable protecting material, e.g. timber, plasterboard, asbestos insulation board, or equivalent. Special attention should be paid to the fixing of such protection to ensure that it remains in position for the required period of fire resistance. Nails, screws or staples may be used in this case to fix this insulation.

c) Any appropriate combination of the methods outlined in a) and b).