UDC 694.01.001.24:624.011.1 DEUTSCHE NORM April 1988

Structural use of timber

Design and construction

DIN 1052 Part 1

Holzbauwerke; Berechnung und Ausführung

This standard, together with DIN 1052 Part 2, April 1988 edition, supersedes October 1969 edition.

In keeping with current practice in standards published by the International Organization for Standardization (ISO), a comma has been used throughout as the decimal marker.

The DIN 1052 series of standards includes:

DIN 1052 Part 1 Structural use of timber; design and construction

DIN 1052 Part 2 Structural use of timber; mechanically fastened joints

DIN 1052 Part 3 Structural use of timber; buildings in timber frame construction; design and construction

References in this standard to DIN 1052 Part 2 are to the April 1988 edition.

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Tr. ation prepared by Fachtechnisches Übersetzungsinstitut Henry G. Freeman, Düsseldorf, edited by *DIN-Sprachendienst*.

1 Field of application

This standard deals with the design and construction of structures and of loadbearing structural members*) and stiffening elements made from timber and wood-based panel products; it also covers temporary structures (see DIN 4112), scaffolding and falsework, bracing and formwork supports (see DIN 4420 Parts 1 and 2 and DIN 4421) and wooden bridges (see DIN 1074) unless otherwise specified in these standards.

The other Parts in this series, Parts 2 and 3, specify requirements relating to mechanically fastened joints and buildings in timber frame construction respectively.

2 Concepts

2.1 Solid timber and glued laminated timber

2.1.1 Solid timber

The term 'solid timber' is understood to cover logs and converted timber (square-sawn timber, planks, boards and battens) of softwood and hardwood.

2.1.2 Glued laminated timber

Glued laminated timber (BSH) is timber that consists of at least three softwood boards or laminations glued together horizontally, parallel to the grain (see also subclause 12.6).

2.2 Wood-based panel products

For the purpose of this standard, a distinction is made between the following types of wood-based panel product.

a) Grade 100 or 100 G plywood conforming to DIN 68705 Part 3 (BFU) and Part 5 (BFU-BU) for panels as specified in clause 11, and, for floor decks, the above and grade 20 plywood conforming to DIN 68705 Part 3 (BFU).
b) Grade 100 or 100 G particle board for timber panels as specified in clause 11 and, for floor sheathing, the above and grade 20 particle board, conforming to DIN 68763 in each case.
c) Fibre building board, i.e. hardboard and mediumboard as specified in DIN 68754 Part 1 (used only for buildings in timber frame construction; see DIN 1052 Part 3).

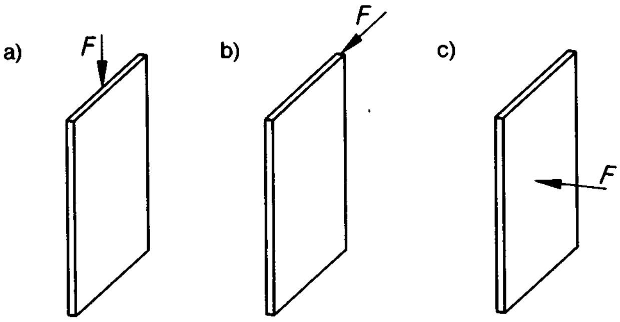
2.3 Timber panels, sheathing and roof decking

2.3.1 Timber panels

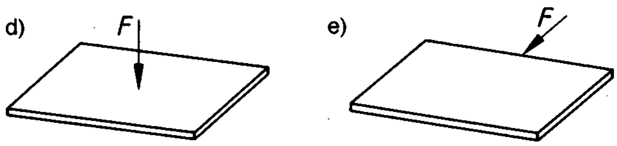
Timber panels are composite members comprising studs made from structural timber, glued laminated timber or

*) Translator's note. In this standard, the term 'structural member' shall be understood to include the terms 'structural component' and 'structural element'.

wood-based panel products and sheathing made of timber or wood-based panel products, which are located on either one or both sides of the studs and contribute to loadbearing strength or are for stiffening. Timber panels ('panels', for short) are used as loadbearing wall, floor or roof panels subject to the loads shown in figure 1.



a) to c) Wall panels



d) and e) Floor or roof panels

Figure 1. Loadbearing panels with their directions of loading

2.3.2 Sheathing

Sheathing is a building element that either contributes to loadbearing strength if it is designed for load resistance and transmission, or has a stiffening function if it is intended only to ensure the lateral stability of the studs.

2.3.3 Roof decking

Roof decking is a plane, loadbearing structural member consisting of solid timber boards or planks or of wood-based panel products which support the roofing and are walked on for cleaning and maintenance purposes only.

3 Structural analysis and drawings

3.1 Structural analysis

3.1.1 The structural analysis, which shall be well set out and easy to check, shall provide particularly the following information:

a) design loads;
b) construction materials to be used;
c) dimensions of loadbearing members, including their cross-sectional shapes and dimensions;
d) stresses to which members and joints are designed to be subjected;
e) any deformations and cambers.

3.1.2 A structural analysis may be dispensed with if it is obvious that members and joints are adequately designed.

3.2 Drawings

3.2.1 It is generally required that the calculations forming the structural analysis be supplied with drawings which include details of the dimensions and cross sections of loadbearing members, the detailing of joints and bracing, the number and arrangement of fasteners, any necessary camber and other information of special relevance.

3.2.2 Where necessary, drawings shall show the positions of fasteners in different planes and, in the case of nails, the side on which the head is located.

3.3 Specification of works

Information required for the execution of the construction work (including transport and erection) or for checking the structural analysis and the drawings, but which is not given in the documents listed in subclauses 3.1 and 3.2, shall be detailed in a specification of works.

3.4 Designation

In the structural analysis, in drawings and, where necessary, in the specification of works, each construction material or member shall be identified by its standard designation.

The designation of timber as set out in table 1 shall fulfil at least the following conditions.

a) Timber as in table 1, line 1, shall be identified by the symbol NH and the appropriate grade.
b) Glued laminated timber as in table 1, line 2, shall be identified by the symbol BSH and the appropriate grade.
c) Timber as in table 1, line 3, shall be identified by the symbol LH and a letter denoting its timber type (i.e. A, B or C).

If, for plywood conforming to DIN 68705 Part 3 or Part 5, or particle board as specified in DIN 68763, design moduli of elasticity or shear higher than those given in table 2 or table 3, footnote 1, are taken as a basis, this shall be stated clearly as well as providing the standard designation of the wood-based panel product.

If, for the purposes of stress analysis in the cases specified in subclause 12.3, the reduction in cross section, v, is not taken into consideration for finger-jointed cross sections, this shall also be made clear in the designation of members as used in the structural analysis and drawings.

The designation of mechanical fasteners shall include details relevant to their design and finish in accordance with DIN 1052 Part 2.

Note. Where construction materials and members have obtained building inspectorate approval, the designation contained in the agrément certificate shall have priority.

4 Material characteristics

4.1 Moduli of elasticity, shear and torsion

4.1.1 The moduli of elasticity and shear, required for calculation of elastic dimensional changes, are given in table 1 for solid timber and glued laminated timber, table 2, for plywood conforming to DIN 68705 Parts 3 and 5 and table 3, for particle board as specified in DIN 68763.

Twisting of solid timber and glued laminated timber may be calculated in a simplified manner by the elastic theory as used for isotropic materials. For this purpose, the modulus of torsion, G_T, may be taken as being equal to ⅔ G for solid wood and equal to G for glued laminated timber.

4.1.2 The moduli of elasticity and shear shall be reduced in the following cases.

a) For solid timber and glued laminated timber in members which are exposed to the weather on all sides or which are liable to be exposed to short-term soaking, a reduction of one-sixth is required.
b) For members exposed to permanent soaking, e.g. such in constant contact with water, a reduction of one-quarter is required.

A reduction for moisture is not necessary for hardwood of type C (see table 1).

Where use is made of BFU 100 G plywood and of V 100 G particle board in which a moisture content (measured in accordance with DIN 52183) of more than 18% is to be expected over a prolonged period (i.e. of several weeks), the moduli of elasticity and shear shall be reduced by one-quarter for BFU 100 G plywood and by one-third for V 100 G particle board (see DIN 68 800 Part 2).

4.2 Moisture and shrinkage

4.2.1 The equilibrium moisture content is deemed to be the level at which the moisture in timber and wood-based panel products in a finished structure remains approximately constant after a given period in service.

The following values shall be taken to represent the equilibrium moisture content:

a) structures enclosed on all sides:
   - heated, (9 ± 3) %;
   - unheated, (12 ± 3) %;

b) open structures with horizontal covering: (15 ± 3) %;

c) structures exposed to the weather on all sides: (18 ± 6) %.

4.2.2 If, at the time of installation, timber has a moisture content higher than that specified in subclause 4.2.1, its use shall be limited to structures in which it is given the opportunity to dry out and to members which are not susceptible to deformation due to shrinkage.

Table 1. **Design moduli of elasticity and shear, in MN/m², for solid timber and glued laminated timber** (with a moisture content of not more than 20%)

	Type of timber	Modulus of elasticity parallel to grain, $E_{\parallel}$	Modulus of elasticity perpendicular to grain, $E_{\perp}$	Modulus of shear, G
1	European whitewood, Scots pine, fir, European larch, Douglas fir, Southern pine, Western hemlock 1)	10 000 2) 3)	300 4)	500
2	Glued laminated timber made from timber types as in line 1	11 000	300	500
3	Hardwood of type			
	A Oak, beech, teak, keruing (Yang)	12 500	600	1 000
	B Afzelia, merbau, basralocus	13 000	800	1 000
	C Ekki, greenheart	17 000 5)	1 200 5)	1 000 5)

1) Botanical names: Picea abies Karst. (European whitewood), Pinus sylvestris L. (Scots pine), Abies alba Mill. (fir), Larix decidua Mill. (European larch), Pseudotsuga menziesii Franco (Douglas fir), Pinus palustris (Southern pine), Tsuga heterophylla Sarg. (Western hemlock).
2) $E_{\parallel}$ for grade III timber is equal to 8000 MN/m².
3) $E_{\parallel}$ for round timber is equal to 12 000 MN/m².
4) $E_{\perp}$ for grade III timber is equal to 240 MN/m².
5) These values are applicable irrespective of the moisture content.

Table 2. **Design moduli of elasticity and shear, in MN/m², for structural plywood conforming to DIN 68 705 Parts 3 and 5**

	Type of stress	Modulus of elasticity, E 1) 2) 3): parallel to grain of face veneer, Number of plies 3	parallel, ≥5	perpendicular to grain of face veneer, Number of plies 3	perpendicular, ≥5	Modulus of shear, G 1) 2) 4): parallel and perpendicular to grain of face veneer, Number of plies ≥3
1	Bending about axis perpendicular to plane of board	8000	5500	400	1500	250 (400)
2	Bending, compression and tension in plane of board	4500		1000	2500	500 (700)

1) Higher values are permissible if validated by a test certificate following third party inspection of the production process.
2) The design moduli of elasticity and shear shall be reduced by one-fifth for plywood made from okoume and poplar.
3) The values specified in Supplement 1 to DIN 68 705 Part 5 shall apply to beech plywood conforming to DIN 68 705 Part 5.
4) Values in brackets apply to beech plywood conforming to DIN 68 705 Part 5.

Table 3. **Design moduli of elasticity and shear, in MN/m², for particle board conforming to DIN 68763**

	Type of stress		Modulus of elasticity, E [1]						Modulus of shear, G [1]					
			Nominal thickness of board, in mm						Nominal thickness of board, in mm					
			Up to 13	Over 13 up to 20	Over 20 up to 25	Over 25 up to 32	Over 32 up to 40	Over 40 up to 50	Up to 13	Over 13 up to 20	Over 20 up to 25	Over 25 up to 32	Over 32 up to 40	Over 40 up to 50
1	Bending	perpendicular to plane of board	3200	2800	2400	2000	1600	1200	200			100		
2	Bending	in plane of board	2200	1900	1600	1300	1000	800	1100	1000	850	700	550	450
3	Tension and compression in plane of board		2200	2000	1700	1400	1100	900	–					

[1]) Higher values are permissible if validated by a test certificate following third party inspection of the production process.

4.2.3 Percentage shrinkage and swell coefficients to be used in the design of timber perpendicular to the grain and for wood-based panel products in the plane of the board are given in table 4.

4.2.4 Shrinkage or swelling of timber parallel to the grain or of wood-based panel products in the plane of the board need be taken into account in special cases only, since coefficients are 0,01 % on average. Shrinkage or swelling perpendicular to the plane of wood-based panel products may be disregarded.

4.2.5 Where shrinkage or swelling is hindered, the figures in table 4 and subclause 4.2.4 may be taken at half their value.

4.2.6 The classification of wood-based panel products shall be on the basis of their anticipated wet exposure stress, as specified in DIN 68800 Part 2.

Table 4. **Design percentage shrinkage and swell coefficients**

	Wood type	Percentage shrinkage and swell coefficient for changes in moisture content of 1 % below grain saturation zone
1	European whitewood, Scots pine, fir, European larch, Douglas fir, Southern pine, Western hemlock, glued laminated timber, oak	0,24 [1])
2	Beech, keruing, basralocus greenheart	0,3 [1])
3	Teak, afzelia, merbau	0,2 [1])
4	Ekki	0,36 [1])
5	Plywood	0,020 [2])
6	Particle board	0,035 [2])

[1]) Mean from coefficients measured tangentially and radially to annual ring or growth zone.

[2]) These values apply in the plane of the board.

4.3 Deformation due to creep

For the analysis of bending as described in subclause 8.5 and for calculating torsion, deformation due to creep induced by self-weight shall, where necessary, be taken into consideration.

Creep deformation in flexural members shall be assumed to be proportional to their elastic deformation, an analysis being required if the self-weight exceeds 50 % of the overall load.

For single-span beams with a self-weight, g, and an overall load, q, the creep index, φ, may be calculated in accordance with equation (1).

$$\varphi = \frac{1}{\eta_k} - 1 \qquad (1)$$

An analogous procedure may be adopted for other structural systems and non-uniform load distribution.

For members of timber or plywood with an equilibrium moisture content of not more than 18 %, η_k shall be taken to be as follows:

$$\eta_k = \frac{3}{2} - \frac{g}{q} \qquad (2)$$

However, in cases where the equilibrium moisture content is more than 18 %, equation (3) shall apply.

$$\eta_k = \frac{5}{3} - \frac{4}{3}\frac{g}{q} \qquad (3)$$

Twice the values of φ shall be adopted for particle board, provided that the moisture content of the timber does not consistently lie below 15 % (see DIN 68800 Part 2).

Where applicable, the moduli of elasticity and shear shall be subject to the reductions specified in subclause 4.1.2.

For roofs, the snow load, expressed by the relationship $0{,}5\,(s_0 - 0{,}75) \cdot s/s_0$, shall be treated as a permanent load, s and s_0 signifying the design snow load and the normal snow load respectively, in kN/m², as described in DIN 1055 Part 5.

Deformation due to creep may be disregarded when making an analysis for bending for roofs of dwellings, except where these are flat.

4.4 Thermal effects

Thermal effects may be ignored in the case of timber and wood-based panel products as part of wooden structures.

5 Permissible stresses

5.1 Solid timber and glued laminated timber

5.1.1 Table 5 gives the permissible stresses for load case H for members made of structural timber as specified in DIN 4074 Parts 1 and 2, of glued laminated timber and of mediumboard (see subclauses 5.1.5 to 5.1.12 for cases in which permissible stresses are to be increased or reduced).

5.1.2 In the case of built-up-members, the properties of the assembly, and not those of their elements, normally determine their grading according to DIN 4074 Part 1.

In the case of members subject to bending or coexistent bending and longitudinal force, the elements in the tension zone, considered separately, shall be of grades in which they are stressed to the permissible limits. Where members are of glued laminated timber, this applies at least to the two outer laminations in the tension zone. In the case of built-up tension members, all elements shall be of the required grade.

5.1.3 Permissible stresses for grade I timber, as shown in table 5, are not applicable to rafters, purlins and floor joists of square-sawn timber or planks, or to other members unless the requirements of DIN 4074 Parts 1 and 2 with regard to marking, selection, etc., are met and they are designed and constructed to the highest standards.

5.1.4 In the case of temporary structures (see DIN 4112), only those timbers which conform to the requirements of grade I as specified in DIN 4074 Parts 1 and 2 may be used for the members making up the main loadbearing system.

5.1.5 Permissible compressive stresses where the force acts obliquely to the direction of the grain (see figure 2) shall be calculated by means of the following equation:

$$zul\ \sigma_{D\measuredangle} = zul\ \sigma_{D\parallel} - (zul\ \sigma_{D\parallel} - zul\ \sigma_{D\perp}) \cdot \sin\alpha \qquad (4)$$

where α is the angle between the line of force and the grain.

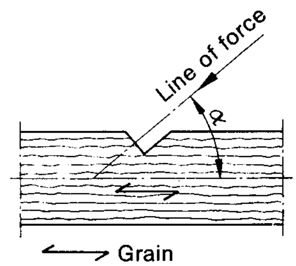


Figure 2. Force acting at an angle to direction of grain

5.1.6 The permissible stresses listed in table 5 may be increased by 25 % for load case HZ (see subclause 6.2.2), by 100 % for impact loads as in DIN 1055 Part 3 and earthquake loads as in DIN 4149 Part 1, and by 50 % to take into account handling loads during transport and erection (see subclause 3.2 of DIN 1052 Part 2 for mechanically fastened joints).

5.1.7 Allowance for wet exposure

The permissible dry exposure stresses given in table 5 shall be reduced

a) by one-sixth for members exposed to the weather on all sides or in which an equilibrium moisture content of more than 18 % is to be expected, but not for scaffolding, and

b) by one-third for members and scaffolding which are in permanent contact with water, and for scaffolding constructed from timber which, at the time of loading, has not yet attained semi-dryness (see DIN 4074 Parts 1 and 2).

Table 5. **Permissible dry exposure stresses for solid timber and glued laminated timber, in MN/m², for load case H**

	Type of stress	Solid timber (of timber types as in table 1, line 1) Grade as in DIN 4074 Parts 1 and 2			Glued laminated timber (of timber types as in table 1, line 1) as specified in subclause 12.6 Grade as in DIN 4074 Part 1		Solid timber (of hardwood as in table 1) Timber type		
							A	B	C
		III	II	I	II	I	Medium grade 1)		
1	Bending, $zul\ \sigma_B$	7	10	13	11	14	11	17	25
2	Tension, $zul\ \sigma_{Z\parallel}$	0	8,5	10,5	8,5	10,5	10	10	15
3	Tension, $zul\ \sigma_{Z\perp}$	0	0,05	0,05	0,2	0,2	0,05	0,05	0,05
4	Compression, $zul\ \sigma_{D\parallel}$	6	8,5	11	8,5	11	10	13	20
5a 5b	Compression, $zul\ \sigma_{D\perp}$	2 2,5 2)	2 2,5 2)	2 2,5 2)	2,5 3,0 2)	2,5 3,0 2)	3 4 2)	4 –	8 –
6	Shear, $zul\ \tau_a$	0,9	0,9	0,9	0,9	0,9	1	1,4	2
7	Shear due to transverse force ('transverse shear', for short), $zul\ \tau_Q$	0,9	0,9	0,9	1,2	1,2	1	1,4	2
8	Torsion 3), $zul\ \tau_T$	0	1	1	1,6	1,6	1,6	1,6	2

1) At least grade II, on the lines of DIN 4074 Parts 1 and 2.

2) Use of these values will normally involve greater compression effects which will need to be taken into due account in the design. These values shall not be used for joints featuring more than one type of connection.

3) The values in line 7 shall be applicable for box sections.

Reductions are not permissible with regard to hardwood of type C or to temporary structures having a protective coating which is to be renewed at intervals of not more than two years.

5.1.8 In the case of continuous beams without hinges, the bending stress above the inner supports may exceed by 10 % the permissible values given in table 5, line 1. This does not, however, apply to rafters of collar beam trusses with collar beams able to move horizontally.

5.1.9 In the case of round timber, the permissible bending and compressive stresses given in table 5, lines 1 and 4, may be increased by 20 % if there is no reduction in cross section at its edge.

5.1.10 In the case of nailed joints in tension, the permissible stresses given in table 5, line 2, shall be reduced by 20 % for those components to be joined, which are not to be designed to sustain 1,5 times the proportionate tensile load component as set out in subclause 7.3.

5.1.11 Where compression acts at right angles to the direction of the grain, the outstand, $\ddot{u}$, of beams and sills beyond the compression area parallel to the grain on one or both sides shall be at least 100 mm where h is greater than 60 mm and at least 75 mm where h is not more than 60 mm. A distance of at least 150 mm shall be allowed between two compression areas.

In the case of compression areas, the length, l, of which, in the direction of the grain, is less than 150 mm (see figure 3), the permissible compressive stress given in table 5, line 5a, may be multiplied by the following factor:

$$k_{\mathrm{D}\perp} = \sqrt[4]{\frac{150}{l}} \qquad (5)$$

which shall on no account exceed 1,8, l being the length of the compression area, in mm.

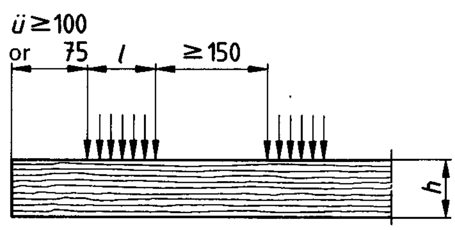


Figure 3. Load configuration for compression areas of short length

If the outstand is less than that stated in the first paragraph, the permissible stresses given in table 5, lines 5a and 5b, shall be reduced by multiplying by a factor, $k_{\mathrm{D}\perp}$ equal to 0,8.

5.1.12 In the case of continuous or cantilever flexural beams made of softwood or hardwood of type A, the permissible transverse shear, $zul\ \tau_{\mathrm{Q}}$, given in table 5, line 7, may be increased to 1,2 MN/m^2 in areas at least 1,50 m from the end.

5.2 Wood-based panel products

5.2.1 For load case H, the stresses specified in table 6 are permitted for components made of wood-based panel products.

For plywood conforming to DIN 68705 Part 3, the permissible stresses in the plane of the board, $\sigma_{\mathrm{Z,D}}$, are equal to 2 MN/m^2, where the angle α between the line of force and the grain at the outer fibre is between 30° and 60°. Where the angle α is between 0° and 30°, values may be linearly interpolated between 8 MN/m^2 and 2 MN/m^2, and where α is greater than 60° but not greater than 90°, between 2 MN/m^2 and 4 MN/m^2.

5.2.2 Subclause 5.1.6 shall apply analogously.

5.2.3 Allowance for wet exposure conditions

Where BFU 100 G plywood and V 100 G particle board are used in which a moisture content of more than 18 % is to be expected over a period of several weeks, the permissible stresses for BFU 100 G plywood shall be reduced by one-quarter and those for V 100 G particle board, by one-third.

5.3 Other materials

5.3.1 Other construction materials shall be subject to the specifications contained in the appropriate standards.

5.3.2 DIN 18 800 Part 7 shall apply to welded steel members.

5.3.3 In the case of straight members of steel flats and rounds, which are not provided with a certificate of compliance with the order as specified in DIN 50 049, subclause 2.1, the tensile and bending stresses for load cases H and HZ shall not exceed 110 MN/m^2, or 100 MN/m^2 in the core cross section of rounds.

5.3.4 DIN 55 928 Parts 1, 2, 4, 5, 6 and 8 shall be complied with as regards the protection of steel components against corrosion, and DIN 4113 Part 1 as regards corrosion protection of aluminium components.

6 Design principles

6.1 General

Special attention shall be paid to the three-dimensional stiffening of construction components and to their stability. The consequences for the stability of the structure as a whole of the failure of single members shall be evaluated and, where necessary, suitable precautions taken for their limitation.

6.2 Design loads

6.2.1 Loads

Design loads for the analysis of stability shall be based on the relevant standards.

Loads acting on a structure are classed as principal, secondary and special loads.

Principal loads are as follows:

a) self-weight;
b) imposed loads (including snow load but excluding wind loads);
c) machinery loads;
d) lateral loads on stiffening elements (see clause 10), where they are due to principal loads.

Secondary loads are as follows:

a) wind loads;
b) braking forces;
c) horizontal lateral loads (e.g., from cranes);
d) imposed deformations caused by thermal gradients and changes in moisture content;
e) lateral loads on constructions providing lateral restraint, where such loads are due to secondary loads.

Special loads are as follows:

a) horizontal impact loads;
b) earthquake loads.

6.2.2 Load cases

For purposes of stability analysis, a distinction is drawn between the following load cases:

a) load case H: total of principal loads;
b) load case HZ: total of principal and secondary loads.

If, besides self-weight, a member is subjected only to secondary loads, the greatest of them shall be deemed to be the principal load.

Concentrated loads as described in DIN 1055 Part 3 shall always be classed as secondary loads.

See subclause 5.1.6 for horizontal impact loads and earthquake loads.

6.3 Minimum cross-sectional areas

6.3.1 Non-composite, loadbearing solid timber members shall be at least 24 mm wide and have a cross-sectional area of at least 14 cm^2 (11 cm^2 for battens). Greater minimum dimensions may be required to suit certain types of fasteners.

See subclause 12.6 for dimensions of the individual plies of plywood.

6.3.2 See subclause 11.1.1 for minimum thickness of panels.

6.3.3 The minimum thickness of loadbearing boards made of wood-based panel products shall be 8 mm for particle board and 6 mm for plywood. Plywood shall comprise at least three plies where it is intended to provide lateral restraint only, and at least five plies for all other loadbearing functions.

6.4 Reduction in cross-sectional area

6.4.1 Wanes not exceeding the permissible limits for width as specified in DIN 4074 Part 1 need not be taken into consideration.

6.4.2 In the stress analysis, any reduction in cross-sectional area for tension members and in the tension zone of flexural members (due to bolt holes, tapering for joints, etc.) shall be taken into account.

Reductions in cross-sectional area located in a line parallel to the grain shall be allowed for once only. This also applies to reductions staggered across the grain provided that their clear spacing is more than 150 mm, or not less than 4 d where joints are dowelled.

In the case of DIN 68140 finger joints, the reduction in cross section at the base of the joint needs to be allowed for once only (see subclause 12.3). Reductions in cross-sectional area due to holes to receive connectors or precision bolts shall be based on their diameter, d_{st}, whereas they shall be based on the bolt hole diameter, d_b (+ 1 mm), for other bolts.

In the case of connectored joints, the corresponding perforation areas, ΔA, shall be deducted alongside that of the bolt hole in question (see figure 4 for an example of a reduction in cross-sectional area for split-ring connectors).

In the case of joints using special connectors, the perforation areas shall be taken from tables 4, 6 and 7 of DIN 1052 Part 2.

Any reduction in cross-sectional area due to nailholes shall be on the basis of the nail diameter where nails are more than 4,2 mm in diameter or where nails are 4,2 mm or less and nailholes are pre-drilled. This also applies to nailholes in plywood irrespective of the size of the nail or whether or not nailholes are pre-drilled.

Table 6. **Permissible stresses for wood-based panel products, in MN/m^2, for load case H**

	Type of stress	Plywood as in DIN 68705 Parts 3 and 5 [1]) – Parallel to the grain of outer veneers – Number of plies: 3	5 or more	Perpendicular to the grain of outer veneers – Number of plies: 3	5 or more	DIN 68763 particle board – Nominal thickness of board, in mm: Up to 13	Over 13 up to 20	Over 20 up to 25	Over 25 up to 32	Over 32 up to 40	Over 40 up to 50
1	Bending, perpendicular to plane of board, zul σ_{Bxy}	13		5		4,5	4,0	3,5	3,0	2,5	2,0
2	Bending, in plane of board, zul σ_{Bxz}	9		6		3,4	3,0	2,5	2,0	1,6	1,4
3	Tension, in plane of board, zul σ_{Zx}	8		4		2,5	2,25	2,0	1,75	1,5	1,25
4	Compression, in plane of board, zul σ_{Dx}	8		4		3,0	2,75	2,5	2,25	2,0	1,75
5	Compression, at right angles to plane of board, zul σ_{Dz}	3 (4,5)		3 (4,5)		2,5	2,5	2,5	2,0	1,5	1,5
6	Shear, in plane of board and on glueline, zul τ_{zx} [2])	0,9 (1,2)		0,9 (1,2)		0,4	0,4	0,4	0,3	0,3	0,3
7	Shear, at right angles to plane of board, zul τ_{yx} [2])	1,8 (3)	3 (4)	1,8 (3)	3 (4)	1,8	1,8	1,8	1,2	1,2	1,2
8	Bearing stress [3]) [4]), zul σ_l	8		4		6,0	6,0	6,0	6,0	6,0	6,0

[1]) The values in brackets apply to plywood conforming to DIN 68705 Part 5 and Supplement 1 to DIN 68705 Part 5. Other values may be obtained by multiplying the strength values specified in DIN 68705 Part 5 by a safety factor of 3.

[2]) The values also apply to transverse shear.

[3]) For bolts and dowels.

[4]) zul $\sigma_l = 2 \cdot$ zul σ_{Dx} for plywood as specified in DIN 68705 Part 5 with at least five plies.

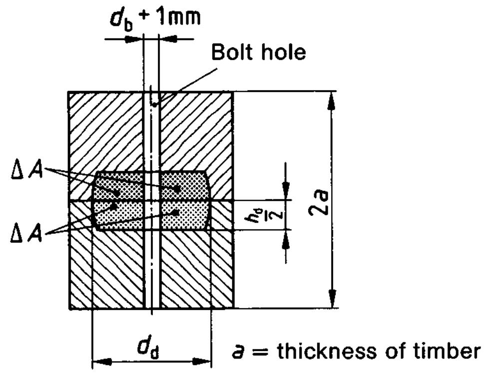


Figure 4. Reductions in cross-sectional area due to split-ring connector joints

In this case, the perforation area shall be calculated as follows:

$$\Delta A = (d_\mathrm{d} - (d_\mathrm{b} + 1)) \cdot \frac{h_\mathrm{d}}{2}$$

Any reduction in cross-sectional area due to screw holes shall be based on the screw shaft diameter.

6.4.3 In the case of compression members and of members subject to bending in the compression zone, any reduction in cross-sectional area need not be taken into account when performing a general stress analysis unless the point at which the cross section is reduced is not completely filled in or if the filling material has a lower modulus of elasticity than the principal material (e.g., if the grain of packing is perpendicular or at an angle to that of the compression member).

6.4.4 If reductions in the cross section result in significant eccentricity, this shall be taken into due account in the analysis.

6.5 Members subject to alternating stresses

6.5.1 Members in which a change from positive to negative stress or vice versa is not solely due to wind and snow loads shall be designed to resist the following stress, $zul\ \sigma'$:

$$zul\ \sigma' = k_\mathrm{w} \cdot zul\ \sigma \tag{6}$$

where

$$k_\mathrm{w} = 1 - 0{,}25\ \frac{min\ |\sigma|}{max\ |\sigma|} \tag{7}$$

$min\ |\sigma|$ and $max\ |\sigma|$ denoting the stress of the smallest or largest absolute magnitude respectively.

6.5.2 Joints shall be designed on the same lines.

6.6 Eccentricity in joints

Special consideration shall be given to stresses resulting from eccentricity in joints.

Where possible, truss members shall be connected axially. Stresses due to eccentricity do not normally need to be verified for nailed joints as shown in figure 5a or made with punched metal plate fasteners or gussets as shown in figure 5b provided that the eccentricity, e_1 or e_2, is not greater than half the height of the chord.

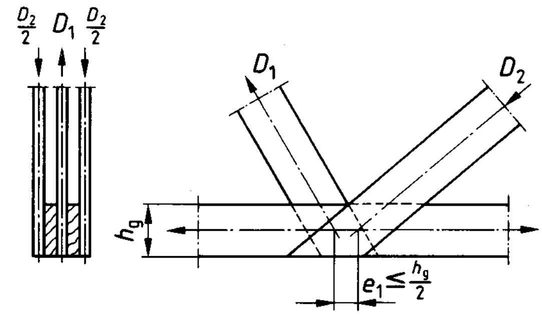
7 Design principles for tension members

7.1 Axial tension

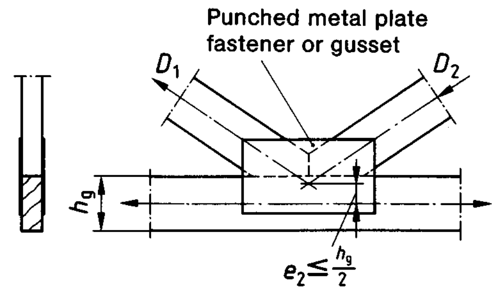
For tension members designed to be axially stressed, maintenance of the permitted stress limits shall be verified in accordance with subclause 6.4, allowance being made for any reduction in cross-sectional area:

$$\frac{\frac{N}{A_\mathrm{n}}}{zul\ \sigma_{\mathrm{Z}\parallel}} \leq 1 \tag{8}$$

where A_n is the effective cross-sectional area, $zul\ \sigma_{\mathrm{Z}\parallel}$ being taken from table 5 or table 6.



a) for nailed board and plank trusses



b) for trusses with punched metal plate fasteners or gussets

Figure 5. Eccentric truss connection

7.2 Eccentric tension (coexistent tension and bending)

For tension members designed to be stressed eccentrically or to be subjected to additional stressing, transversely to the member axis, it shall be demonstrated that the following condition is met.

$$\frac{\frac{N}{A_\mathrm{n}}}{zul\ \sigma_{\mathrm{Z}\parallel}} + \frac{\frac{M}{W_\mathrm{n}}}{zul\ \sigma_\mathrm{B}} \leq 1 \tag{9}$$

where W_n is the effective moment of resistance, $zul\ \sigma_{\mathrm{Z}\parallel}$ and $zul\ \sigma_\mathrm{B}$ being taken from table 5 or 6.

7.3 Joints

Joints shall normally be located symmetrically to the axis or axes of the members. Components made from timber or wood-based panel products subjected to stressing on one side only shall be designed to sustain 1,5 times the tensile load component.

8 Design principles for flexural members

8.1 Principles

8.1.1 Effective spans

8.1.1.1 The effective span, l, shall be taken as the distances between centres of bearings. This also applies where bearings are of masonry or concrete, except where these support single-span beams, in which case l shall be not more than 1,05 times the clear width.

8.1.1.2 Continuous timber boards or planks, or boards made of wood-based panel products shall normally be treated as freely rotating simply supported beams.

For roof decking and floor boarding, continuous timber boards or planks, or boards made of wood-based panel products shall be treated as such for calculation purposes, provided that any joints are individually designed.

8.1.1.3 Subclause 8.2.4 shall apply to purlins and beams fitted with braces or bolsters.

8.1.2 Bearing reactions

Bearing reactions of continuous beams except two-span beams (including purlins) may, as a rule, be calculated as for single-span beams, provided that the ratio of adjacent spans is between 2 : 3 and 3 : 2.

8.1.3 Butt joints

At butt joints, it shall be ensured that action effects are transmitted through connecting elements and fasteners. When performing a deformation analysis, or a structural analysis of hyperstatical systems, the connecting elements and, if necessary, the likelihood of the fasteners to slip at the joint shall be taken into consideration when accounting. In the case of compression chords of solid web beams, the required second moment of area shall be assumed for the connecting elements; here, the fasteners may, where contact joints are provided, be designed to resist half the compressive force.

8.1.4 Effective load width

If evidence is provided that boards of wood-based panel products, used for roof decking and directly loaded floor sheathing, or upper roof or floor boarding, connected together by tongue and groove or similar joints are able to sustain a concentrated load of 1 kN, the largest effective load width, t, as given in table 7 may be taken as the effective board width.

The effective load width may be taken as 0,35 m where roof decking and floor sheathing consist of timber boards or planks connected together by tongue and groove or similar joints, irrespective of the width of the individual member, or as 0,16 m where the boards or planks are not connected.

Table 7. **Effective load width, t, for boards of wood-based panel products**

	Board width, b	Boards	
		joined together	not joined together
1	Not less than 0,35 m [1])	0,35 m	0,35 m
2	Not less than 1 m [1])	0,70 m	0,35 m
3	Greater than effective span, l	0,7 l	0,35 l
4	Not greater than effective span, l	0,7 b	0,35 b

[1]) Any effective span, l, permitted.

8.2 Solid timber and glued laminated timber flexural members

8.2.1 Design analysis

8.2.1.1 Design for bending

For members subject to bending stress, the stress analysis shall be made according to the following condition, any reduction in cross-sectional area being taken into account as specified in subclause 6.4:

$$\frac{\frac{M}{W_n}}{zul\ \sigma_B} \leq 1 \tag{10}$$

where W_n is the effective moment of resistance, $zul\ \sigma_B$ being taken from table 5, line 1.

In addition, in the case of built-up beams, the stress in the centroids in the tension zones of the chord shall not exceed the values given in table 5, line 2.

In addition, proof of lateral stability shall be provided as specified in subclause 8.6.

8.2.1.2 Design for transverse force

In the case of beams supported at their lower edge and loaded on the upper edge, the analysis of shear stresses in the beams and, where appropriate, in the fasteners at the end and intermediate supports need not be based on the full transverse force unless there are notches or penetrations at these points. The transverse force at a distance of $h/2$ from the edge of the bearing may be used, h being the depth of the beam at the centre of the bearing, even if it is chamfered.

For a concentrated load at a distance, a, not less than $2h$ from the centre of the bearing, the design calculations shall be based on the full transverse force; where a is less than $2h$, the distance assumed shall be not a but a_o (this being equal to $2h$), and the shear shall be reduced in a ratio of $a/(2h)$.

The permitted values for shear, given in table 5, line 7, shall be used in the shear analysis.

8.2.1.3 Design for torsion and transverse force

Analysis of the effects of torsion is not required if torsion is unnecessary for the maintenance of equilibrium, i.e. in the case of rafters, purlins and joists forming part of roofs and floors of ordinary design.

A simplified analysis of torsional stresses may be made on the basis of the elastic theory relating to isotropic materials, the shear stresses determined not exceeding the values given in table 5, line 8.

In cases of coexistent shear due to torsion and transverse force, the following condition shall be satisfied:

$$\frac{\tau_T}{zul\ \tau_T} + \left(\frac{\tau_Q}{zul\ \tau_Q}\right)^m \leq 1 \tag{11}$$

where

m is a coefficient, with a value of 2 for softwood and 1 for hardwood;

τ_T is the shear due to torsion;

τ_Q is the transverse shear;

$zul\ \tau_Q$ is the permissible transverse shear as given in table 5, line 7;

$zul\ \tau_T$ is the permissible shear due to torsion as given in table 5, line 8.

8.2.2 Notches and penetrations in rectangular softwood beams

8.2.2.1 Notches and tenons

For beam ends with sloping or right-angled notches and beams with tenon joints as shown in figure 6, the permissible shear force shall be calculated according to equation (12):

$$zul\ Q = \frac{2}{3} \cdot b \cdot h_1 \cdot k_A \cdot zul\ \tau_Q \tag{12}$$

where

b is the beam width;

$zul\ \tau_Q$ is the permissible transverse shear as given in table 5, line 7;

k_A is the reduction factor allowing for coexistent shear and transverse tensile stresses.

The ratio of notch height, a, to depth of beam, h, shall be not greater than 0,5, a being not greater than 0,50 m.

For unreinforced right-angled notches (see figure 6a), the reduction factor, which shall be not less than 0,3, shall be calculated as follows:

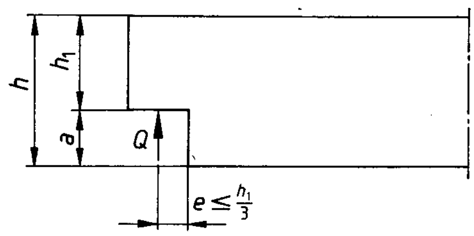
$$k_A = 1 - 2{,}8 \frac{a}{h} \tag{13}$$

For reinforced right-angled notches (see figure 6b), k_A may be taken to be equal to unity. By way of simplification, the reinforcement may be designed to withstand the following tensile force, Z:

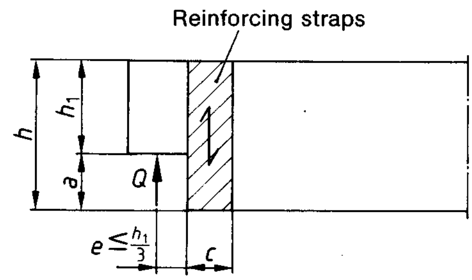
$$Z = 1{,}3\, Q \cdot \left[3 \left(\frac{a}{h} \right)^2 - 2 \left(\frac{a}{h} \right)^3 \right] \tag{14}$$

Straps of grade 100 plywood comprising at least five plies, as specified in DIN 68705 Part 5, may be used as the reinforcement for the notched joint by glueing on both sides with a resorcinal resin adhesive, bonding pressure being generated by nails or staples (see subclause 12.5) if required. The width, c, of these straps shall be between 0,25a and 0,50a.

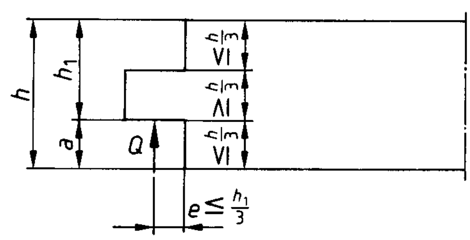
The permissible stresses in the plywood, $zul\ \sigma_{Z\parallel}$, shall be taken as being 4 MN/m^2, and at the glueline, $zul\, \tau_a$, as 0,25 MN/m^2.



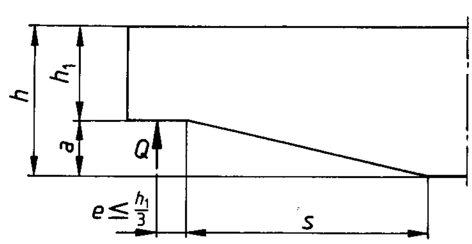
a) Unreinforced right-angled notch



b) Reinforced right-angled notch



c) Tenon



d) Sloping notch

Figure 6. Beams with notches on the underside or tenons

Beams up to 300 mm high with tenons as shown in figure 6c may be designed in accordance with equations (12) and (13), h_1 being assumed to be equal to ⅔ h, unless a more precise analysis is required.

In the case of notches forming an inclined beam edge (see figure 6d), k_A may be taken to be equal to unity, provided that the length, s, is not less than 14 a for grade I and not less than 10 a for grade II, or that s is not less than 2,5 · h, whichever is lower. In this case, however, a need not be more than 0,50 m. The combination of stresses at the inclined edge of the beam shall be taken into account (see subclause 8.2.3.4).

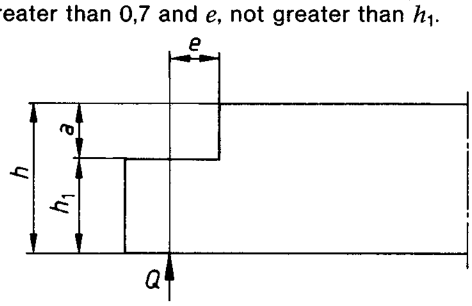
In the case of beams which are notched or chamfered on the top edge, as shown in figure 7, the permissible transverse force shall be calculated in accordance with equation (15):

$$zul\ Q = \frac{2}{3}\, b \cdot \left[h - \frac{a}{h_1} \cdot e \right] \cdot zul\ \tau_Q \tag{15}$$

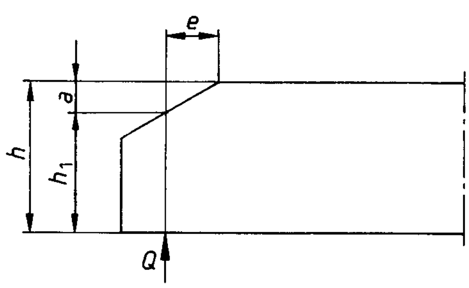
Notching or chamfering shall be subject to the following conditions:

a) where the depth of the beam, h, is greater than 300 mm, $\frac{a}{h}$ shall be not greater than 0,5 and e, not greater than h_1;

b) where h is not greater than 300 mm, $\frac{a}{h}$ shall be not greater than 0,7 and e, not greater than h_1.



a) Right-angled notch



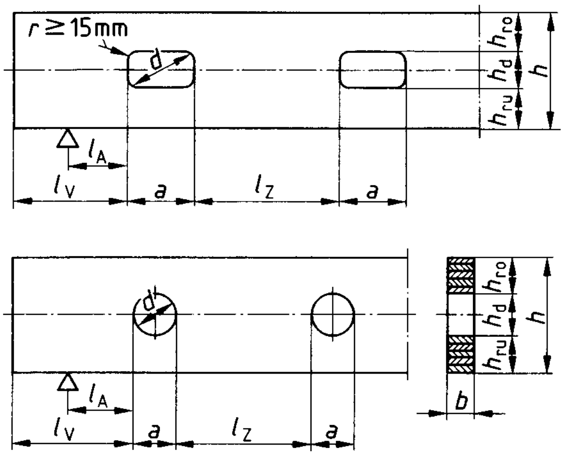
b) Chamfer

Figure 7. Beam notched or chamfered on the top edge

8.2.2.2 Penetrations in glued laminated timber beams

Penetrations are taken, in the present context, to be such openings in beams made of glued laminated timber the clear dimensions, d, of which are greater than 50 mm (see figure 8). Where possible, penetrations shall be sited symmetrically to the beam axis; distance from the top and bottom edges of the beam, h_{ro} and h_{ru} respectively, shall be not less than 0,3h. The distance from the end of the beam, l_V, shall be at least equal to h and the distance, l_o, from the centre of support and from large concentrated loads at least equal to h/2. All corners of glued laminated timber shall be rounded to a radius of at least 15 mm.

Unless a more precise analysis is made, penetrations shall be provided with reinforcement if their largest clear dimension, d, in mm, as a function of the shear stress, τ_Q, occurring in the full cross section of the beam in the centre of the penetration, meets the requirements of either equation (16) or equation (17).



$l_A \geq \frac{h}{2}$; l_V and $l_Z \geq h$; $a \leq h$; h_{ro} and $h_{ru} \geq 0{,}3\,h$; $h_d \leq 0{,}4\,h$

Figure 8. Dimensions and location of penetrations

$$d > 100 - 42\tau_Q \tag{16}$$

$$d > (0{,}1 - 0{,}042\tau_Q) \cdot h \tag{17}$$

where

$$\tau_Q = \frac{1{,}5\,Q}{b \cdot h}, \text{ in MN/m}^2; \tag{18}$$

Q is the transverse force in the centre of the penetration, a reduction as specified in subclause 8.2.1.2 not being permissible;

h is the depth of the glued laminated timber beam;

b is the width of the glued laminated timber beam.

Should a more precise analysis of reinforced penetrations not be required, reinforcement may be provided by glueing on grade 100 plywood as specified in DIN 68705 Part 5, as shown in figure 9. The total thickness of the reinforcement, t ($t/2$ on each side), in mm, as a function of the shear stress, induced in the centre of the penetrations, and on the width of the beam shall be equal to the following, with a minimum of 20 mm:

$$t \geq (0{,}15 + 0{,}4 \cdot \tau_Q) \cdot b \tag{19}$$

Further required dimensions relevant to the reinforcement of penetrations by means of plywood may be taken from figure 9. The grain of the outer veneer shall be parallel to that of the laminations of the beam. Resorcinal resin adhesive shall be used for glueing, bonding pressure being generated by nails or staples if required. Further specifications are given in subclause 12.5.

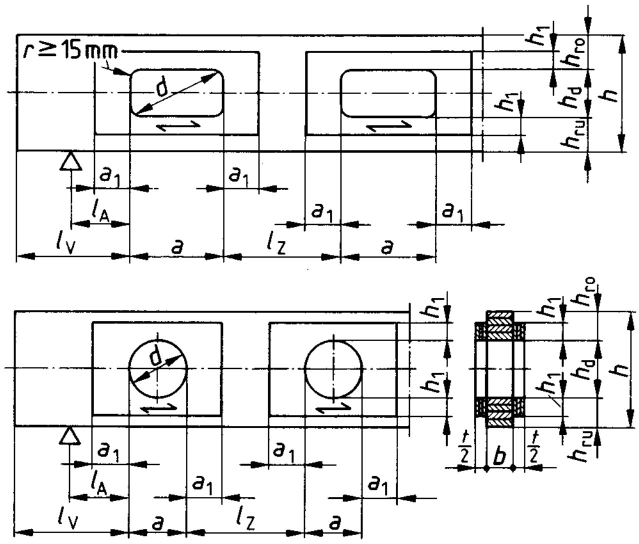
8.2.3 Curved and pitched cambered glued laminated beams

8.2.3.1 General

In the case of curved and pitched cambered glued laminated beams as shown in figures 10 to 12, an analysis shall be made of the transverse and longitudinal stresses at the apex, a verification of stress combinations being in addition required for pitched cambered beams as illustrated in figures 11 and 12.

In the case of rectangular beams, the maximum transverse and longitudinal stresses resulting from the moment at the apex of beams as shown in figures 10, 11 and 12 shall be calculated in accordance with subclauses 8.2.3.2 and 8.2.3.3 for γ not greater than 20°, unless a more detailed analysis is carried out.

Verification of the stress combination as specified in subclause 8.2.3.4, shall be based on the greatest longitudinal stress occurring outside the apex.



$l_A \geq \frac{h}{2}$; l_V and $l_Z \geq h$; $a \leq h$;

$a_1 \geq 0{,}25\,a$ and $\geq h_1$; h_{ro} and $h_{ru} \geq 0{,}3\,h$;

$h_d \leq 0{,}4\,h$; $h_1 \geq 0{,}25\,h_d$ and $\geq 0{,}1\,h$; $b \leq 220$ mm

Figure 9. Dimensions and location of reinforcement

8.2.3.2 Transverse stresses

The transverse stress, $\sigma_\perp$, shall be determined on the basis of the following:

$$max\ \sigma_\perp = \varkappa_q \cdot \frac{M}{W_m} \tag{20}$$

$\varkappa_q$ being determined as follows:

$$\varkappa_q = A_q + B_q \cdot \left[\frac{h_m}{r_m}\right] + C_q \cdot \left[\frac{h_m}{r_m}\right]^2 \tag{21}$$

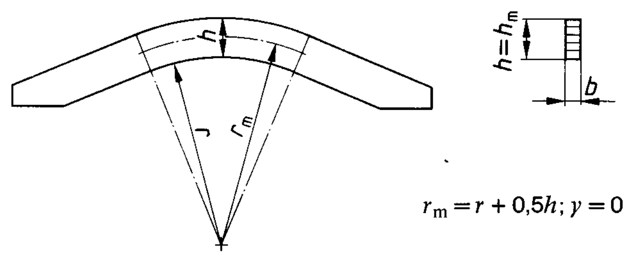
where

$$A_q = 0{,}2 \cdot \tan\gamma \tag{22}$$

$$B_q = 0{,}25 - 1{,}5 \cdot \tan\gamma + 2{,}6 \cdot \tan^2\gamma \tag{23}$$

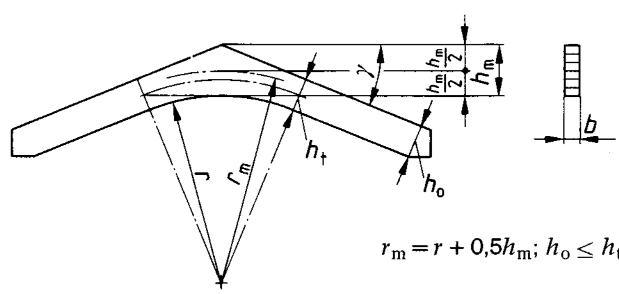
$$C_q = 2{,}1 \cdot \tan\gamma - 4 \cdot \tan^2\gamma \tag{24}$$

The transverse stresses determined by means of equation (20) shall not exceed the values given in table 5, lines 3 and 5a.



$r_m = r + 0{,}5h$; $\gamma = 0$

Figure 10. Curved beam of constant depth of section



$r_m = r + 0{,}5h_m$; $h_o \leq h_t$

Figure 11. Pitched cambered beam with curved lower chord

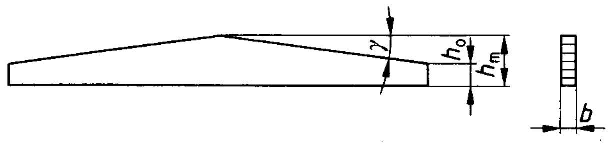


Figure 12. Pitched cambered beam with straight lower chord

8.2.3.3 Axial stresses at the inner/lower beam edge

The axial stress, $\sigma_{\parallel}$, shall be calculated from the following equation:

$$max\ \sigma_{\parallel} = \varkappa_l \cdot \frac{M}{W_m} \tag{25}$$

$\varkappa_l$ being determined as follows:

$$\varkappa_l = A_l + B_l \cdot \left[\frac{h_m}{r_m}\right] + C_l \cdot \left[\frac{h_m}{r_m}\right]^2 + D_l \cdot \left[\frac{h_m}{r_m}\right]^3 \tag{26}$$

where

$$A_l = 1 + 1{,}4 \cdot \tan\gamma + 5{,}4 \cdot \tan^2\gamma \tag{27}$$

$$B_l = 0{,}35 - 8 \cdot \tan\gamma \tag{28}$$

$$C_l = 0{,}6 + 8{,}3 \cdot \tan\gamma - 7{,}8 \cdot \tan^2\gamma \tag{29}$$

$$D_l = 6 \cdot \tan^2\gamma \tag{30}$$

The axial stresses at the outer/upper beam edge may be calculated taking $\varkappa_l$ to be equal to unity.

The axial stresses determined by means of equation (25) shall not exceed the values given in table 5, line 1.

8.2.3.4 Combination of stresses

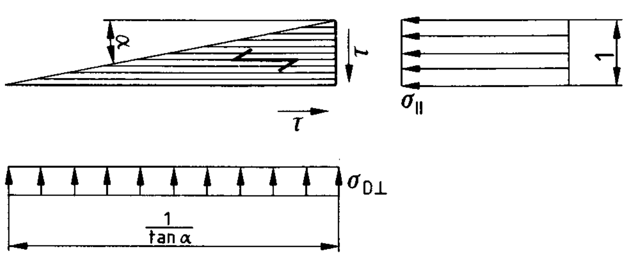
If the grain of glued laminated timber beams does not run parallel to the edge of the beam, resulting in transverse stresses, $\sigma_{\perp}$ and shear stresses, τ, in addition to axial stresses, $\sigma_{\parallel}$ (see figure 13), the following condition shall be met for the edge subject to tensile bending:

$$\left[\frac{\sigma_{\parallel}}{zul\ \sigma_B}\right]^2 + \left[\frac{\sigma_{Z\perp}}{1{,}25\ zul\ \sigma_{Z\perp}}\right]^2 + \left[\frac{\tau}{1{,}33\ zul\ \tau_a}\right]^2 \leq 1 \tag{31}$$

and for the edge subject to compressive bending:

$$\left[\frac{\sigma_{\parallel}}{zul\ \sigma_B}\right]^2 + \left[\frac{\sigma_{D\perp}}{zul\ \sigma_{D\perp}}\right]^2 + \left[\frac{\tau}{2{,}66\ zul\ \tau_a}\right]^2 \leq 1 \tag{32}$$

The appropriate permissible stresses for grade I glued laminated timber as in table 5 shall be entered as the denominator. Where sloping edges are subject to compression, the stress combination need not be taken into account provided that α is not more than 3°.



$\tau = \sigma_{\parallel} \cdot tan\ \alpha$
$\sigma_{D\perp} = \sigma_{\parallel} \cdot tan^2\alpha$

(α denotes the angle between edge of beam and direction of grain, and ⟷ denotes the direction of grain)

Figure 13. Axial, transverse and shear stresses in a triangular element of an edge subject to compressive bending

8.2.4 Braced beams

If purlins or beams braced in each bay are required to bear a predominantly uniformly distributed load or equal concentrated loads a relatively short distance apart (i.e. rafters) and the centre-to-centre spacing, l, of adjacent supports (see figure 14) does not vary by more than one-fifth, the largest bay widths (l_1, l_2, l_3 or l_4) may be taken into account and the member regarded as a freely rotating, simply supported beam. Where imposed loading patterns change from span to span and the spacing of supports varies by more than one-fifth of the smallest distance, a more precise calculation, also of the supports, shall be made and the beams detailed accordingly.

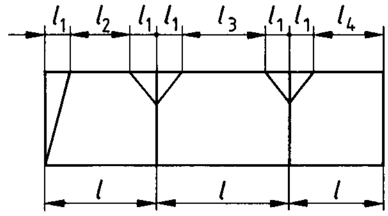


Figure 14. Bay widths of braced beams

In the case of purlins and beams with bolsters but without braces, the centre-to-centre spacing of supports shall always be taken as the effective span.

8.3 Built-up beams with non-rigid interconnection of chord and web

8.3.1 When calculating the stresses induced in built-up beams, consideration may need to be given to the influence of slip occurring in the joints.

For beams with cross sections shaped as type 5, symmetrical about a single axis (see table 8 and figure 15d), the stresses shall be calculated as follows:

$$\sigma_{si} = \pm \frac{M}{ef\ I} \cdot \gamma_i \cdot a_i \cdot \frac{A_i}{A_{in}} \cdot n_i \tag{33}$$

$$\sigma_{ri} = \pm \frac{M}{ef\ I} \cdot \left(\gamma_i \cdot a_i \cdot \frac{A_i}{A_{in}} + \frac{h_i}{2} \cdot \frac{I_i}{I_{in}}\right) \cdot n_i \tag{34}$$

where

M is the bending moment, which is positive in the presence of compressive stress in the upper, and tensile stress in the lower outer fibre of the beam;

σ_{si} and σ_{ri} are centroidal or edge stresses in the separate parts of the cross section (i.e. chords or web); the appropriate signs are shown in figure 15d;

a_i is the distance of the centroidal axes of the full cross-sectional areas from the relevant neutral axis y – y, provided that a_2 is not less than zero and not greater than $h_2/2$;

h_i is the thickness or depth of each part of the cross sections;

γ_i is a reduction coefficient for calculating $ef\ I$ using equation (36) or equation (37);

I_i and I_{in} are the second moments of area of the full and reduced parts of the cross section ($I_i = b_i \cdot h_i^3/12$);

$ef\ I$ is the effective second moment of area of the full cross section in accordance with equation (35);

A_i and A_{in} are the areas of the full and reduced parts of the cross section ($A_i = b_i \cdot h_i$);

b_i is the cross-sectional width;

E_i is the modulus of elasticity of each part of the cross section;

E_v is a freely selected comparative modulus of elasticity;

n_i is equal to E_i/E_v.

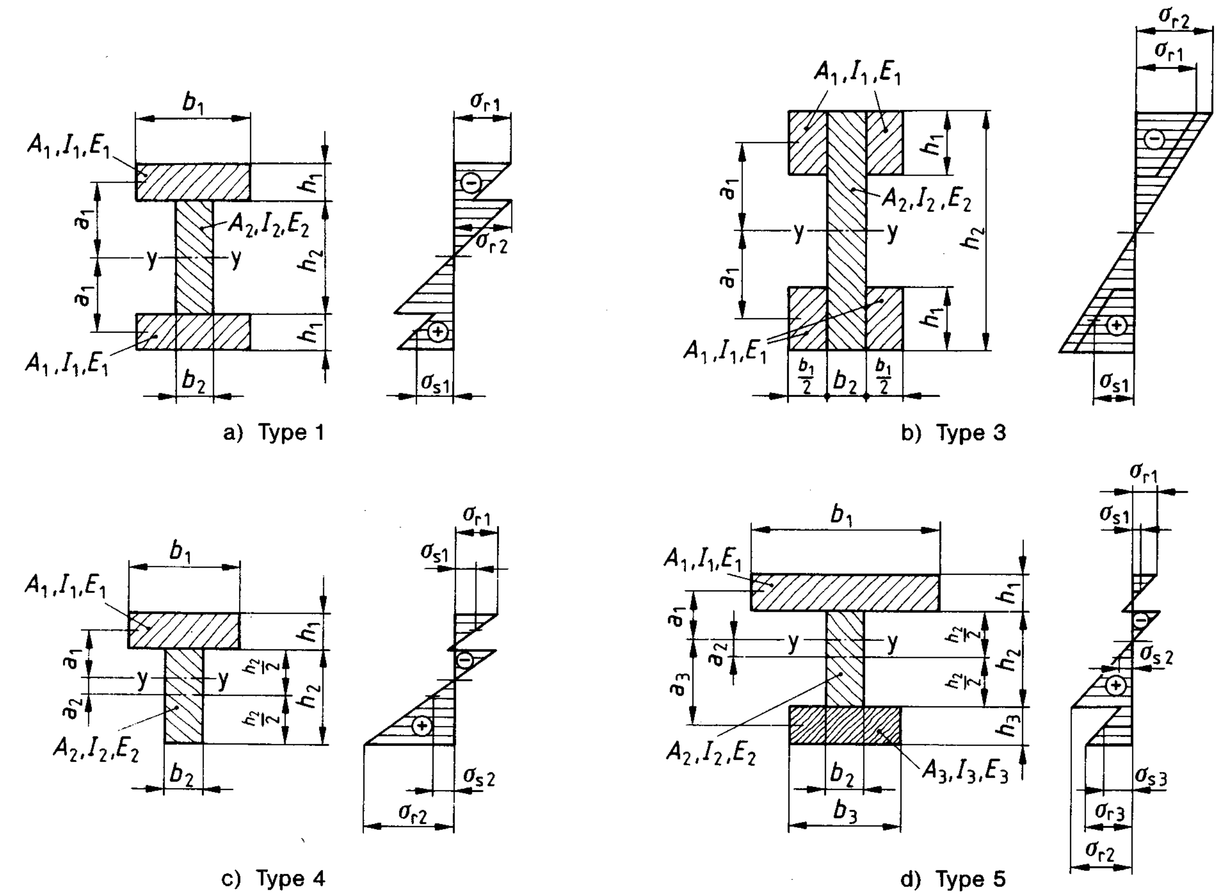
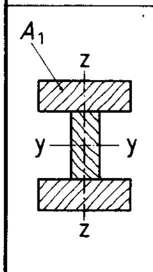
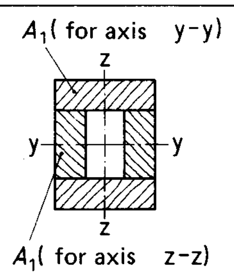
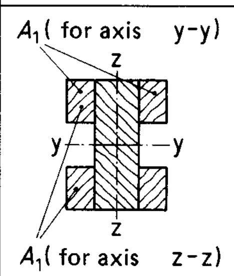
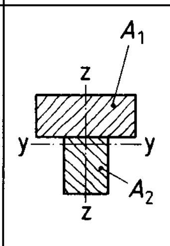
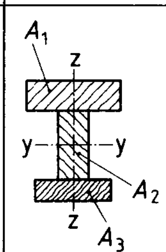


Figure 15. Types of cross section of built-up beams and stress distribution diagrams in connection with positive bending moment

Table 8. **Beam cross sections and design slip moduli, C, in N/mm**

Centroidal axis significant for bending or buckling	Fastener	Type 1	Type 2	Type 3	Type 4	Type 5
						
y – y	Nail (in single shear)	600	600	900	600	600
	Nail (in double shear)	700	700	900 per joint	–	700
z – z	Nail (in single shear)	–	900	600	–	–
	Nail (in double shear)	–	900 per joint	700	–	–
y – y and z – z	Connector as in DIN 1052 Part 2	15000 for permissible loads [1]) up to 16 kN				
		22500 for permissible loads [1]) over 16 up to 30 kN				
		30000 for permissible loads [1]) over 30 kN				
y – y and z – z	Dowel, precision bolt	0,7 · *zul N* per joint, *zul N* being equal to the permissible load, in N, per joint between web and chord [2])				

[1]) The permissible load is that specified per connector for load case H (see tables 4, 6 and 7 of DIN 1052 Part 2).
[2]) For type C hardwood: 1,0 · *zul N*.

Second moments of area of reduced parts of the cross section may be related to the centroidal axes of such parts.

Taking into account subclause 8.2.1.1, the edge stresses, σ_{ri}, shall not exceed the values of bending stress given in table 5, line 1, nor shall the centroidal stresses, σ_{si}, in those parts of the cross section in tension exceed the values of tensile stress given in table 5, line 2. In addition, subclause 5.1.7 shall be taken into account.

Calculations shall be based on the effective second moment of area of the full cross section, *ef I*, obtained as follows:

$$ef\ I = \sum_{i=1}^{3} (n_i \cdot I_i + \gamma_i \cdot n_i \cdot A_i \cdot a_i^2) \quad (35)$$

where

$$\gamma_{1,3} = \frac{1}{1 + k_{1,3}} \quad (36)$$

$$\gamma_2 = 1 \quad (37)$$

$$k_{1,3} = \frac{\pi^2 \cdot E_{1,3} \cdot A_{1,3} \cdot e'_{1,3}}{l^2 \cdot C_{1,3}} \quad (38)$$

and

$$a_2 = \frac{1}{2} \cdot \frac{\gamma_1 \cdot n_1 \cdot A_1 (h_1 + h_2) - \gamma_3 \cdot n_3 \cdot A_3 (h_2 + h_3)}{\sum_{i=1}^{3} \gamma_i \cdot n_i \cdot A_i} \quad (39)$$

In the above equations:

e'_1 and e'_3 are the mean distances between fasteners (notionally in a single row (see figure 16)), by means of which the chords are joined to the web;

C_1 and C_3 are the slip moduli of the fasteners connecting the chords to the web (to be taken from table 8);

l is the relevant effective span.

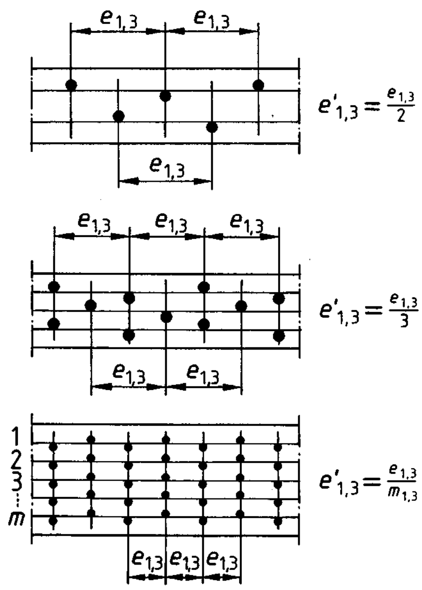


Figure 16. Spacing $e'_{1,3}$ of fasteners arranged in more than one row

For the calculation of the k values in accordance with equation (38), the reductions described in subclause 4.1.2 shall not be taken into account as regards the modulus of elasticity and the slip modulus. In the case of DIN 96, DIN 97 and DIN 571 wood screws and of staples conforming to DIN 1052 Part 2, the slip moduli shall be as those for nails as specified in table 8.

For beams with cross sections shaped as types 1 to 3, symmetrical about both axes (see table 8 and figures 15a and 15b), the following conditions shall apply: $A_3 = A_1$, $E_3 = E_1$, $n_3 = n_1$, $e'_1 = e'_3 = e'$ and $C_1 = C_3 = C$. Using equation (38) or (36), this results in equal values for k_1, k_3 and k and for γ_1, γ_3 and γ, and also, using equation (39), results in a_2 being equal to zero. If equation (33) is applied, the stresses σ_{s1} and σ_{s3} are shown to be equal and σ_{s2} is equal to zero, whilst from equation (34), σ_{r1} is equal to σ_{r3}.

For beams with cross sections shaped as type 4, symmetrical about a single axis (see table 8 and figure 15c), equations (38) and (39), with A_3, equal to zero, may be taken as a basis. The stresses are obtained analogously from equations (33) and (34).

8.3.2 Unless a more precise calculation is made, four-fifths of the relevant effective span of continuous beams shall be taken into account when determining k. For the purposes of verifying the stresses above intermediate supports, the smaller value over the two adjacent spans shall be used.

For cantilever beams, the effective width shall be taken to be equal to $2 \cdot l_K$, l_K being the cantilever length.

8.3.3 Fasteners shall normally be designed to resist the greatest transverse force, *max Q*, taking account of the effective second moment of area in accordance with equation (35). In the case of beams with a cross section shaped as type 5, symmetrical about a single axis, the maximum linear shear, *ef* $t_{1,3}$, in the joints connecting the chords are found to be

$$ef\ t_{1,3} = \frac{max\ Q}{ef\ I} \cdot \gamma_{1,3} \cdot n_{1,3} \cdot S_{1,3} \quad (40)$$

and the required spacing, $e'_{1,3}$, of fasteners to be

$$erf\ e'_{1,3} = \frac{zul\ N_{1,3}}{ef\ t_{1,3}} \quad (41)$$

As a rule, fasteners shall be evenly spaced over the length of the beam, irrespective of the shear distribution over the length of the beam.

If, however, fasteners are staggered to correspond to the transverse force distribution and if the maximum spacing, $e'_{1,3}$, is not more than $4 \cdot min\ e'_{1,3}$, the corresponding spacing of fasteners may be substituted in equation (38) as the following:

$$\bar{e}_{1,3} = 0{,}75 \cdot min\ e'_{1,3} + 0{,}25 \cdot max\ e'_{1,3} \quad (42)$$

Shear stresses in the neutral axis shall be verified for *max Q*, again taking *ef I* into account. For beams with cross sections shaped as type 5, the greatest shear stress in the relevant plane of zero stress y – y shall be taken to be

$$max\ \tau = \frac{max\ Q}{b_2 \cdot ef\ I} \cdot \sum_{i=1}^{2} \gamma_i \cdot n_i \cdot S_i \quad (43)$$

In equations (40) to (43),

S_1 and S_3 are the section moduli of the chords as a function of the relevant plane of zero stress y – y ($S_{1,3} = b_{1,3} \cdot h_{1,3} \cdot a_{1,3}$);

S_2 is the section modulus of the web above the relevant plane of zero stress y – y, in relation to the plane of zero stress y – y ($S_2 = b_2 \cdot (h_2/2 - a_2)^2/2$);

zul N_1 and N_3 are the permissible loads on the fasteners used.

In the case of beams with cross-sectional shapes as types 1 to 3, symmetrical about two axes (see table 8 and figures 15a and 15b), and of beams with cross-sectional shapes as of type 4, symmetrical about a single axis (see table 8 and figure 15c), equations (40) to (43) shall be applied as appropriate (cf. subclause 8.3.1).

If, in the case of beams of types 2 and 3 (see table 8), the relevant centroidal axis is axis z – z, an analogous procedure shall be followed.

8.3.4 Verification of deflection in accordance with subclause 8.5 shall be effected using $ef\ I$ (obtained from equation (35)) and E_V. For this purpose, the slip modulus, C, to be substituted in equation (38) shall be equal to 1,25 times the values shown in table 8 or the values specified in table 13 of DIN 1052 Part 2, whichever is greater.

8.4 Solid-web, trussed and lattice beams

8.4.1 Solid-web beams with webs made from wood-based panel products

Solid-webbed beams as shown in figure 17, the webs of which are made of plywood or particle board and which are either in one piece or of several pieces glued end to end, shall be designed taking into account the different moduli of elasticity of the materials making up the web and the chord. If splices are of the nailed type, consideration may also need to be given to the occurrence of slip.

Where the connection between chord and web is non-rigid, the beam shall be designed as specified in subclause 8.3.

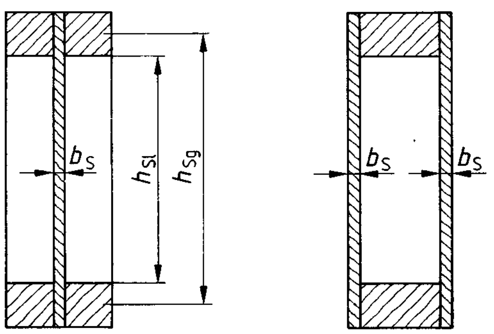
Unless a more precise analysis of buckling is made, more or less uniformly loaded glued solid-web beams (see figure 17) of plywood consisting of at least five plies as specified in DIN 68705 Part 3 or Part 5 shall meet the following condition:

$$\frac{h_{Sl}}{b_S} \leq 35 \tag{44}$$

if the webs are made of particle board as specified in DIN 68763, the following condition shall be met:

$$\frac{h_{Sl}}{b_S} \leq 50 \tag{45}$$

For nailed solid-web beams with webs made from wood-based panel products, h_{Sg} shall be substituted for h_{Sl} in equations (44) and (45).



a) I beam b) Box beam

Figure 17. Solid-web beams with webs made from wood-based panel products.

In figure 17 and equations (44) and (45),

h_{Sl} is the clear height of the web;

h_{Sg} is the distance between the mid-heights of the chord cross sections;

b_S is the web thickness.

Stiffening is required at least in the area of support and at the point where concentrated loads act. In the case of beams more than 500 mm in depth, the spacing of the stiffening shall not exceed three times the depth of the beam.

8.4.2 Solid-web beams with timber webs

8.4.2.1 In the case of box beams, I beams, or box beams consisting of timber boards nailed at right angles to each other, the web shall not be taken into account for the determination of the effective second moment of area. The boards making up the web, and their connections to the chords shall be designed to provide adequate resistance to shear. The verification of stresses in the chords shall take due account of the influence of slip occurring in the joints. Where fasteners are in staggered arrangement, equation (41) may be applied as appropriate.

The resistance to buckling of web boards subject to compression shall also be verified if they are not adequately connected to the tension chords. Verification shall be made of the capability of box beams to withstand the torsional moments generated by the arrangement of plywood boards at right angles to each other.

8.4.2.2 If an I beam is produced as two separate halves (i.e. in single shear), proof shall be provided of its capability to resist the coupling forces between the two halves of the beam.

8.4.2.3 Timber webs of solid-web beams shall not be taken into account as providing resistance to additional compressive or tensile forces (e.g. as in frames).

8.4.2.4 If the chords are made up of several separate boards (see figure 18), then, provided that a more precise analysis is not made, the cross sections of the separate boards shall be taken into consideration with the following factor, ζ:

Board 1: $\zeta = 1{,}0$.

Board 2: $\zeta = 0{,}8$.

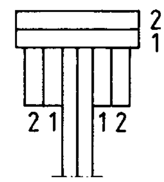


Figure 18. Cross section of a built-up chord of a nailed solid-web beam

The number of boards used fastened on top of each other shall not exceed two except where chords are of glued laminated timber, in which case the number of boards is not limited and multiplication by a reduction coefficient is not neccessary.

8.4.3 Trussed and lattice beams

In the case of trussed beams with parallel chords and trapezoidal lattice beams, composed of members with joints in which slip is likely to occur, bending stresses in the chords shall be verified if the height of the chord is more than 1/7 of the depth of the beam as a whole.

8.5 Deflection and camber

8.5.1 In particular the serviceability of the structure and of individual members is dependent on certain limits of deflection due to imposed loads (including wind and snow loads, but with no allowance being made for vibration and impact) and also due to the total load (i.e. self-weight and imposed loads including wind and snow loads but with no allowance being made for vibration and impact) not being exceeded.

The type or use of a structure may require a maximum permissible deflection less than specified in table 9 or in subclauses 8.5.7 and 8.5.8.

8.5.2 The full cross section may be used for calculating deflection. In the case of built-up beams, the analysis shall be made as specified in subclause 8.3.4.

8.5.3 The values given in table 9 shall apply to the design permissible deflection of glued laminated timber beams, built-up beams, solid-web beams and trussed and lattice beams. See clause 10 for stiffening elements.

When determining the deflection of trussed and lattice beams, a distinction shall be drawn between a simplified calculation, in which only the elastic deformation of the chord is taken into account, and a more precise calculation, in which the elastic deformation of all members and the possibility of slip occurring in the joints are considered. This also applies to thick ply-webbed beams with the boarding in only one direction. In the case of flat roofs with span ratios, l/h, greater than 10, the more precise method shall normally be employed.

8.5.4 In the case of beams with webs made from timber or wood-based panel products, sag due to shear deformation shall be taken into consideration. For solid-web beams, it will normally suffice, unless a more precise analysis is made, to establish the design sag due to shear deformation by a simplified method assuming a representative and uniformly distributed load. For simply-supported solid-web beams of constant cross section, this sag in the centre of the beam, $max\ f_\tau$, may be assumed to be:

$$max\ f_\tau = \frac{q \cdot l^2}{8\,G \cdot A_{Steg}} \tag{46}$$

where G is the modulus of shear of the web material.

In the case of continuous beams, $max\ f_\tau$ may be calculated in the same way, with l as the overall width of the bay in question.

8.5.5 In the case of glued laminated timber beams, built-up beams and trussed and lattice beams, the whole beam shall normally be given a parabolic camber which shall correspond at least to the design deflection under overall load, taking creep deformation into account. Any likelihood of slip occurring in the joints due to the type of fastener used shall be taken into consideration. If the depth of camber is not subject to calculation, it shall be not less than $l/200$ where semi-dried or fresh timber is used, $l/150$ for cantilever beams, and not less than $l/300$ in all other cases. A similar procedure shall be adopted for frames.

8.5.6 In the case of cantilever beams, the design deflection of the cantilevered ends may exceed the values given in table 9 by 100 % in terms of the length of cantilever.

8.5.7 The design deflection of floors of rooms in dwellings, offices, etc., and of rooms in factories and workshops, under overall load, shall not normally exceed $l/300$. This shall generally also apply with regard to purlins, rafters and beams of living rooms, rooms in offices, etc.

8.5.8 The design deflection of purlins and rafters, beams forming part of floors of buildings in which animals are kept, or such in barns and the like, and solid-webbed and trussed or lattice beams without camber in other agricultural buildings under overall load, shall not exceed $l/200$ and $l/400$, when a simplified analysis of trussed or lattice beams is performed.

8.5.9 The design deflection of posts and horizontal frame members in the outer walls of buildings closed on all sides, under horizontal load, such as wind loads as specified in DIN 1055 Part 4, shall not normally be more than $l/200$.

8.5.10 The design deflection of roof decking and of floor decks subject to direct loading, and of upper roof and floor sheathing shall not be more than $l/200$, with a maximum of 10 mm, under overall load or more than $l/100$, with a maximum of 20 mm, under self-weight and a concentrated load of 1 kN, the sag due to shear deformation being disregarded in both cases. Subclause 10.3.1 shall be complied with as regards the deflection of stiffening plates made from wood-based panel products.

8.6 Lateral stability of flexural members

8.6.1 Adequate provision shall be made to prevent flexural members deflecting sideways.

If rectangular beams of depth h and width b, spaced at a distance s apart are fixed so that it is almost impossible for them to deflect sideways, verification of bending stress resulting from a bending moment, M (which is here assumed to be constant), shall be based on the following condition:

$$\frac{\dfrac{M}{W}}{k_B \cdot 1{,}1 \cdot zul\ \sigma_B} \leq 1 \tag{47}$$

$$\text{with } k_B = \begin{cases} 1 & \text{and} \quad \lambda_B \leq 0{,}75 \quad (48) \\ 1{,}56 - 0{,}75 \cdot \lambda_B & \text{and } 0{,}75 \leq \lambda_B \leq 1{,}4 \quad (49) \\ 1/\lambda_B^2 & \text{and} \quad \lambda_B > 1{,}4 \quad (50) \end{cases}$$

where λ_B is the slenderness ratio to be calculated as follows:

$$\lambda_B = \sqrt{\frac{s \cdot h \cdot \gamma_1 \cdot zul\ \sigma_B}{\pi \cdot b^2 \cdot \sqrt{E_\parallel \cdot G_T}}} \tag{51}$$

The load for load cases H and HZ shall be increased by multiplying by a factor, γ_1, of 2,0.

A more precise analysis is not required for solid-webbed box or I beams in cases where the compression chord is punctually fixed and incapable of deflecting laterally, and where the ratios of inertia, i, of chord cross section, relative to the critical centroidal axis of the beam, is more than $s/40$ (s denoting the spacing of the points of fixing). If i is less than $s/40$, then, unless a more precise analysis is made, the centroidal stress in the part of the cross section in compression shall not exceed a value equal to $k_S \cdot zul\ \sigma_k$. For this purpose, $zul\ \sigma_k$ shall be determined from equation (59), ω being the buckling coefficient derived, in accordance with table 10, from the slenderness ratio, λ, which in turn is equal to s/i. k_S shall be given the same magnitude as the buckling coefficient, ω, derived from a slenderness ratio of 40, as given in table 10. If required, *ef* I shall be determined in accordance with equations (35) to (39) (cf. subclause 9.3.3.2).

Table 9. **Permissible deflection of flexural members**

Load	Precambered beams as in subclause 8.5.5			Beams without camber		
	Glued laminated beams, built-up beams, solid-web beams	Trussed or lattice beams [1])		Glued laminated beams, built-up beams, solid-web beams	Trussed or lattice beams [1])	
		Simplified method	Precise method		Simplified method	Precise method
Imposed load	$l/300$	$l/600$	$l/300$	–	–	–
Overall load	$l/200$	$l/400$	$l/200$	$l/300$	$l/600$	$l/300$

[1]) Including solid-web beams boarded in one direction.

8.6.2 In place of an analysis as specified in subclause 8.6.1, evidence of structural adequacy under working load may also be provided with the aid of second order theory. The likelihood of slip occurring in the joints due to the type of fasteners used, and deformations due to creep shall also, where necessary, be considered.

The structural analysis shall be on the basis of loading multiplied by factor γ_1. Proof of structural adequacy shall be deemed to be provided if γ_1 times the permissible stresses and γ_1 times the permissible loads on the fasteners are exceeded in no part of the beam.

In the case of beams which, as seen from above, are designed to be straight, an initial lateral curvature of the longitudinal axis, which may be either sinusoïdal or parabolic, shall be taken into account in the calculation by assuming a design lateral eccentricity as in equation (73), s being taken to be the spacing of the stiffening elements.

Stresses in the corners of cross sections, due to unintentional bending in two directions may exceed by 10 % the permissible bending stress specified in table 5, line 1. Analysis of bending in one direction shall be made in addition. See subclause 9.6.3 for details.

9 Design of compression members

9.1 Effective length

9.1.1 If the compression member is supported at its ends by members (e.g. bracing, plates) which prevent it from deflecting sideways, it may be considered pin-jointed at the ends. If the compression member is supported at intermediate points which abut against other fixed points, the distance between these supports shall be taken as the effective length, s_k, with regard to buckling in the direction in which the support is effective. If the conditions are other than described above, greater effective lengths shall be assumed. See also subclause 8.6 for compression chords of solid-web beams.

9.1.2 The effective length of chord members in trusses, when considering buckling in the truss plane, shall generally be taken to be the projected length. The effective length of web members shall be taken to be equal 0,8 · s (s being equal to the projected length) unless the web member is only connected by means of framed joints, or by means of bolt and connectors, or connected by bolts *) only, in which case the effective length shall be the same as the projected length.

When considering buckling from the truss plane, the effective length of chord members shall be the spacing of nogging and, in the case of web members, the length of the projected length.

See subclause 10.5 for other relevant information.

9.1.3 Unless a more precise analysis is required when considering buckling in the plane of the system, the effective length of rafters in collar roofs shall be taken as being equal to 0,8 · s in the case of collars which are able to move in horizontal direction, where the length, s_u, of the lower rafter section is less than 0,7 · s but greater than 0,3 · s, s denoting the total length of the rafter. In all other cases, the effective length shall be taken to be equal to s. Where collars cannot move in horizontal direction, the effective length shall be assumed to be equal to s_u or s_o and the check for buckling safety based on the greatest compressive strength in the upper and lower rafter section.

The check for buckling from the plane of systems shall be based on the distance between noggings (cf. subclause 10.5).

9.1.4 Unless a more precise analysis is required, the effective length of posts of frames with truss horizontal members shall be calculated by a simplified method, as shown in figure 19, when considering buckling in the plane of the frame, using the following equation:

$$s_k = 2h_u \cdot \left(1 + 0{,}35\,\frac{h_o}{h_u}\right) \qquad (52)$$

An idealized situation shall be assumed in that the greater of the two axial forces, N_o or N_u, shall be taken to act over the whole length, h (this being the sum of h_o and h_u).

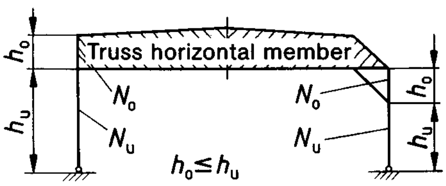


Figure 19. Two-hinged frame with truss horizontal member

9.1.5 Unless a more precise analysis is made for two and three-hinged arches with a rise-to-span ratio, $f : l$ (see figure 20), of between 0,15 and 0,5 and which are of more or less constant cross section, their effective length, when considering buckling in the plane of the arch, shall equal 1,25 · s (53), where s is equal to half of the arch length. The check for safety against buckling shall be based on the compressive forces acting at each quarter point.

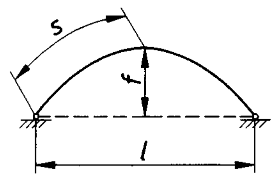


Figure 20. Arch

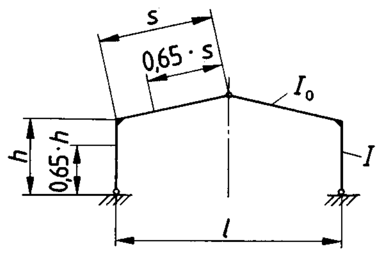


Figure 21. Frame

9.1.6 Unless a more precise analysis is required for symmetrical two- and three-hinged frames as shown in figure 21, the effective length of the post, when considering buckling in the plane of the horizontal member, shall be calculated as follows:

$$s_k = 2h \cdot \sqrt{1 + 0{,}4c} \qquad (54)$$

with

$$c = \frac{I \cdot 2s}{I_o \cdot h} \qquad (55)$$

where

I is the second moment of area of the post;
I_o is the second moment of area of the horizontal member;
h is the height of the post;
s is the length of the horizontal member.

Unless a more precise analysis is required, the effective length of the horizontal member shall be assumed to be equal to $2h \cdot \sqrt{1 + 0{,}4c} \cdot \sqrt{k_R}$ (56), with

$$k_R = \frac{I_o \cdot N}{I \cdot N_o} \qquad (57)$$

where

N is the mean force acting in the post axis;
N_o is the mean force acting in the horizontal member axis.

*) Translator's note. Both studs and bolts will be covered by the single term 'bolt'.

If second moments of area are variable, calculations shall be based on those present at $0{,}65 \cdot h$ or $0{,}65 \cdot s$, the radii of gyration, i, also being calculated from the second moments of area and areas of cross section at these points.

The analysis of structural stability shall be made using equation (72), inserting $max\ N$ and $max\ M$ in the part of the frame considered.

9.1.7 Where appropriate, consideration shall be given to the likelihood of slip occurring in the joints and the effect this may have on the effective length.

9.1.8 In the case of truss frames where the inner corners of the frame are not held laterally, the effective length of the inner compression members in the posts shall be equal to the distance between the base of the post and the underside of the roof membrane when considering buckling from the plane of the frame. In addition, a lateral force equal to 1/100 of the maximum tensile or compressive force acting in the inner corner of the frame shall be taken into account.

9.2 Slenderness ratio

A slenderness ratio, λ, of up to 150 is permissible for non-composite compression members and effective slenderness ratio, $ef\ \lambda$, up to 175 for built-up but not glued, compression members, and up to 200 for bracing members and for tension members which are subject to only minor compressive forces from secondary loads. In the case of temporary structures (see DIN 4112), slenderness ratios up to 200 are permissible for compression members subject to predominantly static loading. Masts intended to reduce sagging of membranes may have a slenderness ratio up to 250.

9.3 Axial compression

9.3.1 General

In the case of members designed to be straight in shape and subject to axial compression, the check for safety against buckling shall be made following the procedure set out in subclauses 9.3.2 to 9.3.3.4 and the normal stress analysis shall be performed provided that reductions in cross-sectional area as specified in subclause 6.4 are restricted to the effective load zone.

9.3.2 Check for safety against buckling of non-composite members

The following check for safety against buckling shall be carried out for non-composite members:

$$\frac{\frac{N}{A}}{zul\ \sigma_k} \leq 1 \tag{58}$$

with

$$zul\ \sigma_k = \frac{zul\ \sigma_{D\parallel}}{\omega} \tag{59}$$

where

N is the maximum compressive force acting in the member;

A is the full cross section of the member;

$zul\ \sigma_{D\parallel}$ is the permitted compressive stress according to table 5, line 4, or table 6, line 4, taking into consideration subclauses 5.1.6, 5.1.7 and 5.1.9 or 5.2.3;

ω is the buckling coefficient, taken, as a function of the slenderness ratio, from table 10; intermediate values may be obtained by linear interpolation;

λ is the larger slenderness ratio of the member, i.e. either λ_y, equal to s_{ky}/i_y, or λ_z, equal to s_{kz}/i_z, s_{ky} and s_{kz} denoting the effective lengths of the member at right angles to the relevant centroidal axes (see subclause 9.1) and i_y and i_z, denoting the appropriate radii of gyration.

9.3.3 Check for safety against buckling of composite members

9.3.3.1 General

A distinction shall be made firstly, between straddled composite members (i.e. such having cross sections as shown in table 8) and those which are not straddled (as shown in figure 22) (the straddling distance being defined as the clear distance, a, between components divided by the thickness of each component, h_1) and secondly, between the buckling directions (i.e. perpendicular to y- or z-axis).

In the case of non-straddled members having cross sections of types 1, 4 and 5 (see table 8) and of straddled members, composite members shall, when considering buckling perpendicular to the centroidal $z-z$ axis, be treated as non-composite members the second moments of area, I_z, of which are equal to the sum of the second moments of area of the components, as follows:

$$I_z = \sum_{i=1}^{n} I_{zi} \tag{60}$$

where I_{zi} is the second moment of area of each component in relation to the centroidal $z-z$ axis of the cross-sectional area. Subclause 9.3.3.2 shall apply by analogy if components are of different materials.

In the case of both straddled and non-straddled members, it cannot always be presumed that each component makes a full structural contribution to buckling perpendicular to the centroidal y – y axis, the check for safety against buckling then being carried out on the basis of an effective slenderness ratio larger than λ_{rigid}.

Where non-straddled members have cross sections of types 2 and 3 (see table 8), the condition specified in the above paragraph also applies with regard to buckling perpendicular to the centroidal $z-z$ axis.

9.3.3.2 Built-up, non-straddled members with continuous joints (cross-sectional types in table 8)

For glued members, λ shall be taken as equal to λ_{rigid} and I equal to I_{rigid}, the latter being calculated by analogy with equations (35) and (39), γ_i being taken as equal to unity.

Where there is likelihood of slip occurring at the joints, $ef\ I$ may need to be calculated as for built-up beams by means of equations (35) and (39), but inserting the appropriate effective length, s_k, (see subclause 9.1) instead of the effective span, l, and taking the values of C from subclause 8.3.1. Using $ef\ I$, the effective slenderness ratio, $ef\ \lambda$, shall be calculated on the basis of which the associated buckling coefficient can be taken from table 10. If more than one type of material is used, the largest buckling coefficient shall be taken unless a more precise analysis is required.

The following condition shall be met by all members with a cross section of type 5, symmetrical about one axis (see table 8):

$$\frac{\frac{N}{\bar{A}} \cdot n_i}{zul\ \sigma_k} \leq 1 \tag{61}$$

where

$$\bar{A} = \sum_{i=1}^{3} n_i \cdot A_i \tag{62}$$

$zul\ \sigma_k$ shall be calculated for each cross-sectional part in accordance with equation (59). An analogous procedure shall be adopted for members with cross sections of types 1 to 4 (see table 8).

The design of fasteners shall generally be based on the assumption of shear force acting over the whole length of the member, this being calculated as follows:

$$Q_i = \frac{ef\ \omega \cdot N}{60} \tag{63}$$

Table 10. **Buckling coefficients, ω**

Slenderness ratio	Solid softwood timber as in table 1, line 1	Glued softwood laminated timber as in table 1, line 1		Solid hardwood timber as in table 1			Plywood conforming to DIN 68705 Parts 3 and 5, with compression parallel to direction of grain of outer veneer		Particle board conforming to DIN 68783	
	Grade	Grade		Type			Number of plies		Board thickness, in mm	
λ	I to III	I	II	A	B	C	3	≥ 5	≤ 25	> 25
0	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
10	1,04	1,00	1,00	1,04	1,03	1,03	1,02	1,01	1,03	1,02
20	1,08	1,00	1,00	1,08	1,08	1,07	1,05	1,04	1,07	1,07
30	1,15	1,00	1,00	1,15	1,15	1,15	1,11	1,12	1,15	1,16
40	1,26	1,03	1,03	1,25	1,27	1,29	1,22	1,28	1,28	1,34
50	1,42	1,13	1,11	1,40	1,45	1,50	1,38	1,54	1,49	1,61
60	1,62	1,28	1,25	1,59	1,69	1,79	1,61	1,91	1,78	1,99
70	1,88	1,51	1,45	1,83	2,00	2,17	1,92	2,53	2,15	2,48
80	2,20	1,92	1,75	2,13	2,38	2,67	2,30	3,30	2,60	3,24
90	2,58	2,43	2,22	2,48	2,87	3,38	2,87	4,18	3,22	4,10
100	3,00	3,00	2,74	2,88	3,55	4,17	3,55	5,16	3,98	5,07
110	3,63	3,63	3,32	3,43	4,29	5,05	4,29	6,24	4,82	6,13
120	4,32	4,32	3,95	4,09	5,11	6,01	5,11	7,43	5,73	7,30
130	5,07	5,07	4,63	4,79	5,99	7,05	5,99	8,72	6,73	8,56
140	5,88	5,88	5,37	5,56	6,95	8,18	6,95	10,11	7,80	9,93
150	6,75	6,75	6,17	6,38	7,98	9,39	7,98	11,61	8,96	11,40
160	7,68	7,68	7,02	7,26	9,08	10,68	9,08	13,20	10,19	12,97
170	8,67	8,67	7,92	8,20	10,25	12,06	10,25	14,91	11,50	14,64
175	9,19	9,19	8,39	8,69	10,86	12,78	10,86	15,80	12,19	15,52
180	9,72	9,72	8,88	9,19	11,49	13,52	11,49	16,71	12,90	16,41
190	10,83	10,83	9,89	10,24	12,80	15,06	12,80	18,62	14,37	18,29
200	12,00	12,00	10,96	11,35	14,18	16,69	14,18	20,63	15,92	20,26
210	13,23	13,23	12,08	12,51	15,64	18,40	15,64	22,75	17,55	22,34
220	14,52	14,52	13,26	13,73	17,16	20,19	17,16	24,97	19,27	24,52
230	15,87	15,87	14,50	15,01	18,76	22,07	18,76	27,29	21,06	26,80
240	17,28	17,28	15,78	16,34	20,43	24,03	20,43	29,71	22,93	29,18
250	18,75	18,75	17,13	17,73	22,16	26,08	22,16	32,24	24,88	31,66

In cases where $ef\,\lambda$ is less than 60, the above value of Q_i may be modified by the factor $\lambda/60$ which shall not, however, be more than 0,5.

In equation (63),

$ef\,\omega$ is the appropriate buckling coefficient taken from table 10, as a function of the effective slenderness ratio;

N is the compressive force acting in the member.

The linear shear, $ef\,t$, and the required spacing of fasteners, $e'_{1,3}$, shall be obtained by means of equations (40) and (41).

9.3.3.3 Composite straddled compression members (spaced and lattice members)

In order to check for buckling perpendicular to the centroidal $y-y$ axis of spaced members as shown in figures 22a to 22e, the effective slenderness ratio shall be calculated as follows:

$$ef\,\lambda = \sqrt{\lambda_y^2 + \frac{m}{2} \cdot c \cdot \lambda_1^2} \qquad (64)$$

where

λ_y is equal to s_{ky}/i_y, defined as the design slenderness ratio of the gross cross section, the radius of gyration, i_y, being calculated from the second moment of area, $I_{y,\mathrm{rigid}}$, of the gross cross section (in relation to the centroidal $y-y$ axis);

m is the number of components;

c is a factor depending on the type of bridging, taken from table 11;

λ_1 is equal to s_1/i_1, defined as the slenderness ratio of the component in the centroidal axis parallel to the centroidal $y-y$ axis.

The centre-to-centre spacing of the bridging shall be taken as the effective length, s_1, of each flange, with λ_1 being not more than 60 and s_1 not more than ⅓ s_{ky}.

Where the centre-to-centre spacing of the bridging is less than $30 \cdot i_1$, λ_1 shall be given the value of 30 in equation (64).

If shafts are only connected by bolts, c may be assumed to be equal to 3,0 if the structures in question are temporary structures covered by DIN 4112 or scaffolding, provision being made to enable the subsequent tightening of bolts. In all other cases, bolted composite compression members shall be regarded, for calculation purposes, as consisting of components not acting in cooperation to resist loads.

Where components are considerable distances apart, lattice members as shown in figures 22f and 22g shall be given preference to spaced members with battens, the effective

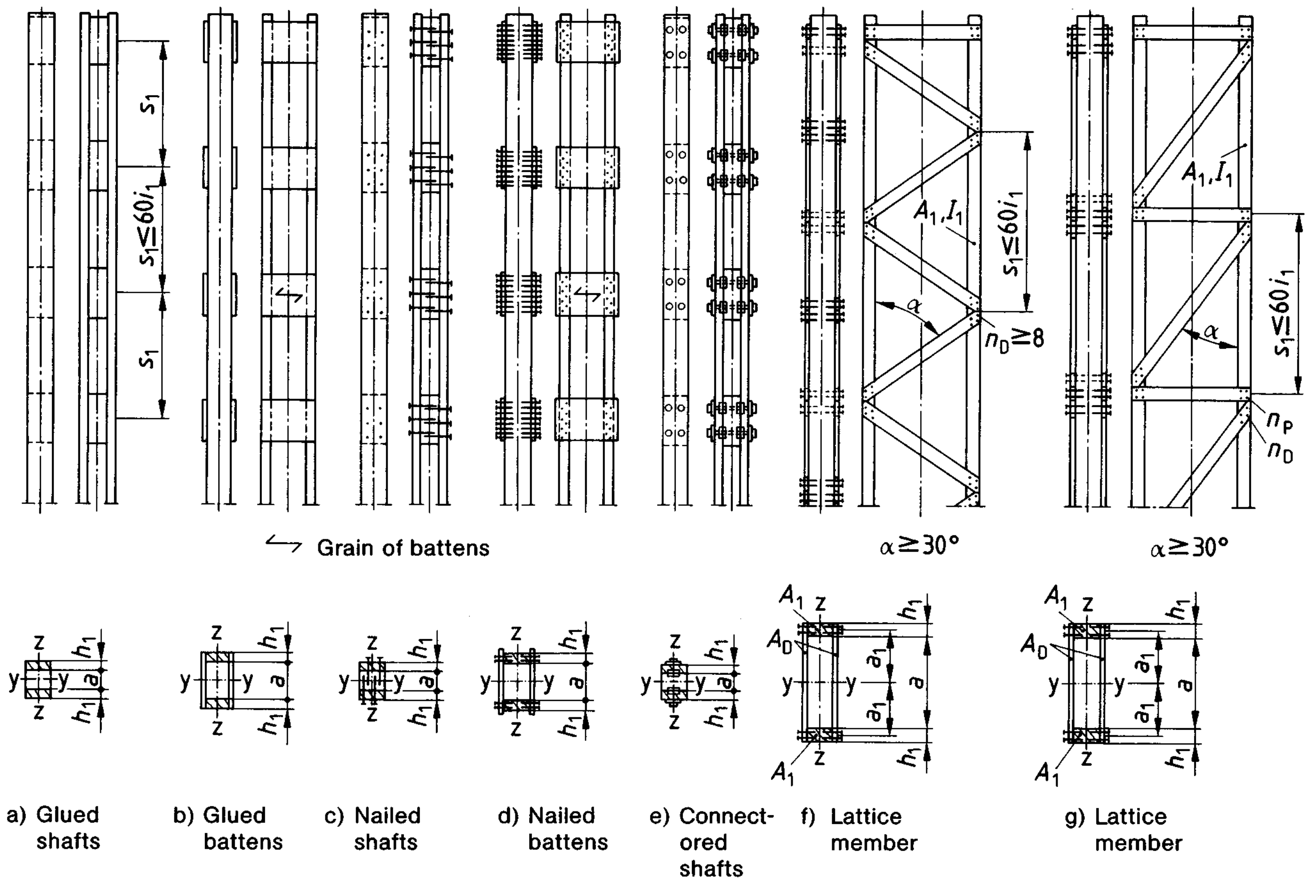


Figure 22. Spaced members (a to e) and lattice member (f and g)

slenderness ratio, $ef\ \lambda$, being calculated according to equation (64) but substituting for $c \cdot \lambda_1^2$ the following formula for members as shown in figure 22f:

$$\frac{4\,\pi^2 \cdot E \cdot A_1}{a_1 \cdot n_D \cdot C_D \cdot \sin 2\,\alpha} \tag{65}$$

and the following formula for members as shown in figure 22g:

$$\frac{4\,\pi^2 \cdot E \cdot A_1}{a_1 \cdot \sin 2\,\alpha} \cdot \left[\frac{1}{n_D \cdot C_D} + \frac{\sin^2 \alpha}{n_P \cdot C_P}\right] \tag{66}$$

where

A_1 is the cross section of the component;

C_D and C_P are the moduli of slip of the fasteners (taken from table 8) used for connection of struts or webs;

α is the angle of the strut;

n_D and n_P are the total number of fasteners used to connect all struts and webs.

Table 11. **Factor c for spaced members as shown in figures 22a to 22e**

Type of bridging	Type of joint	Factor c
Shafts	Bonded	1,0
	Connectored	2,5
	Nailed, screwed, stapled or dowelled	3,0
Battens	Bonded	3,0
	Nailed, screwed or stapled	4,5

9.3.3.4 Structural detailing and analysis of bridging

All shafts and battens, and all packing and joints between these shall be designed to resist the transverse force, Q_i, given in subclause 9.3.3.2, equation (63).

Spaced members with shafts as shown in figures 22a, 22c and 22e, usually having a relationship of a to h_1 of 3 or less, and spaced members columns with battens (see figures 22b and 22d), having a relationship of a to h_1 greater than 3 but not more than 6, shall be provided with bridging designed to resist a shear force, T (see figure 23), which shall be calculated as follows unless a more precise analysis is required:

$$T = \frac{Q_i \cdot s_1}{2a_1} \tag{67}$$

for a member consisting of two components ($m = 2$);

$$T = \frac{0{,}5 \cdot Q_i \cdot s_1}{2a_1} \tag{68}$$

for a member consisting of three components ($m = 3$);

$$T' = \frac{0{,}4 \cdot Q_i \cdot s_1}{2a_1} \text{ and} \tag{69}$$

$$T'' = \frac{0{,}3 \cdot Q_i \cdot s_1}{2a_1} \tag{70}$$

for a member consisting of four components ($m = 4$).

The number of interfaces in the spaced members shall be not less than three, so that there shall be bridging in at least each third of the member. Spaced and lattice members shall also be provided with bridging if they are not joined together by at least two connectors, one behind the other, or by four nails one behind the other.

Bridging pieces shall be connected to each component by at least two connectors or four nails. Where shafts are glued, the length of each one shall be at least twice the clear distance between components. Provided that the relationship a/h_1 is not more than 2, it is not necessary to check whether that shafts exhibit sufficient resistance to the bending moment due to shear force, T.

Struts or webs in lattice columns as shown in figures 22f and 22g shall be designed to withstand the gross force in the struts, N_D (calculated as $Q_i/\sin\alpha$), or gross force in the webs, N_P (equal to Q_i, Q_i being the notional effective transverse force obtained by means of equation (63)). Each strut or web shall be connected by at least four nails in single shear (cf. subclause 6.2.1 of DIN 1052 Part 2).

9.4 Eccentric compression (coexistent bending and compression)

Members that are designed to be subjected to compression at a certain distance from their centroid or to exhibit a certain curvature even before they are in their loaded state, or members which are not only in compression but are also stressed perpendicular to their longitudinal axis, shall be deemed as being subjected to design eccentricity of compression.

Such members shall initially undergo the standard check for coexistent bending and compression, the effect of deformation being neglected on the basis of the following condition:

$$\frac{\dfrac{N}{A_n}}{zul\ \sigma_{D\parallel}} + \frac{\dfrac{M}{W_n}}{zul\ \sigma_B} \leq 1 \qquad (71)$$

where the appropriate values of $zul\ \sigma_{D\parallel}$ or $zul\ \sigma_B$ shall be taken from table 5 or 6, taking into account subclauses 5.1 and 5.2. Due consideration shall be given to reductions in cross section as set out in subclause 6.4.

Unless a more rigorous treatment is required, the stability analysis shall be carried out according to the following equation:

$$\frac{\dfrac{N}{A}}{zul\ \sigma_k} + \frac{\dfrac{M}{W}}{k_B \cdot 1{,}1 \cdot zul\ \sigma_B} \leq 1 \qquad (72)$$

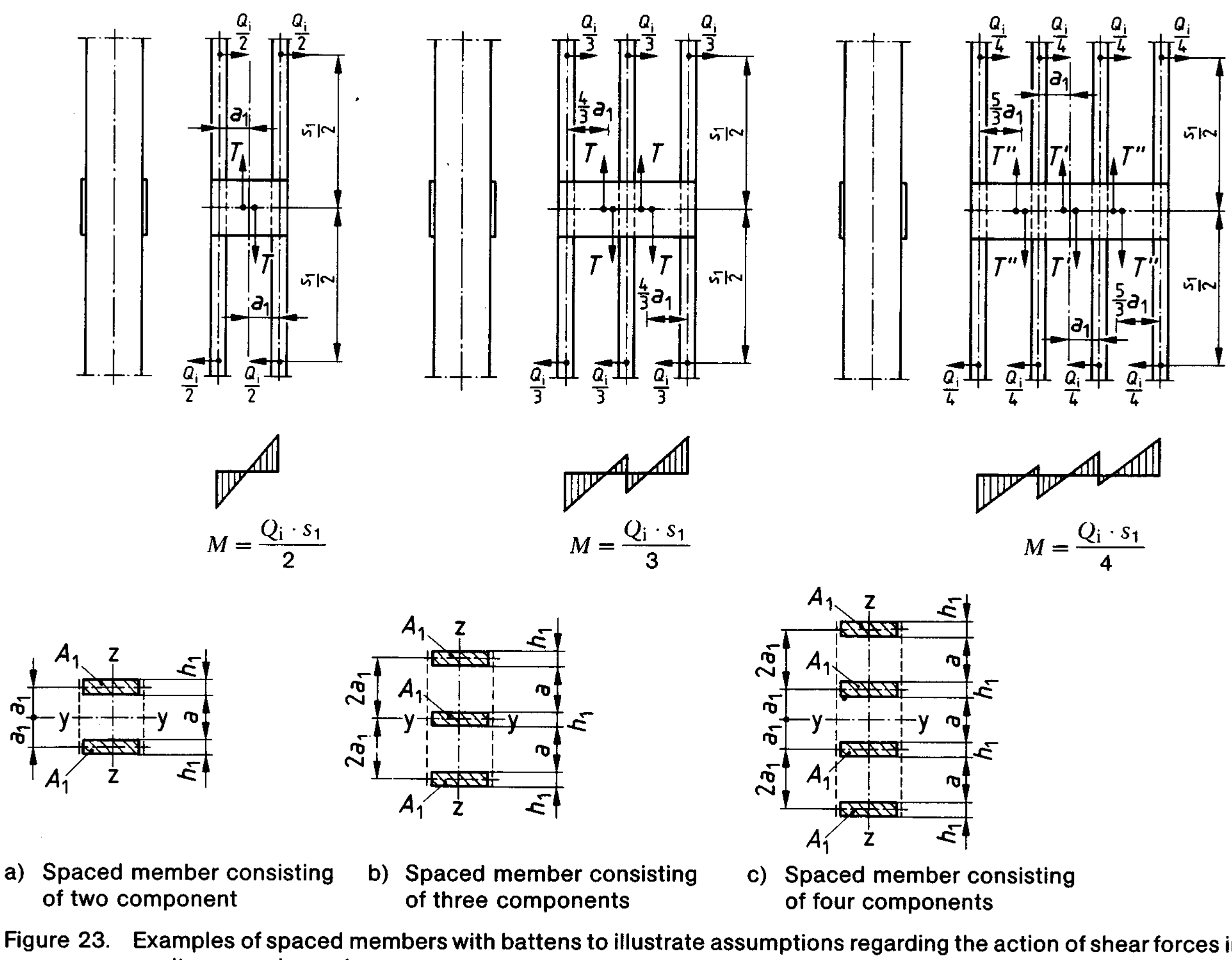
where $zul\ \sigma_k$ is to be obtained from equation (59); the maximum value of ω shall be substituted, irrespective of the direction of eccentricity, and k_B is to be obtained using equations (48) to (50).

In the case of composite members connected so as to prevent, as far as possible, any slip occurring, the magnitude of bending stresses shall be calculated as set out in subclause 8.3, taking into account the effective second moment of area, $ef\ I$. Frame and lattice members as shown in figure 22 shall normally only be subjected to centric loading. Perpendicular to their void axis, they may be subjected to wind and other additional loads subject to their effects being investigated.

9.5 Splices (butt joints)

Where compression members are designed to be centrically loaded and the joints take the form of perfect butt joints (made, if need be, using appropriate fitting pieces), it is sufficient to keep the components in place by means of straps. However, this is only permitted for the first and last quarters of the effective length, the fasteners being designed to accommodate half the design compression (considered without a buckling coefficient).

In all other cases, the second moments of area about both axes of the compression member shall be assumed to be the connecting elements and the total compression is required to be resisted by the fasteners, any likelihood of slip occurring at the joints being taken into consideration.



a) Spaced member consisting of two component

b) Spaced member consisting of three components

c) Spaced member consisting of four components

Figure 23. Examples of spaced members with battens to illustrate assumptions regarding the action of shear forces in composite spaced members

9.6 Check for structural adequacy under working load by second order theory

9.6.1 For structural systems which are not stiffened in their plane by bracing, plates or the like (e.g. frames as shown in figure 25), a check for structural adequacy under working load by second order theory may be carried out instead of the check for safety against buckling as set out in subclauses 9.1 to 9.4. Either of these checks will suffice.

The likelihood of slip occurring at the joints or of deformations due to creep may need to be taken into account and a linear correlation between the stiffness of the system and the deformation behaviour may be assumed.

The resistance to bending, strain and shear shall be determined using the moduli of elasticity and shear given in tables 1 to 3, and the spring stiffness of joints at which slip is likely to occur shall be determined using eight times the moduli of slip given in clause 13 of DIN 1052 Part 2.

The creep coefficient may be determined as specified in subclause 4.3, a suitable proportion of the imposed load being assumed to act as a permanent load, if required.

9.6.2 The structural analysis by second order theory for load cases H and HZ shall be based on loads multiplied by a factor of γ_1 (equal to 2,0) or γ_2 (equal to 3,0), allowance being made for any prestraining, which may include deformations due to creep, as specified in subclauses 9.6.3 to 9.6.6.

Structural adequacy shall be deemed to be attained if the following conditions are met.

a) Under loads multiplied by a factor of γ_1, the permissible stresses given in clause 5, multiplied by a factor of γ_1, and the permissible loads on the fasteners specified in DIN 1052 Part 2, also multiplied by a factor of γ_1, are exceeded at no place in the structure.

b) Under loads multiplied by a factor of γ_2, the relevant deformations (in particular maximum deflection and horizontal slip) are not greater than 4,5 times the associated deformations under γ_1 times these loads.

c) The smallest radius of gyration of non-composite and non-built up linear members in the plane of the structural system shall be at least 1/150 of their length, that of composite, non-bonded members, 1/175, and that of bracing members and tension members that are subjected to negligible additional loads, 1/200 of their length.

9.6.3 Even where members are intended to be without curvature and to be subjected to centric compression, a certain degree of technical imperfection is inevitable in practice and shall be allowed for by assuming a sinusoidal or parabolic curvature of the member axis, the design accidental eccentricity at mid-height with the member in the unloaded state, e (see figure 24), being calculated as follows:

$$e = \eta \cdot k \cdot \frac{s}{i} \qquad (73)$$

where

s is the projected length of the member;

i and k are the radii of gyration and the core width of the cross section respectively, no allowance being made for any likelihood of slip at the joints of composite members;

η is an initial curvature coefficient, equal to 0,003 for glued laminated members and 0,006 for solid timber members made of grade I or II softwood or of medium-grade hardwood.

Where cross sections are asymmetrical, the larger value of k shall be taken.

9.6.4 In the case of framed structures, additional allowance shall be made for an accidental slope of the posts in the least favourable direction when the frame is in its unloaded condition. The same applies, by analogy, to columns standing alone or in rows (see figure 25).

The design deviation of the post or column from its intended position, ψ, shall be assumed to be the following:

$$\psi = \pm \frac{1}{100 \cdot \sqrt{h}} \qquad (74)$$

where

h is the height of the post or column, or, in the case of multi-storey frames, the overall height of the structure, in m.

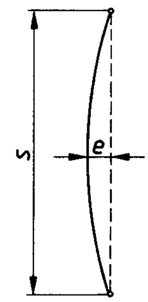


Figure 24. Member with accidental eccentricity, e, in unloaded condition

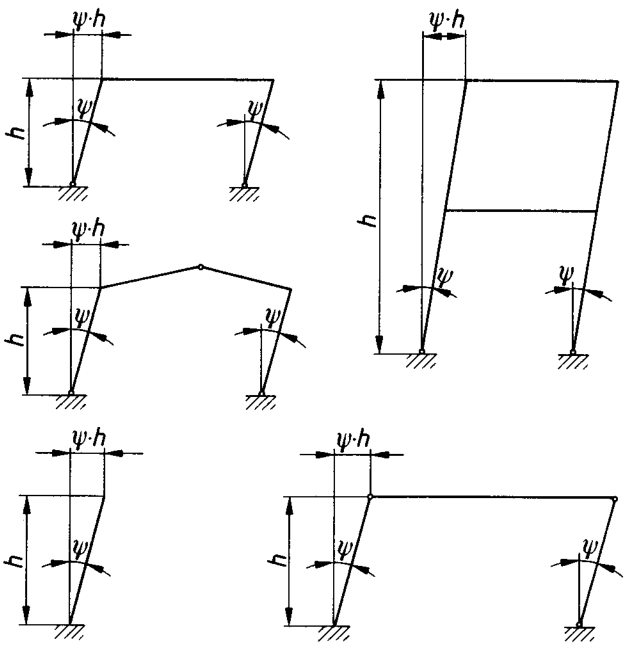


Figure 25. Frames, columns and rows of columns having accidental slope

9.6.5 Where members are intended to be subjected to eccentric compression, allowance shall also be made for the design eccentricity as given in equation (73) except where the intended eccentricity, M/N, in the cross section at the end or in the middle of the member, whichever is relevant, is not less than $20 \cdot e$.

9.6.6 Where the posts of frames exhibit an intended eccentricity of not less than $1/5 \cdot \sqrt{h}$ (both quantities expressed in m), the slope of the posts may be disregarded. This rule applies, by analogy, to columns standing alone or in rows.

9.6.7 The analysis of deflection as set out in subclause 8.5 in the service condition may be carried out by first order theory.

10 Bracing, plates and lateral supports

10.1 Stiffening of compression flanges of flexural members

Beams and compression flanges of trussed or lattice beams shall be protected from lateral deflection, the appropriate method of analysis being outlined in subclause 8.6 for beams and in subclause 9.3, taking into account subclause 9.1.2 or 9.4, for the compression flange of trussed or lattice beams.

10.2 Design principles

10.2.1 General

Unless direct lateral support is provided in the form of fixed objects, linear members, L-frames, etc., beams, plates or bracing shall ensure adequate stiffening.

10.2.2 Compression flanges of trussed and lattice beams

Unless a more detailed treatment is required, the stiffening of compression flanges of trussed and lattice beams shall be designed on the basis of a uniformly distributed lateral load equal to the following, assumed to act perpendicular to the plane of the beam in both directions:

$$q_s = \frac{m \cdot N_{\text{Flange}}}{30 \cdot l} \tag{75}$$

where

m is the number of compression flanges requiring stiffening;

N_{Flange} is the mean force acting in the flange in the least favourable load case;

l is the effective span of the stiffening element.

10.2.3 Box beams

Unless a more detailed treatment is required, the stiffening elements of box beams with a ratio of height to width of not more than 10 may be designed on the basis of a uniformly distributed lateral load equal to the following, assumed to act perpendicular to the plane of the beam in both directions, whilst this is mandatory for beams the ratio of height to width of which is greater than 10:

$$q_s = \frac{m \cdot max\ M}{350 \cdot l \cdot b} \tag{76}$$

where

m is the number of beams requiring stiffening;

$max\ M$ is the maximum bending moment in each beam as a result of vertical loading;

b is the width of the beam;

l is the effective span of the stiffening element.

The stiffening elements shall be connected to the compression flanges of the beams.

10.2.4 Coexistent wind and lateral loading

Where members serve both to stiffen compression flanges of beams or trussed or lattice beams and to provide resistance to wind, the full design wind load as specified in DIN 1055 Part 4 shall be assumed in cases where effective spans are not less than 40 m, whereas half of the design wind load shall be taken in the case of effective spans of 30 m or less, the wind load being superposed on the lateral load in each case. The analysis shall be based on the permissible stresses for load case HZ. Loading due to wind and lateral loading for effective spans between 30 m and 40 m may be obtained by linear interpolation.

Where members provide resistance to only either wind or lateral loads, the analysis shall be based on the permissible stresses for load case H (incorporating principal loads only).

10.2.5 Control of deflection and structural detailing

The design horizontal deformation of the stiffening elements when using equation (75) or (76) shall not exceed 1/1000 of the effective span. Analysis of deflection is not generally required in cases where the ratio of the height of the stiffening element to its span is not less than 1/6.

In order to limit the risk of deformation of the parts of the structure located between the stiffening elements and to take into account any likelihood of slip occurring at joints, at least two stiffening elements shall be provided for structures over 25 m in length. The clear distance between these elements shall not generally be greater than 25 m unless the necessity of such has been demonstrated in a more detailed analysis.

10.3 Plates

10.3.1 General

Plates conforming to the specifications outlined below shall be assumed to resist and transmit predominantly static loads (including wind loads) and earthquake loads acting in the plane of the plate. Plates comprise either an assembly of boards made from wood-based panel products frictionally connected to a substructure (such as a beam or binder with purlins) or, for effective spans of 30 m or less, of a system of panels (see subclause 11.3). It is recommended that the upper faces of the substructure all be in the same plane.

If, parallel to the direction of span of a plate made from wood-based panel products, there are more than two unsupported splices (see figure 26), then the effective span of the plate, l_s, shall be limited to 12,50 m.

The design deflection of plates made from wood-based panel products as a result of a vertical area load comprising self-weight and snow load or self-weight and imposed load shall be not more than 1/400 of the effective span of the plate.

10.3.2 Plates requiring analysis

The stress analysis of boards made from wood-based panel products and their substructure shall take into account the results of all types of stressing (i.e. including stresses due to their function as a plate). The permissible deflection of such plates shall be 1/1000 of the effective span of the plate.

10.3.3 Plates not requiring analysis

The minimum thickness of boards made from wood-based panel products shall be taken from table 12 as a function of the effective span of the plate. Their smallest plane dimension, h_s, shall be at least 1,0 m.

A stress analysis is not required for plates with a ratio of height, h_s, to effective span, l_s, of 0,25 or more.

An analysis of the effectiveness of the plate and of the deflection in its plane is not required if the specifications given in table 12 and figure 26 and the detailing provisions specified in subclause 10.3.1 are followed. For an analysis of the deflection perpendicular to the plane of the plate, the stresses due to the wood-based panel products acting as a plate and to the substructure need not be taken into account.

Nails in the area of the plate taken as having a stiffening function shall be spaced as specified in table 12, variations not being allowed.

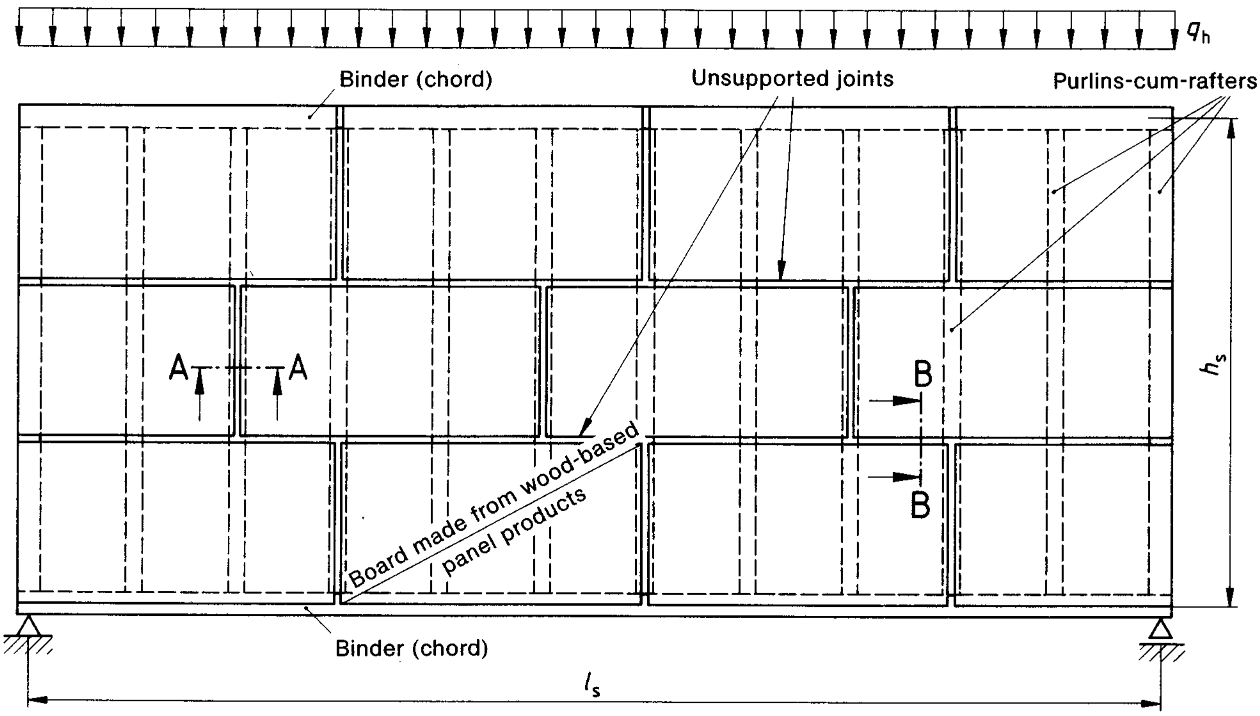
Figure 26 illustrates the required spacing of nails fastened perpendicular to the plane of the board, the joints between panels being positioned over the timber of the substructure.

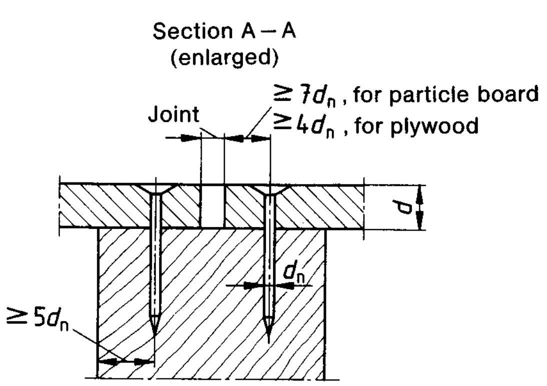
The purlins-cum-rafters at the edge of the slab (see figure 26) shall be at least 1,5 times wider than the inner joists.

Table 12. **Specifications for plates not requiring analysis**

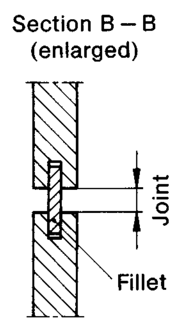
Uniformly distributed horizontal load, q_h, in kN/m	Effective span of plate, l_s, in m	Minimum board thickness, in mm		Required spacing of nails, e, in mm, for nail diameters of 3,4 mm[1]) and a plate height, h_s, of			
		Particle board	Plywood	$0{,}25 l_s$ or more	$0{,}50 l_s$ or more	$0{,}75 l_s$ or more	$1{,}0 l_s$
$\leq 2{,}5$	≤ 25	19	12	60	120	180	200
$\leq 3{,}5$	≤ 30	22	12	40	90	130	180

[1]) Where other nail diameters up to 4,2 mm are used, the required nail spacing shall be calculated by converting the given values on the basis of the permissible nail loads, a maximum spacing of 200 mm being permitted.





Supported joint between boards



Unsupported joint between boards

Figure 26. Stiffening plate with supported butt joints in the direction of loading and unsupported butt joints parallel to the span

10.4 Lateral restraint from battens and roof decking

Battens shall not be considered as providing adequate lateral restraint for flanges in compression except in cases where they support rafters which are otherwise liable to buckle or truss top chords at least 40 mm in width, subject to the span of the roof not exceeding 15 m, the spacing of rafters or trusses being 1,25 m or less and the cross-sectional height of the rafters, not more than four times their width of cross section.

In the case of roof trusses with chords 40 mm or more in width, subjected to a permanent load constituting less than 50 % of the overall load, roof decking composed of timber boards at least 120 mm in width, laid at right angles to the chords may be considered as providing lateral restraint if each board can be correctly fastened to each chord by means of at least two nails, even where boards are joined over it (see DIN 1052 Part 2), trusses are 1,25 m or less apart, the span of the truss is 12,50 or less and the length of the roof surface is at least 0,8 times the truss span but 25 m at the most. The joints between boards shall be staggered by an amount equal to at least twice the distance between trusses, and the width of the joint shall be not more than 1,0 m. The roof decking shall be frictionally connected to the wind bracing or other stiffening elements.

Separate bracing shall be provided to resist wind loads parallel to battens or boards.

10.5 Punctual lateral restraint for division of effective length into sections

Elements serving to provide lateral restraint for a compression member and thus divide its effective length into sections, as specified in subclause 9.1.1, shall be designed to resist the following concentrated loads when made of solid timber:

$$K = N/50 \tag{77}$$

and the following when made of glued laminated timber:

$$K = N/100 \tag{78}$$

where N is the maximum axial force (without buckling coefficient) acting in the compression members given punctual support.

If a single element supports more than one compression member (see figure 27), the appropriate bearing reactions in each section shall be taken into account.

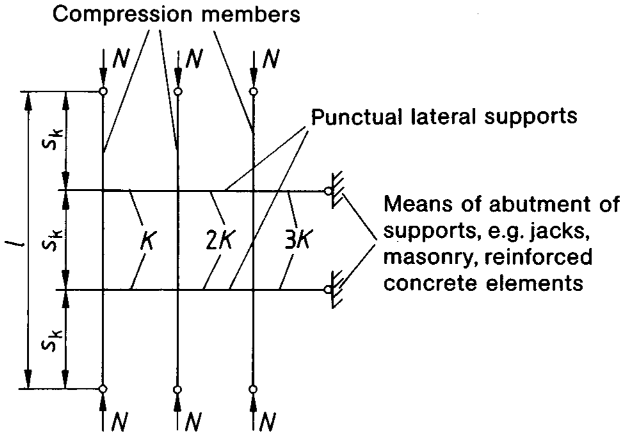


Figure 27. Punctual lateral support of compression members

Elements which support a compression member against bracing as set out in subclause 10.2 and thus reduce into sections the effective length of the member, as specified in subclause 9.1.1, shall be designed to withstand the lateral load component, q_s, and at least one concentrated load as given by equation (77) or (78), whichever is the least favourable, and shall be connected as appropriate.

11 Timber panels

11.1 General

11.1.1 Materials, minimum thicknesses and reductions in cross-sectional area

Grade 20 wood-based panels products complying with DIN 68 000 Part 2 may be used as panel sheathing unless other grades are required for timber preservation purposes.

The minimum thickness of panels shall remain within the limits set out in table 13, taking local reductions in cross section into account. Studs made from converted timber shall be of at least grade II, cutting grade A, as set out in DIN 4074 Part 1 and shall be free of wane over a minimum thickness of 24 mm. Where sheathing boards are joined over studs, the faces bordering on the stud shall be square edged over a minimum of 24 mm except where panels are glued (and there is no pressure generated by nails or staples), in which case a minimum length of square edge of 12 mm is required.

Table 13. **Minimum thickness of panels**

Material	Minimum thickness, in mm	
	Studs [1])	Sheathing
Converted timber, glued laminated timber	24	–
Plywood	15	6
Particle board	16	8

[1]) The minimum cross-sectional area of converted timber shall be 14 cm² and that of wood-based panel products, 10 cm².

Recesses in boarding with a loadbearing function may be neglected in the stress analysis if, over a panel area of 2,5 m², the total recess area is 300 cm² at the most and, in addition, no single recess is more than 200 mm long or the sum of the widths of all the recesses in the panel cross section is greater than 200 mm.

11.1.2 Moisture content

Panels shall be made of timber the moisture content of which shall be not greater than 18 %, or not more than 15 % where intended for glueing.

11.1.3 Loadbearing connections

Connections with the end-grain of wood or with interfaces of boards made from wood-based panel products shall not be considered loadbearing except where wood-based sheathing is glued to the interfaces of studs also made from wood-based panel products.

In the case of glued joints, the bond between the stud and the sheathing shall be at least 10 mm in width. The bond between solid timber studs and sheathing may incorporate the generation of pressure by nails or staples, subject to the specifications of subclause 12.5 being met.

11.2 Panels subjected to compression or bending (see figures 1a, 1c and 1d)

11.2.1 General

Panels with sheathing made from wood-based panel products having a loadbearing function do not require a lining. The same applies to sheathing with a stiffening function provided that the ratio of height to breadth of the stud is not greater than 4.

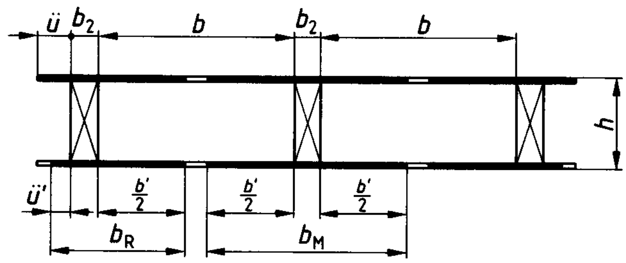
Where cross sections are a combination of timber and wood-based panel products, analyses shall be based on the buckling coefficients appropriate to the stud.

The stresses at the bending edge of studs shall not exceed the permissible limits and the centroidal stresses in the sheathing, the permissible compression or tension.

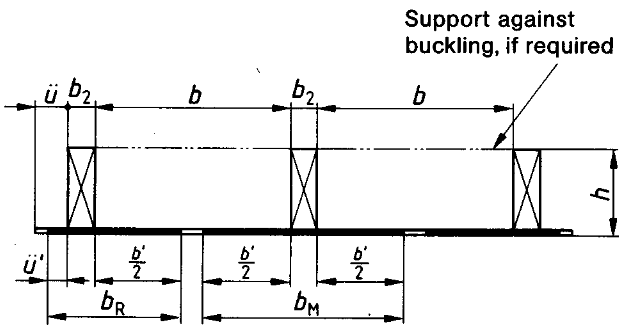
The increase in the permissible bending stress, as specified in subclause 5.1.8, shall only be applicable to timber studs.

Sag due to shear strain may be neglected in the case of panels with timber studs.

Butt joints between sheathing boards shall be taken into account for the purposes of the stress analysis, but only from a distance b (denoting the clear spacing of studs) onwards from the joint in the case of glued panels or, in the case of sheathing boards which are non-rigidly connected, from the point of application of the longitudinal force to be withstood by the sheathing. Joints between sheathing boards may usually be neglected for the analysis of deflection and the check for safety against buckling.



a) Panels with sheathing and lining



b) Panels with sheathing only

Figure 28. Effective width of sheathing

11.2.2 Effective width of sheathing

The analysis of sheathing made from wood-based panel products may assume the widths b_M and b_R, as shown in figure 28, as the effective widths of sheathing per stud at the middle and edge of the sheathing respectively:

$$b_M = b' + b_2 \tag{79}$$

$$b_R = b'/2 + b_2 + ü' \tag{80}$$

where

b is the clear distance between studs;

b' is the effective width between two studs;

b_2 is the width of the stud;

$ü$ is the lateral projection of the sheathing;

$ü'$ is the effective width of the lateral projection;

h is the overall cross-sectional height;

l is the length of span or span section.

The length of span, l, of floor panels is the distance between the points of zero moment assuming an even load distribution between all spans (l corresponding to the effective span in the case of simply supported panels without cantilever) or, in the case of panels requiring a check for buckling strength, the appropriate effective length. Should board sections feature recesses or other interruptions transverse to the direction of span of the panel (e.g. joints between sheathing boards), only the span sections up to the interruption may be taken as l.

b' and $ü'$ shall be determined for each span or span section, l, and clear width, b, a distinction being made between uniform line loads in the direction of span of the panel and concentrated loads (or line loads transverse to the direction of span).

Where a uniformly distributed load can be assumed transverse to the direction of span of the panel or in all directions (e.g. due to the presence of transverse nogging of approximately the same cross section as the longitudinal studs), the effective edge and inner zones of a panel may be considered together as a single cross section. In all other cases, each part shall be analyzed separately.

Unless a more detailed treatment is required, the following conditions may apply in cases where the b/l ratio is not greater than 0,4 and the panel is subjected to a uniform line load,

for plywood:

$$b'/b = 1{,}06 - 1{,}4 \cdot b/l \tag{81}$$

for particle board:

$$b'/b = 1{,}06 - 0{,}6 \cdot b/l \tag{82}$$

b' being not greater than b in both cases.

In cases where the b/l ratio is not greater than 0,4, the effective width under concentrated load, b'_F, can be obtained by approximation as follows:

for plywood:

$$b'_F/b = 1 - 1{,}8 \cdot b/l, \text{ where } l/c_F \leq 5 \tag{83}$$

$$b'_F/b = 1 - 2{,}6 \cdot b/l, \text{ but not less than 0,2, where } 5 < l/c_F \leq 20 \tag{84}$$

for particle board:

$$b'_F/b = 1 - 0{,}9 \cdot b/l, \text{ where } l/c_F \leq 5 \tag{85}$$

$$b'_F/b = 1 - 1{,}4 \cdot b/l, \text{ where } 5 < l/c_F \leq 20 \tag{86}$$

Lateral projections, $ü$, which are not supported by adjacent elements shall be assumed to have an effective width, $ü'$, equal to b_2, whilst in all other cases $ü'/ü$ shall be determined in the same way as b'/b but substituting $2 \cdot ü/l$ for b/l.

c_F shall be taken to be the sum of that length of panel (in the direction of its span) over which the load is applied, and twice the gross cross-sectional height of the panel, h.

If the line of load action is at a distance less than b from a point of zero moment, or if the ratio of l to c_F is greater than 20, then b'_F shall be taken to be equal to zero.

The stress analysis of the loads acting in the vicinity of support moments in continuous or cantilever panels shall always be assumed to be concentrated.

The effective width in combination with a constant line load may be used for the analysis of deflection and the structural analysis.

11.2.3 Geometrical characteristics

The geometrical characteristics of the inner or edge zones of panels with or without a lining shall be determined taking into account the condition that n_i is to be equal to E_i/E_v. See figure 29 for an example of a composite cross section consisting of three parts, in which:

E_1 and E_3 are the elastic moduli of the sheathing in compression and tension respectively;

E_2 is the elastic modulus of studs made from solid or glued laminated timber, the elastic modulus in bending of studs made from wood-based panel products mainly subject to bending or the elastic modulus in compression of studs made from wood-based panel products mainly subject to compression;

E_v is any elastic modulus used for reference purposes.

Sheathing may be taken to make a structural contribution over its effective width, b_M or b_R, as specified in subclause 11.2.2.

If sheathing or studs are to be jointed by glueing, their connection may be deemed to be rigid (i.e. there is no likelihood of slip).

When using fasteners conforming to DIN 1052 Part 2, due consideration shall be taken to the likelihood of slip occurring at joints. The geometrical characteristics shall be calculated according to subclause 8.3 irrespective of whether cross sections are asymmetrical (see figure 29).

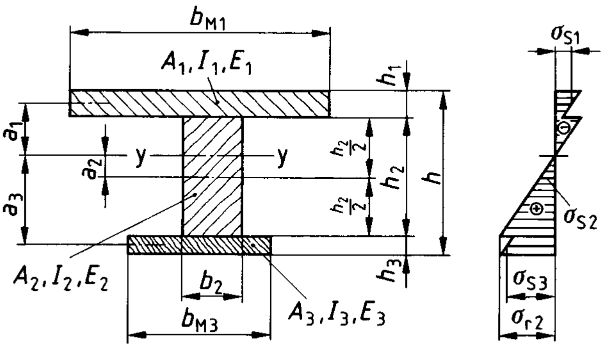


Figure 29. Asymmetrical cross section with sheathing on both sides

11.2.4 Spacing of studs

Sheathing made from wood-based panel products shall be stiffened by means of longitudinal studs spaced a clear distance, b, apart, this being calculated as follows:

$$b \leq 1{,}25 \cdot h_{1,3} \cdot \sqrt{E_{Bv}/\sigma_{Dx}} \quad (87)$$

where

h_1 and h_3 are the thicknesses of the sheathing;

E_{Bv}, equal to $\sqrt{E_{Bx} \cdot E_{By}}$, is the reference elastic modulus in bending of the sheathing;

σ_{Dx} is the compression acting in the sheathing (considered without a buckling coefficient).

However, b shall be not more than $50 \cdot h_1$ or $50 \cdot h_3$. Where h_1 and h_3 are not equal, the smaller value of b shall be taken.

11.3 Floor and roof plates of panel construction

11.3.1 General

Floor and roof plates complying with the following specifications may be taken to be effective in resisting and transmitting predominantly static loads (including wind and earthquake loads) acting in their plane over effective spans extending up to 30 m. For simplification, they may be treated as beams.

The height of plates, h_s, shall be equal to at least one quarter of their effective span, l_s (see figure 30). In those cases where the height of plates is greater than their effective span, the value assigned to h_s shall be assumed to be not more than l_s.

11.3.2 Deflection

The maximum permissible deflection shall be 1/1000 of the effective span, shear strain being taken into account. An analysis of deflection is not required for plates the effective span of which is not more than twice their height.

Butt joints between sheathing sections in each panel need not be taken into consideration provided that they are oriented parallel to the load and they are spaced apart or from the point of support of the plate at a distance of not less than $l_s/4$.

Where joints are spaced at distances ranging from $l_s/4$ to $l_s/8$, the design stiffnes of the gross cross section shall be reduced by one third. Joint spacing less than $l_s/8$ is not permitted.

11.4 Wall plates of panel construction

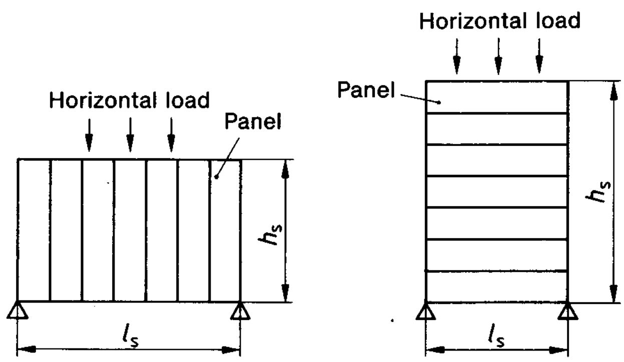
11.4.1 General

Wall plates of panel construction are subject to stressing from horizontal loads acting in their plane, as illustrated in figure 1b, sometimes in combination with vertical loads as shown in figure 1a or horizontal loads as shown in figure 1c.

Panels subject only to the loading pattern illustrated in figure 1a or 1c shall be designed in accordance with the specifications of subclause 11.2

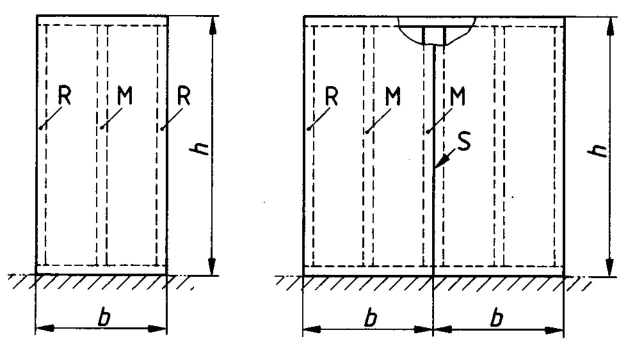
The specifications given in subclause 11.4 are applicable to wall plates not featuring openings. Unless a more detailed treatment is required, the design analysis of such plates shall be in accordance with subclauses 11.4.2 and 11.4.3. If wall plates featuring openings such as windows are to be counted as making a contribution to load transmission, due account of these openings shall be taken when determining their structural performance.

A distinction shall be made between panels comprising one sheathing board (see figure 31a) and composite panels comprising more than one sheathing board (see figure 31b). The width, b, of a board is defined as the distance between the outer edges of the end studs or, in panels comprising more than one board, as the distance between the outer edges of the end stud and the joint between boards or the distance between vertical joints between boards, or as a combination of both, or a maximum of 0,5 times the height of the panel.



a) $h_s < l_s$ b) $h_s > l_s$

Figure 30. Examples of roof and floor plates of panel construction (plan view); dimensions and loading direction



R End stud
M Inner stud

a) Panel comprising one sheathing board b) Composite panel comprising two sheathing boards with joint (S)

Figure 31. Examples of panels comprising one and more than one sheathing board

11.4.2 Design of wall plates to accommodate a horizontal load acting in their plane

11.4.2.1 Wall plates consisting of panels comprising one sheathing board

The specifications below are applicable to panel widths of 0,60 m or more.

An analysis of the capability to resist and transmit the following forces shall be carried out.

a) Compressive stress, D_1, in lower horizontal timber (as shown in figure 32):

$$D_1 = \alpha_1 \cdot F_H \cdot h/b_{s1}, \text{ where } \alpha_1 \text{ is to be taken from table 14.} \tag{88}$$

b) Tensile anchor force:

$$Z_A = F_H \cdot h/b_{s1} \tag{89}$$

If the lower horizontal timber does not extend beyond the end stud in compression, Z_A shall be increased by 10% for panels comprising one sheathing board.

c) In the case of panels without a lining, the tensile force, Z, resulting from the notional strutting effect in the sheathing shall be calculated and shall be verified as being resisted by the notional strut having a width b_Z as shown in figure 33. A value of b_Z equal to 0,50 m may be assumed for panels at least 1,20 m in width without further analysis being required. The components Z_H and Z_V shall be taken to act in the upper and lower horizontal members and end studs over the lengths b and h' respectively.

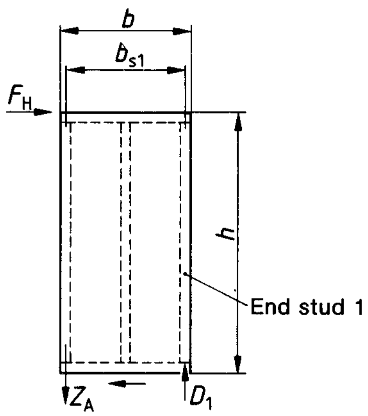


Figure 32. Tensile anchor force, Z_A, and compressive force, D_1, in lower horizontal member

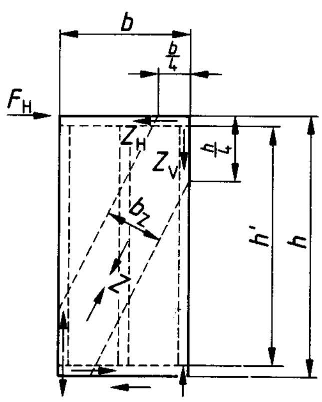
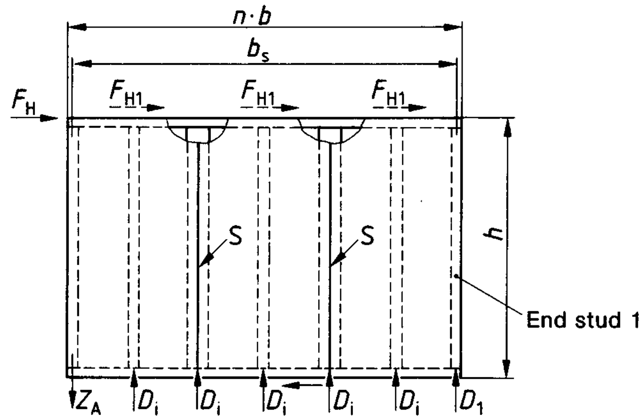


Figure 33. Resolution of tensile force, Z, of notional strut in sheathing of panels without a lining

Sheathing boards and their connection need not be analyzed for panels which are lined on both sides and are of a width, b, of at least 1,0 m. The maximum spacing of fasteners shall be kept within the limits specified.

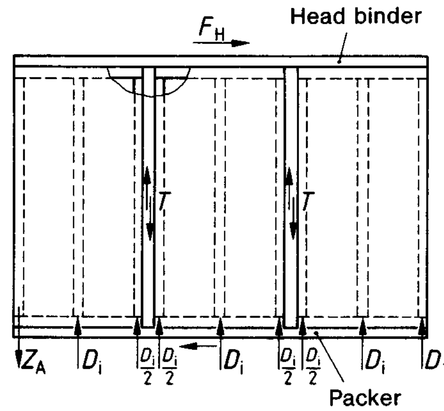
The permissible lateral deflection in the upper horizontal timber shall be 1/500 of the panel height, h. If the relationship of panel height to width is 3,0 or less, an analysis is not required irrespective of whether panels are provided with a lining.

11.4.2.2 Wall plates consisting of composite panels comprising more than one sheathing board



S = joint between sheathing boards

a) Tensile anchor force, Z_A, and compressive forces, D_i, acting in lower horizontal timber



b) Tensile anchor force, Z_A, compressive forces, D_i, in lower horizontal timber and shear force, T, acting in composite panels comprising more than one sheathing board

Figure 34. Composite panels comprising more than one sheathing board

Panels comprising n sheathing boards (see figure 34) shall be designed by analogy with subclause 11.4.2.1.

The compressive forces, D_i, in the lower horizontal timber shall be calculated as follows:

$$D_i = \alpha_i \cdot F_H \cdot h/b_s \tag{90}$$

α_i being taken from table 14.

Resistance to Z_A, which is equal to $F_H \cdot h/b_s$, need be accommodated only at the tension edge of the panel (viewed as a whole).

Table 14. **Factors α_1 and α_i for panels comprising sheathing boards of width, b, 1,20 m and more**

Panel	Number of boards, n	α_1 (end stud 1)	α_i (other studs)
With lining	1 2 >2	2/3 [1] 2/3 1/2	0 1/5 1/5
Without lining	1 ≥ 2	3/4 [1] 3/4	0 2/5

[1] α_1 shall be taken as 1,0 for b equal to 0,60 m, intermediate values being obtained by linear interpolation for b between 0,60 m and 1,20 m.

If panels of the composite type are formed by combining single boards, the boards shall be interconnected so as to provide adequate resistance to shear. Unless a more detailed treatment is required, fasteners shall be designed to resist a shear force, T, equal to Z_A (see figure 34b). In addition, continuous head binders shall be provided at the top and, if required, packers at the base, the connections of which shall be designed to transmit the horizontal load, F_H.

11.4.3 Analysis of pressure in lower horizontal timbers as a result of vertical loading

11.4.3.1 Composite panels comprising one sheathing board

Transmission of vertical loads, F_{Vi}, into the substructure is effected by the studs, which exert pressure on the lower horizontal timber, and by the sheathing by being directly connected to the lower horizontal timber (see figure 35). The calculation of each compressive force acting in the lower horizontal timber shall be effected by dividing the overall load, $\sum F_{Vi}$, in the ratio of the relevant permissible compressive force, D_i, to the total permissible compression, $zul\ D$ (which is equal to $\sum(zul\ D_i) + zul\ D_{Shea}$), $zul\ D_{Shea}$ denoting the maximum permissible load to which the entity of fasteners in the lower horizontal timber may be subjected. Where panels are fastened by glueing, the compressive stress in the sheathing shall not exceed the permitted limit.

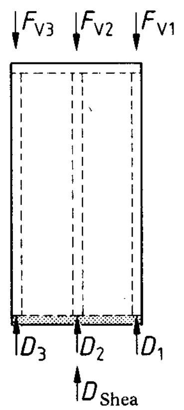


Figure 35. Panels comprising one sheathing board under vertical load, F_{Vi}, compressive force, D_i, in the lower horizontal timber and force D_{Shea} transmitted from sheathing to lower horizontal timber

11.4.3.2 Composite panels comprising more than one sheathing board

Panels with n number of sheathing boards shall be notionally divided into panels consisting of only one board. D_i shall be calculated separately for each board as specified in subclause 11.4.3.1. Where adjacent boards share a common stud, F_{Vi} and the cross section of the stud shall be notionally allocated half each to either board.

11.4.4 Analysis of pressure in lower horizontal timbers due to coexistent horizontal and vertical loads

The compression in the lower horizontal timber due to F_H as specified in subclause 11.4.2 and that due to F_V as specified in subclause 11.4.3 shall be combined for verification as being within the permissible limits.

11.4.5 Distribution of horizontal loads from floors or roofs

Horizontal loads from floors and roofs may be assumed to be evenly distributed between the boards where panel cross sections are uniform and proportionately where they are not (see subclause 11.4.2.2). Wall panels shall be suitably connected to floors or roofs.

11.5 Structural detailing of panels

Sheathing in the direction parallel to the studs shall be jointed above studs made of solid timber or glued laminated timber. Such joints shall not be provided directly above studs where wood-based panel products would be in direct contact with the sheathing. The glueline between stud and sheathing shall be a minimum of 10 mm on each side of the joint.

At the unsupported edges near joints between sheathing boards, the sheathing shall be prevented from being subject to varying degrees of deflection due to the action of loads perpendicular to the panel by provision of suitable means of connection, e.g. dovetailing.

Wall panels acting as plates shall be provided at the top and base with horizontal frame members.

It shall be ensured that during construction work on the structure, this remains stable prior to completion of the floor or roof plate.

12 Glued joints

12.1 Qualification approval

Glued loadbearing timber members may only be used if supplied by an approved manufacturer (see appendix A).

Note. A register of approved manufacturers is kept at the *Institut für Bautechnik* (Institute of Building Technology), Reichpietschufer 74–76, D-1000 Berlin 30. This register is regularly updated and published in the bulletin of the Institute.

The use of building inspectorate approved timber members will also involve taking into account the specific conditions set out in the agrément certificate and, in some cases, an additional inspection certificate.

12.2 Moisture content of timber at the time of glueing

Only timber with a maximum moisture content of 15 % may be glued.

12.3 Longitudinal joints

Longitudinal joints shall be either scarf joints with tapers of 1 : 10 or finger joints of stress grade I as specified in DIN 68 140.

The stress analysis of the finger jointed cross section of members shall be performed using the following reduced cross section:

$$red\ A = (1 - v) \cdot A \qquad (91)$$

where v is the reduction coefficient as specified in DIN 68 140.

By way of exception to the above rule, a stress analysis of grade II solid timber as specified in table 1, line 1, and glued laminated timber as specified in table 1, line 2, having geometrical characteristics of section up to 300 mm may be performed without making allowance for the reduction coefficient if the following conditions are satisfied:

a) the calculated stresses are within the permissible stresses for grade II timber;

b) the finger joints are made by a duly approved manufacturer as specified in subclause 12.1.

In the case of members made of glued laminated timber, allowance need not be made for the reductions in area in each board due to finger joints.

12.4 Adhesives

The use of adhesives in connection with loadbearing members shall be subject to their passing the tests specified in DIN 68141.

Where members are intended for service outdoors, or indoors under climatic conditions which involve the possibility of the member being at an equilibrium moisture of over 20% or at a temperature of over 50°C over prolonged periods or at frequent intervals, only use of synthetic resins (e.g. resorcin or melamine resin adhesive) the resistance of which to climatic influences has been proven in appropriate tests, is permitted.

12.5 Glueing and bonding pressure

Bonding pressure shall be applied as uniformly as possible over the area to be glued.

Bonding pressure may also be generated by nails when fastening timber laminations up to 33 mm in thickness or boards made from wood-based panel products up to 50 mm thick by glueing. The nails used shall conform to DIN 1052 Part 2, their lengths being approximately 2,5 times the thickness of the lamination or board. Nails shall be spaced not more than 100 mm apart with a density of at least one nail per 65 cm^2.

Where plywood boards are over 20 mm thick, nailholes of a diameter equal to 85% of that of the nail shall be pre-drilled except where nailing guns are used.

If several boards or laminations are used in conjunction, each board or lamination shall be nailed separately, the nails being arranged in a staggered pattern.

12.6 Form and lay-up of glued laminated members

The thickness of the boards consisting of glued laminated members shall be between 6 mm and 33 mm except where members are straight but are not subjected to extreme changes in climatic conditions in service, in which case the thickness of boards may be increased to 40 mm.

With regard to curved members, the radius of curvature of each board, r_1, shall be at least equal to $200 \cdot a$, a denoting the thickness of the board, radii of curvature between $200 \cdot a$ and $150 \cdot a$ being permissible if a, in mm, satisfies the following condition:

$$a \leq 13 + 0{,}4 \left[\frac{r_1}{a} - 150\right] \tag{92}$$

The bending stresses due to the curvature of the boards prior to glueing may be neglected.

Where glued laminated members have a cross-sectional width of more than 220 mm, boards shall be provided with at least one groove (to compensate for stresses) running the length of the board. The depth of the groove (which shall be either sawn or sunk), shall at no point be greater than one quarter to one fifth of the thickness of the board, and the width of the groove shall be no more than 4 mm. Where glued laminated members consist of boards with a width of more than 220 mm but are not grooved, each lamination shall consist of at least two sections. The longitudinal joints of superposed laminations shall be staggered by an amount equal to either at least the board thickness or 25 mm (whichever is higher) provided that the boards making up the laminations are not glued along their narrow sides.

Where members are intended for direct exposure to the weather, even though they may be in reception of a protective coating, either at least the outer laminations in the tension and compression zones shall be parallel to the outer face of the member or appropriate laminations shall be added after cutting to size.

12.7 Transport and erection

During transport, storage and erection of the members, suitable precautions shall be taken to ensure that their moisture content is not disproportionately affected both by soil moisture or precipitation and severe drying out (see DIN 68800 Part 2).

13 Workmanship

13.1 Assembly and erection

13.1.1 All parts of a structure shall be assembled and erected so as to exclude the occurrence of unpermitted indirect action-effects or similar undesirable phenomena.

13.1.2 Loadbearing bolts and binding bolts forming part of connector units shall be tightened if considerable shrinkage of the timber is expected. Their threaded section shall be of sufficient length, and they shall remain accessible until termination of the shrinkage process.

13.1.3 Where outer metal parts are joined by means of precision bolts, adequate bearing capacity shall be ensured by the full cross section of the shank being present over the required length in order to enable the bolts to resist bearing forces.

13.2 Roof decking

13.2.1 Roof decking as base for tiling

For roof decking on which to fasten tiling, timber of grade II or better, as specified in DIN 4074 Part 1 and wood-based panel products of grade 100 or 100 G (see DIN 68800 Part 2) may be used.

Butt joints parallel to the supports shall only be located above the supporting members (such as rafters or purlins), with a bearing depth of at least 20 mm.

The unsupported edges of timber boards or planks or of wood-based panel products positioned perpendicular to the supports shall be dovetailed or interconnected by other appropriate means if the ratio of their clear width, l_w, to thickness, d, is greater than 30.

13.2.2 Roof decking as base for waterproofing sheeting

The following specifications shall be satisfied in addition to those set out in subclause 13.2.1.

Wood-based panel products of grade 100 G shall be used.

The design of joints shall take into account the changes in length and width to be expected as a result of swelling. This is usually achieved by allowing 2 mm/m for particle board and 1 mm/m for plywood.

Unsupported edges perpendicular to the bearings shall be dovetailed or interconnected by other appropriate means.

The pitch of the roof shall be not less than 2% except in the following circumstances.

a) The roofing sheet is, and will remain, absolutely waterproof even if there are temporary pools of water on the roof.

b) Where necessary, allowance for trapped water is made when designing the roof decking including the substructure.

14 Marking

The following members shall be permanently and clearly marked.

a) Members made from timber types as in table 1, line 1, of grades I and III shall be marked with the grade, the mark of the sorting works and that of the responsible examiner; where grade I members are prefabricated from more than one timber, the marking associated with grade I timber may be limited to those areas in which grade I characteristics are used as a basis for the design calculations.

b) Glued laminated timber as in table 1, line 2, of grade I and in the case of members over 10 m in length, of grade II, shall be marked with the grade, the date of production and the timber producer's mark.

c) Members made from hardwood as in table 1, line 3, shall be marked with the timber type (A, B or C), the mark of the sorting or production works and that of the responsible examiner.

Glued loadbearing members combining timber and wood-based panel products shall be marked with the date of production and the mark of the production works even if grade II solid timber or glued laminated timber is used.

Appendix A

Qualification approval for production of glued loadbearing members

A.1 Members may be considered to be of approved production as set out in subclause 12.1 when the producer can submit a certificate as specified in subclause A.3 stating that the works is qualified to produce glued members intended to have a loadbearing function.

A.2 Such certificates are issued by accredited testing agencies registered with the *Institut für Bautechnik* following due inspection of the relevant plant, key personnel and methods. A certificate is valid for five years (during which, given good grounds, it may be withdrawn at any time) and is renewable for further periods of five years upon application, subject to a renewed inspection of the works. The holder of the certificate shall inform the testing agencies of any change of key personnel or plant and any change in gluing methods.

A.3 A distinction is made between the following types of certificate.

a) Type A certificates are issued to manufacturers qualified to produce all types of glued structural member.

b) Type B certificates are issued to manufacturers qualified to produce glued structural members of simple construction (e.g. beams with effective spans of up to 12 m, three-hinged trusses with spans up to 15 m and L beams with a total length of up to 12 m); if members also qualify for a C certificate, this shall be stated.

c) Type C certificates are issued to manufacturers qualified to receive general building inspectorate approval for the production of glued structural members of special construction.

d) Type D certificates are issued to manufacturers qualified to produce timber panels for buildings in timber frame construction. Manufacturers with A or B certificates automatically carry this qualification.

In the certificates it shall also be stated if the manufacturer is qualified to produce finger joints as set out in subclause 12.3.

Standards referred to

DIN	96	Slotted round head wood screws
DIN	97	Slotted countersunk head wood screws
DIN	571	Hexagon head wood screws
DIN	1052 Part 2	Structural use of timber; mechanically fastened joints
DIN	1052 Part 3	Structural use of timber; buildings in timber frame construction; design and construction
DIN	1055 Part 3	Design loads for buildings; imposed loads
DIN	1055 Part 4	Design loads for buildings; imposed loads; wind loads in structures not susceptible to vibration
DIN	1055 Part 5	Design loads for buildings; imposed loads; snow load and ice load
DIN	1074	Wooden bridges; design and construction
DIN	4074 Part 1	Structural timber; grading of converted timber (softwood)
DIN	4074 Part 2	Structural timber; grading of round timber (softwood)
DIN	4112	Temporary structures; design and construction
DIN	4113 Part 1	Aluminium structures subject to predominantly static loading; design and construction
DIN	4149 Part 1	Buildings in German earthquake zones; design loads; design and construction of ordinary, commonly occurring buildings
DIN	4420 Part 1	Scaffolding and working platforms (excluding ladder scaffolding); design and construction
DIN	4420 Part 2	Scaffolding and working platforms; ladder scaffolding
DIN	4421	Falsework; design and construction
DIN	18800 Part 7	Steel structures; fabrication and verification of suitability for welding
DIN	50049	Materials testing certificates
DIN	52183	Testing of timber; determination of moisture content
DIN	55928 Part 1	Corrosion protection of steel structures; general
DIN	55928 Part 2	Corrosion protection of steel structures; proper design for prevention of corrosion
DIN	55928 Part 4	Corrosion protection of steel structures; preparation and testing of surfaces
DIN	55928 Part 5	Corrosion protection of steel structures; coating materials and protective systems
DIN	55928 Part 6	Corrosion protection of steel structures; workmanship and inspection of corrosion protective work
DIN	55928 Part 8	Corrosion protection of steel structures; corrosion protection of thin-walled structural members (light gauge steel construction)
DIN	68140	Wood finger jointing
DIN	68141	Wood joints; testing of adhesives and glued joints for timber structural members; grading
DIN	68705 Part 3	Plywood; structural veneer plywood
DIN	68705 Part 5	Plywood; structural plywood made from beech
Supplement 1 to DIN	68705 Part 5	Structural plywood made from beech; relation between lay-up, elastic properties and strength
DIN	68754 Part 1	Fibre building board (hardboard and mediumboard); wood-based panel product grade 20
DIN	68763	Particle board for structural purposes; concepts, requirements, testing and inspection
DIN	68800 Part 2	Timber preservation in building construction; structural preservation
DIN	68800 Part 3	Timber preservation in building construction; chemical preservation

Previous editions

DIN 1052: 07.33, 05.38, 10.40x, 10.47, 08.65; DIN 1052 Part 1: 10.69.

Amendments

In comparison with the October 1969 edition, the standard has been completely revised, especially with respect to the following.

a) Some non-European types of timber and particle board have been introduced as well as specifications relating to panels, sheathing and roof decking.

b) More information on material characteristics is now included and consideration given to creep deformation due to bending.

c) $zul\ \sigma$ has been increased by 25% for load case HZ, by 100% for impact and earthquake loads and by 50% for handling loads during transport and erection.

d) Details are provided of permissible torsional stresses and transverse tensile stresses in solid timber and in glued laminated timber (with reductions from 0,25 to 0,2 MN/m^2). In the case of plywood, higher values have been specified for shear ($zul\ \tau$) perpendicular to the plane of boards and partly also for plywood made from beech.

e) Reference is made to the possibility of designing flexural beams with reduced transverse force; included are provisions relating to torsion and transverse force, notching, beam penetrations, and curved beams and pitched cambered beams, and stress patterns at the sloping edge.

f) More elaborate formulae for the design of built-up beams (with cross sections symmetrical about a single axis or having differing E moduli) have been included.

g) C values are now given for dowels.

h) A stability analysis by second order theory has been introduced.

i) The basic equations for the design of members subject to tensile, compressive and bending stresses have been revised in order to improve their clarity and logical sequence.

j) New specifications have been introduced for verifying the stability of flexural beams of rectangular cross section.

k) The values of ω for determining the relevant permitted buckling stress are now given separately for various types of solid timber, glued laminated timber and wood-based panel products.

l) More information is provided on the design of stiffening elements for solid-web flexural beams of rectangular cross section, as this differs from that for trussed or lattice beams.

m) Specifications have been introduced for floor, roof and wall plates made of wood-based panel products and wood panels.

Explanatory notes

The symbols used in this standard differ in part from those employed in DIN 1080 Part 5, March 1980 edition, a revision of which is pending.

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