CIB
STRUCTURAL TIMBER DESIGN CODE

CIB Report 1983 Publication 66

WORKING GROUP W18
TIMBER STRUCTURES

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Sixth edition, January 1983

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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 four 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 and representatives of Confédération Européen des Industries du Bois (CEI-bois), on the basis of comments from CIB-W18, ISO/TC 165 - Timber Structures, and CEI-bois.

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.

RELATED DOCUMENTS

The Code makes reference to several documents produced by a joint 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 A: Punched Metal Plate Fasteners.
Annex B: Nails.

Other documents relating to the sampling of test specimens and the analysis of test data to produce characteristic values 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 11. Notes on the deviation and interpretation of some particular clauses in the Code are contained in Annex 02, published under separate cover (August 1980).

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 among others that

- the actual conditions of use of the structure during its life do not depart significantly from those specified during the design stage,
- the structure, by design or use of suitable materiale or by impregnation, is protected against attack by fungi, insects, shipworm, gribble, etc., and its integrity is ensured by correct maintenance.

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, strength and elastic moduli	Pascal	Pa ($1\,Pa = 1 N/m^2$, $1 MPa = 1 N/mm^2$)

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
u	Displacement
x y z	Coordinates
α	Angle
β	Factor
γ	Partial coefficient (load factor, material factor)
η	Factor
λ	Slenderness ratio
ν	Poisson's ratio
ρ	Relative density

Main symbols (continued)

σ	Normal stress
τ	Shear stress
φ	Ratio

Subscripts

c	Compression
crit	Critical
d	Design
E	Euler
ef	Effective
f	Flange Load (on γ)
in	Inner
inst	Instability
k	Characteristic
m	Bending Material property (on γ)
ul	Outer
t	Tension
tor	Torsion
v	Shear
vol	Volume
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 b): $\sigma_t \leqslant k_{vol,90} \cdot f_{t,0}$ is to be read as $\sigma_{t,d} \leqslant k_{vol,90} \cdot f_{t,0,d}$.

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.

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.

At normal temperatures the equilibrium moisture content in most softwoods will not exceed 0.12 in Moisture class 1 and 0.18 in Moisture class 2.

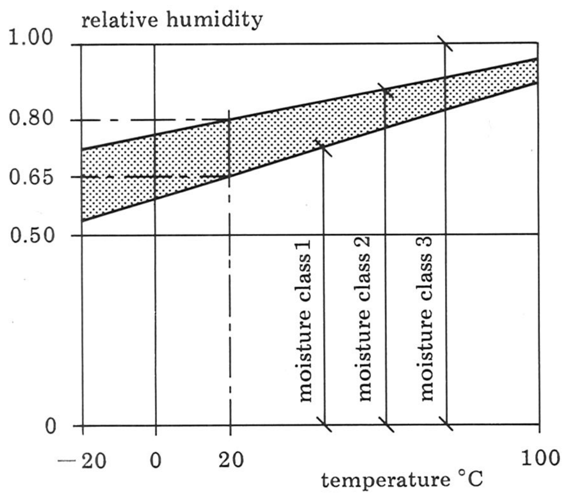


Fig. 2.2

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	Order of duration	
Long-term	$> 10^5$ hours	(> 10 years)
Medium-term	10^4 hours	(1 year)
Short-term	10^2 hours	(1 week)
Very short-term	< 10 hours	
Instantaneous		

For an intermittent load the load duration class may in certain cases be determined corresponding to the accumulated loading time.

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),
- 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

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

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 among others 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

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

or

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

The design load combination should be given as

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

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

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

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

For serviceability limit states $\gamma_m = 1.0$ 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 57 - TSB).

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 be sampled in accordance with Annex 41 (in preparation), and orientated at random in the testing machine.

4.1.1 Standard classes

Annex 42 describes a system of standard strength classes and standard density classes based on ISO/TC 165 . . . (in preparation).

4.1.2 Characteristic values and mean elastic moduli

Annex 43 contains characteristic values and mean elastic moduli for a number of structural species and grades commonly used in Western Europe.

Annex 42 contains similar values for the standard classes mentioned in section 4.1.1.

4.2 Finger jointed structural timber

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.

4.3 Glued laminated timber

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

4.4 Wood-based sheet materials

Testing should 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-57-TSB (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 ⩽ 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 are found by

- multiplying the characteristic values or the mean elastic moduli by a modification factor k_{mod} taking into account the influence of moisture content and loading time, and
- dividing by the partial coefficient γ_m, see section 3.2.2.

For the standard classes covered by Annex 42 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.

Design values of strength and stiffness should be used in the design equations given in this chapter, unless specifically stated otherwise.

This means e.g. that equation (5.1.1 a) should be read as $\sigma_{t,d} \leqslant f_{t,0,d}$.

Deformations greater than 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 k_{mod} to characteristic and mean values

	Values for strength calculations 1) 2)		Values for deformation calculations1)		
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	0.70 (0.50)	0.60 (0.40)	1	0.8	0.7
Short-term	0.80 (0.70)	0.70 (0.60)	1	0.8	0.7
Very short-term	0.95 (0.90)	0.80 (0.75)	1	0.8	0.7
Instantaneous	1.1 (1.1)	0.95 (0.95)			

1) Where a load case consists of loads belonging to different load-duration classes the factors corresponding to the shortest load-duration may be used by strength calculations. By deformations the deformations are calculated for each load with the appropriate factor.

2) 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:

$$\sigma_t \leqslant f_{t,0} \qquad (5.1.1\,a)$$

and for tension perpendicular to the grain

$$\sigma_t \leqslant k_{vol,90} f_{t,90} \qquad (5.1.1b)$$

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

$$k_{size,90} = \begin{cases} 1 & \text{for} \quad V \leqslant 0.02\ m^3 \\ \dfrac{0.45}{V^{0.2}} & \text{for} \quad V > 0.02\ m^3 \end{cases} \qquad (5.1.1c)$$

5.1.2 Compression without column effect

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

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

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.

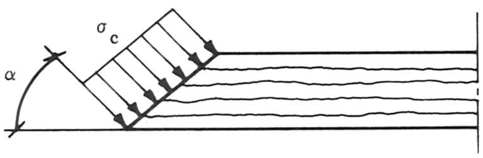
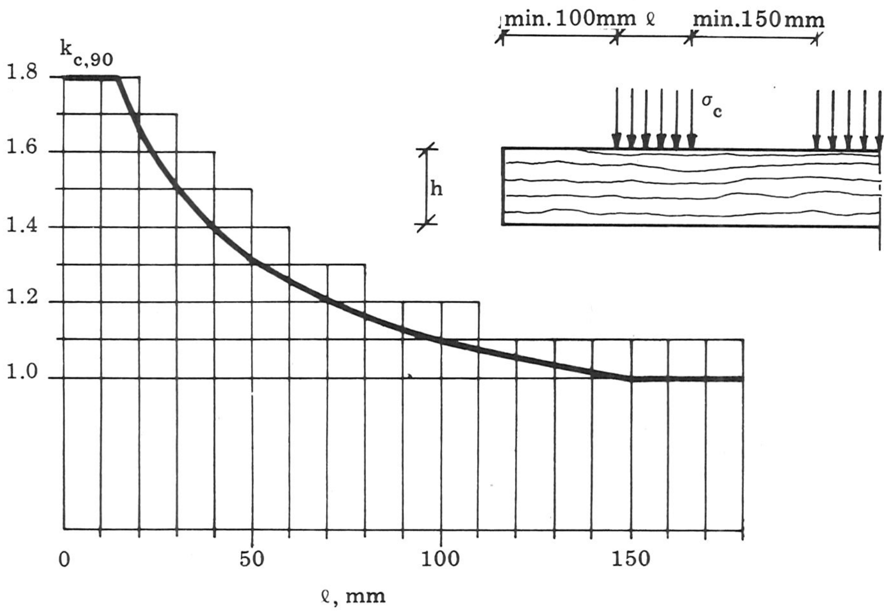


Fig. 5.1.2a



$$k_{c,90} = \sqrt[4]{150/\ell}$$

$$1 \leqslant k_{c,90} \leqslant 1.8$$

Fig. 5.1.2b. The factor $k_{c,90}$.

For bearings on the side grain ($\alpha = 90°$) the stresses should satisfy the following condition:

$$\sigma_c \leqslant k_{c,90} f_{c,90} \tag{5.1.2b}$$

For bearings located at least 100 mm from the end and 150 mm from other loads, $k_{c,90}$ may be taken from fig. 5.1.2b. In other cases $k_{c,90} = 1$.

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

The deformation perpendicular to the grain u can be estimated from

$$u = k_{u,90} \frac{\sigma_{c,90}}{E_{90}} h$$

where

$$k_{u,90} = \begin{cases} \dfrac{\ell}{\ell + 2h} & \text{for } a \geqslant h \\ \dfrac{\ell}{\ell + 2h}\left(3 - 2\left(1 - \dfrac{a}{h}\right)^2\right) & \text{for } a < h \end{cases} \tag{5.1.2c}$$

$k_{u,90}$ is shown in figure 5.1.2c for $a \geqslant h$.

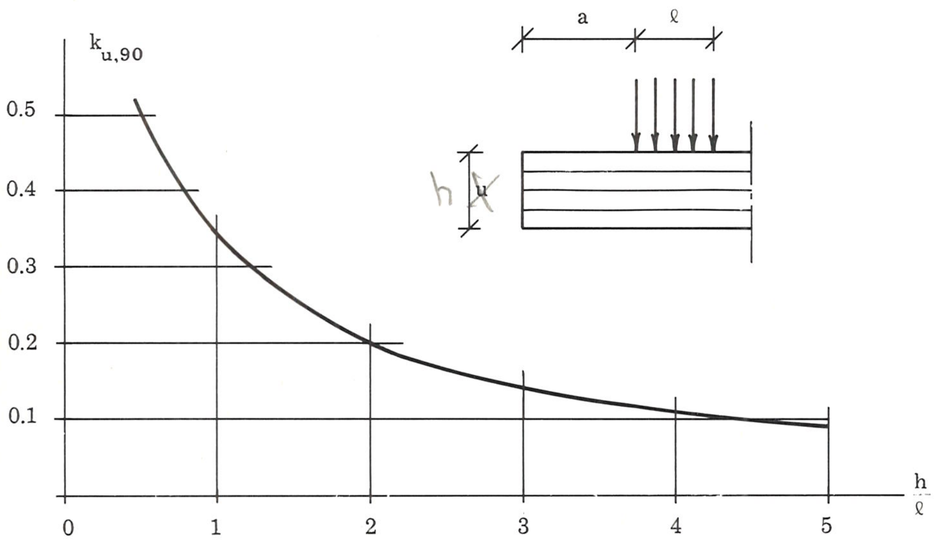


Fig. 5.1.2c. $k_{u,90}$ for $a \geqslant h$.

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 on the supporting structure.

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.

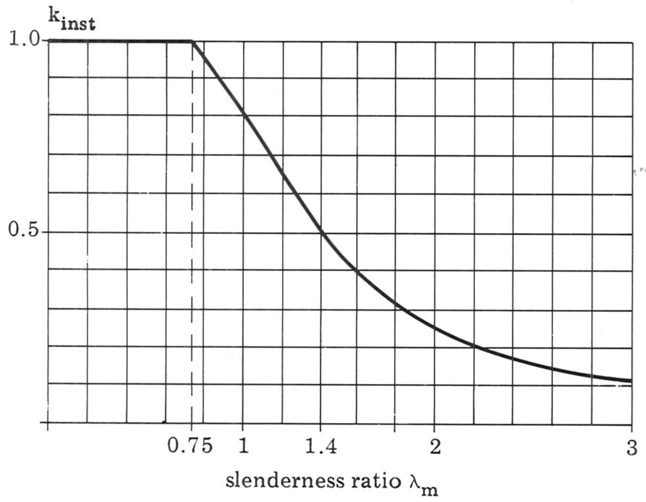
The strength reduction may be disregarded, i.e. $k_{inst} = 1$, if displacements and torsion are prevented at the supports and if

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

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

$$\lambda_m = \sqrt{\frac{\ell_e h}{\pi b^2}\frac{f_{m,d}}{E_{0,d}}\sqrt{\frac{E_{0,mean}}{G_{mean}}}} \qquad (5.1.3c)$$

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, the shear force may be calculated according to the reduced influence line shown in figure 5.1.4a.

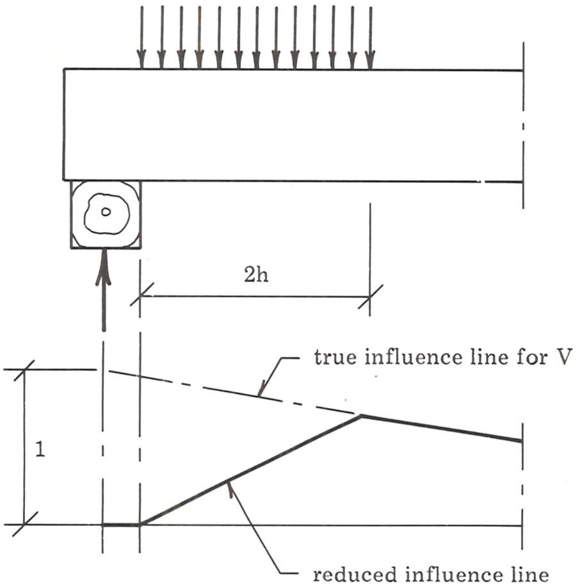


Fig. 5.1.4a

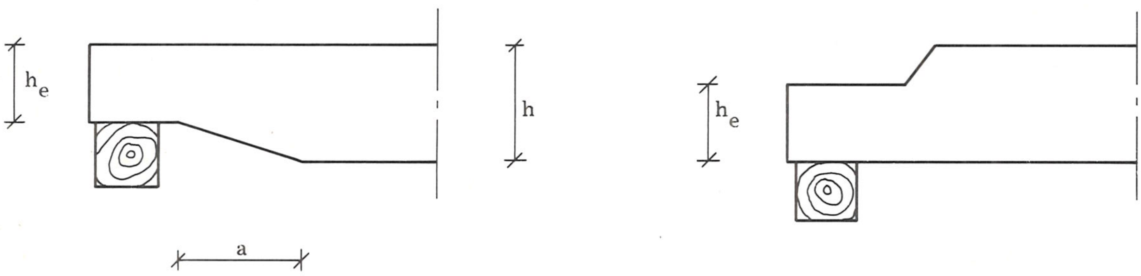


Fig. 5.1.4b

For beams notched at the ends, see fig. 5.1.4b, 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

$$\tau \leqslant [1 - \frac{h - h_e}{h}(1 - \frac{a}{3(h - h_e)})] f_v = (\frac{h_e}{h} + \frac{a}{3h}) f_v \qquad (5.1.4b)$$

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 Tapered beams

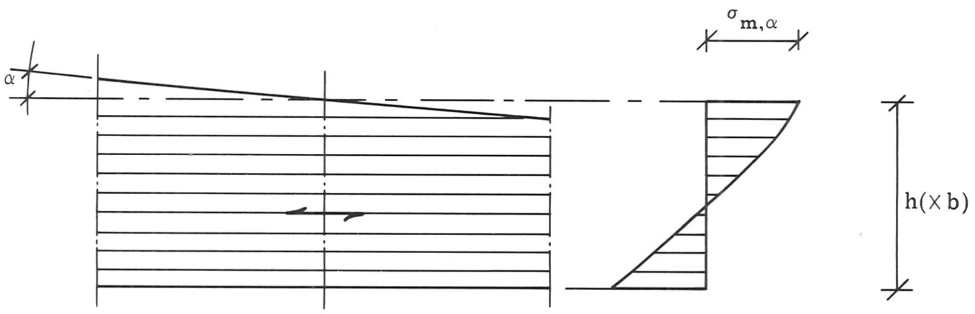


Fig. 5.1.6.1

For tapered beams with rectangular cross-section the bending stresses in the outmost fibres should satisfy the following conditions:

If $\sigma_{m,\alpha}$ is a tensile stress

$$\sigma_{m,\alpha} \leqslant \frac{f_m}{\sqrt{\cos^4\alpha + \sin^2\alpha\cos^2\alpha(\frac{f_m}{f_v})^2 + \sin^4\alpha(\frac{f_m}{f_{t,90}})^2}} \qquad (5.1.6.1a)$$

If $\sigma_{m,\alpha}$ is a compressive stress

$$\sigma_{m,\alpha} \leqslant \frac{f_m}{\sqrt{\cos^4\alpha + \frac{1}{2}\sin^2\alpha\cos^2\alpha(\frac{f_m}{f_v})^2 + \sin^4\alpha(\frac{f_m}{f_{c,90}})^2}} \qquad (5.1.6.1b)$$

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

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

where σ_m are the bending stresses calculated without regard to initial curvature and deflections, and k_c 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

$$e = \eta r \lambda \qquad (5.1.7b)$$

where r is the core radius.

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

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

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

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

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.

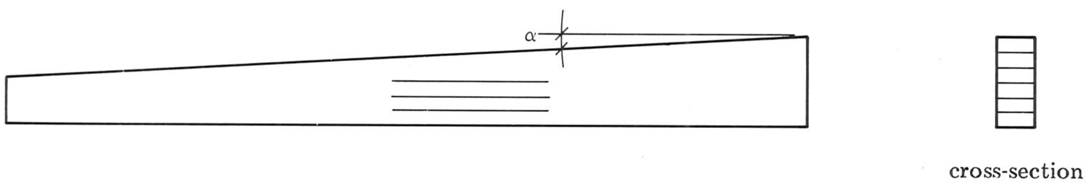


Fig. 5.2.1a. Single-tapered beam.

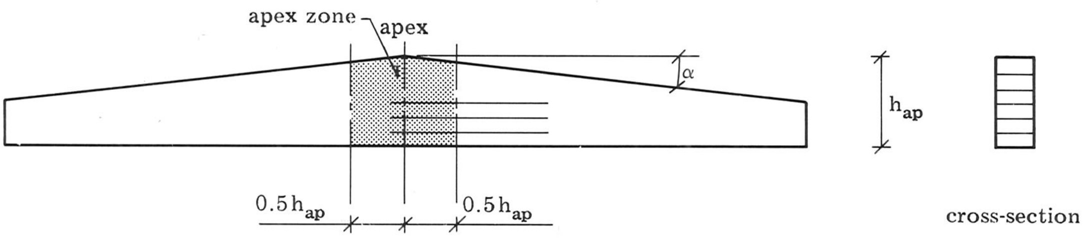


Fig. 5.2.1b. Double-tapered beam.

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

$$\sigma_m \leqslant f_m \qquad (5.2.1a)$$

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

$$\sigma_t \leqslant k_{vol,90} f_{t,90} \qquad (5.2.1b)$$

where

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

V is the volume in m^3 of the apex zone (corresponding to the shaded area in fig. 5.2.1b).

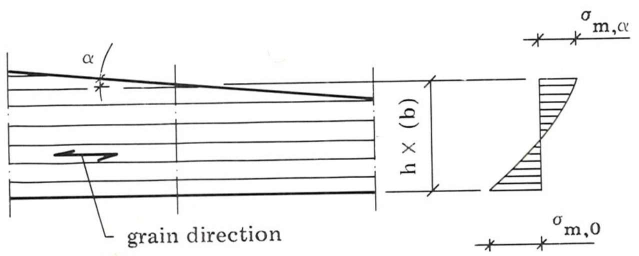


Fig. 5.2.1c

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 $\sigma_{m,\alpha}$ should satisfy the conditions (5.1.6.1a) - (5.1.6.1b).

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

$$\sigma_m = (1 + 1.4\tan\alpha + 5.4\tan^2 \alpha) \frac{6M_{ap}}{bh_{ap}^2} \qquad (5.2.1f)$$

and the greatest tensile stresses perpendicular to the grain as

$$\sigma_t = 0.2\tan\alpha \frac{6M_{ap}}{bh_{ap}^2} \qquad (5.2.1g)$$

M_{ap} is the bending moment at the apex.

5.2.2 Curved beams

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

The ratio between the radius of curvature, r_t, and the laminae thickness, t, should be greater than 125. For $r_t/t < 240$ 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_r, where

$$k_r = 0.76 + 0.001 \frac{r_t}{t} \qquad (5.2.2a)$$

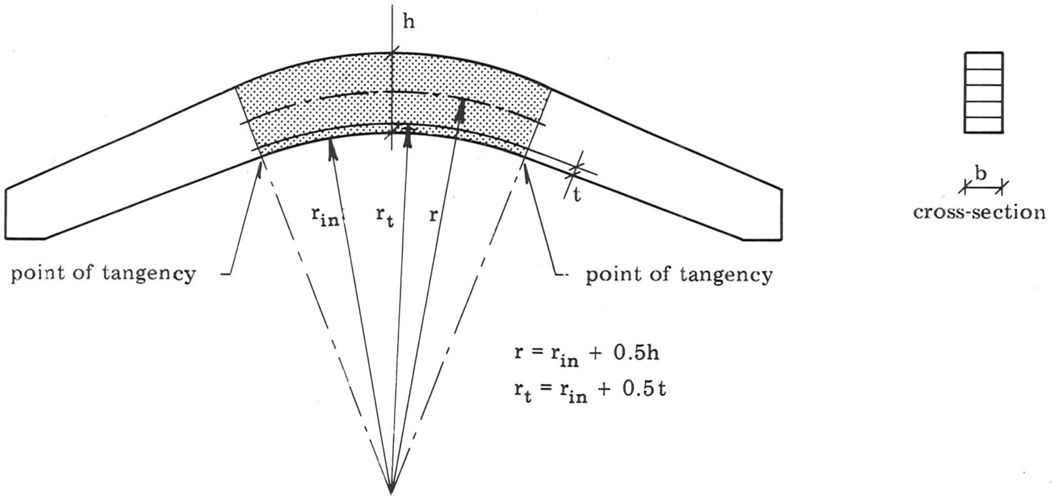


Fig. 5.2.2

In sharply curved beams (i.e. the ratio between minimum mean-radius of curvature, r, 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

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

where

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

The bending stresses in the outermost fibre may be calculated by the usual expression ($\sigma_m = \frac{6M}{bh^2}$).

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

$$\sigma_t \leqslant k_{vol,90} f_{t,90} \tag{5.2.2d}$$

where

$$k_{vol,90} = \begin{cases} \frac{0.65}{V^{0.2}} & \text{for uniformly distributed load } \leq 1{,}0 \quad \text{oder } V > 0{,}12\ m^3 \\ \frac{0.5}{V^{0.2}} & \text{for other loading } \leq 1{,}0 \quad V > 0{,}03\ m^3 \end{cases} \tag{5.2.2e}$$

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

$$\sigma_t = \frac{1.5\,M}{r\,bh} \tag{5.2.2f}$$

5.2.3 Cambered beams

This section applies to cambered beams with rectangular cross-section as shown in fig. 5.2.3a and with $r/h_t < 30$. For $r/h_t \geqslant 30$ section 5.2.1 applies. The shear forces are assumed to be small near the apex.

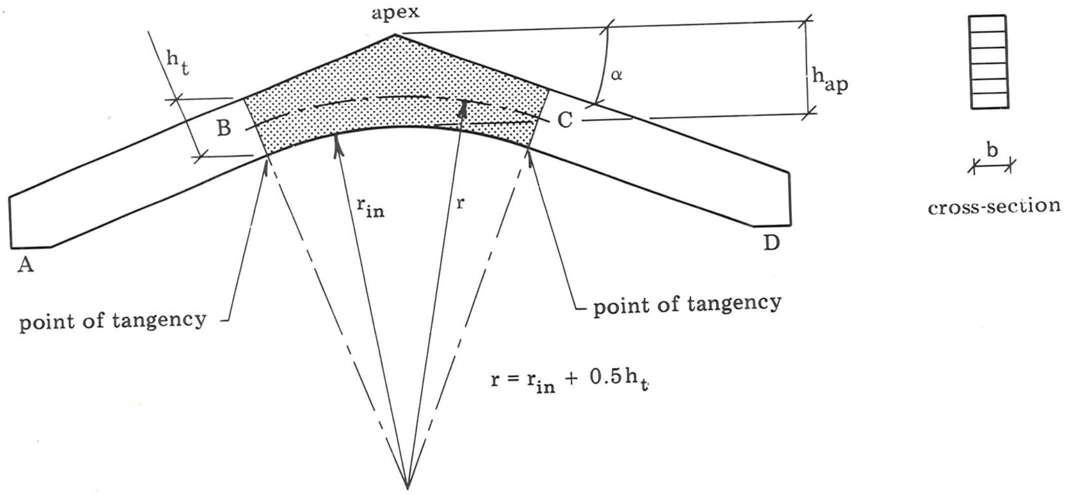


Fig. 5.2.3a. Cambered beam.

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

$$k_{vol,90} = \begin{cases} \dfrac{0.9}{V^{0.2}} & \text{for uniformly distributed load} \quad \leq 1{,}0 \text{ oder } V > 0{,}24 \\ \dfrac{0.6}{V^{0.2}} & \text{for other loading} \quad \leq 1{,}0 \qquad V > 0{,}08 \end{cases} \tag{5.2.3a}$$

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 at the apex may be calculated as

$$\sigma_{m,ap} = k_m k_{in} \frac{6M_{ap}}{bh_t^2} \tag{5.2.3b}$$

$$\sigma_{t,ap} = k_t \frac{1.5M_{ap}}{rbh_t} \tag{5.2.3c}$$

M_{ap} is the bending moment at the apex, and k_{in} is given in equation (5.2.2c). k_m and k_t are given in figures 5.2.3b and 5.2.3c.

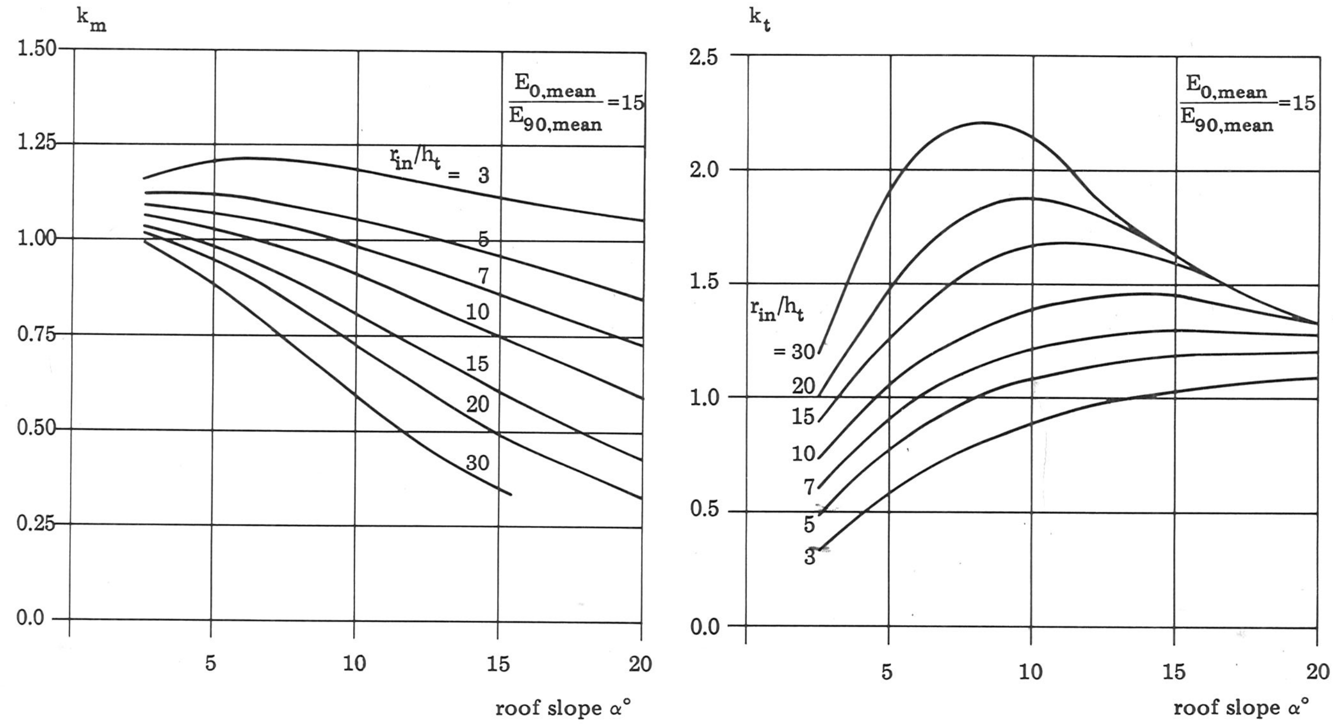


Fig. 5.2.3b. k_m and k_t for $E_{0,mean}/E_{90,mean}$ = 15.

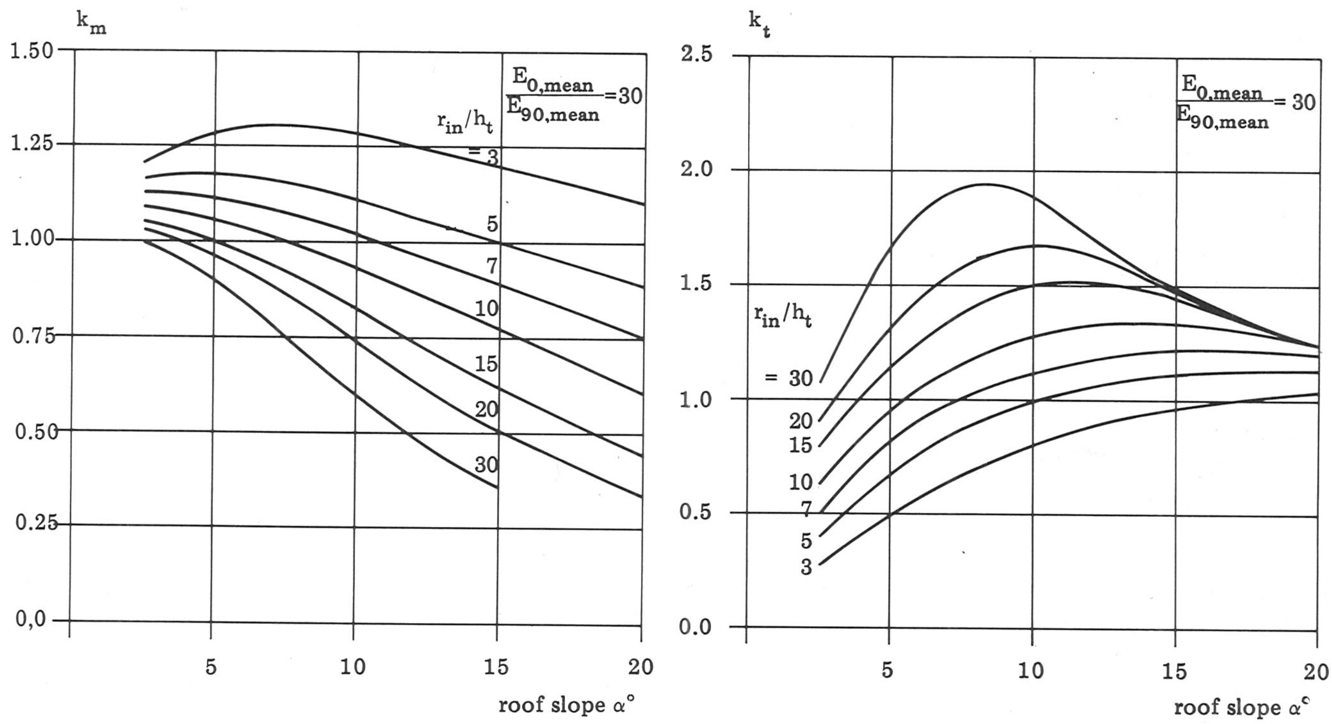


Fig. 5.2.3c. k_m and k_t for $E_{0,mean}/E_{90,mean}$ = 30.

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

$$F_k = k d^{\beta} \qquad (6.1.1.1a)$$

where d (in mm) is the diameter for round nails and the side measurement for square nails. The parameters k and β 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 in accordance with Annex B to the RILEM/CIB 3TT-1 document mentioned in section 6.1.0.

There should normally 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_{ef} is

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

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

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

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

$\beta = 1.7$

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

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

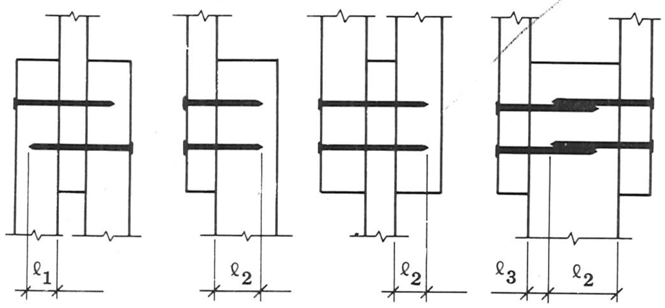


Fig. 6.1.1.1a

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 $d \leq 5$ 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)	$\ell_1 \geq 8d$
Other cases	
smooth nails	$\ell_2 \geq 12d$
annularly and spirally grooved nails	$\ell_2 \geq 8d$

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.

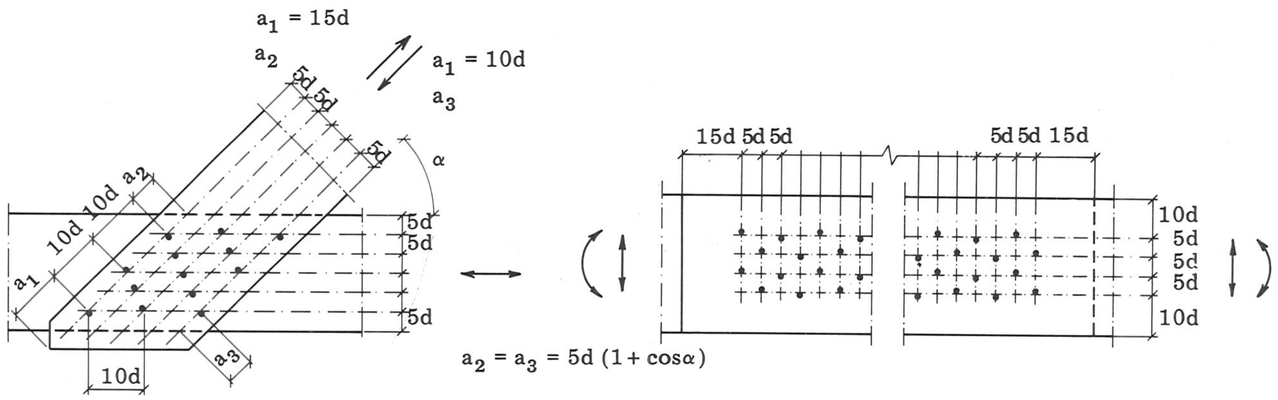


Fig. 6.1.1.1b. Minimum distances for timber-to-timber joints.

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 with characteristic density of 0.36 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

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.

$$F = \min \begin{cases} f_1 d \cdot \ell & (6.1.1.2a) \\ f_1 dh + f_2 d^2 \quad \text{for smooth nails} & (6.1.1.2b) \\ f_2 d^2 \quad \text{for annularly and spirally grooved nails} & (6.1.1.2c) \end{cases}$$

head pull-through

$\ell \geqslant 4d$ is assumed.

The parameters f_1 and f_2 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.

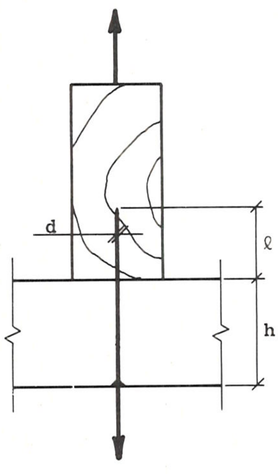


Fig. 6.1.1.2a

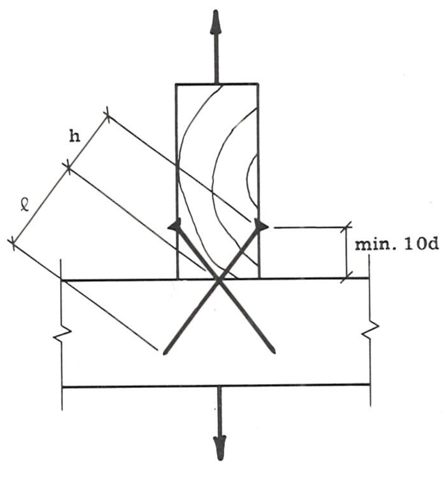


Fig. 6.1.1.2b

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 $f_1 = 6\rho^2$ and $f_2 = 400\rho^2$ may be assumed for ordinary round nails.

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).

$$F = \min \begin{cases} 18\rho(k_{\alpha,1}t_1 + k_{\alpha,2}t_2)d & \text{(only for two-member joints)} \quad (6.1.2.1a) \\ 35\rho k_{\alpha,2}t_2 d & \text{(only for three-member joints)} \quad (6.1.2.1b) \\ 70\rho k_{\alpha,1}t_1 d & (6.1.2.1c) \\ 75d^2\sqrt{\rho}\sqrt{(k_{\alpha,1} + k_{\alpha,2})/2}\sqrt{f_y/240} & (6.1.2.1d) \end{cases}$$

where

t_1 and t_2 are timber thicknesses in mm

d is the diameter in mm

$k_{\alpha,1}$ and $k_{\alpha,2}$ are factors taking into consideration the influence of the angle, α, between force and the direction of the grain

$$k_{\alpha,1}(\text{or } k_{\alpha,2}) = \frac{k_{90}}{k_{90}\cos^2\alpha + \sin^2\alpha} \qquad (6.1.2.1e)$$

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

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 $k_{\alpha,1}t_1 \leqslant k_{\alpha,2}t_2$.

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_{ef} is

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

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 furthest bolt, see fig. 6.1.2.1a.

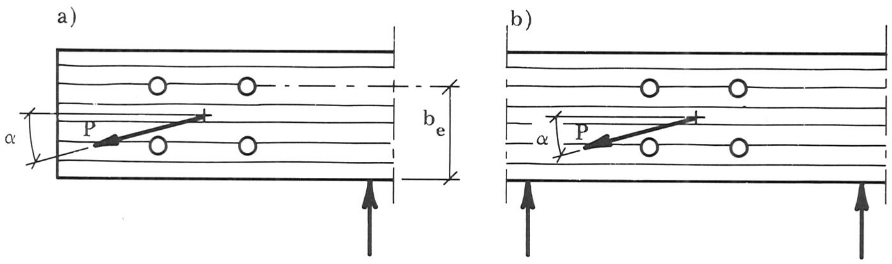


Fig. 6.1.2.1a. In case a) $V = P\sin\alpha$. In case b) $V = \frac{1}{2}P\sin\alpha$.

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

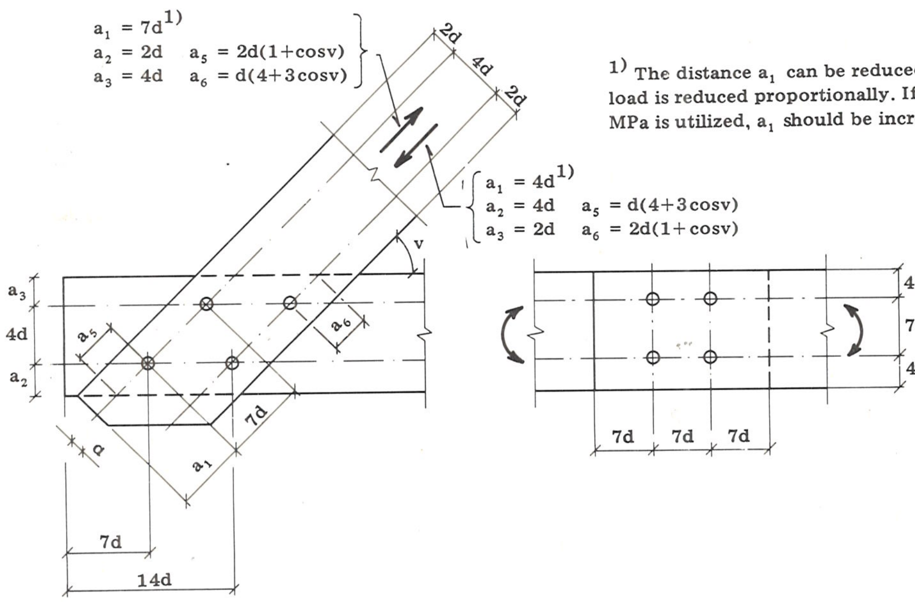
The slip at one third of the characteristic load is approximately 0.1 d + 1 mm.

Steel-to-timber joints

Where the side members are steel plates the loads calculated from the above formulae 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.

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.

6.1.2.2 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 (6.1.3.1a) - (6.1.3.1b)

$$F = \min \begin{cases} 70\rho k_{\alpha,1} td & (6.1.3.1a) \\ 75d^2 \sqrt{\rho}\sqrt{(k_{\alpha,1} + k_{\alpha,2})/2}\sqrt{f_y/240} & (6.1.3.1b) \end{cases}$$

where

- t is the thickness in mm of the timber
- d is the diameter in mm of the screw, measured on the smooth shank
- $k_{\alpha,1}$, $k_{\alpha,2}$ are factors, obtained from formula (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 ($k_{\alpha,1}$) and the member receiving the point ($k_{\alpha,2}$).

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

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

It is assumed that

- screws are turned, not hammered 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.

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

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

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

$$F = f_3(\ell_t - d)d \qquad (6.1.3.2a)$$

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_3 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

$$f_3 = (20 + \frac{50}{d})\sqrt{\rho}$$

$$F = (50 + 20d)\sqrt{\rho}\,(\ell_t - d) \qquad (6.1.3.2b)$$

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.

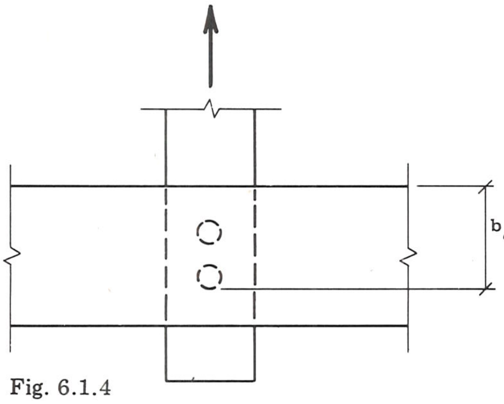


Fig. 6.1.4

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 is

$$F = F_1 + F_2$$

where F_1 is the load-carrying capacity of the bolt (or screw) calculated as stated in 6.1.2 (or 6.1.3), and F_2 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 A 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.

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.

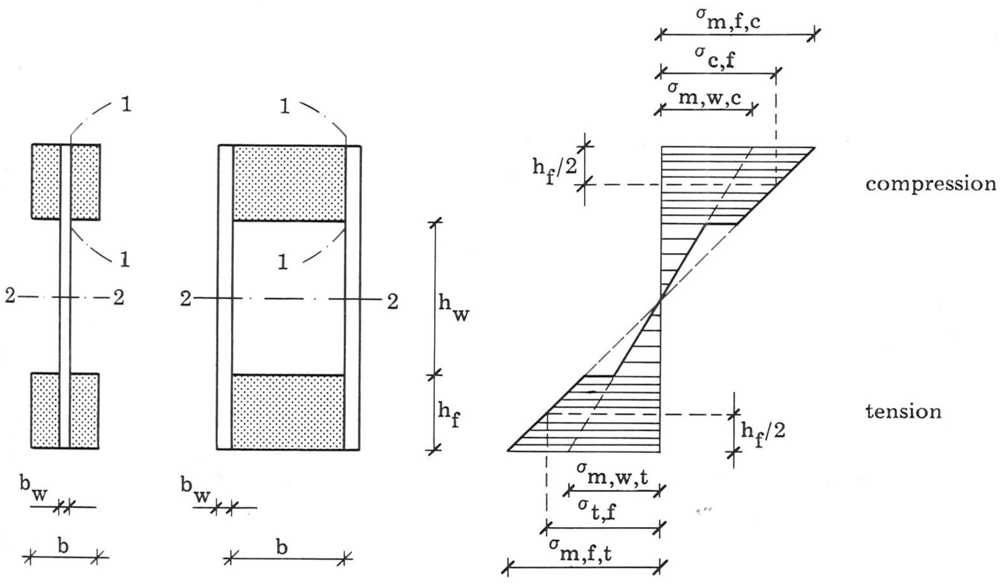


Fig. 7.1.1a

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

$$|\sigma_{m,f,c}| \leqslant f_m \tag{7.1.1a}$$

$$|\sigma_{c,f}| \leqslant k_c f_{c,0} \tag{7.1.1b}$$

$$\sigma_{t,f} \leqslant f_{t,0} \tag{7.1.1c}$$

$$\sigma_{m,f,t} \leqslant f_m \tag{7.1.1d}$$

k_c is determined according to section 5.1.7 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, $k_c = 1$ may be assumed.

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

$$|\sigma_{m,fc}| \leqslant k_{inst} f_m \tag{7.1.1e}$$

where k_{inst} is determined according to section 5.1.3.

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 $\varphi < 0.5$	$57\ b_w$
Plywood with $\varphi \geqslant 0.5$	$\frac{60}{1 + 0.1\varphi}\ b_w$

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. 7.1.1 b 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

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

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

τ_{crit} should be determined as

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

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

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. 7.1.1 b.

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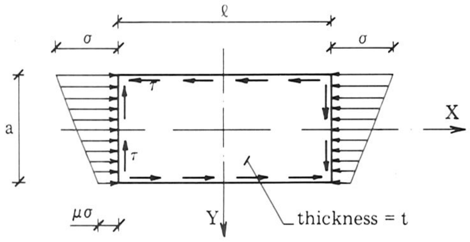


Fig. 7.1.1 b

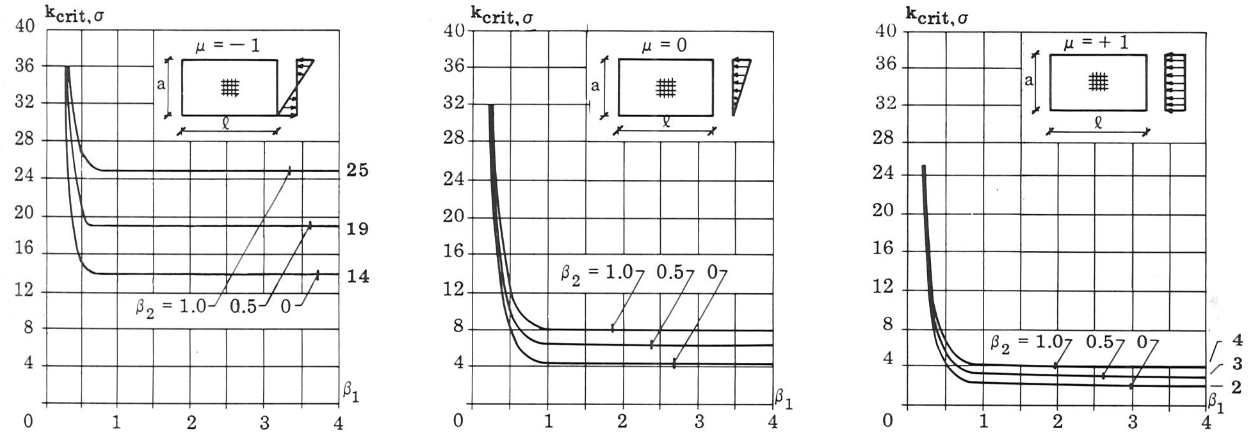


Fig. 7.1.1 c

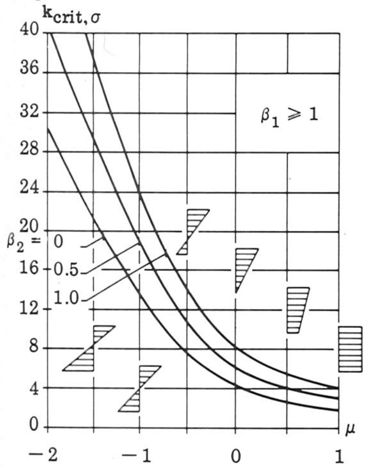


Fig. 7.1.1 d

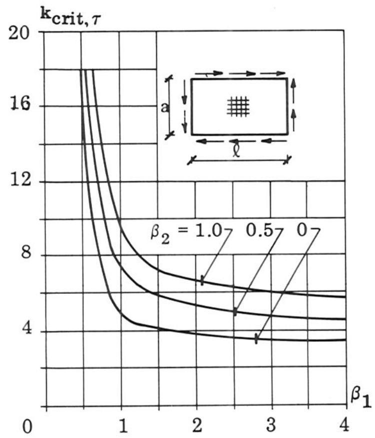


Fig. 7.1.1 e

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

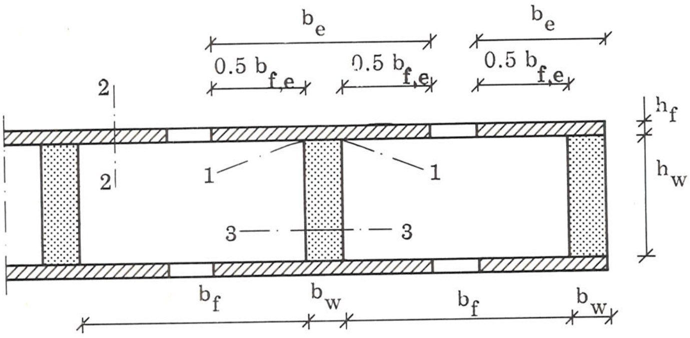


Fig. 7.1.2

$$b_e = b_{f,e} + b_w \tag{7.1.2a}$$

or

$$b_e = 0.5\, b_{f,e} + b_w \tag{7.1.2b}$$

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.

The shear stresses may be assumed uniformly distributed over the width of the sections 1-1, 2-2 and 3-3 shown in fig. 7.1.2.

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

The relevant parts of sections 5.1.7, 7.1.1 and 7.1.2 apply to I- and box columns.

The requirements for solid columns (section 5.1.7) 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 72 and for lattice columns in Annex 73.

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. k_{mod} should be taken from section 5.1.0.

Table 7.2 *Provisional*

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

E_0 is the modulus of elasticity of the timber in N/mm^2. d is the diameter in mm.

For beams a design method for a number of cross-sections is given in Annex 71 and for columns in Annex 71 - 72 - 73.

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 74 is permitted (in preparation).

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. The nails should be staggered in the best possible way.

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 a diameter not greater than 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.

In cases where the fire resistance cannot be calculated from the rules given in this chapter use should be made of more detailed design methods or a fire test. The same applies in such cases where the timber is protected against fire by sheathing, paint or impregnation.

Charring may be assumed to occur at a steady rate and the timber beneath the charred layer - except for a shallow 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 Solid members of structural timber

Calculation should be based on an effective residual cross-section by subtracting from the appropriate faces the notional amount of charring determined according to a notional charring rate of $0.25/\rho$ mm/minute, where ρ is the characteristic relative density.

The notional charring rate includes the effect of charring and weakening of the wood in a zone below the surface due to elevated temperatures.

Rounding of corners is disregarded.

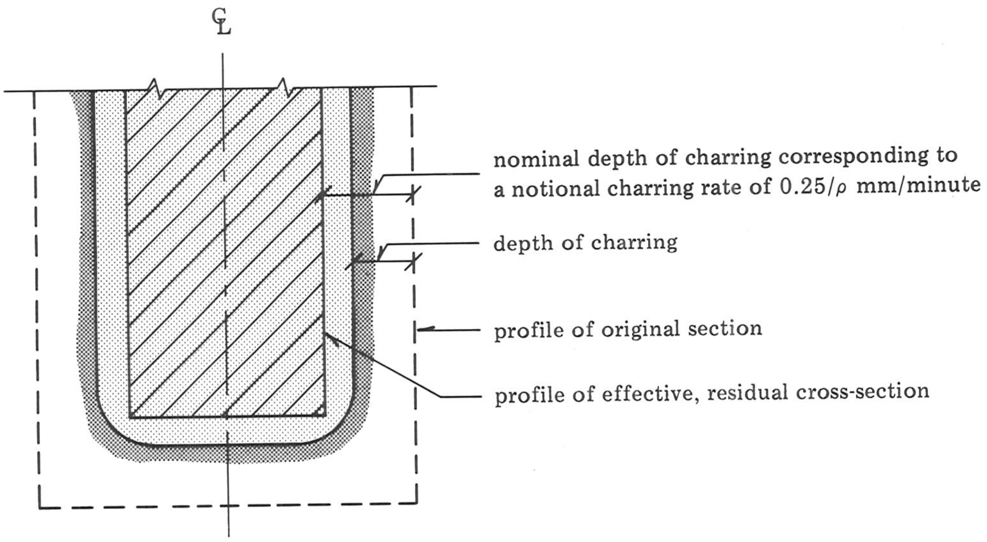


Fig. 9.1a. Determination of effective residual cross-section.

A member that is exposed to 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 9.1c and 9.1e) should be assumed to char equally on all faces during the whole period of fire exposures.

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 above (see figure 9.1b and 9.1d). 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.

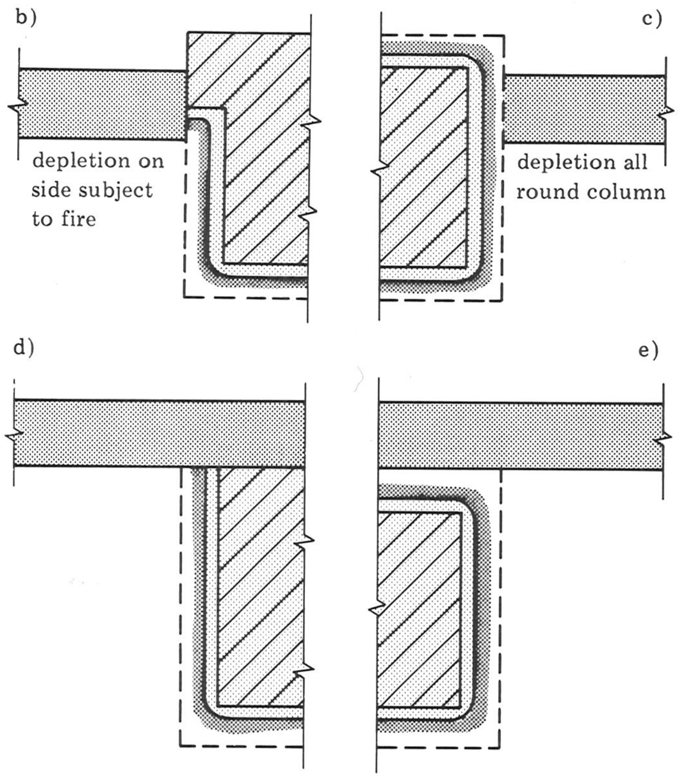


Fig. 9.1. b) and d) Wall having fire resistance not less than the column.
c) and e) Wall having less fire resistance than the column.

9.1.2 Glued laminated members

The charring rates given in 9.1.1 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.

9.1.3 Finger joints

Finger joints manufactured using the adhesives given in 9.1.2 may be considered to char at the rates given in 9.1.1.

9.1.4 Joints

Joints between members may be particularly vulnerable to the effects of fire and require special consideration.

Where a compression 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.

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. As assessment should 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 9.1.1.

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. 9.1f. 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.

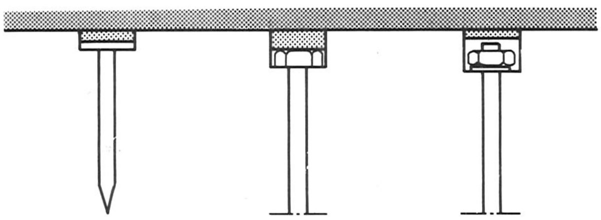


Fig. 9.1f. Sections and joints with metal fasteners.

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).

9.1.5 Built-up sections with metal fasteners

The charring rates in 9.1.1 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 9.1.4). 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.

9.2 Characteristic strength

The characteristic strength and deflection of a member should be calculated in accordance with the relevant parts of clause 5.1.1 using the effective residual section, the characteristic stress and the mean or characteristic modulus of elasticity, as appropriate.

For columns the maximum slenderness ratio based on the residual section should not exceed 200 and the stress modification factor for the slenderness ratio of the residual column should be derived from clause 5.1.7.

ANNEX 11
CURRENT LIST OF CIB-W18 PAPERS

Technical papers presented to CIB-W18 are identified by a code CIB-W18/a - b - c, where:

a denotes the meeting at which the paper was presented. Meetings are classified in chronological order:

1 Princes Risborough, England; March 1973
2 Copenhagen, Denmark; October 1973
3 Delft, Netherlands; June 1974
4 Paris, France; February 1975
5 Karlsruhe, Federal Republic of Germany; October 1975
6 Aalborg, Denmark; June 1976
7 Stockholm, Sweden; February/March 1977
8 Brussels, Belgium; October 1977
9 Perth, Scotland; June 1978
10 Vancouver, Canada; August 1978
11 Vienna, Austria; March 1979
12 Bordeaux, France; October 1979
13 Otaniemi, Finland; June 1980
14 Warsaw, Poland; May 1981
15 Karlsruhe, Federal Republic of Germany; June 1982

b denotes the subject:

1 Limit State Design
2 Timber Columns
3 Symbols
4 Plywood
5 Stress Grading
6 Stresses for Solid Timber
7 Timber Joints and Fasteners
8 Load Sharing
9 Duration of Load
10 Timber Beams
11 Environmental Conditions
12 Laminated Members
13 Particle and Fibre Building Boards
14 Trussed Rafters
15 Structural Stability
16 Fire
17 Statistics and Data Analysis

100 CIB Timber Code
101 Loading Codes
102 Structural Design Codes
103 International Standards Organisation
104 Joint Committee on Structural Safety
105 CIB Programme, Policy and Meetings
106 International Union of Forestry Research Organisations

c is simply a number given to the papers in the order in which they appear:

Example: CIB-W18/4-102-5 refers to paper 5 on subject 102 presented at the fourth meeting of W18.

Listed below, by subjects, are all papers that have to date been presented to W18. When appropriate some papers are listed under more than one subject heading.

LIMIT STATE DESIGN

1-1-1 Limit State Design, *H. J. Larsen*

1-1-2 The Use of Partial Safety Factors in the New Norwegian Design Code for Timber Structures, *O. Brynildsen*

1-1-3 Swedish Code Revision Concerning Timber Structures, *B. Norén*

TIMBER COLUMNS

SYMBOLS

PLYWOOD

7-4-2 Comments Received on Paper 7-4-1

7-4-3 The Effect of Rate of Testing Speed on the Ultimate Tensile Stress of Plywood, *C. R. Wilson and A. V. Parasin*

7-4-4 Comparison of the Effect of Specimen Size on the Flexural Properties of Plywood Using the Pure Moment Test, *C. R. Wilson and A. V. Parasin*

8-4-1 Sampling Plywood and the Evaluation of Test Results, *B. Norén*

9-4-1 Shear and Torsional Rigidity of Plywood, *H. J. Larsen*

9-4-2 The Evaluation of Test Data on the Strength Properties of Plywood, *L. G. Booth*

9-4-3 The Sampling of Plywood and the Derivation of Strength Values (Second Draft), *B. Norén*

9-4-4 On the Use of the CIB/RILEM Plywood Plate Twisting Test: a progress report, *L. G. Booth*

10-4-1 Buckling Strength of Plywood, *J. Dekker, J. Kuipers and H. Ploos van Amstel*

11-4-1 Analysis of Plywood Stressed Skin Panels with Rigid or Semi-Rigid Connections, *I. Smith*

11-4-2 A Comparison of Plywood Modulus of Rigidity Determined by the ASTM and RILEM CIB/3-TT Test Methods, *C. R. Wilson and A. V. Parasin*

11-4-3 Sampling of Plywood for Testing Strength, *B. Norén*

12-4-1 Procedures for Analysis of Plywood Test Data and Determination of Characteristic Values Suitable for Code Presentation, *C. R. Wilson*

14-4-1 An Introduction to Performance Standards for Wood-base Panel Products, *D. H. Brown*

14-4-2 Proposal for Presenting Data on the Properties of Structural Panels, *T. Schmidt*

STRESS GRADING

1-5-1 Quality Specifications for Sawn Timber and Precision Timber, *Norwegian Standard NS 3080*

1-5-2 Specification for Timber Grades for Structural Use, *British Standard BS 4978*

4-5-1 Draft Proposal for an International Standard for Stress Grading Coniferous Sawn Softwood, *ECE Timber Committee*

STRESSES FOR SOLID TIMBER

4-6-1 Derivation of Grade Stresses for Timber in the UK, *W. T. Curry*

5-6-1 Standard Methods of Test for Determining some Physical and Mechanical Properties of Timber in Structural Sizes, *W. T. Curry*

5-6-2 The Description of Timber Strength Data, *J. R. Tory*

5-6-3 Stresses for EC1 and EC2 Stress Grades, *J. R. Tory*

6-6-1 Standard Methods of some Physical and Mechanical Properties of Timber in Structural Sizes (Third Draft), *W. T. Curry*

7-6-1 Strength and Long-term Behaviour of Lumber and Glued Laminated Timber under Torsion Loads, *K. Möhler*

9-6-1 Classification of Structural Timber, *H. J. Larsen*

9-6-2 Code Rules for Tension Perpendicular to Grain, *H. J. Larsen*

9-6-3 Tension at an Angle to the Grain, *K. Möhler*

9-6-4 Consideration of Combined Stresses for Lumber and Glued Laminated Timber, *K. Möhler*

11-6-1 Evaluation of Lumber Properties in the United States, *W. L. Galligan and J. H. Haskell*

11-6-2 Stresses Perpendicular to Grain, *K. Möhler*

11-6-3 Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to paper CIB-W18/9-6-4), *K. Möhler*

12-6-1 Strength Classifications for Timber Engineering Codes, *R. H. Leicester and W. G. Keating*

12-6-2 Strength Classes for British Standard BS 5268, *J. R. Tory*

13-6-1 Strength Classes for the CIB Code, *J. R. Tory*

13-6-2 Consideration of Size Effects and Longitudinal Shear Strength for Uncracked Beams, *R. O. Foschi and J. D. Barrett*

13-6-3 Consideration of Shear Strength on End-Cracked Beams, *J. D. Barrett and R. O. Foschi*

15-6-1 Characteristic Strength Values for the ECE Standard for Timber, *J. G. Sunley*

TIMBER JOINTS AND FASTENERS

1-7-1 Mechanical Fasteners and Fastenings in Timber Structures, *E. J. Stern*

4-7-1 Proposal for a Basic Test Method for the Evaluation of Structural Timber Joints with Mechanical Fasteners and Connectors, *RILEM 3TT Committee*

4-7-2 Test Methods for Wood Fasteners, *K. Möhler*

5-7-1 Influence of Loading Procedure on Strength and Slip-Behaviour in Testing Timber Joints, *K. Möhler*

5-7-2 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures, *RILEM 3TT Committee*

5-7-3 CIB-Recommendations for the Evaluation of Results of Tests on Joints with Mechanical Fasteners and Connectors used in Load-Bearing Timber Structures, *J. Kuipers*

6-7-1 Recommendations for Testing Methods for Joints with Mechanical Fasteners and Connectors in Load-Bearing Timber Structures (Seventh Draft), *RILEM 3TT Committee*

6-7-2 Proposal for Testing of Integral Nail Plates as Timber Joints, *K. Möhler*

6-7-3 Rules for Evaluation of Values of Strength and Deformation from Test Results - Mechanical Timber Joints, *M. Johansen, J. Kuipers, B. Norén*

6-7-4 Comments to Rules for Testing Timber Joints and Derivation of Characteristic Values for Rigidity and Strength, *B. Norén*

7-7-1 Testing of Integral Nail Plates as Timber Joints, *K. Möhler*

7-7-2 Long Duration Tests on Timber Joints, *J. Kuipers*

7-7-3 Tests with Mechanically Jointed Beams with a Varying Spacing of Fasteners, *K. Möhler*

7-100-1 CIB-Timber Code Chapter 5.3 Mechanical Fasteners; CIB-Timber Standard 06 and 07, *H. J. Larsen*

9-7-1 Design of Truss Plate Joints, *F. J. Keenan*

9-7-2 Staples, *K. Möhler*

11-7-1 A Draft Proposal for an International Standard: *ISO Document ISO/TC 165N 38E*

12-7-1 Load-Carrying Capacity and Deformation Characteristics of Nailed Joints, *J. Ehlbeck*

12-7-2 Design of Bolted Joints, *H. J. Larsen*

12-7-3 Design of Joints with Nail Plates, *B. Norén*

13-7-1 Polish Standard BN-80/7159-04: Parts 00-01-02-03-04-05. »Structures from Wood and Wood-based Materials. Methods of Test and Strength Criteria for Joints with Mechanical Fasteners»

13-7-2 Investigation of the Effect of Number of Nails in a Joint on its Load-Carrying Ability, *W. Nozynski*

13-7-3 International Acceptance of Manufacture, Marking and Control of Finger-Jointed Structural Timber, *B. Norén*

13-7-4 Design of Joints with Nail Plates - Calculation of Slip, *B. Norén*

13-7-5 Design of Joints with Nail Plates - The Heel Joint, *B. Källsner*

13-7-6 Nail Deflection Data for Design, *H. J. Burgess*

13-7-7 Test on Bolted Joints, *P. Vermeyden*

13-7-8 Comments to paper CIB-W18/12-7-3 »Design of Joints with Nail Plates», *B. Norén*

13-7-9 Strength of Finger Joints, *H. J. Larsen*

13-100-4 CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates, *N. I. Bovim*

14-7-1 Design of Joints with Nail Plates (Second Edition), *B. Norén*

14-7-2 Method of Testing Nails in Wood (Second Draft, August 1980), *B. Norén*

14-7-3 Load-Slip Relationship of Nailed Joints, *J. Ehlbeck and H. J. Larsen*

14-7-4 Wood Failure in Joints with Nail Plates, *B. Norén*

14-7-5 The Effect of Support Eccentricity on the Design of W- and WW-Trusses with Nail Plate Connectors, *B. Källsner*

14-7-6 Derivation of the Allowable Load in Case of Nail Plate Joints Perpendicular to Grain, *K. Möhler*

14-7-7 Comments on CIB-W18/14-7-1, *T. A. C. M. van der Put*

15-7-1 Final Recommendation TT-1A: Testing Methods for Joints with Mechanical Fasteners in Load-Bearing Timber Structures. Annex A Punched Metal Plate Fasteners, *Joint Committee RILEM/CIB-3TT*

LOAD SHARING

3-8-1 Load Sharing - An Investigation of the State of Research and Development of Design Criteria, *E. Levin*

4-8-1 A Review of Load-Sharing in Theory and Practice, *E. Levin*

4-8-2 Load Sharing, *B. Norén*

DURATION OF LOAD

3-9-1 Definitions of Long Term Loading for the Code of Practice, *B. Norén*

4-9-1 Long Term Loading of Trussed Rafters with Different Connection Systems, *T. Feldborg and M. Johansen*

5-9-1 Strength of a Wood Column in Combined Compression and Bending with Respect to Creep, *B. Källsner and B. Norén*

6-9-1 Long Term Loading for the Code of Practice (Part 2), *B. Norén*

6-9-2 Long Term Loading, *K. Möhler*

6-9-3 Deflection of Trussed Rafters under Alternating Loading during a Year, *T. Feldborg and M. Johansen*

7-6-1 Strength and Long-Term Behaviour of Lumber and Glued Laminated Timber under Torsion Loads, *K. Möhler*

7-9-1 Code Rules Concerning Strength and Loading Time, *H. J. Larsen and E. Theilgaard*

TIMBER BEAMS

4-10-1 The Design of Simple Beams, *H. J. Burgess*

4-10-2 Calculation of Timber Beams Subjected to Bending and Normal Force, *H. J. Larsen*

5-10-1 The Design of Timber Beams, *H. J. Larsen*

9-10-1 The Distribution of Shear Stresses in Timber Beams, *F. J. Keenan*

9-10-2 Beams Notched at the Ends, *K. Möhler*

11-10-1 Tapered Timber Beams, *H. Riberholt*

13-6-2 Consideration of Size Effects in Longitudinal Shear Strength for Uncracked Beams, *R. O. Foschi and J. D. Barrett*

13-6-3 Consideration of Shear Strength on End-Cracked Beams, *J. D. Barrett and R. O. Foschi*

ENVIRONMENTAL CONDITIONS

5-11-1 Climate Grading for the Code of Practice, *B. Norén*

6-11-1 Climate Grading (2), *B. Norén*

9-11-1 Climate Classes for Timber Desing, *F. J. Keenan*

LAMINATED MEMBERS

6-12-1 Directives for the Fabrication of Load-Bearing Structures of Glued Timber, *A. van der Velden and J. Kuipers*

8-12-1 Testing of Big Glulam Timber Beams, *H. Kolb and P. Frech*

8-12-2 Instruction for the Reinforcement of Apertures in Glulam Beams, *H. Kolb and P. Frech*

8-12-3 Glulam Standard Part 1; Glued Timber Structures; Requirements for Timber (Second Draft)

9-12-1 Experiments to Provide for Elevated Forces at the Supports of Wooden Beams with Particular Regard to Shearing Stresses and Long-term Loadings, *F. Wassipaul and R. Lackner*

9-12-2 Two Laminated Timber Arch Railway Bridges Built in Perth in 1849, *L. G. Booth*

9-6-4 Consideration of Combined Stresses for Lumber and Glued Laminated Timber, *K. Möhler*

11-6-3 Consideration of Combined Stresses for Lumber and Glued Laminated Timber (addition to Paper CIB-W18/9-6-4), *K. Möhler*

12-12-1 Glulam Standard Part 2: Glued Timber Structures; Rating (3rd Draft)

12-12-2 Glulam Standard Part 3: Glued Timber Structures; Performance (3rd Draft)

13-12-1 Glulam Standard Part 3: Glued Timber Structures; Performance (4th Draft)

14-12-1 Proposals for CEI-Bois/CIB-W18 Glulam Standards, *H. J. Larsen*

14-12-2 Guidelines for the Manufacturing of Glued Load-Bearing Timber Structures, *Stevin Laboratory*

14-12-3 Double Tapered Curved Glulam Beams, *H. Riberholt*

14-12-4 Comment on CIB-W18/14-12-3, *E. Gehri*

PARTICLE AND FIBRE BUILDING BOARDS

7-13-1 Fibre Building Boards for CIB Timber Code (First Draft), *O. Brynildsen*

9-13-1 Determination of the Bearing Strength and the Load-Deformation Characteristics of Particle Board, *K. Möhler, T. Budianto and J. Ehlbeck*

9-13-2 The Structural Use of Tempered Hardboard, *W. W. L. Chan*

11-13-1 Tests on Laminated Beams from Hardboard under Short- and Longterm Load, *W. Nozynski*

11-13-2 Determination of Deformation of Special Densified Hardboard under Long-term Load and Varying Temperature and Humidity Conditions, *W. Halfar*

11-13-3 Determination of Deformation of Hardboard under Long-term Load in Changing Climate, *W. Halfar*

14-4-1 An Introduction to Performance Standards for Wood-Base Panel Products, *D. H. Brown*

14-4-2 Proposal for Presenting Data on the Properties of Structural Panels, *T. Schmidt*

TRUSSED RAFTERS

4-9-1 Long-term Loading of Trussed Rafters with Different Connection Systems, *T. Feldborg and M. Johansen*

6-9-3 Deflection of Trussed Rafters under Alternating Loading During a Year, *T. Feldborg and M. Johansen*

7-2-1 Lateral Bracing of Timber Struts, *J. A. Simon*

9-14-1 Timber Trusses - Code Related Problems, *T. F. Williams*

9-7-1 Design of Truss Plate Joints, *F. J. Keenan*

10-14-1 Design of Roof Bracing - The State of the Art in South Africa, *P. A. V. Bryant and J. A. Simon*

11-14-1 Design of Metal Plate Connected Wood Trusses, *A. R. Egerup*

12-14-1 A Simple Design Method for Standard Trusses, *A. R. Egerup*

13-14-1 Truss Design Method for CIB Timber Code, *A. R. Egerup*

13-14-2 Trussed Rafters, Static Models, *H. Riberholt*

13-14-3 Comparison of 3 Truss Models Designed by Different Assumptions for Slip and E-Modulus, *K. Möhler*

14-14-1 Wood Trussed Rafter Design, *T. Feldborg and M. Johansen*

14-14-2 Truss-Plate Modelling in the Analysis of Trusses, *R. O. Foschi*

14-14-3 Cantilevered Timber Trusses, *A. R. Egerup*

14-7-5 The Effect of Support Eccentricity on the Design of W- and WW-Trusses with Nail Plate Connectors, *B. Källsner*

15-14-1 Guidelines for Static Models of Trussed Rafters, *H. Riberholt*

15-14-2 The Influence of Various Factors on the Accuracy of the Structural Analysis of Timber Roof Trusses, *F. R. P. Pienaar*

15-14-3 Bracing Calculations for Trussed Rafter Roofs, *H. J. Burgess*

15-14-4 The Design of Continuous Members in Timber Trussed Rafters with Punched Metal Connector Plates, *P. O. Reece*

15-14-5 A Rafter Design Method Matching U. K. Test Results for Trussed Rafters, *H. J. Burgess*

STRUCTURAL RELIABILITY

8-15-1 Laterally Loaded Timber Columns: Tests and Theory, *H. J. Larsen*

13-15-1 Timber and Wood-Based Products Structures. Panels for Roof Coverings. Methods of Testing and Strength Assessment Criteria. *Polish Standard BN-78/7159-03*

FIRE

12-16-1 British Standard BS 5268 The Structural Use of Timber: Part 4 Fire Resistance of Timber Structures

13-100-2 CIB Structural Timber Design Code. Chapter 9. Performance in Fire

STATISTICS AND DATA ANALYSIS

13-17-1 On Testing Whether a Prescribed Exclusion Limit is Attained, *W. G. Warren*

CIB TIMBER CODE

2-100-1 A Framework for the Production of an International Code of Practice for the Structural Use of Timber, *W. T. Curry*

5-100-1 Design of Solid Timber Columns (First Draft), *H. J. Larsen*

5-100-2 A Draft Outline of a Code for Timber Structures, *L. G. Booth*

6-100-1 Comments on Document 5-100-1; Design of Solid Timber Columns, *H. J. Larsen and E. Theilgaard*

6-100-2 CIB Timber Code: CIB Timber Standards, *H. J. Larsen and E. Theilgaard*

7-100-1 CIB Timber Code Chapter 5.3 Mechanical Fasteners;
CIB Timber Standard 06 and 07, *H. J. Larsen*

8-100-1 CIB Timber Code - List of Contents (Second Draft), *H. J. Larsen*

9-100-1 The CIB Timber Code (Second Draft)

11-100-1 CIB Structural Timber Design Code (Third Draft)

11-100-2 Comments Received on the CIB Code
- a *U. Saarelainen*
- b *Y. M. Ivanov*
- c *R. H. Leicester*
- d *W. Nozynski*
- e *W. R. A. Meyer*
- f *P. Beckmann; R. Marsh*
- g *W. R. A. Meyer*
- h *W. R. A. Meyer*

11-100-3 CIB Structural Timber Design Code, Chapter 3, *H. J. Larsen*

12-100-1 Comment on the CIB Code, *Sous-Commission Glulam*

12-100-2 Comment on the CIB Code, *R. H. Leicester*

12-100-3 CIB Structural Timber Design Code (Fourth Draft)

13-100-1 Agreed Changes to CIB Structural Timber Design Code

13-100-2 CIB Structural Timber Design Code. Chapter 9: Performance in Fire

13-100-3a Comments on CIB Structural Timber Design Code

13-100-3b Comments on CIB Structural Timber Design Code, *W. R. A. Meyer*

13-100-3c Comments on CIB Structural Timber Design Code, *British Standards Institution*

13-100-4 CIB Structural Timber Design Code. Proposal for Section 6.1.5 Nail Plates, *N. I. Bovim*

14-103-2 Comments on the CIB Structural Timber Design Code, *R. H. Leicester*

15-103-1 Resolutions of TC 165-meeting in Athens 1981-10-12/13

LOADING CODES

4-101-1 Loading Regulations, *Nordic Committee for Building Regulations*

4-101-2 Comments on the Loading Regulations, *Nordic Committee for Building Regulations*

STRUCTURAL DESIGN CODES

1-102-1 Survey of Status of Building Codes, Specifications etc., in USA, *E. G. Stern*

1-102-2 Australian Codes for Use of Timber in Structures, *R. H. Leicester*

1-102-3 Contemporary Concepts for Structural Timber Codes, *R. H. Leicester*

1-102-4 Revision of CP 112 - First Draft, July 1972, *British Standards Institution*

4-102-1 Comparison of Codes and Safety Requirements for Timber Structures in EEC Countries, *Timber Research and Development Association*

4-102-2 Nordic Proposals for Safety Code for Structures and Loading Code for Design of Structures, *O. A. Brynildsen*

4-102-3 Proposal for Safety Codes for Load-Carrying Structures, *Nordic Committee for Building Regulations*

4-102-4 Comments to Proposal for Safety Codes for Load-Carrying Structures, *Nordic Committee for Building Regulations*

4-102-5 Extract from Norwegian Standard NS 3470 »Timber Structures»

4-102-6 Draft for Revision of CP 112 »The Structural Use of Timber», *W. T. Curry*

8-102-1 Polish Standard PN-73/B-03150: Timber Structures; Statistical Calculations and Designing

8-102-2 The Russian Timber Code: Summary and Contents

9-102-1 Svensk Byggnorm 1975 (2nd Edition); Chapter 27: Timber Construction

11-102-1 Eurocodes, *H. J. Larsen*

13-102-1 Program of Standardisation Work Involving Timber Structures and Wood-Based Products in Poland

INTERNATIONAL STANDARDS ORGANISATION

3-103-1 Method for the Preparation of Standards Concerning the Safety of Structures (ISO/DIS3250), *International Standards Organisation ISO/TC98*

4-103-1 A Proposal for Undertaking the Preparation of an International Standard on Timber Structures, *International Standards Organisation*

5-103-1 Comments on the Report of the Consultation with Member Bodies Concerning ISO/TS/P129 - Timber Structures - Dansk Ingeniørforening

7-103-1 ISO Technical Committees and Membership of ISO/TC 165

8-103-1 Draft Resolutions of ICO/TC 165

12-103-1 ISO/TC 165 Ottawa, September 1979

13-103-1 Report from ISO/TC 165, *A. Sørensen*

14-103-1 Comments on ISO/TC 165 N52 »Timber Structures; Solid Timber in Structural Sizes; Determination of Some Physical and Mechanical Properties»

14-103-2 Comments on the CIB Structural Timber Design Code, *R. H. Leicester*

JOINT COMMITTEE ON STRUCTURAL SAFETY

3-104-1 International System on Unified Standard Codes of Practice for Structures, *Comitè Européen du Bèton (CEB)*

7-104-1 Volume 1: Common Unified Rules for Different Types of Construction and Material, *CEB*

CIB PROGRAMME, POLICY AND MEETINGS

1-105-1 A Note on International Organisations Active in the Field of Utilisation of Timber, *P. Sonnemans*

5-105-1 The Work and Objectives of CIB-W18 - Timber Structures, *J. G. Sunley*

10-105-1 The Work of CIB-W18 Timber Structures, *J. G. Sunley*

15-105-1 Terms of Reference for Timber, *Framed Housing Sub-Group of CIB-W18*

INTERNATIONAL UNION OF FORESTRY RESEARCH ORGANISATIONS

7-106-1 Time and Moisture Effects, *CIB-W18/IUFRO 55.02-03 Working Party*

ANNEX 42
STANDARD STRENGTH AND DENSITY CLASSES

42.0 General

The following two bases will be used for grouping timber with respect to its structural properties.

- Strength class grouping, based on the structural properties in bending, tension and compression parallel to the grain, and shear.
- Density class grouping, based on the density.

42.1 Strength class grouping

The standard strength classes SC5, SC6 . . . SC60, and SC 75 shown in table 42.1 are used.

The target characteristic values are given in table 42.1.

A given grade may be assigned to one of the standard strength classes if the characteristic values of bending strength, tension strength and modulus of elasticity are not less than the target values and if the values of compression strength and shear strength exceed the requirements for the nearest lower class.

Table 42.1 Standard strength classes. Characteristic values in MPa *Provisional*

		SC5	SC6	SC8	SC10	SC12	SC15	SC19
Bending	f_m	5.0	6.0	7.5	9.5	12	15	19
Tension parallel to grain	$f_{t,0}$	2.5	3.2	4.1	5.4	7.0	9.1	11.8
Compression parallel to grain	$f_{c,0}$	5.8	7.0	8.4	10.0	12.0	14.5	17.5
Shear parallel to grain	f_v	1.0	1.1	1.3	1.5	1.7	2.0	2.2
Modulus of elasticity	E_0	2300	2600	3000	3500	4000	4600	5300

Table 42.1 (continued)

		SC24	SC30	SC38	SC48	SC60	SC75
Bending	f_m	24	30	38	48	60	75
Tension parallel to grain	$f_{t,0}$	15.5	20	25	34	44	54
Compression parallel to grain	$f_{c,0}$	21	25	30	36	43	52
Shear parallel to grain	f_v	2.6	3.0	3.5	4.0	4.6	5.2
Modulus of elasticity	E_0	6100	7000	8100	9300	10600	12200

42.2 Density class grouping

The standard ~~strength~~ density classes D300, . . . , D800 shown in table 42.2 are used.

A given grade can be assigned to one of the density classes if the characteristic, relative density is not less than the minimum values given in table 42.2.

Table 42.2 Standard density classes. Minimum characteristic relative densities

	Standard density class				
	D300	D400	D500	D600	D800
Minimum, characteristic relative density	0.32	0.40	0.50	0.63	0.78

42.3 Characteristic values and mean values

For the standard strength classes the characteristic values should be taken to be the values given in table 42.1.

For E_{mean} a value of 1.4 E_0 may be assumed.

For G_{mean} a value of 0.095 E_0 may be assumed.

For the standard density classes the characteristic values should be taken to be the values given in table 42.3.

For rolling shear the strength may be assumed to be equal to $f_{t,90}$.

Table 42.3 Standard density classes. Characteristic values in MPa

		Standard density class				
		D300	D400	D500	D600	D800
Tension perpendicular to grain	$f_{t,90}$	0.40	0.50	0.65	0.85	1.10
Compression perpendicular to grain	$f_{c,90}$	2.0	3.0	4.5	6.8	10.1

ANNEX 43
CHARACTERISTIC STRENGTHS AND STIFFNESSES

43.0 General

This annex provides examples of characteristic strengths and stiffnesses accepted by national codes authorities for specific grading rules.

The definition of a characteristic value given in clause 2.1.1 applies to the values tabulated in this annex unless otherwise qualified.

43.1 United Kingdom

The characteristic values given in table 43.1 have formed the basis for the grade stresses which are to be published*) in British Standard BS 5268 The Structural Use of Timber; Part 2 Permissible Stress Design, Materials and Workmanship. The grades to which the values relate are the visual grades specified in British Standard BS 4978 Timber Grades for Structural Use.

The values given in table 43.1 for bending, tension, compression parallel to the grain and modulus of elasticity were derived from an analysis of results from structural sized tests. The analysis estimated 5-percentile values from 3-parameter Weibull distribution functions without associated confidence levels. The values for shear parallel to grain were derived from small clear specimen tests and for compression perpendicular to grain from small clear bending strength tests. For these properties 5-percentile values were estimated from normal distribution functions without associated confidence levels.

BS 5269 will define stresses for a dry environmental condition approximating to a moisture content of 18% or less in softwoods (20°C 80 RH) and it is to this condition that the tabulated values relate.

BS 5268 will recognize that bending strength is dependent on cross-section depth and tension strength on cross-section width. The bending and tension strengths given in table 43.1 are therefore, also related to a greatest cross-section dimension of 200 mm.

*) Publication anticipated in 1983/84.

Table 43.1 Characteristic strengths and stiffnesses for timbers graded to British Standard BS4978 rules (in MPa)

Species	Source	Grade	f_m	$f_{t,0}$	$f_{c,0}$	$f_{c,90}$	f_v	E_0
Redwood/whitewood	Europe	SS	21.6	13.0	19.4	5.6	2.17	7000
Scots pine, larch	UK	GS	15.3	9.2	16.6	4.8	2.17	5800
Corsican pine	UK	SS	21.6	13.0	19.4	5.6	2.17	6300
		GS	15.3	9.2	16.6	4.8	2.17	5200
Douglas fir	UK	SS	17.9	10.7	16.1	6.4	2.33	7100
		GS	12.7	7.6	13.8	5.6	2.33	5900
European spruce	UK	SS	16.5	9.9	14.9	4.2	1.70	5100
Sitka spruce	UK	GS	11.7	7.0	12.7	3.7	1.70	4300
Parana pine		SS	25.9	15.5	23.3	6.4	2.73	7400
		GS	18.4	11.0	19.9	5.8	2.73	6100
Pitch pine		SS	30.0	18.0	27.0	8.5	3.07	8900
		GS	21.3	12.8	23.1	7.4	3.07	3400
Western red cedar		SS	16.5	9.9	14.9	4.5	1.67	5600
		GS	11.7	7.0	12.7	4.2	1.67	4600
Doug-fir-larch	Canada	SS	21.6	13.0	19.4	6.4	2.25	7400
		GS	15.3	9.2	16.6	5.8	2.25	6200
Doug-fir-larch	USA	SS	21.6	13.0	19.4	6.4	2.25	7400
		GS	15.3	9.2	16.6	5.8	2.25	6200
Hem-fir	Canada	SS	21.6	13.0	19.4	5.0	1.80	7400
		GS	15.3	9.2	16.6	4.5	1.80	6100
Hem-fir	USA	SS	21.6	13.0	19.4	5.0	1.80	7400
		GS	15.3	9.2	16.6	4.5	1.80	6100
Spruce-pine-fir	Canada	SS	21.6	13.0	19.4	4.8	1.80	6700
		GS	15.3	9.2	16.6	4.2	1.80	5500
Western whitewoods	USA	SS	19.1	11.5	17.2	4.5	1.75	5800
		GS	13.6	8.2	14.7	4.0	1.75	4800
Southern pines	USA	SS	27.8	16.7	25.0	6.6	2.60	8400
		GS	19.7	11.8	21.4	5.8	2.60	6900

The values for SS also apply to ECE grade S8 and for GS to ECE grade 6.

ANNEX 71
MECHANICALLY JOINTED MEMBERS WITH I-, T- OR BOX CROSS-SECTIONS

71.1 Scope

Members with cross-sections as shown in fig. 71.1 are dealt with. The individual parts are full length and connected to each other by nails, bolts with toothed metal plate connectors or similar non-rigid fasteners.

A method is given to determine stresses, deflections and load on the fasteners of beams and the load-carrying capacity of columns, including the necessary requirements to the fasteners.

71.2 Notations

Reference is made to fig. 71.1. In all cases the Z-axis is a symmetry axis. For cross-sections of type 1 the Y-axis is a gravity axis, while for type 2 and 3 it is a symmetry axis.

For beams bending about the Y-axis is assumed.

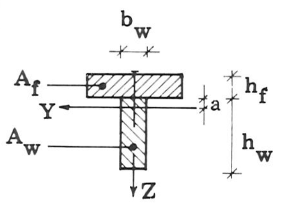
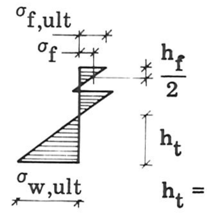
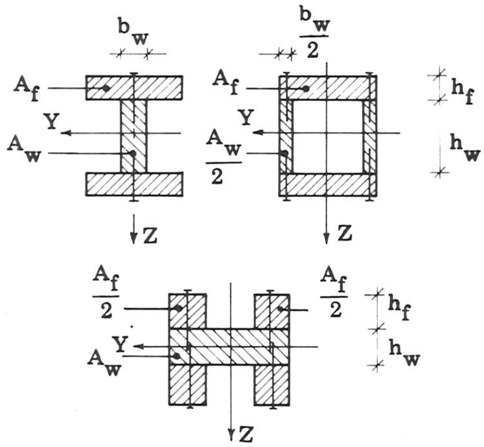
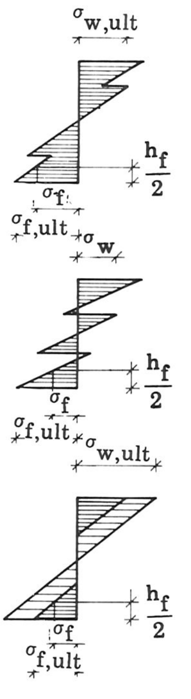
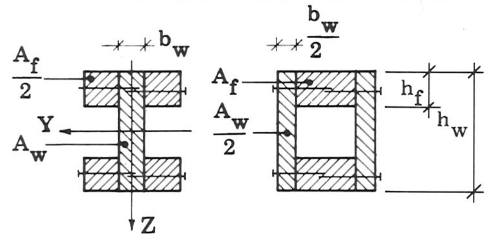
Type No.	Cross-section	A_r	Stress distribution
1		$\frac{A_f A_w}{A_f + A_w}$	 $h_t = \frac{h_w}{2} + \gamma\,(\frac{h_w}{2} - a)$
2		A_f	
3			

Fig. 7.1.1

A	Area	
	A_{tot}	Total area
	A_f	Flange area
	A_w	Web area
	A_r	See fig. 71.1
E	Modulus of elasticity	
F	Axial load on column	
	F_{crit}	Critical value
F_1	Load on one fastener	
G	Shear modulus	
I	Moment of inertia (second moment of area)	
	I_{tot}	Total value calculated around the geometric gravity axis (Y-axis)
	I_0	Sum of moments of inertia for the individual parts around own gravity axis
	I_{ef}	Effective moment of inertia, see formula (71.2)
K	Slip modulus, see section 71.3	
M	Bending moment (about Y-axis)	
S	Static moment (first moment of area)	
	S_f	Static moment of flange about Y-axis
V	Shear force (in the direction of the Z-axis)	
a	Distance of gravity axis from flange	
b	Width	
	b_w	Thickness of web
f	Strength	
	$f_{c,0}$	Compressive strength
h	Depth	
	h_f	Depth of flange
	h_w	Depth of web
	h_t	Depth of tension zone in web
k_c	Column factor	
ℓ	Span of beam or free length of column	
s	Spacing of fasteners ($1/s$ is number of fasteners per unit length). If s is varying evenly in the longitudinal direction according to the shear force between s_{min} and $s_{max} \leqslant 4s_{min}$ an effective value of s equal to $s = 0.75\ s_{min} + 0.25\ s_{max}$ may be used.	
u	Slip between jointed members	
γ	Effectiveness factor, see section 71.4	
λ	Slenderness ratio	
	λ_e	Effective slenderness ratio
σ	Axial stresses	
	$\sigma_{f,ult}$	σ in outermost fibre of flange
	σ_f	σ in the middle of the flange
	$\sigma_{w,ult}$	σ in outermost fibre of web
τ	Shear stress	

71.3 Assumptions

The member is assumed either loaded in the Z-direction giving moments about the Y-axis, or with an axial load acting in the gravity axis.

The case where both bending moment and axial force are acting (beam columns) at the same time is only treated in a special case, cf. section 71.7.

The conditions are assumed linear-elastic and the following relation between the load on a fastener and the slip is assumed to apply.

$$F_1 = Ku \tag{71.1}$$

All members are assumed to have the same modulus of elasticity, but the expressions may be extended to apply also to cross-sections where the cross-section members have different properties by transforming of the cross-section sizes in relation to their stiffness after the usual methods.

71.4 Effective moment of inertia

The effective moment of inertia is determined by

$$I_{ef} = I_0 + \gamma(I_{tot} - I_0) \tag{71.2}$$

where

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} \frac{E}{K} s} \tag{71.3}$$

For beams with thin webs of plywood, particle boards or fibre boards, however,

$$\gamma = \frac{1}{1 + \frac{\pi^2 A_r}{\ell^2} \left(\frac{E}{K} s + \frac{E}{2Gb_w}\right)} \tag{71.4}$$

71.5 Beams

Calculation of stresses

Fig. 71.1 is referred to.

The stresses in cross-sections of type 1 are calculated from the following expressions:

$$|\sigma_{w,ult}| = \frac{M}{I_{ef}} h_t \tag{71.5}$$

$$|\sigma_f| = \frac{M}{I_{ef}} \gamma\left(\frac{h_f}{2} + a\right) \tag{71.6}$$

$$|\sigma_{f,ult}| = |\sigma_f| + \frac{M}{I_{ef}} \frac{h_f}{2} \tag{71.7}$$

in cross-sections of type 2 from:

$$|\sigma_{w,ult}| = \frac{M}{I_{ef}} \frac{h_w}{2} \tag{71.8}$$

$$|\sigma_f| = \frac{M}{I_{ef}} \gamma \frac{(h_w + h_f)}{2} \tag{71.9}$$

$$|\sigma_{f,ult}| = |\sigma_f| + \frac{M}{I_{ef}} \frac{h_f}{2} \tag{71.10}$$

and in cross-sections of type 3 from:

$$|\sigma_{w,ult}| = \frac{M}{I_{ef}} \frac{h_w}{2} \tag{71.11}$$

$$|\sigma_f| = \frac{M}{I_{ef}} \gamma \frac{(h_w - h_f)}{2} \tag{71.12}$$

$$|\sigma_{f,ult}| = |\sigma_f| + \frac{M}{I_{ef}} \frac{h_f}{2} \tag{71.13}$$

Calculation of maximum shear stresses

For cross-sections of type 1 the maximum shear stresses occur where the stresses in the web are zero and can be calculated from

$$\max\tau = \frac{Vh_t^2}{2I_{ef}} \tag{71.14}$$

For cross-sections of types 2 and 3 the maximum shear stresses occur in the middle of the web. For type 2 they can be calculated from

$$\max\tau = \frac{V}{I_{ef}b_w}(\gamma A_f \frac{(h_w + h_f)}{2} + \frac{1}{8} A_w h_w) \tag{71.15}$$

and for type 3 from:

$$\max\tau = \frac{V}{I_{ef}b_w}(\gamma A_f \frac{(h_w - h_f)}{2} + \frac{1}{8} A_w h_w) \tag{71.16}$$

Calculation of load on fasteners

The load per fastener can be determined from

$$F_1 = \gamma \frac{VS_f}{I_{ef}} s \tag{71.17}$$

Deflections

The deflections induced by the moment are calculated as usual, using the effective moment of inertia I_{ef}.

71.6 Concentrically loaded columns

Load-carrying capacity

The load-carrying capacity corresponding to deflection along the Z-axis can be determined as

$$F_{crit} = k_c f_{c,0} A_{tot} \tag{71.18}$$

The column factor k_c is determined as for a corresponding column with rigid joints between the cross-section members, but the effective slenderness ratio:

$$\lambda_{ef} = \ell \sqrt{\frac{A_{tot}}{I_{ef}}} \tag{71.19}$$

is used.

For the T-cross-section and the I-cross-section, type 2, the load-carrying capacity for deflection in the Y-direction is found as the sum of the load-carrying capacity of the individual members, i.e. the stiffening effect that the members might have on each other is not taken into account.

Load on fasteners

The load on the fasteners can be calculated by eq. (71.17), assuming

$$V = \left\{ \begin{array}{ll} \frac{F}{60}\frac{1}{k_c} & \text{for } 60 \leqslant \lambda_{ef} \\ \frac{\lambda_{ef}}{60}\frac{F}{60}\frac{1}{k_c} & \text{for } 30 \leqslant \lambda_{ef} \leqslant 60 \\ \frac{F}{120}\frac{1}{k_c} & \text{for } \lambda_{ef} \leqslant 30 \end{array} \right\} \quad (71.20)$$

71.7 Combined loads

In cases where small moments resulting from e.g. own weight are acting apart from axial load, the usual interaction formulas can be used for the stresses determined above.

71.8 References

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up timber columns. CIB-W18/3-2-1.

ANNEX 72
SPACED COLUMNS WITH NAILED OR GLUED PACKS OR BATTENS

72.1 Scope

Columns as shown in fig. 72.1 are dealt with, i.e. columns with two or in certain cases three or four identical shafts jointed with packs or battens. The joints may be either nailed or glued or bolted with toothed metal plate connectors. Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio, whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

It is assumed that the construction rules given in section 72.3 are observed, and that the joints are designed for forces as stated in section 72.6.

Only concentrically loaded columns are dealt with.

72.2 Notations

Reference is made to fig. 72.1.

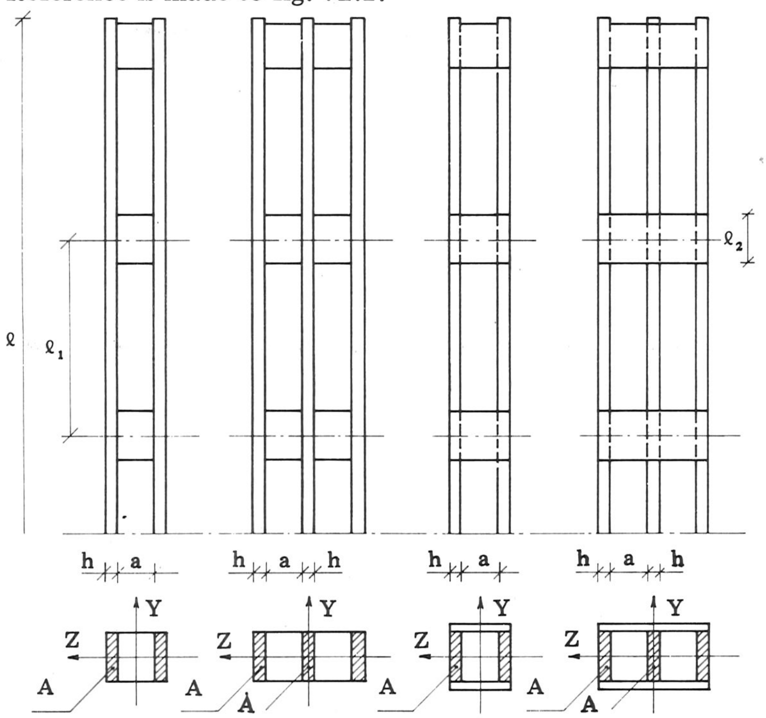


Fig. 72.1

A	Area of one shaft
	$A_{tot} = nA$ Total area
F	Column load
	F_{crit} Load-carrying capacity
I	Moment of inertia (second moment of area) of one shaft
	I_{tot} Total moment of inertia about Y-axis
T_0	See fig. 72.2

V	Shear force	
a	Distance	
	a	Free distance between shafts
	$a_1 = a + h$ See fig. 72.2	
$f_{c,0}$	Compressive strength parallel to the grain	
k_c	Column factor	
ℓ	length	
	ℓ	Free length of column
	ℓ_1	Distance between midpoints of packs or battens
	ℓ_2	Length of packs or battens
n	Number of shafts	
λ	Slenderness ratio	
	$\lambda = \ell\sqrt{\frac{A_{tot}}{I_{tot}}}$	Slenderness ratio for a solid column with the same cross-section
	$\lambda_1 = \ell_1\sqrt{\frac{A}{I}}$	Slenderness ratio for the shafts
	λ_{ef}	Effective slenderness ratio
η	Factor, see table 72.1	

72.3 Assumptions

- The cross-section is composed of 2, 3 or 4 identical shafts.
- The Y- and Z-axes are symmetry axes.
- The number of free fields are at least 3, i.e. the shafts are at least jointed in the ends and in the third points.
- The free distance between the shafts is not greater than 5 times the lamella thickness ($a/h \leqslant 5$).
- The joints and packs and battens are designed for a shear force V as stated in section 72.5.
- The length of the packs should satisfy the condition $\ell_2/a \geqslant 1.5$.
- For nailed joints there should in each section be at least 4 nails or 2 bolts with metal plate connectors. For nailed joints at the ends apply that there should be at least 4 nails in a row in the longitudinal direction of the column.
- Battens should be made of structural plywood and their length satisfy the condition $\ell_2/a \geqslant 2$.
- The columns are solely subjected to concentrical axial loads.

72.4 Load-carrying capacity

For deflection in the Y-direction the load-carrying capacity can be determined as the sum of the load-carrying capacity of the individual members.

For deflection in the Z-direction the load-carrying capacity are determined as

$$F_{crit} = k_c f_{c,0} A_{tot} \tag{72.1}$$

The column factor k_c is determined as for solid columns, but the usual slenderness ratio

$$\lambda = \ell\sqrt{\frac{A_{tot}}{I_{tot}}} \tag{72.2}$$

is replaced by the effective slenderness ratio

$$\lambda_{ef} = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \qquad (72.3)$$

η is given in table 72.1.

Table 72.1. η

	Packs			Battens	
	glued	nailed	bolted*	glued	nailed
Long-term loading	1	4	3.5	2	6
Short-term loading	1	3	2.5	2	4.5

* with toothed metal plates.

72.5 Shear forces

The load on the fasteners and battens or packs can be calculated as stated in fig. 72.2, assuming

$$V = \begin{cases} \frac{F}{60} \frac{1}{k_c} & \text{for } 60 \leqslant \lambda_{ef} \\ \frac{\lambda_{ef}}{60} \frac{F}{60} \frac{1}{k_c} & \text{for } 30 \leqslant \lambda_{ef} \leqslant 60 \\ \frac{F}{120} \frac{1}{k_c} & \text{for } \lambda_{ef} \leqslant 30 \end{cases}$$

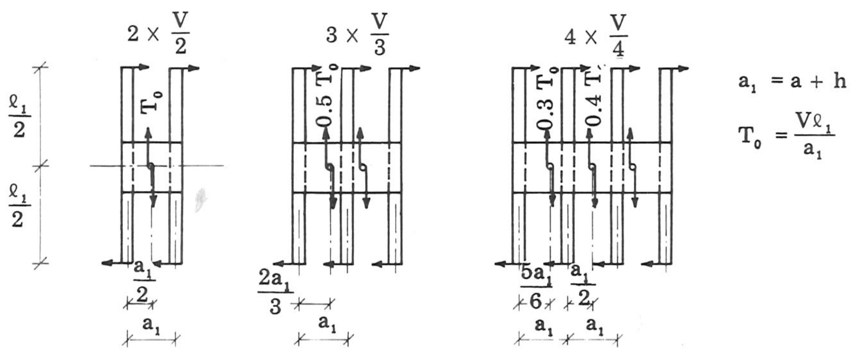


Fig. 72.2

72.6 References

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

The design of built-up columns. CIB-W18/3-2-1.

ANNEX 73
LATTICE COLUMNS WITH GLUED OR NAILED JOINTS

73.1 Scope

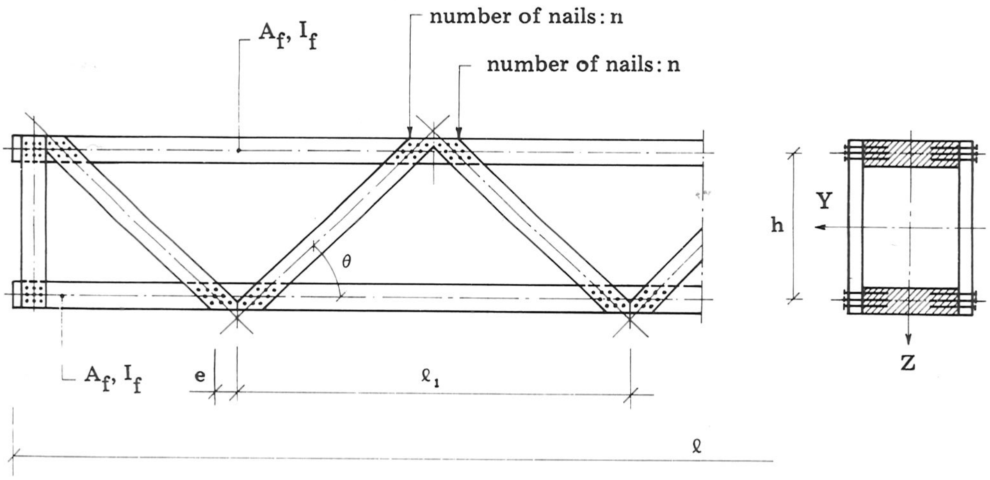
Lattice columns with N- or V-lattice and with glued or nailed joints are dealt with.

Expressions are given to determine an effective moment of inertia and thus an effective slenderness ratio whereupon the critical column stress is determined as for a column of solid timber with the same slenderness ratio.

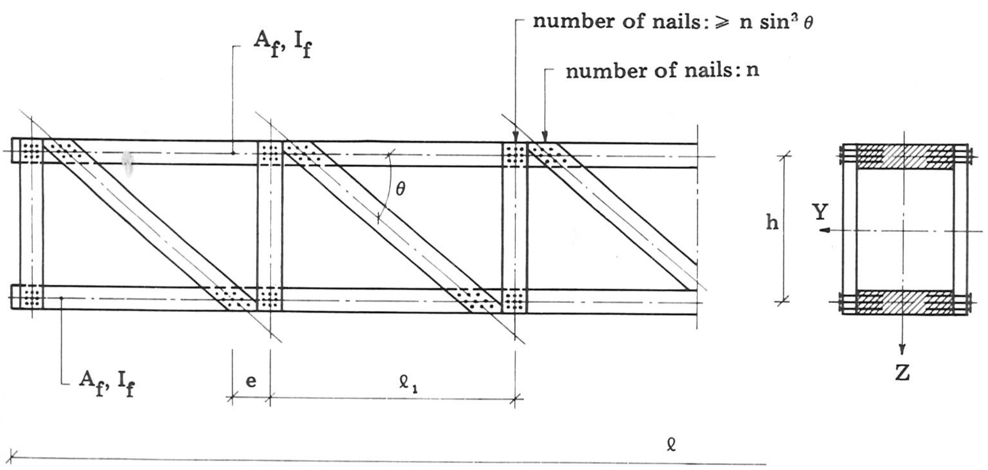
It is assumed that the joints are designed for forces as stated in sections 73.3 and 73.5.

73.2 Notations

Reference is made to fig. 73.1.



V-truss



N-truss

Fig. 73.1

A Area

A_f Area of one flange

F Axial load on column

I Moment of inertia (second moment of area)

I_f I for one flange about own axis

I_{tot} Total value: $I_{tot} = 2I_f + \frac{1}{2} A_f h^2$

K Slip modulus for one nail, i.e. the force per nail that will cause a slip of 1

V Shear force

e Eccentricity

$f_{c,0}$ Compression strength

h Depth of column (flange centre distance)

i_f Radius of gyration ($= \sqrt{I_f/A_f}$)

k_c Column factor

ℓ_1 Joint distance

n Number of nails per diagonal in a joint. If the diagonal consists of two or more pieces, n is the sum of nails, not the number of nails per shear face

θ Angle between a flange and a diagonal

λ Slenderness ratio

λ_{ef} Effective slenderness ratio

λ_f Slenderness ratio of a flange (ℓ_1/i_f)

μ Parameter, see section 73.4.

73.3 Assumptions

The structure is assumed to be symmetric about the Y- and Z-axes of the cross-section. However, the lattice of the two sides are allowed to be staggered the length $\ell_1/2$.

There should be at least 3 fields, i.e. $\ell >$ about $3\ell_1$.

The column should be designed for a shear force V, as stated in section 73.5, i.e. the diagonals and joints should be designed for $V/\sin\theta$.

In nailed structures at least 4 nails per shear should be used in each diagonal in each nodal point. At each end bracings should be used.

The slenderness ratio for the individual flange corresponding to the length ℓ_1 must not exceed 60, i.e.

$$\lambda_f \leqslant 60$$

Besides, it assumed that no local rupture is occurring in the flanges corresponding to the column length λ_1.

73.4 Load-carrying capacity

The load-carrying capacity corresponding to deflection in the Y-direction is equal to the sum of the load-carrying capacity of the flanges for deflection in this direction.

For deflection in the Z-direction the load-carrying capacity is assumed to be equal to

$$F_{crit} = 2 k_c f_{c,0} A_f$$

where the column factor k_c is determined as for a corresponding column of solid timber, but instead of the geometrical slenderness ratio of the column

$$\lambda = \ell \sqrt{\frac{2A_f}{I_{tot}}}$$

the effective slenderness ratio

$$\lambda_{ef} = \lambda \sqrt{1 + \mu} \qquad = \sqrt{\lambda^2 + \mu \lambda^2} \qquad = \sqrt{\lambda^2 + \mu \cdot e^2 \frac{2A_f}{2J_F + \frac{1}{2}A_f \ell^2}}$$

is used, where μ is determined as stated below:

Glued V-truss:

$$\mu = 4 \left(\frac{e}{i_f}\right)^2 \left(\frac{h}{\ell}\right)^2$$

λ_{ef} is not to be taken less than 1.05 λ.

Glued N-truss:

$$\mu = \left(\frac{e}{i_f}\right)^2 \left(\frac{h}{\ell}\right)^2$$

λ_{ef} is not to be taken less than 1.05 λ.

Nailed V-truss:

$$\mu = 25 \frac{hEA_f}{\ell^2 nK \sin 2\theta} \qquad \sim \lambda_{ef} = \sqrt{\lambda^2 + \frac{50 \cdot A_f \cdot \ell}{2J_F + \frac{1}{2}A_f \ell^2} \cdot \frac{EA_f}{nK \sin 2\theta}}$$

Nailed N-truss:

$$\mu = 50 \frac{hEA_f}{\ell^2 nK \sin 2\theta}$$

≙ DIN 1052 $\qquad a_1 = \frac{\ell}{2}$

73.5 Shear forces

The column should be designed for a shear force V given by

$$V = \begin{cases} \frac{F}{60} \frac{1}{k_c} & \text{for } 60 \leqslant \lambda_{ef} \\ \frac{\lambda_{ef}}{60} \frac{F}{60} \frac{1}{k_c} & \text{for } 30 \leqslant \lambda_{ef} \leqslant 60 \\ \frac{F}{120} \frac{1}{k_c} & \text{for } \lambda_{ef} \leqslant 30 \end{cases}$$

73.6 References

K. Möhler, J. Ehlbeck, G. Hempel & P. Köster: Erläuterungen zu DIN 1052, Blatt 1 und 2 - Holzbauwerke - Ausgabe Oktober 1969 with further references.

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