UNIVERSITY OF JOS FACULTY OF ENGINEERING

DEPARTMENT OF CIVIL ENGINEERING

CVE 306: DESIGN OF TIMBER NOTES

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Introduction

- The technological and scientific knowledge of engineering materials is essential for design.
- Extensive research into properties of timber has established that it is sensitive to environmental conditions E.g. moisture content, which has a direct bearing on the strength and stiffness as well as the swelling and shrinkage of timber.
- Therefore, adequate understanding of the physical characteristics of wood is necessary for the construction of safe and durable timber structures.

The structure of timber

- Structural timber is sourced from matured trees, thus adequate knowledge of the nature and growth patterns of trees is essential for understanding its behaviour under various loading conditions.
- Basically a tree has three subsystems; roots, trunk and crown. Each subsystem has a role to play in the growth pattern of the tree.
 - 1. Roots. They act as a foundation by spreading through the soil, enabling the tree to withstand wind forces. They also absorb moisture containing minerals from the soil and transfer it via the trunk to the crown.
 - 2. Trunk. This provides rigidity, mechanical strength and height to maintain the crown. It also transports moisture and minerals up to the crown and sap down from the crown.
 - 3. Crown. This provides as large as possible a catchment area covered by leaves. These produce chemical reactions that form sugar and cellulose which cause the growth of the tree.
- Engineers we are mainly concerned with the trunk of the tree. Consider a cross-section of a trunk as shown in Fig. 1.
- The figure shows that wood, in general, is composed of long thin tubular cells. The cell walls are made up of cellulose and the cells are bound together by a substance known as lignin.
- Most cells are oriented in the direction of the axis of the trunk, except for cells known as **rays** which run radially across the trunk. Rays are present in all trees but are more pronounced in some species, such as oak.
- In temperate countries, a tree produces a new layer of wood just under the bark in the early part of every growing season. This growth ceases at the end of the growing season or during winter months.
- This process results in clearly visible concentric rings known as **annular rings, annual rings** or **growth rings.**
- In tropical countries where trees grow throughout the year, a tree produces wood cells that are essentially **uniform**.
- The age of a tree may be determined by counting its growth ring.
- The annular band of cross-section nearest to the bark is called **sapwood**. The central core of the wood which is inside the sapwood is **heartwood**.

- The sapwood is lighter in colour compared to heartwood and is 25-170mm wide, depending on the species. It acts as a medium of transportation for sap from the roots to the leaves.
- The heartwood functions mainly to give mechanical support or stiffness to the trunk. In general, the moisture content, strength and weights of the two are nearly equal.
- Sapwood has a lower natural resistance to attacks by fungi and insects and accepts preservatives more easily than heartwood.



Fig. 1 Cross-section of a trunk of a tree

Defects in timber

- Because wood is a naturally occurring material, there are many defects which are introduced during the growing period and during the conversion and seasoning process.
- Any of these defects can result in either reducing the strength of timber or impairing its appearance.
- Defects may be classified as: natural defects, chemical defects, conversion defects and seasoning defects.

Natural defects

These are defects that occur during the growing period. Examples of natural defects are illustrated in Fig. 2(a). These may include:

- Cracks and fissures. They may occur in various parts of the tree and may even indicate the presence of decay or the beginnings of decay.
- Knots. These are common features of the structure of wood. A knot is a portion of a branch embedded by the natural growth of the tree, normally originating at the centre of the trunk or a branch.

- Grain defects. Wood grain refers to the general direction of the arrangement of fibres in wood. Grain defects can occur in the form of twisted-grain, cross-grain, flat-grain and spiral-grain, all of which can induce subsequent problems of distortion in use.
- Fungal decay. This may occur in growing mature timber or even in recently converted timber, and in general it is good practice to reject such timber.
- Annual ring width. This can be critical in respect of strength in that excess width of such rings can reduce the density of the timber.



Shake

Knot

Wane







Diagonal-grain

Cross-grain

Flat-grain

(a) Natural and conversion defects







Cupping

End splitting

Honeycombing



(b) Seasoning defects

Fig. 2 Defects in timber

Chemical defects

- These may occur in particular instances when timber is used in unsuitable positions or in association with other materials.
- Timbers such as oak and western red cedar contain tannic acid and other chemicals which corrode metals.
- Gums and resins can inhibit the working properties of timber and interfere with the ability to take adhesives.

Conversion defects

- These are due basically to unsound practice in the use of milling techniques or to undue economy in attempting to use every possible piece of timber converted from the trunk.
- A wane is a good example of a conversion defect.

Seasoning defects

- Seasoning defects are directly related to the movement that occurs in timber due to changes in moisture content.
- Excessive or uneven drying, exposure to wind and rain, and poor stacking and spacing during seasoning can all produce defects or distortions in timber.
- Examples of seasoning defects are illustrated in Fig. 2(b). All such defects have an effect on structural strength as well as on fixing, stability, durability and finished appearance.

Types of timber

Trees and commercial timbers are divided into two groups: **softwoods** and **hardwoods**. This terminology has no direct bearing on the actual softness or hardness of the wood.

Softwoods

- Softwoods are generally evergreen with needle-like leaves comprising single cells called **tracheids**, which are like straws in plan, and they fulfil the functions of conduction and support.
- Rays, present in softwoods, run in a radial direction perpendicular to the growth rings. Their function is to store food and allow the convection of liquids to where they are needed.
- E.g. Fir, Larch, Spruce, Hemlock, Pine.

Softwood characteristics

- Quick growth rate; trees can be felled after 30 years, resulting in low density timber with relatively low strength.
- Generally poor durability qualities, unless treated with preservatives.
- Due to speed of felling, they are readily available and comparatively cheap.

Hardwoods

• Hardwoods are generally broad-leaved (deciduous) trees that lose their leaves at the end of each growing season.

- The cell structure of hardwoods is more complex than that of softwoods, with thick walled cells, called **fibres**, providing the structural support and thin walled cells, called **vessels**, providing the medium for food conduction.
- E.g. Ash, Elm, Oak, Teak, Iroko, Ekki, Greenheart.

Hardwood characteristics

- Hardwoods grow at a slower rate than softwoods. This generally results in a timber of high density and strength which takes time to mature over 100 years in some instances.
- There is less dependency on preservatives for durability qualities.
- Due to time taken to mature and the transportation costs of hardwoods, as most are tropical, they tend to be expensive in comparison to softwoods.

Physical properties of timber

Because of the variability of timber its strength is influenced by many factors, among which the following are most important.

Moisture content

- The structural behaviour of timber is greatly influenced by moisture content unlike most other structural materials.
- The moisture content as determined by oven drying of test piece is defined as: $w = 100 (m_1 - m_2)/m_2$ (1)

where m_1 is the mass of the test piece before drying (g) m_2 is the mass of the test piece after drying (g)

- Moisture contained in green timber is held both within the cells (free water) and within the cell walls (bound water). The condition where all the free water has been removed but the cell walls are still saturated is known as the Fibre Saturation Point (FSP).
- At levels of moisture above the FSP, most physical and mechanical properties remain constant.
- While variations of moisture content below the FSP cause considerable changes to properties such as weight, strength, elasticity, shrinkage and durability.
- Thus, timber is described as being hygroscopic meaning that it attempts to attain an equilibrium moisture content with its surrounding environment. Fig. 3 shows the relationship between timber strength/stiffness and its moisture content.
- It is for this reason that timber is dried in a controlled environment in a process known as seasoning. Two methods are generally used; Air seasoning and kiln drying.

Density

- Density is the best single indicator of the properties of a timber and is a major factor determining its strength.
- Specific gravity or relative density is a measure of timber's solid substance. It is generally expressed as the ratio of the oven-dry weight to the weight of an equal volume of water.

- Since water volume varies with the moisture content of the timber, the specific gravity of timber is expressed at a certain moisture content.
- specific gravity of commercial timber ranges from 0.29 to 0.8 1, most falling between 0.35 and 0.60.



Fig. 3 General relationship between strength and/or stiffness and moisture content.

Slope of grain

- Grain is the longitudinal direction of the main elements of timber, these main elements being fibres or tracheids, and vessels in the case of hardwoods.
- In many instances the angle of the grain in a cut section of timber is not parallel to the longitudinal axis.
- It is possible that this variation is due to poor cutting of the timber, but more often than not the deviation in grain angle is due to irregular growth of the tree.
- This effect is of lesser consequence when timber is axially loaded, but leads to a significant drop in bending resistance.

Direction of grain

- Anisotrophy is a characteristic of timber because of the long fibrous nature of the cells and their orientation and the variations from early to late wood and the differences between sapwood and heartwood.
- The elastic modulus of a fibre in a direction along its axis is considerably greater than that across it, resulting in the strength and elasticity of timber parallel to the grain being much higher than in the radial and tangential directions.

Timber defects

Defects in timber, whether natural or caused during conversion or seasoning, will have an effect on structural strength as well as on fixing, stability, durability and finished appearance of timber.

Structural Timber Design to BS 5268: Part 2: 1996

- The strength of timber is a function of several parameters including the moisture content, density, duration of the applied load, size of members and presence of various strength-reducing characteristics such as slope of grain, knots, fissures and wane.
- To overcome this difficulty, the stress grading method of strength classification has been devised.
- Guidance on the use of timber in building and civil engineering structures is given in BS 5268: Structural use of timber.
- This has now been replaced by Eurocode 5 as of 1st of April 2010.

Design Philosophy

- The structural design of timber members is related to Part 2 of BS 5268, and is based on **permissible stress** design philosophy in which design stresses are derived on a statistical basis and deformations are also limited.
- Elastic theory is used to analyse structures under various loading conditions to give the worst design case. Then timber sections are chosen so that the permissible stresses are not exceeded at any point of the structure.
- Permissible stresses are calculated by multiplying the 'grade stresses', given in Tables 7 to 12a of BS 5268 : Part 2, by the appropriate modification factors, *K-factors*, to allow for the effects of parameters such as load duration, moisture content, load sharing, section size, etc.
- Applied stresses which are derived from the service loads should be less than or equal to the permissible stresses.
- A summary of the K-factors used for the calculation of permissible stresses is given in Table 1.
- The **limit states** design philosophy of Eurocode 5 has two basic requirements. The first is **ultimate limit states** (i.e. **safety**) which is usually expressed in terms of load-carrying capacity and is achieved by factoring-up of load values and factoring-down of material strength properties by partial safety factors.
- The second is **serviceability limit states** (i.e. **deformation and vibration limits**) which refers to the ability of a structural system and its elements to perform satisfactorily in normal use.
- In **permissible stress** design philosophy partial safety factors (**i.e. modification factors**) are **applied only to material properties**, i.e. for the calculation of **permissible stresses**, and **not to the loading**!!!

Stress grading of timber

- Once timber has been seasoned it is stress graded; this grading will determine the strength class of the timber to satisfy the design requirements of **BS** 5268 : Part 2.
- Strength grading takes into account defects within the timber such as slope of grain, existence and extent of knots and fissures, etc.
- All timber used for structural work needs to be strength graded by either visual inspection or by an approved strength grading machine. Clause 2.5 of BS 5268 : Part 2 deals with strength grading of timber.

K-factor	Description or application	BS 5268 : Part 2 : 1996
K2	Timber grade stresses and moduli for service class 3	Table 13
K ₃	Duration of loading	Table 14
K_4	Bearing stress	Table 15
K5	Shear at notched ends	Clause 2.10.4
K ₆	Form factor: bending stress for non-rectangular sections	Clause 2.10.5
K7	Depth factor: bending stress for beams other than 300 mm deep	Clause 2.10.6
K ₈	Load sharing systems	Clause 2.9
K9	To modify E_{\min} for deflection in trimmer beams and lintels	Table 17
K 12	Slenderness in compression members	Table 19/Annex B
K ₁₃	Effective length of spaced columns	Table 20
K ₁₄	Width factor for tension members	Clause 2.12.2
K ₁₅₋₂₀	Single grade glued laminated members and horizontally laminated beams	Table 21
K ₂₇₋₂₉	Vertically glued laminated members	Table 22
K ₃₀₋₃₂	Individually designed glued end joints in horizontally glued laminated members	Table 23
K ₃₃₋₃₄	Curved glued laminated beams	Clause 3.5.3
K ₃₅	Stress factor in pitched cambered softwood beams	Clause 3.5.4.2
K ₃₆	Plywood grade stresses for duration of loading and service classes	Table 33
K ₃₇	Stress concentration factor for ply-webbed beams	Clause 4.6
K ₃₈₋₄₁	For tempered hardboards	Section 5
Kser	Fastener slip moduli	Table 52
K43-50	Nailed joints	Clause 6.4
K52-54	Screwed joints	Clause 6.5
K56, 57	Bolted and dowelled joints	Clause 6.6
K _{S,C,58-61}	Toothed-plate connector joints	Clause 6.7
KS,C,D,62-65	Split-ring connector joints	Clause 6.8
KS,C,D,66-69	Shear-plate connector joints	Clause 6.9
K ₇₀	Glued joints	Clause 6.10

Table 1 Summary of K-factors used for calculation of permissible stresses

Visual grading

- Visual grading is a manual process carried out by an approved grader. The grader examines each piece of timber to check the size and frequency of specific physical characteristics or defects, e.g. knots, slope of grains, rate of growth, wane, resin pockets and distortion, etc.
- The required specifications are given in **BS** 4978 and BS 5756 to determine if a piece of timber is accepted in to one of the two visual stress grades or rejected.
- These are General Structural (GS) and Special Structural (SS)grades. Table 2 of **BS** 5268 : Part 2 (reproduced here as Table 2) refers to main softwood combinations of species visually graded in accordance with **BS** 4978.

Timber species Strength classes C14 C16 C18 C22 C24 C27 C30 Imported: Parana pine GS SS Caribbean pitch pine GS SS Redwood GS SS Whitewood GS SS Western red cedar GS SS Douglas fir-larch (Canada and USA) GS SS Hem-fir (Canada and USA) GS SS Spruce-pine-fir (Canada and USA) GS SS Sitka spruce (Canada) GS SS Western white woods (USA) GS SS Southern pine (USA) GS SS British grown: Douglas fir GS SS Larch GS SS British pine SS GS British spruce GS SS

Table 2 Softwood combinations of species and visual grades which satisfy the requirements for various strength classes. Timber graded in accordance with BS4978 (Table 2, BS 5268 : Part 2)

Machine grading

- Machine grading of timber sections is carried out on the principle that strength is related to stiffness.
- The machine exerts pressure and bending is induced at increments along timber length. The resulting deflection is then automatically measured and compared with preprogrammed criteria, which leads to the grading of timber section.
- In general less material is rejected if it is machine graded, however timber is also visually inspected during machine grading to ensure major defects do not exist.

Strength classes

- There are a total of 16 strength classes, C14 to C40 for softwoods and D30 to D70 for hardwoods as given in Table 7 of BS 5268: Part 2: 1996 (reproduced here as Table 3).
- The number in each strength class refers to its 'characteristic bending strength' value, for example, C40 timber has a characteristic bending strength of 40N/mm².
- It is to be noted that characteristic strength values are considerably larger than the grade stress values used in BS 5268: Part 2, as they do not include effects of long-term loading and safety factors.

Table 3 Grade stresses and moduli of elasticity for various strength classes: for service classes 1 and 2 (Table 7, BS 5268: Part 2)

ATBU Bauchi CE321- Structural Design in Timber Engr. Dr. AG. Mohammed

Structural Timber Design to BS 5268: Part 2: 1996

Strength	Bending	Tension #	Compression	Comp	ression	Shear //	Modulus of	elasticity	Densi	ty ^e
class	\parallel to grain, $\sigma_{m.g.\parallel}$ (N/mm ²)	to grain, $\sigma_{t,g,l}$ (N/mm^2)	$\sigma_{c,g,\sharp}$ (N/mm ²)	$\pm \cos \theta$ σ_{c_s} (N/n)	grain, ^{g,⊥} nm ²)	$\tau_{g,\parallel}$ (N/mm ²)	E_{mean} (N/mm ²)	E_{\min}	$\frac{\rho_k}{(kg/m^3)}$	$\rho_{\rm mean}$
C14	4.1	2.5	5.2	2.1	1.6	0.60	6 800	4 600	290	350
C16	53	3.2	6.8	2.2	1.7	0.67	8 800	5 800	310	370
C18	5.8	3.5	7.1	2.2	1.7	0.67	9100	6 000	320	380
C22	6.8	4.1	7.5	2.3	1.7	0.71	9 700	6 500	340	410
C24	7.5	4.5	7.9	2.4	1.9	0.71	10800	7 200	350	420
TR26d	10.0	6.0	8.2	2.5	2.0	1.10	11 000	7 400	370	450
C27	10.0	6.0	8.2	2.5	2.0	1.10	12 300	8 200	370	450
C30	11.0	6.6	8.6	2.7	2.2	1.20	12 300	8 200	380	460
C35	12.0	7.2	8.7	2.9	2.4	1.30	13 400	9 000	400	480
C40	13.0	7.8	8.7	3.0	2.6	1.40	14 500	10 000	420	500
D30	9.0	5.4	8.1	2.8	2.2	1.40	9 500	6 000	530	640
D35	11.0	6.6	8.6	3.4	2.6	1.70	10 000	6 500	560	670
D40	12.5	7.5	12.6	3.9	3.0	2.00	10800	7 500	590	700
D50	16.0	9.6	15.2	4.5	3.5	2.20	15000	12 600	650	780
D60	18.0	10.8	18.0	5.2	4.0	2.40	18 500	15600	700	840
D70	23.0	13.8	23.0	6.0	4.6	2.60	21 000	18 000	900	1080

^a Strength classes C14 to C40 and TR26 are for softwoods, D30 to D70 are for hardwoods

^b If wane is allowed, the lower value should be used

^cCharacteristic density values are used only when designing joints. For calculation of dead load, the average density should be used

^d The strength class TR26 is essentially for the manufacture of trussed rafters but may be used for other applications with the grade stresses and moduli given above. For joints, the tabulated permissible loads for strength class C27 should be used. When used with the provisos given in BS 5268 : Part 3 the grade stresses are similar to the former M75 redwood/whitwood so timber and trussed rafter designs to this M75 grade/species combination are interchangeable with timber and trussed rafter designs using the TR26 strength class.

Design considerations (factors affecting timber strength)

I. Loading

For the purpose of design, loading should be in accordance with **BS** 6399 : Parts 1, 2, and 3 and CP 3: Chapter V : Part 2 or other relevant standards, where applicable.

II. Service classes

Due to the effects of moisture content on mechanical properties of timber, the permissible property values should be those corresponding to one of the three service classes described in Clause 1.6.4 and given in Table 1 of BS 5268 : Part 2 : 1996. These are summarised below:

(1) *Service class I* refers to timber used internally in a continuously heated building. The average moisture content likely to be attained in service condition is 12%.

(2) *Service class* 2 refers to timber used in a covered building. The average moisture content likely to be attained in service condition if building is generally heated is 15%, and if unheated, 18%.

(3) *Service class* 3 refers to timber used externally and fully exposed. The average moisture content likely to be attained in service condition is over 20%.

Grade stress and elastic moduli values given in Tables 7 to 12a of **BS** 5268 : Part 2 apply to various strength classes and timber species in service classes 1 and 2. For service class 3 condition they should be multiplied by the modification factor K_2 from Table 13 of the code (reproduced here as Table 4).

Table 4 Modification factor K_2 for obtaining stresses and moduli applicable to service class 3 (Table 13, BS 5268 : Part 2)

Property	<i>K</i> ₂
Bending parallel to grain	0.8
Tension parallel to grain	0.8
Compression parallel to grain	0.6
Compression perpendicular to grain	0.6
Shear parallel to grain	0.9
Mean and minimum modulus of elasticity	0.8

III. Moisture content

Moisture content affects the structural properties of timber significantly, BS 5268 : Part 2 : 1996 recommends that in order to reduce movement and creep under load the moisture content of timber and wood-based panels when installed should be close to that likely to be attained in service.

IV. Duration of loading

- Duration of load affects timber strength and therefore the permissible stresses.
- The grade stresses (Tables 7 to 12a) and the joint strengths given in BS 5268 :Part 2 are applicable to long-term loading.
- Because timber and wood-based materials can sustain a much greater load for a short period (a few minutes) than for a long period (several years), the grade stresses and the joint loads may be increased for other conditions of loading by the modification factors given in the appropriate sections of **BS** 5268 : Part **2**.
- Table 14 of **BS** 5268: Part 2 (reproduced here as Table 5) gives the modification factor **K**₃ by which all grade stresses (excluding moduli of elasticity and shear moduli) should be multiplied for various durations of loading.

Duration of loading	
Long-term: i.e. dead + permanent imposed ^a	1.00
Medium-term: i.e. dead + temporary imposed + snow	1.25
Short-term: i.e. dead + imposed + wind, ^b dead + imposed + snow + wind ^b	1.50
Very short-term: i.e. dead + imposed + wind (gust) ^c	1.75

Table 5 Modification factor K₃ for duration of loading (Table 14, BS 5268 : Part 2)

^a For uniformly distributed imposed floor loads $K_3 = 1$ except for type 2 and type 3 buildings (see Table 5 of BS 6399: Part 1:1984²) where, for corridors, hallways, landings and stairways only, K_3 may be assumed to be 1.5.

^b For wind, short-term category applies to class C (15 s gust) as defined in CP 3: Chapter V: Part 2^3 or, where the largest diagonal dimension of the loaded area *a*, as defined in BS 6399: Part 2,² exceeds 50 m.

^c For wind, very short-term category applies to classes A and B (3 s or 5 s gust) as defined in CP 3: Chapter V: Part 2^3 or, where the largest diagonal dimension of the loaded area *a*, as defined in BS 6399: Part 2,² does not exceeds 50 m.

V. Section size

- The bending, tension and compression and moduli of elasticity given in Part 2 of BS 5268 are applicable to materials 300 mm deep (or wide, for tension).
- These properties of timber are dependent on section size and size related grade effects.
- The grade stresses should be modified for section sizes other than 300 mm deep by the modification factor K_7 specified in the appropriate sections of the code.

VI. Load sharing systems

- The grade stresses given in Part 2 of **BS** 5268 are applicable to individual pieces of structural timber.
- Where a number of pieces of timber (in general four or more) at a maximum spacing of 610 mm centre to centre act together to support a common load, then the grade stresses can be modified (increased) in accordance with the appropriate sections of the code.
- In a load-sharing system such as rafters, joists, trusses or wall studs spaced at a maximum of 610 mm centre to centre, and which has adequate provision for the lateral distribution of loads by means of purlins, binders, boarding, battens, etc., the appropriate grade stresses can be multiplied by the load-sharing modification factor **K**₈ which has a value of 1.1.
- In addition, **BS** 5268 :Part 2 recommends that the mean modulus of elasticity should be used to calculate deflections and displacements induced by static loading conditions.
- Therefore in a load-sharing system:

Modification factor $K_8 = 1.1$ Modulus of elasticity $E = E_{mean}$

Design of Flexural Members

Flexural members are those that are under the action of bending forces. There are various types of flexural timber members and they include; solid rectangular beams, floor joists, rafters and purlins, glulam (vertical and horizontal glued laminated beams), ply-web beams (I- beams and box beams) and simple composites (Tee and I shaped beams).

Design Considerations

The main design considerations for flexural members are:

- 1. Bending stress and prevention of lateral buckling
- 2. Deflection
- 3. Shear stress
- 4. Bearing stress.

In the design, the cross-sectional properties of all flexural members have to satisfy elastic strength and service load requirements. In general, bending is the most critical criterion for medium span beams, deflection for long-span beams and shear for heavily short-span beams. In practice, design checks are carried out for all the criteria enumerated above.

The design is carried out according to the permissible stress design philosophy by calculating the permissible stress value as the product of the grade stress and the appropriate modification factors for particular service and loading conditions. This is then compared with the applied stress in a member or a component in structural design calculations. Generally;

Permissible stress (= grade stress x K- factors) \geq applied stress

Bending stress and prevention of lateral buckling

The design of timber beams in flexure requires the application of the elastic theory of bending as expressed by:

$$\sigma = \frac{M \cdot y}{I} \tag{2.1}$$

The term I/y is referred to as section modulus and is denoted by Z. The applied bending stress about the major (**x-x**) axis of the beam (say) (see Figure 2.1), is calculated from:

$$\sigma_{m,a,\parallel} = \frac{M \cdot y}{I_{xx}} = \frac{M}{Z_{xx}}$$
(2.2)



Figure. 2.1 Cross-section of a rectangular beam,

where:

σ_{m,a,∥} = applied bending stress (in N/mm²)
 M = maximum bending moment (in Nmm)
 Z_{xx} = section modulus about its major (x-x) axis (in mm³). For rectangular sections

$$Z_{xx} = \frac{I_{xx}}{y} = \frac{\frac{bh^3}{12}}{\frac{h}{2}} = \frac{bh^2}{6}$$

(2.3)

 I_{xx} = second moment of area about x-x axis (in mm⁴)

y = distance from the neutral-axis of the section to the extreme fibres (in mm)

h =depth of the section (in mm)

b = width of the section (in mm).

The permissible bending stress $\sigma_{m,adm,\parallel}$ is calculated as the product of grade bending stress parallel to grain $\sigma_{m,g,\parallel}$ and any relevant modification factors (K-factors). These are K_2 for wet exposure condition (if applicable), K_3 for load-duration, K_6 for solid timber members other than rectangular sections (if applicable), K_7 for solid timber members other than 300 mm deep, and K_8 for load-sharing systems (if applicable). Hence:

$$\sigma_{m,adm,\parallel} = \sigma_{m,g,\parallel} \times K_2 \times K_3 \times K_6 \times K_7 \times K_8$$
(2.4)

 K_2 , K_3 and K_8 are general modification factors as described previously. K_6 and K_7 specifically relate to the calculation of permissible bending, $\sigma_{m,adm}$, \parallel and are described in the following sections.

Effective Span Le

Clause 2.10.3 of BS 5268:Part 2 recommends that the span of flexural members should be taken as the distance between the centres of bearings.



Figure 2.2.- Effective Span (Baird and Ozelton)

Where members extend over bearings, which are longer than is necessary, the spans may be measured between the centres of bearings of a length which should be adequate in accordance with Part 2 of the code (see Figure 2.2).

In determining the effective span, *L*e, it is usually acceptable to assume an addition of 50mm to the clear span, between the supports, for solid timber beam and joists and 100mm for built up beams on spans up to around 12m.

Form Factor, K₆

Grade bending stress values given in the code apply to solid timber members of rectangular crosssection. For shapes other than rectangular (see Figure 2.3) the grade bearing stress value should be multiplied by the modification factor K_6 where:

 $K_6 = 1.18$ for solid circular sections, and

 $K_6 = 1.41$ for solid square sections loaded diagonally (see Figure 2.3)

Depth Factor, K7

The grade bending stresses given in Tables 7-12a of BS 5268: Part 2 apply to beams having a depth, h, of 300mm (Clause 2.10.6) For other depths of beams, the grade bending stress should be multiplied by the depth modification factor, K_7 , where:



Figure 2.3- Form Factor, K6

Selection of a suitable section size

There are two methods commonly used in selecting an appropriate trial section:

- 1. By utilising engineering judgement based on experience.
- 2. Using the permissible bending stress criterion in equation 2.2. Thus the expression for calculation of the required section modulus Z_{xx} for timber members, incorporating all the relevant factors, is as follows:

$$Z_{xx,required} = \frac{M}{\sigma_{m,adm,//}} = \frac{M}{\sigma_{m,g,//} K_2 K_3 K_6 K_7 K_8}$$
(2.5)

Thus a suitable section size having a $Z_{xx} \ge Z_{xx,required}$ can be selected. The chosen section should then be checked for lateral stability, deflection, shear and bearing.

Lateral Stability

BS 5268: Part 2:1996 recommends that the depth to breadth ratio of solid and laminated rectangular beams should not exceed the values given in Table 16 (shown here as Table 2.1) of the code corresponding to the appropriate degree of lateral support. Examples of provisions of lateral support are shown in Figure 2.4.

7

Degree of lateral support	Maximum depth to breadth ratio
No lateral support	2
Ends held in position	3
Ends held in position and member held in line as by purlins or tie rods at centres not more than 30 times breadth of the member	4
Ends held in position and compression edge held in line, as by direct connection of sheathing, deck or joists	5
Ends held in position and compression edge held in line, as by direct connection of sheathing, deck or joists, together with adequate bridging or blocking spaced at intervals not exceeding 6 times the depth	6

Table 2.1. Maximum depth to breadth ratio for solid and laminated members, (Table 16, BS 5268:part 2)

Ends held in position and both edges held firmly in line



(c) Herring bone strutting (bridging)



Example 2.1

Determine the value of permissible bending stress parallel to grain $\sigma_{m,adm,//}$ and magnitude of maximum bending moment for a main beam of 50 mm × 200 mm deep Canadian Douglas fir-larch grade SS under service class 2 and short-duration loading.

Solution in Tabular form:

BS 5268 : Part 2	Description	Output
Table 2	strength classification	strength class = $C24$
Table 7	grade stress // to grain	$\sigma_{m.g./l} = 7.5 \mathrm{N/mm^2}$
Clause 1.6.4	service class 2	$K_2 = 1$
Table 14	short-duration loading	$K_3 = 1.5$
Clause 2.10.5	rectangular section	$K_6 = 1$
Clause 2.10.6	depth factor	$K_7 = \left(\frac{300}{200}\right)^{0.11} = 1.045$
Clause 2.9	no load-sharing	$K_8 = 1$

Permissible bending stress

$$\sigma_{m,adm,//} = \sigma_{m,g,//} K_2 K_3 K_6 K_7 K_8 = 7.5 \times 1.0 \times 1.5 \times 1.0 \times 1.045 \times 1.0$$

$$\sigma_{m,adm,//} = 11.75 \text{ N/mm}^2$$

Allowable maximum bending moment is obtained by rearranging equation 2.5

$$M = Z_{xx, provided} \times \sigma_{m, adm, ||} = \frac{bh^2}{6} \times \sigma_{m, adm, ||} = \frac{50 \times 200^2}{6} \times 11.75$$
$$M = 3.91 \text{ kNm}$$

Deflection

BS 5268: Part, Clause 2.10.7 recommends that "the dimensions of flexural members should be such as to restrict deflection within limits appropriate to the type of structure, having regards to the possibility of damage to surfacing ,materials, ceilings, partitions and to the functional needs as wells as aesthetic requirements.

Deflection limits

In most cases, including domestic flooring, the combined deflection due to bending, Δ_m , and shear, Δ_s , should not exceed 0.003 of the span to satisfy this recommendation. In addition, for domestic floor joists, the deflection under full load should not exceed the lesser of 0.003 times the span or 14 mm. This is to avoid undue vibration under moving or impact loading.

In general $\Delta_{total} = (\Delta_m + \Delta_s) \le (0.003 \times \text{span})$

and for domestic joists $\Delta_{total} \leq \text{lesser of } (0.003 \times \text{span or } 14 \text{ mm})$

Bending Deflection

The maximum bending deflection induced by the two most common load cases is given below:

(1) For a simply supported beam carrying a uniformly distributed load of W_{total}



(2) For a simply supported beam carrying a concentrated load at mid-span of P



(2.7)

where:

 $\Delta_m = \frac{PL^3}{48EI}$

 $W_{total} =$ total uniformly distributed load

- P =concentrated load acting at mid-span
- L = effective span
- I = second moment of area about axis of bending, usually beam's major (x-x) axis
- $E = E_{min}$ for a beam acting on its own
 - $=E_{mean}$ for a beam in a load-sharing system.

Design of Flexural Members

For a single-span simply supported beam subjected to a maximum bending moment of M_{max} , irrespective of the loading type, the maximum deflection of the beam may be estimated using:

$$\Delta_m \simeq \frac{0.104 M_{max} L^2}{EI} \tag{2.8}$$

BS 5268 :Part 2, Clause 2.10.11 recommends that for trimmer joists and lintels which comprise two or more pieces connected together in parallel and acting together to support the loads, the minimum modulus of elasticity modified by the modification factor K_9 , given in Table 17 (shown here as Table 2.2) of the code, should be used for calculations of deflections.

Table 2.2. Modification factor K_9 used to modify the minimum modulus of elasticity for trimmer joists and lintels (Table 17, BS 5268: part 2)

Number of pieces	Value of K_9			
	Softwoods	Hardwoods		
1	1.00	1.00		
2	1.14	1.06		
3	1.21	1.08		
4 or more	1.24	1.10		

Shear Deflection

Since in timber and wood based structural materials the shear modulus is considerably lower as a proportion of the modulus of elasticity, compared to other structural materials such as steel, the effect of shear deflection can be significant and should be considered in the design calculations.

The maximum shear deflection, Δ_s , induced in a single-span simply supported beam of either rectangular or square cross-section, may be determined from the following equation:

$$\Delta_s = \frac{19.2M_{max}}{AE} \tag{2.9}$$

where A is the cross-sectional area of the beam, M_{max} is the maximum bending moment in the beam and E is as defined above.

Bearing Stress

The bearing stresses in timber beams are developed due to compressive forces applied in a direction perpendicular to the grain and occur in positions such as points of support or applied concentrated loads. Possible bearing failure positions are shown in Figure 2.5.

Design of Flexural Members

The applied bearing stress, $\sigma_{c,a,\perp}$ is calculated from the following equation:

$$\sigma_{c,a,\perp} = \frac{F}{A_{bearing}} \tag{2.10}$$

where:

F = bearing force (usually maximum reaction or concentrated load) $A_{bearing} =$ bearing area (= bearing length × breadth of the section).

In general, the value of applied bearing stress, $\sigma_{c,a,\perp}$ should not exceed the permissible bearing stress, $\sigma_{c,adm,\perp}$ determined from:

$$\sigma_{c,adm,\perp} = \sigma_{c,g,\perp} \times K_2 \times K_3 \times K_4 \times K_8 \tag{2.11}$$

 K_2, K_3 , and K_8 are general modification factors, which have been described previously. K_4 relates to the calculation of permissible bearing stress, $\sigma_{c,adm,\perp}$, and is described below.



Figure 2.5- Possible bearing failures.

Length and Position of Bearings

BS 5268: Part 2, clause 2.10.2 stipulates that the grade stresses for compression normal to the grain apply to bearings of any length at the ends of a member, and bearings 150mm or more in length at any position. For, bearing less than 150mm long located 75mm or more from the end of a member, as shown if Figure 2.6, the grade stresses should be multiplied by the modification factor K_4 given in table 15 of the code (shown here as Table 2.3).



Figure 2.6- Bearing length and position.

Table 2.3.	Modification	factor K4 fo	r bearing stress	(Table 15	. BS	5268: Part	2)
1 4010 2.01	1.10 anite actor	Inclusion III 10	i ocanig birebb	(14010 10	$, - \sim$	02001 I uit	-/

Length of bearing (mm)	<i>K</i> ₄	
10	1.74	
15	1.67	
25	1.53	
40	1.33	
50	1.20	
75	1.14	
100	1.10	
150 or more	1.00	

Shear Stress

The critical position for shear stress is usually at supports where maximum the reaction occurs. The applied shear stress, τ , is calculated as the maximum (not average) shear stress from the following equations:

$$\tau = \frac{F_v \cdot Q}{I \cdot b} \tag{2.12}$$

For a rectangular timber beam, the maximum applied shear stress parallel to grain, $\tau_{a,//}$ occurs at the neutral axis and is calculated from:

$$\tau_{a,//} = 1.5 \, \frac{F_v}{A} \tag{2.13}$$

where:

- F_v = maximum vertical shear force (usually maximum reaction)
- A = cross-sectional area
- Q = first moment of area about neutral axis above the position where shear stress is required
- I = second moment of area
- b = breadth of the section at the position where shear stress is required.

In general, the value of applied shear stress, $\tau_{a,//}$, should not exceed the permissible shear stress parallel to grain, $\tau_{adm,//}$, determined from:

$$\tau_{adm, \parallel} = \tau_{g, \parallel} \times K_2 \times K_3 \times K_5 \times K_8 \tag{2.14}$$

 K_2, K_3 , and K_8 are general modification factors, which have been described previously. K_5 relates to the calculation of the permissible shear stress, T_{adm} , # for members with notched ends and is described below.

Shear at Notched Ends

 K_5 is a modification factor which allows for stress concentration induced at square-cornered notched at the ends of flexural member (Clause 2.10.4, BS 5368: Part 2) where:

1. For a notch on the underside of a beam, see Figure 2.7(a)

$$K_5 = \frac{h_{\rm e}}{h}$$

2. For a notch on the top edge, see Figure 2.7(b),



$$K_5 = \frac{h(h_e - a) + ah_e}{h_e^2} \quad \text{for } a \le h_e$$
$$= 1.0 \qquad \qquad \text{for } a > h_e$$

Figure 2.7 Notched Beams

Suspended Timber Floors

A suspended timber flooring system generally consists of a series of joists closely spaced, being rather simply supported at their ends or continuous over load bearing partition walls. The floor boarding or decking is applied on the top of the joists and underneath ceiling linings are fixed. A typical suspended floor arrangement is shown in Figure 2.8(a).

Joist spacing is normally dependent on the size of the decking and ceiling boards. In addition, the choice of joist spacing may also be affected by the spanning capacity of the flooring material, joist span and other geometrical constraints such as opening for a stairwell.

The most common floor decking in domestic dwellings and timber framed buildings uses some form of wood based panel products, e.g. chipboard or plywood. Solid timber decking such as softwood tongued and grooved (t &g) decking (Figure 2.8(b)) is often used in roof constructions, in conjunction with glu-lam members, to produce a pleasant, natural timber ceiling with clear spans between the main structural members.

The supports for joists are provided in various forms depending on the type of construction. Timber wall plates are normally used to support joists on top of masonry walls and foundations, Figure 2.8 (e). In situations where joists are to be supported on load bearing timber frame walls or internal partitions, header beams or spreader beams are provided to evenly distribute the vertical loads. Joist hangers are often used to attach and support joists onto the main timber beams, trimmer members or masonry walls (Figure 2.8 (c)).

Timber trimmer joist are frequently used within timber floors of all types in domestic buildings, see Figure 2.8(a). They are mainly used for two reasons, first is to trim around an opening such as s stairwell or loft access (trimmer joists A), and to support incoming joists (trimmer joists B), and second is to reduce the span of floor joists over long open spans (trimmer joists C) as indicated in Figure 2.8 (a). Trimmers can be formed by using two or more joists nailed together to form a trimmer beam, Figure 2.8 (c), or by using a single but larger timber section if construction geometry permits. Alternatively, trimmers can be of hardwood or glu-lam timber, boxed ply-webbed beams or composite timber and steel fitched beams, Figure 2.8 (d).



Figure 2.8- Typical components of suspended timber floors.

Example 2.2 Design of a main beam

A main beam of 3m length spans over an opening 2.8m wide Figure 2.9 and supports a flooring system which exerts a long duration loading of 3.9 kN/m, including its own self-weight, over its span. The beam is supported by 50mm wide walls on either side. Carry out design checks to show that a 75mm x 225mm deep sawn section whitewood grade SS under service class 1 is suitable.



Figure 2.9- Beam details of Example 2.2

Definitions

Force, kN	N := newton
Length, m	$kN := 10^3 \cdot N$
Cross-sectional dimensions, mm	Direction parallel to grain, //
Stress, Nmm ⁻²	Direction perpendicular to grain, pp

1. Geometrical properties

$L := 2.8 \cdot m$
$bw := 50 \cdot mm$
$L_e := L + bw$
$L_e = 2.85 \circ \mathrm{m}$
$b := 75 \cdot \mathrm{mm}$
$h := 225 \cdot \mathrm{mm}$
$A := b \cdot h$
$A = 16875 \circ \mathrm{mm}^2$
$I_{xx} := \frac{1}{12} \cdot b \cdot h^3$
$I_{xx}=7.12\times10^7\circ\mathrm{mm}^4$

Applied uniformly distributed load, w Total load, W $w := 3.9 \cdot kN \cdot m^{-1}$ $W := w \cdot L_e$ $W = 11.11 \circ kN$

3. K-factors

Service class 1 (K_2 , Table 13) Load duration (K_3 , Table 14) Bearing: 50 mm, but located < 75 mm from end of member (K_4 , Table 15) Notched end effect (K_5 , Clause 2.10.4) Form factor (K_6 , Clause 2.10.5)

Depth factor (K7, Clause 2.10.6)

No load sharing (K_8 , Clause 2.9)

4. Grade stresses

BS 5268 : Part 2, Tables 2 and 7Whitewood grade SSStrength class = C24Bending parallel to grain $\sigma_{m\cdot g\cdot//} := 7.5 \cdot N \cdot mm^{-2}$ Compression perpendicular to grain $\sigma_{c\cdot g \cdot pp} := 2.4 \cdot N \cdot mm^{-2}$ Shear parallel to grain $\tau_{g\cdot//} := 0.71 \cdot N \cdot mm^{-2}$ Minimum modulus of elasticity $E_{min} := 7200 \cdot N \cdot mm^{-2}$

5. Bending stress

Applied bending moment

Section modulus

Applied bending stress

Permissible bending stress

$$K_{2} := 1$$

$$K_{3} := 1 \quad \text{for long-term}$$

$$K_{4} := 1$$

$$K_{5} := 1 \quad \text{for no notch}$$

$$K_{6} := 1$$

$$K_{7} := \left(\frac{300 \cdot \text{mm}}{h}\right)^{0.11}$$

$$K_{7} = 1.03$$

$$K_{8} := 1$$

$$M := \frac{W \cdot L_e}{8}$$

$$M = 3.96 \circ \text{kN} \cdot \text{m}$$

$$Z_{provided} := \frac{b \cdot h^2}{6}$$

$$Z_{provided} = 632\,812.5 \circ \text{mm}^3$$

$$\sigma_{m.a.//} := \frac{M}{Z_{provided}}$$

$$\sigma_{m.a.//} = 6.26 \circ \text{N} \cdot \text{mm}^{-2}$$

 $\sigma_{m.adm./\!/} := \sigma_{m.g./\!/} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_8$ $\sigma_{m.adm./\!/} = 7.74 \circ N \cdot mm^{-2}$ Bending stress satisfactory

6. Lateral stability

BS 5268 : Part 2, Clause 2.10.8 and Table 16

Maximum depth to breadth ratio, h/b

 $\frac{h}{b} = 3$

 $F_{v} := \frac{W}{2}$

 $F_v = 5.56 \circ \mathrm{kN}$

 $\tau_a := \frac{3}{2} \cdot \left(\frac{F_v}{b \cdot h}\right)$

 $\tau_a = 0.49 \circ \mathrm{N} \cdot \mathrm{mm}^{-2}$

 $\tau_{adm.//} := \tau_{g.//} \cdot K_2 \cdot K_3 \cdot K_5 \cdot K_8$ $\tau_{adm.//} = 0.71 \circ N \cdot mm^{-2}$ Shear stress satisfactory

Ends should be held in position

7. Shear stress

Applied shear force

Applied shear stress

Permissible shear stress, no notch

8. Bearing stress

Applied load

Applied bearing stress

Permissible shearing stress

9. Deflection

No load sharing

Deflection due to bending

Deflection due to shear

Total deflection

$$F_{v} := \frac{W}{2}$$

$$F_{v} = 5.56 \circ \text{kN}$$

$$\sigma_{c.a.pp} := \left(\frac{F_{v}}{b \cdot bw}\right)$$

$$\sigma_{c.a.pp} = 1.48 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\sigma_{c.adm.pp} := \sigma_{c.g.pp} \cdot K_{2} \cdot K_{3} \cdot K_{4} \cdot K_{8}$$

$$\sigma_{c.adm.pp} = 2.4 \circ \text{N} \cdot \text{mm}^{-2}$$
Bearing stress satisfactory

$$E := E_{min}$$

$$\Delta_m := \frac{5 \cdot W \cdot L_e^3}{384 \cdot E \cdot I_{xx}}$$

$$\Delta_m = 6.54 \circ mm$$

$$\Delta_s := \frac{19.2 \cdot M}{(b \cdot h) \cdot E}$$

$$\Delta_e = 0.63 \circ mm$$

 $\begin{array}{l} \Delta_{total} := \Delta_m + \Delta_s \\ \Delta_{total} = 7.16 \circ \mathrm{mm} \end{array}$

Permissible deflection

Design of Flexural Members

 $\Delta_{adm} := 0.003 \cdot L_e$ $\Delta_{adm} = 8.55 \circ \text{mm}$ Deflection satisfactory

Therefore a $75 \text{ mm} \times 225 \text{ mm}$ sawn section whitewood C24 is satisfactory

Example 2.3 Design of Floor Joists

A timber floor spanning 3.8m centre to centre is to be designed using timber joists at 400 mm centres. The floor is subjected to a domestic imposed load of 1.5 kN/m^2 and carries a dead loading, including self-weight of 0.35 kN/m². Carry out design checks to show that a series of 44mm x 200mm deep sawn section British spruce grade SS under service class 1 is suitable.



Figure 2.10- Timber floor joists (Example 2.3)

Definitions

Joist spacing, Js

Force, kN	N := newton		
Length, m	$kN := 10^3 \cdot N$		
Cross-sectional dimensions, mm	Direction parallel to grain, //		
Stress, Nmm ⁻²	Direction perpendicular to grain, pp		
1. Geometrical properties			
Effective span, L_e	$L_e := 3.8 \cdot \mathrm{m}$		

 $Js := 0.4 \cdot m$

Joist dimensions: Breadth of section, b $b := 44 \cdot \mathrm{mm}$ Depth of section, h $h := 200 \cdot \text{mm}$ $A := b \cdot h$ Cross-sectional area, A $A = 8.8 \times 10^3 \circ \mathrm{mm}^2$ $I_{xx} := \frac{1}{12} \cdot b \cdot h^3$ Second moment of area, I_{xx}

2. Loading

Dead load, DL Imposed load, IL Total load, W

$I_{xx} = 2.93 \times 10^7 \circ \mathrm{mm^4}$

 $DL := 0.35 \cdot kN \cdot m^{-2}$ $IL := 1.5 \cdot kN \cdot m^{-2}$ $W := (DL + IL) \cdot Js \cdot L_e$ $W = 2.81 \circ kN$

 $K_7 = 1.05$

 $K_8 := 1.1$

3. K-factors

Service class 1 (K_2 , Table 13) $K_2 := 1$ Load duration (K_3 , Table 14) $K_3 := 1$ for long-term Bearing: assume 50 mm, but located < 75 mm $K_4 := 1$ from the end of the member $(K_4, \text{ Table 15})$ Notched end effect (K_5 , Clause 2.10.4) $K_5 := 1$ for no notch Form factor (K_6 , Clause 2.10.5) $K_6 := 1$ $K_7 := \left(\frac{300 \cdot \mathrm{mm}}{h}\right)^{0.11}$

Depth factor (K7, Clause 2.10.6)

Load sharing applies (K_8 , Clause 2.9)

4. Grade stresses

BS 5268 : Part 2, Tables 2 and 7	
British spruce grade SS	Strength class $=$ C18
Bending parallel to grain	$\sigma_{m.g.//} := 5.8 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Compression perpendicular to grain	$\sigma_{c.g.pp} := 2.2 \cdot \text{N} \cdot \text{mm}^{-2}$ no wane
Shear parallel to grain	$\tau_{g.//} := 0.67 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Mean modulus of elasticity, load sharing	$E_{mean} := 9100 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$

5. Bending stress

 $M := \frac{W \cdot L_e}{8}$ Applied bending moment $M = 1.34 \circ kN \cdot m$ $Z_{provided} := \frac{b \cdot h^2}{6}$ Section modulus $Z_{provided} = 2.93 \times 10^5 \circ \text{mm}^3$

3Z

Design of Flexural Members

Applied bending stress

Permissible bending stress

$$\sigma_{m.a.//} := \frac{M}{Z_{provided}}$$

$$\sigma_{m.a.//} = 4.55 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\sigma_{m.adm.//} := \sigma_{m.g.//} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_7$$

$$\sigma_{m.adm.//} = 6.67 \circ \text{N} \cdot \text{mm}^{-2}$$
Bending stress satisfactory

6. Lateral stability

BS 5268 : Part 2, Clause 2.10.8, and Table 16

Maximum depth to breadth ratio, h/b

$$\frac{h}{b} = 4.55$$

Ends should be held in position and compression edges held in line

7. Shear stress

Applied shear force

Applied shear stress

Permissible shear stress, no notch

$$F_{v} := \frac{W}{2}$$

$$F_{v} = 1.41 \circ \text{kN}$$

$$\tau_{a} := \frac{3}{2} \cdot \left(\frac{F_{v}}{b \cdot h}\right)$$

$$\tau_{a} = 0.24 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\tau_{adm.//} := \tau_{g.//} \cdot K_{2} \cdot K_{3} \cdot K_{5} \cdot K_{8}$$

$$\tau_{adm.//} = 0.74 \circ \text{N} \cdot \text{mm}^{-2}$$
Shear stress satisfactory

8. Bearing stress

Applied load

Assume bearing widths, bw, of 50 mm either side

Applied bearing stress

Permissible bearing stress

$$F_{v} := \frac{W}{2}$$
$$F_{v} = 1.41 \circ \text{kN}$$
$$bw := 50 \cdot \text{mm}$$

$$\sigma_{c.a.pp} := \left(\frac{F_v}{b \cdot bw}\right)$$
$$\sigma_{c.a.pp} = 0.64 \circ \mathrm{N} \cdot \mathrm{mm}^{-2}$$

 $\sigma_{c.adm.pp} := \sigma_{c.g.pp} \cdot K_2 \cdot K_3 \cdot K_4 \cdot K_8$ $\sigma_{c.adm.pp} = 2.42 \circ \text{N} \cdot \text{mm}^{-2}$ Bearing stress satisfactory

9. Deflection

Load sharing system	$E := E_{mean}$
Deflection due to bending	$\Delta_m := \frac{5 \cdot W \cdot L_e^3}{384 \cdot E \cdot I_{xx}}$
	$\Delta_m = 7.53 \circ \mathrm{mm}$
Deflection due to shear	$\Delta_s := \frac{19.2 \cdot M}{(b \cdot h) \cdot E}$
	$\Delta_s = 0.32 \circ \text{mm}$
Total deflection	$\Delta_{total} := \Delta_m + \Delta_s$
	$\Delta_{total} = 7.85 \circ \text{mm}$
Permissible deflection	$\Delta_{adm}:=0.003\cdot L_e$
	$\Delta_{adm} = 11.4 \circ \text{mm}$
	Deflection satisfactory

Therefore 44 mm \times 200 mm sawn sections in C18 timber are satisfactory
Design of Armany Bouded Members

Design of compression members

Examples of compression members are posts or columns, vertical wall studs, and struts in trusses in trusses and girders. The relevant sections of BS 5268 that govern timber compression members are stipulated in Clauses 2.6.2, 2.8 and 2.9 which relate to different service class conditions, duration of loading and load sharing systems respectively. Additional factors given in Clause 2.11 of the code also govern the design of compression members and it deals principally with their design in two categories:

- 1. Members subjected to axial compression only, and
- 2. Members subjected to combined axial compression and bending (mainly due to applied eccentric compressive force).

Design considerations

The main design considerations for compression members are:

- Slenderness ratio. This relates to positional restraint of ends, lateral restraint along the length and cross-sectional dimensions of the member.
- (2) Axial compression and bending stresses.

Slenderness ratio, λ

The load-carrying capacity of compression members is a function of the slenderness ratio, λ , which is calculated as the effective length, L_e divided by the radius of gyration, *i*:

$$\lambda = \frac{L_e}{i} \tag{3.1}$$

Design of Axially Loaded Members

The radius of gyration, *i*, is given by

$$i = \sqrt{\left(\frac{I}{A}\right)}$$

Where I is the second moment of area and A is the cross-sectional area of the member. For rectangular sections, where b is the least lateral dimension (Figure 3.1) the value of i simplifies to:

$$i = \frac{b}{\sqrt{12}} \tag{3.2}$$

Clause 2.11.4 of BS 5268 : Part 2:1996 recommends that the slenderness ratio should not exceed a value of:

- $\lambda = 180$, for compression members carrying dead and imposed loads other than loads resulting from wind,
- $\lambda = 250$, for any members subject to reversal of axial stress solely from the effect of wind and any compression member carrying self-weight and wind loads only.

The effective length, L_e , of a column (given in Clause 2.11.3) should be derived from either:

- (1) The deflected form of compression member as affected by any restraint and or fixing moment(s). Then the effective length is considered as the distance between adjacent points of zero bending moment, or
- (2) Table 18 of the code for the particular end conditions at the column ends. Then the effective length is obtained by multiplying a relevant coefficient taken from this table by the actual length, L.

 $L_e = \text{coefficient} \times L$

(3.3)

The end condition defined in Table 18 of BS 5268: Part 2 are reproduced here as Table 5.1 and are shown in Figure 3.2.



Figure 3.1 Cross-section of a column

Table 3.1 Effective length of compression members (Table 18, BS 5268: Part 2)

End conditions	L_e/L
Restrained at both ends in position and in direction	0.7
Restrained at both ends in position and one end in direction	0.85
Restrained at both ends in position but not in direction	1.0
Restrained at one end in position and in direction and at the other end in direction but not in position	1.5
Restrained at one end in position and in direction and free at the other end	2.0

For compression members with slenderness ratios equal to or greater than 5, Clause 2.11.5 of the code requires that the permissible compressive stresses are further modified by K_{12} the modification factor for compression members.



Figure 3.2 Effective lengths and end conditions

Modification factor for compression members *K*₁₂

The modification factor for compression members, K_{12} , can be determined using either Table 19 of BS 5268 or calculated from equation given in Annex B of the code. For either method, the minimum value of modulus of elasticity E (i.e. E_{min}) should b used in all cases including when load sharing is present. The value of for use in either method, should be the grade compressive stress, σ_{max} (given in Tables 7-12a) of the code) modified only for moisture content, duration of loading and size as appropriate.

It is to be noted that for members comprising two or more pieces connected together in parallel and acting together to support the loads, the minimum modulus of elasticity should be modified by K_9 (see Table 17) or K_{28} (see Table 22) of the code. For horizontally laminated members, the modified mean modulus of elasticity should be used (see Clauses 3.2 and 3.6 of the code).

Table 19 of BS 5268 is shown here as Table 3.2 and the equation given in Annex B for calculation of K_{12} is in the form following form:

$$K_{12} = \left\{\frac{1}{2} + \frac{(1+\eta)\pi^2 E}{3\lambda^2 \sigma_c}\right\} - \left[\left\{\frac{1}{2} + \frac{(1+\eta)\pi^2 E}{3\lambda^2 \sigma_c}\right\}^2 - \frac{\pi^2 E}{1.5\lambda^2 \sigma_c}\right]^{1/2}$$
(3.4)

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Design of Axially Loaded Members

Table 3.2 Modification factor K_{12} for compression members (Table 19, BS 5268: Part 2)

$E/\sigma_{c,f}$	Value of K_{12}																			
	Value of slenderness ratio $\lambda(=L_e/i)$																			
	<5	5	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	250
	Equivalent L_e/b (for rectangular sections)																			
	<1.4	1.4	2.9	5.8	8.7	11.6	14.5	17.3	20.2	23.1	26.0	28.9	34.7	40.5	46.2	52.0	57.8	63.6	69.4	72.3
400	1.000	0.975	0.951	0.896	0.827	0.735	0.621	0.506	0.408	0.330	0.271	0.225	0.162	0.121	0.094	0.075	0.061	0.051	0.043	0.040
500	1.000	0.975	0.951	0.899	0.837	0.759	0.664	0.562	0.466	0.385	0.320	0.269	0.195	0.148	0.115	0.092	0.076	0.063	0.053	0.049
600	1.000	0.975	0.951	0.901	0.843	0.774	0.692	0.601	0.511	0.430	0.363	0.307	0.226	0.172	0.135	0.109	0.089	0.074	0.063	0.058
700	1.000	0.975	0.951	0.902	0.848	0.784	0.711	0.629	0.545	0.467	0.399	0.341	0.254	0.195	0.154	0.124	0.102	0.085	0.072	0.067
800	1.000	0.975	0.952	0.903	0.851	0.792	0.724	0.649	0.572	0.497	0.430	0.371	0.280	0.217	0.172	0.139	0.115	0.096	0.082	0.076
900	1.000	0.976	0.952	0.904	0.853	0.797	0.734	0.665	0.593	0.522	0.456	0.397	0.304	0.237	0.188	0.153	0.127	0.106	0.091	0.084
1000	1 000	0.076	0.052	0.004	0.855	0 801	0 742	0 677	0 600	0 542	0 478	0 420	0.325	0.255	0.204	0.167	0.138	0.116	0.099	0.092
11000	1.000	0.976	0.952	0.904	0.855	0.801	0.742	0.687	0.603	0.559	0.497	0.440	0.344	0.272	0.219	0.179	0.149	0.126	0.107	0.100
1200	1.000	0.976	0.952	0.905	0.857	0.807	0.753	0.659	0.634	0.573	0.513	0.457	0.362	0.288	0.233	0.192	0.160	0.135	0.116	0.107
1300	1 000	0.976	0.952	0.905	0.858	0.809	0.757	0.701	0.643	0.584	0.527	0.472	0.378	0.303	0.274	0.203	0.170	0.144	0.123	0.115
1400	1.000	0.976	0.952	0.906	0.859	0.811	0.760	0.707	0.651	0.595	0.539	0.486	0.392	0.317	0.259	0.214	0.180	0.153	0.131	0.122
1500	1.000	0.976	0.952	0.906	0.860	0.813	0.763	0.712	0.658	0.603	0.550	0.498	0.405	0.330	0.271	0.225	0.189	0.161	0.138	0.129
		0.076	0.050	0.000	0.001	0.014	0.766	0.716	0 664	0.611	0.550	0.509	0 417	0 342	0 282	0 235	0 198	0.169	0.145	0.135
1600	1.000	0.976	0.952	0.906	0.861	0.814	0.760	0.710	0.669	0.611	0.559	0.508	0.417	0.342	0.202	0.235	0.207	0.177	0.152	0.142
1700	1.000	0.976	0.952	0.906	0.861	0.815	0.708	0.719	0.009	0.618	0.50/	0.516	0.428	0.363	0.302	0.254	0.215	0.184	0.159	0.148
1800	1.000	0.976	0.952	0.900	0.862	0.810	0.770	0.722	0.677	0.624	0.574	0.520	0.447	0.373	0.312	0.262	0.223	0.191	0.165	0.154
2000	1.000	0.976	0.952	0.907	0.863	0.818	0.773	0.728	0.681	0.634	0.587	0.541	0.455	0.382	0.320	0.271	0.230	0.198	0.172	0.160

where:

 σ_c = permissible stress for a very short column ($\lambda < 5$), $\sigma_{c,g,//} \times K_2 K_3$ E = minimum modulus of elasticity, E_{min}

$$\lambda = \text{slenderness ratio}, \frac{L_e}{i}$$
$$n = 0.005 \lambda$$

Members subjected to axial compression only (Clause 2.11.5)

An axially loaded column has its line of action of load passing through the centroidal axis of the column as shown in Figure 3.3(a).

(1) The axial compressive stress, $\sigma_{c,a,l}$, is given by:

$$\sigma_{c,a,f} = \frac{P}{A} \tag{3.5}$$

where P is the axial compressive load and A the cross-sectional area.

(2) The permissible compressive stress, $\sigma_{c,adm,//}$, is calculated as the product of grade compressive stress parallel to grain, $\sigma_{c,g,//}$, and any relevant modification factors (K-factors). These are K_2 for wet exposure condition (if applicable), K_3 for load-duration, K_8 for load-sharing systems (if applicable) and the modification factor for compression members, K_{12} , if appropriate:

if
$$\lambda < 5$$
 $\sigma_{c,adm,//} = \sigma_{c,g,//} \times K_2 K_3 K_8$ (3.6a)

and for $\lambda \ge 5$ $\sigma_{c,adm,\parallel} = \sigma_{c,g,\parallel} \times K_2 K_3 K_8 K_{12}$ (3.6b)

The compression member is designed so that the applied compressive stress, $\sigma_{c,a,//}$, does not exceed the permissible compressive stress parallel to the grain, $\sigma_{c,adm,//}$

$$\sigma_{c,a,//} \leq \sigma_{c,adm,//}$$



Figure 3.3 Axial concentric and eccentric loads

Members subjected to axial compression and bending (Clause 2.11.6)

This includes compression members subjected to eccentric loading, where load acts through a point at a certain distance from the centroidal axis, which can be equated to an axial compression force and bending moment as shown in 3.3(b), (c) or (d). Members which are restrained at both ends in position but not direction, which covers most real situations, should be so proportioned that:

$$\frac{\sigma_{m,a,m}}{\sigma_{m,adm,m}} \left(1 - \frac{1.5\sigma_{c,a,m}}{\sigma_{e}} K_{12}\right) + \frac{\sigma_{c,a,m}}{\sigma_{c,adm,m}} \le 1$$
(3.7)

where:

$$\sigma_{m,a,//} = \text{applied bending stress} = \frac{M}{Z}$$

 $\sigma_{m,adm,//} = \text{permissible bending stress} = \sigma_{m,g,//} \times K_2 K_3 K_6 K_7 K_8$

$$\sigma_{c,a,//} = \text{applied compression stress} = \frac{P}{A}$$

 $\sigma_{c,adm,//} = \text{permissible compression stress} = \sigma_{c,g,//} \times K_2 K_3 K_8 K_{12}$
 $\sigma_e = \text{Euler critical stress} = \frac{\pi^2 EI}{(L_e/i)^2} \text{ and } E = E_{min}.$

Equation 3.7 is the interaction formular used to ensure that lateral instability does not arise in compression members subject to axial force and bending. Thus, if the column is subjected to compressive loading only, then $\sigma_{m,a,\parallel} = 0$, and hence the equation simplifies to $\sigma_{c,a,\parallel} / \sigma_{c,adm,\parallel} \leq 1$. Alternatively, if the column is subjected to bending only, i.e. $\sigma_{c,a,\parallel} = 0$, then the equation simplifies to $\sigma_{m,a,\parallel} / \sigma_{m,adm,\parallel} \leq 1$.

Design of tension members (Clause 2.12)

The permissible stresses for timber members in tension are stipulated in Clauses 2.6.2, 2.8 and 2.9 of the code, and relate to different service class conditions, duration of loading and load-sharing systems respectively. Additional factors that govern design in tension are also outlined in Clause 2.12 of the code and are discussed here.

Design considerations

The main design considerations for tension members are:

- (1) Tensile stress for members subjected to axial loading only.
- (2) Combined bending and tensile stresses for members subjected to lateral loading as well as the axial tension.

In both cases, as before, the permissible stress value is calculated as the product of the grade stress and the appropriate modification factors for a particular service and loading conditions, and is compared with the applied stress in the member, where:

Permissible stress (= grade stress \times K-factors) \geq applied stress

BS 5268: Part 2 recommends that in the calculation of the permissible tensile stress value the effect of the width of the timber section K_{14} should be considered.

Width factor, K₁₄

The grade tension stresses given in Tables 7-12a of the code apply to members having a width(i.e. the greater transverse dimension), h, of 300mm as shown in Figure 3.1. For other widths of members assigned to a strength class, the grade tension stresses should be multiplied by the width modification factor, K_{14} , where:

for $h \le 72 \text{ mm}$ $K_{14} = 1.17$ and for h > 72 mm $K_{14} = \left(\frac{300}{h}\right)^{0.11}$

Members subjected to axial tension only

An axially loaded column has its line of action of load passing through the centroidal axis of the column.

 The applied tensile stress, σ_{t,a,//}, in an axially loaded timber member is calculated from the following equation:

$$\sigma_{t,a,//} = \frac{T}{A_{net}}$$
(3.8)

where:

T = tensile force $A_{net} =$ net cross-sectional area.

Design of Axially Loaded Members

(2) The *permissible tensile stress*, $\sigma_{t,adm,//}$, is calculated as the product of the grade tensile stress, $\sigma_{t,g,//}$, and any relevant modification factors (*K*-factors) as follows:

$$\sigma_{i,adm,\#} = \sigma_{i,g,\#} \times K_2 K_3 K_8 K_{14}$$

$$(3.9)$$

In general, the value of applied tensile stress, $\sigma_{t,a,//}$, should not exceed the permissible tensile stress, $\sigma_{t,adm,//}$, hence:

 $\sigma_{t,a,/\!\!/} \leq \sigma_{t,adm,/\!\!/}$

Combined bending and tensile stresses

In members that are subjected to lateral loading as well as the axial tension, the position of maximum stress occurs at the point of maximum bending moment (Figure 3.5). Clause 2.12.3 of the code stipulates that the sum of the ratios of the applied tensile and bending stresses to those of the permissible ones must not exceed unity:

$$\frac{\sigma_{m,a,ll}}{\sigma_{m,adm,ll}} + \frac{\sigma_{t,a,ll}}{\sigma_{t,adm,ll}} \le 1$$
(3.10)

where: applied tensile stress, $\sigma_{t,a,//} = \frac{T}{A_{net}}$ permissible tensile stress, $\sigma_{t,adm,//} = \sigma_{t,g,//} \times K_2 K_3 K_8 K_{14}$ applied bending stress, $\sigma_{m,a,//} = \frac{M \cdot y}{I} = \frac{M}{Z}$ permissible bending stress, $\sigma_{m,adm,//} = \sigma_{m,g,//} \times K_2 K_3 K_6 K_7 K_8$



Figure 3.5 Tension member subjected to lateral loading.

Design Examples

Example 3.1 Load capacity of a timber column

A timber column in strength class C18 is 4m in height with a rectangular crosssection of 97mm x 145mm as shown in Figure 3.6. The column is restrained at both ends in position but not in direction and is subjected to service class 2 conditions.

- (a) Determine the maximum axial long-term load that the column can support.
- (b) Check that the column is adequate to resist a long-term axial load of 12 kN and a bending moment of 0.8 kNm about its x-x axis.



Figure 5.6 Details of column in Example 3.1

Definitions

Force, kN	N := newton
Length, m	$kN := 10^3 \cdot N$
Cross-sectional dimensions, mm	Direction parallel to grain, //
Stress, Nmm ⁻²	Direction perpendicular to grain, pp

1. Geometrical properties

BS 5268 : Part 2, Table 18	
Column length, L	$L := 4.0 \cdot m$
Effective length, L_e	$L_c := 1.0 \cdot L$
	$L_e = 4 \circ \mathrm{m}$
Width of section, b	$b := 97 \cdot \mathbf{mm}$
Depth of section, h	$h := 145 \cdot \mathrm{mm}$
Cross-sectional area, A	$A := b \cdot h$
	$A = 14065 \circ \mathrm{mm}^2$
Second moment of area, I_{xx}	$I_{xx} := \frac{1}{12} \cdot b \cdot h^3$
	$I_{xx} = 2.46 \times 10^7 \circ \text{mm}^4$

Second moment of area, Iyy

$$I_{yy} := \frac{1}{12} \cdot h \cdot b^3$$

$$I_{yy} = 1.1 \times 10^7 \circ \text{mm}^4$$

For a rectangular section

Radius of gyration, iyy

$$i_{yy} := \frac{b}{\sqrt{12}}$$

$$i_{yy} = 28 \circ \text{mm}$$

$$Z_{xx} := \frac{b \cdot h^2}{6}$$

$$Z_{xx} = 339\,904.17 \circ \text{mm}^3$$

Section modulus, Zxx

2. Check slenderness ratio, λ

$$\lambda := \frac{L_e}{i_{yy}}$$

$$\lambda = 142.85$$

<180, satisfactory

3. Grade stresses

BS 5268: : Part 2, Table 7	
Timber strength class, C18	
Bending parallel to grain	$\sigma_{m.g.//} := 5.8 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Compression parallel to grain	$\sigma_{c.g.//} := 7.1 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Minimum modulus of elasticity	$E_{min} := 6000 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$

4. K-factors

Service class 2 (K_2 , Table 13)	$K_2 := 1$
Load duration $(K_3, \text{ Table 14})$	$K_3 := 1$ for long-term
Form factor (K_6 , Clause 2.10.5)	$K_6 := 1$
Depth factor (K_7 , Clause 2.10.6)	$K_7 := \left(\frac{300 \cdot \mathrm{mm}}{h}\right)^{0.11}$
	$K_7 = 1.08$
No load sharing	$K_8 := 1$
(K ₈ , Clauses 2.9 and 2.11.5)	
a Na ang kanalan na kanalan kanalan kanalan ka	

The modification factor for compression member, K_{12} , can be calculated using either one of the following methods:

(1) Using equation in Annex B:

$$\sigma_c := \sigma_{c.g.//} \cdot K_2 \cdot K_3 \qquad \eta := 0.005 \cdot \lambda \qquad E := E_{min}$$

$$K_{12} := \frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c} - \sqrt{\left[\frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c}\right]^2 - \frac{\pi^2 \cdot E}{1.5 \cdot \lambda^2 \cdot \sigma_c}}$$

$$K_{12} = 0.22$$

(2) Using Table 19:

For
$$\frac{E_{min}}{\sigma_{c.g.//} \cdot K_2 \cdot K_3} = 845.07$$
 and $\lambda = 142.85$ thus, $K_{12} = 0.22$

Part (a)

5. Permissible compressive stress

$$\sigma_{c.adm, \#} := \sigma_{c.g.\#} \cdot K_2 \cdot K_3 \cdot K_8 \cdot K_{12}$$

$$\sigma_{c.adm, \#} = 1.55 \circ \mathrm{N} \cdot \mathrm{mm}^{-2}$$

Hence the axial long-term load capacity of the column is calculated by rearranging, $\sigma = P/A$. Thus

 $P_{capacity} := \sigma_{c.adm.//} \cdot A$ $P_{capacity} = 21.79 \circ kN$

Part (b)

6. Loading

Axial load	$P := 12 \cdot kN$			
Applied moment	$M := 0.8 \cdot kN \cdot m$			

7. Compression and bending stresses

Applied compressive stress	$\sigma_{c.a./\!\!/} := rac{P}{A}$
Permissible compressive stress	$\sigma_{c.a.//} = 0.85 \circ \text{N} \cdot \text{mm}^{-2}$ $\sigma_{c.adm.//} := \sigma_{c.g.//} \cdot K_2 \cdot K_3 \cdot K_8 \cdot K_{12}$ $\sigma_{c.adm.//} = 1.55 \circ \text{N} \cdot \text{mm}^{-2}$
Applied bending stress	$\sigma_{m.a.//} := \frac{M}{Z_{xx}}$
Permissible bending stress	$\sigma_{m.a.\#} = 2.35 \circ \text{N} \cdot \text{mm}^{-2}$ $\sigma_{m.adm.\#} := \sigma_{m.g.\#} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_8$ $\sigma_{m.adm.\#} = 6.28 \circ \text{Nmm}^{-2}$
Check interaction quantity	
Euler critical stress	$\sigma_e := rac{\pi^2 \cdot E_{min}}{\lambda^2}$
	$\sigma_e = 2.9 \circ \text{N} \cdot \text{mm}^{-2}$

$$\frac{\sigma_{m.a.//}}{\sigma_{m.a.dm.//} \cdot \left(1 - \frac{1.5 \cdot \sigma_{c.a.//}}{\sigma_e} \cdot K_{12}\right)} + \frac{\sigma_{c.a.//}}{\sigma_{c.a.dm.//}} = 0.97 < 1$$

Interaction quantity is satisfactory

Therefore a $97 \text{ mm} \times 145 \text{ mm}$ timber section in strength class C18 is satisfactory

Example 3.2 Design of an eccentrically loaded column

For the design data given below, check that a 100mm x 250mm rough sawn section, as shown in Figure 3.7, is adequate as a column if the load is applies 90mm eccentric to its x-x axis. The column is 3.75m in height and has its ends restrained in position but not in direction.

Design data:	
Timber	Whitewood grade SS
Service class	2
Design load (long-term)	25 kN
Design load (medium-term)	30 kN





Definitions

Force, kN	N := newton
Length, m	$kN := 10^3 \cdot N$
Cross-sectional dimensions, mm	Direction parallel to grain, //
Stress, Nmm ⁻²	Direction perpendicular to grain, pp

1. Geometrical properties

$L := 3.75 \cdot \mathbf{m}$ $L_e := 1.0 \cdot L$
$L_{e} := 1.0 \cdot L$
$L_e = 3.75 \circ m$
$b := 100 \cdot \text{mm}$
$h := 250 \cdot \mathrm{mm}$
$A := b \cdot h$
$A = 25000 \circ \mathrm{mm}^2$
$I_{xx} := \frac{1}{12} \cdot b \cdot h^3$
$I_{xx} = 1.3 \times 10^8 \circ \mathrm{mm}^4$
$I_{yy} := \frac{1}{12} \cdot h \cdot b^3$
$I_{yy} = 2.08 \times 10^7 \circ \mathrm{mm}^4$
$i = \frac{b}{b}$
√12
$i_{yy} = 28.87 \circ \mathrm{mm}$
$b \cdot h^2$
$Z_{xx} := \frac{6}{6}$
$Z_{xx} = 1.04 \times 10^6 \circ \mathrm{mm}^3$

2. Check slenderness ratio, λ

$$\lambda := \frac{L_e}{i_{yy}}$$

$$\lambda = 129.9$$
<180, satisfactory

3. Grade stresses

BS 5268 : Part 2, Tables 2 and 7 Strength classification Bending parallel to grain Compression parallel to grain Minimum modulus of elasticity

Whitewood grade SS = C24 $\sigma_{m.g.//} := 7.5 \cdot N \cdot mm^{-2}$ $\sigma_{c.g.//} := 7.9 \cdot N \cdot mm^{-2}$ $E_{min} := 7200 \cdot N \cdot mm^{-2}$

4. K-factors

Service class 2 (K_2 , Table 13) Load duration (K_3 , Table 14) for long- and medium-terms Form factor (K_6 , Clause 2.10.5) Depth factor (K_7 , Clause 2.10.6) No load sharing (K_8 , Clauses 2.9 & 2.11.5) $K_2 := 1$ $K_{3.long} := 1$ and $K_{3.med} := 1.25$ $K_6 := 1$ $K_7 := \left(\frac{300 \cdot \text{mm}}{h}\right)^{0.11}$ $K_7 = 1.02$ $K_8 := 1$

Modification factor for compression member, K_{12} , can be calculated using either one of the following methods:

Using equation in Annex B:

(a) Long-term loading: $\sigma_c := \sigma_{c.g./!} \cdot K_2 \cdot K_{3.long}$ $\eta := 0.005 \cdot \lambda$ $E := E_{min}$

$$K_{12.long} := \frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c} - \sqrt{\left(\frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c}\right)^2 - \frac{\pi^2 \cdot E}{1.5 \cdot \lambda^2 \cdot \sigma_c}}$$
$$K_{12.long} = 0.27$$

(b) Medium-term loading: $\sigma_c := \sigma_{c.g.//} \cdot K_2 \cdot K_{3.med}$ $\eta := 0.005 \cdot \lambda$ $E := E_{min}$

$$K_{12.med} := \frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c} - \sqrt{\left(\frac{1}{2} + \frac{(1+\eta) \cdot \pi^2 \cdot E}{3 \cdot \lambda^2 \cdot \sigma_c}\right)^2 - \frac{\pi^2 \cdot E}{1.5 \cdot \lambda^2 \cdot \sigma_c}}$$
$$K_{12.med} = 0.23$$

- (2) Using Table 19:
 - (a) Long-term loading:

$$\frac{E_{min}}{\sigma_{c.g.//} \cdot K_2 \cdot K_{3.long}} = 911.39 \text{ and } \lambda = 129.9 \text{ thus, } K_{12.long} = 0.27$$

(b) Medium-term loading:

$$\frac{E_{min}}{\sigma_{c.g.//} \cdot K_2 \cdot K_{3.med}} = 729.11 \text{ and } \lambda = 129.9 \text{ thus, } K_{12.med} = 0.23$$

5. Loading

Eccentricity, e_y	$e_y := 90 \cdot \mathrm{mm}$
(a) Long-term loading:	
Axial load	$P_{long} := 25 \cdot kN$
Moment = axial load × eccentricity	$M_{long} := P_{long} \cdot e_y$
	$M_{long} = 2.25 \circ kN \cdot m$
(b) Medium-term loading:	
Axial load	$P_{med} := 30 \cdot kN$
Moment = axial load × eccentricity	$M_{med} := P_{med} \cdot e_y$
	$M_{med} = 2.7 \circ kN \cdot m$

6. Compression and bending stress

(a) Long-term loading: $K_3 := K_{3,long}$ $K_{12} := K_{12,long}$ $P := P_{long}$ $M := M_{long}$ Applied compressive stress $\sigma_{c.a.//} := \frac{P}{A}$ Permissible compressive stress $\sigma_{c.a.//} = 1 \circ N \cdot mm^{-2}$ $\sigma_{c.adm.//} := \sigma_{c.g.//} \cdot K_2 \cdot K_3 \cdot K_8 \cdot K_{12}$ $\sigma_{c.adm.//} = 2.13 \circ N \cdot mm^{-2}$ Compression satisfactory

Applied bending stress $\sigma_{m.a.//} := \frac{M}{Z_{xx}}$ Permissible bending stress $\sigma_{m.a.//} = 2.16 \circ N \cdot mm^{-2}$ $\sigma_{m.adm.//} := \sigma_{m.g.//} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_8$ $\sigma_{m.adm.//} := 7.65 \circ N \cdot mm^{-2}$ Bending satisfactory

Check interaction quantity

Euler critical stress

$$\sigma_e := \frac{\pi^2 \cdot E_{min}}{\lambda^2}$$

$$\sigma_e = 4.21 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\frac{\sigma_{m.a.//}}{\sigma_{m.adm.//} \cdot \left(1 - \frac{1.5 \cdot \sigma_{c.a.//}}{\sigma_e} \cdot K_{12}\right)} + \frac{\sigma_{c.a.//}}{\sigma_{c.adm.//}} = 0.78 < 1$$

Interaction quantity satisfactory

(b) Medium term loading: $K_3 :=$	$K_{3.med}$ $K_{12} := K_{12.med}$ $P := P_{med}$ $M := M_{med}$
Applied compressive stress	$\sigma_{c.a.//} := \frac{P}{A}$
Permissible compressive stress	$\sigma_{c.a.//} = 1.2 \circ \text{N} \cdot \text{mm}^{-2}$ $\sigma_{c.adm.//} := \sigma_{c.g.//} \cdot K_2 \cdot K_3 \cdot K_8 \cdot K_{12}$ $\sigma_{c.adm.//} = 2.26 \circ \text{N} \cdot \text{mm}^{-2}$ Compression satisfactory
Applied bending stress	$\sigma_{m.a./\!/} := \frac{M}{Z_{xx}}$
Permissible bending stress	$\sigma_{m.a.//} = 2.59 \circ \text{N} \cdot \text{mm}^{-2}$ $\sigma_{m.adm.//} := \sigma_{m.g.//} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_8$ $\sigma_{m.adm.//} = 9.56 \circ \text{N} \cdot \text{mm}^{-2}$ Bending satisfactory

Check interaction quantity

Euler critical stress

$$\sigma_e := \frac{\pi^2 \cdot E_{min}}{\lambda^2}$$

$$\sigma_e = 4.21 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\frac{\sigma_{m.a.//}}{\sigma_{m.adm.//} \cdot \left(1 - \frac{1.5 \cdot \sigma_{c.a.//}}{\sigma_e} \cdot K_{12}\right)} + \frac{\sigma_{c.a.//}}{\sigma_{c.adm.//}} = 0.83 < 1$$

Interaction quantity satisfactory

Therefore a 100 mm \times 250 mm timber section in strength class C24 is satisfactory

Design of glued laminated members

Glued laminated timber, commonly known as *glulam*, is a structural material obtained by gluing together at least four laminations of timber sections of usually 19-50 mm basic thickness so that the grain directions of all are parallel. Normally the laminations are dried to around 12-18% moisture content before being machined and assembled. This leads to a reduction in laminations' thickness of around 3-5mm. Assembly is commonly carried out by applying carefully controlled adhesive mix to the faces of the laminations. They are then placed in mechanical or hydraulic jigs of the appropriate shape and size, and pressure is applied at right angles to the glue lines and is held until curing of the adhesive is complete. Glulam is then cut, shaped and any specified preservative and finishing treatments are applied. In Figure 4.1 the cross-section of a typical glulam beam is shown.



Figure 4.1 Cross-section of a glulam beam.

Since the length of glulam members normally exceeds the length of commercially available solid timber, the individual laminations are finger-jointed together to make laminations of the required length. These finger-joints are then placed randomly throughout the glulam component.

UNIJOS CVE 306- Design in Timber Prof. A. Mohammed

Design of Glued Laminated Members

A glulam beam designed to resist loads applied perpendicular to the plane of the laminations is referred to as *horizantally laminated*, and when it is designed to resist loads applied parallel to the plane of the laminations it is *vertically laminated*, see Figure 4.2.

Construction with glulam members offers the advantages of excellent strength and stiffness-to-weight ratio, which are usual with timber materials. Glued laminated members can also be designed to have substantial fire resistance. A summary of the advantages of glulam members over solid timber sections is as follows:

- 1. Glulam sections are built up from thin members and it is therefore possible to manufacture complicated shapes. They can be produced in any size, length, and shape, and manufacturing and transportation facilities remain as the only practical limiting factors affecting dimensions.
- 2. The use of a number of laminates enables an even distribution of defects and the location of better quality timber where it is most efficiently utilised.
- 3. It is easily possible to build-in a camber at the time of assembly.



Figure 4.2 Horizontally and vertically laminated glulam beams.

Design considerations

The main design considerations for flexural and axially loaded glulam members are the same as those described in the last two lecture notes. BS 5268 : Part 2 : 1996, Section 3 deals with the design of glued laminated timber and specifies that such timber should be manufactured in accordance with Clause 3.4 of the code and also with the recommendations of BS EN 386 : 1995 *Glued laminated timber*. *Performance requirements and minimum production requirements*. It also requires that all timber used for laminated softwood members should be strength graded visually or mechanically in accordance with BS EN 518 or BS EN 519, respectively.

Single grade members

If a glulam section is manufactured using the same grade throughout it is referred to as *single-grade* glued laminated member. The grade stresses given in Tables 7-12 of **BS** 5268: Part 2 apply primarily to solid structural timber. In order to determine the grade stresses for a horizontally glued laminated softwood or hardwood member, Clause 3.2 of the code recommends that the strength class stresses given in its Tables 7-12 for the relevant grade and species are multiplied by the modification factors K_{15} to K_{20} according to the nature of stress and the number of laminations. The modification factors K_{15} to K_{20} , detailed in Table 21 of the code, are for *single-grade horizontally glued laminated members* (Table 21 is shown here as Table 4.1).

Combined grade members

For economic reasons there are many instances where members may be horizontally laminated with two different strength class timber sections. Higher grade laminations are normally placed in the outer portions of the member where they are effectively used, and lower-grade laminations are placed in the inner portion (core) where the lower strength will not greatly affect the overall strength. The combination is permitted provided that the strength classes are not more than three classes apart (as given in Table 7 of the code.) For example C24 and C16 can be horizontally laminated but C24 and C14 should not. The code also requires that in horizontally laminated members where combination grades are used, they should be fabricated so that not less than 25% of the depth at both the top and bottom of the member is of the higher strength class, see Figure 4.3. For such members the grade stresses should be taken as the product of the strength class stresses (given in Table 7) for the *higher strength class laminations* and the modification factors from Table 21 of the code. In addition, for bending,tension and compression parallel to grain, the grade stresses should be multiplied by 0.95.

Table 4.1 Modification factors K_{15} , K_{16} , K_{17} , K_{18} , K_{19} , and K_{20} for single-grade glued laminated members and horizontally glued laminated beams (Table 21, BS 5268)

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Strength classes	Number of laminations ^a	Bending // to grain	Tension // to grain	Compression // to grain	Compression ⊥ to grain ^b	Shear // to grain	Modulus of elasticity ^e
		$(\sigma_{m,g,//}) \atop K_{15}$	$(\sigma_{t,g,//}) \atop K_{16}$	$(\sigma_{c,g, \parallel}) \atop K_{17}$	$\substack{(\sigma_{c,g,\perp})\ K_{18}}$	$(au_{g, \emptyset}) \ K_{19}$	E _{mean} K ₂₀
C27, C30, D50, D60, D70	4 or more	1.39	1.39	1.11	1.49	1.49	1.03
C22, C24,	4	1.26	1.26	1.04	1.55	2.34	1.07
D35, D40	5	1.34	1.34				
-	7	1.39	1.39				
	10	1.43	1.43				
	15	1.48	1.48				
	20 or more	1.52	1.52				
C16, C18,	4	1.05	1.05	1.07	1.69	2.73	1.17
D30	5	1.16	1.16				
	7	1.29	1.29				
	10	1.39	1.39				
	15	1.49	1.49				
	20 or more	1.57	1.57				

^a Interpolation is permitted for intermediate numbers of laminations

^b K_{18} should be applied to the lower value given in Table 7 for compression perpendicular to grain

 $^{c}K_{20}$ should be applied to the mean value of modulus of elasticity



Figure 4.3 Horizontally glued laminated members.

Permissible stresses for horizontally glued laminated members

The *permissible stresses* for a horizontally glued laminated member are determined as the product of grade stresses, given in Table 7, by any relevant modification factors, i.e. K_2, K_3, \ldots, K_{14} and modification factors K_{15} to K_{20} appropriate to the number of laminations. These are summarised below:

Bending parallel to grain:

for a single-grade glulam, $\sigma_{m,adm,//} = \sigma_{m,g,//} \times K_2 K_3 K_6 K_7 K_8 K_{15}$ for a combined-grade glulam, $\sigma_{m,adm,//} = \sigma_{m,g,//} \times K_2 K_3 K_6 K_7 K_8 K_{15} \times 0.95$

Tension parallel to grain:

for a single-grade glulam, $\sigma_{t,adm,//} = \sigma_{t,g,//} \times K_2 K_3 K_8 K_{14} K_{16}$ for a combined-grade glulam, $\sigma_{t,adm,//} = \sigma_{t,g,//} \times K_2 K_3 K_8 K_{14} K_{16} \times 0.95$

Compression parallel to grain: for a single-grade glulam, $\sigma_{c,adm,//} = \sigma_{c,g,//} \times K_2 K_3 K_8 K_{12} K_{17}$ for a combined-grade glulam, $\sigma_{c,adm,//} = \sigma_{c,g,//} \times K_2 K_3 K_8 K_{12} K_{17} \times 0.95$

Compression perpendicular to grain: for both single- and combined-grade glulam members,

$\sigma_{c,adm,\perp} = \sigma_{c,g,\perp} \times K_2 K_3 K_4 K_8 K_{18}$

Shear parallel to grain: for both single- and combined-grade glulam members,

$\tau_{adm, //} = \tau_{g, //} \times K_2 K_3 K_5 K_8 K_{19}$

Modulus of elasticity: for both single- and combined-grade glulam members,

$E_{glulam} = E_{mean} \times K_{20}$

It should also be noted that for tension perpendicular to grain and torsional shear, permissible stresses should be calculated in accordance with Clause 2.7 of the code, and factors K_{19} and K_{27} should be disregarded. For example,

For tension perpendicular to grain:

for both single- and combined-grade glulam members

 $\sigma_{t,adm,\perp} = \frac{1}{3}\tau_{g,\parallel} \times K_2 K_3 K_8$

Grade stresses for vertically glued laminated beams

A glulam beam designed to resist loads applied parallel to the plane of laminations is referred to as a *vertically laminated beam*. This is illustrated in Figure 4.2. BS 5268 : Part 2, Clause 3.3 specifies that the permissible stresses of vertically glued laminated beams are governed by the particular conditions of service and loading defined in its Clauses 2.6.2, 2.8 and 2.9 which relate to service class 3 (wet-exposure) condition, duration of loading and load-sharing systems respectively; and also by the additional modification factors K_{27} , K_{28} and K_{29} given in Table 22 of the code appropriate to the number of laminations. Table 22 of the code is reproduced here as Table 4.2.

In general it is not recommended that vertically laminated beams be designed using different grade laminates (i.e. combined strength classes). When a designer is considering a glulam beam with vertical laminations, it is often because a horizontally laminated member with bending about the axis is also being subjected to bending on its y" axis, for example, due to lateral loading. If the vertically laminated beam is made from different grade laminations it can be assumed that such a member has grade stresses equal to the product of *weighted* strength class stresses, and the modification factors K_{27} , K_{28} and K_{29} , given in Table 22, appropriate to the total number of laminations in the beam.

A *weighted* strength class stress, σ_w , may be obtained from:

$$\sigma_w = \sigma_{g,lower} \times \frac{N_{lower}}{N_{total}} + \sigma_{g,higher} \times \frac{N_{higher}}{N_{total}}$$
(4.1)

Where $\sigma_{g,lower}$ and $\sigma_{g,higher}$ refer to appropriate Stress or modulus of elasticity for the lower-grade and higher-grade laminations respectively, as given in Table 7 of the code; and similarly N refers to the appropriate number of laminations in the member. For example, the weighted bending stress parallel to grain for a vertically laminated beam of 12 laminations, where 3 laminations at each end are in C24 and 6 inner ones are in C18 softwood timber, may be calculated as follows:

From Table 7 of the code (Table 3 of the lecture notes):

for C18 timber, $\sigma_{m.g.//} = 5.8 \text{ N/mm}^2$ for C24 timber, $\sigma_{m.g.//} = 7.5 \text{ N/mm}^2$

Therefore the weighted grade bending stress is:

$$\sigma_{w,m,g,ll} = 5.8 \times \frac{6}{12} + 7.5 \times \frac{6}{12} = 6.65 \,\mathrm{N/mm^2}$$

Hence the weighted bending stress parallel to grain for this glulam beam is calculated as:

From Table 22 of the code (Table 4.2 of the lecture notes), for 12 laminations, $K_{27} = 1.25$.

 $\sigma_{w,m,l/l} = 6.65 \times 1.25 = 8.31 \text{ N/mm}^2$

Permissible stresses for vertically glued laminated members

The permissible stresses for a vertically glued laminated member are determined as the product of grade stresses (or weighted grade stresses in the case of a member with combined-grade laminations) by any relevant modification factors, i.e. K_{27} , K_{3} and K_{29} appropriate to the number of laminations). These are summarized below:

Bending parallel to grain: $\sigma_{m,adm,//} = \sigma_{m,g,//} \times K_2 K_3 K_6 K_7 K_8 K_{27}$ Tension parallel to grain: $\sigma_{t,adm,//} = \sigma_{t,g,//} \times K_2 K_3 K_8 K_{14} K_{27}$ Shear parallel to grain: $\tau_{adm,//} = \tau_{g,//} \times K_2 K_3 K_5 K_8 K_{27}$ Compression parallel to grain: $\sigma_{t,adm,//} = \sigma_{t,g,//} \times K_2 K_3 K_8 K_{12} K_{28}$ Compression perpendicular to grain: $\sigma_{c,adm,\perp} = \sigma_{c,g,\perp} \times K_2 K_3 K_4 K_8 K_{29}$ Modulus of elasticity: $E_{ghulam} = E_{mean}$

For vertically laminated beams, the E value (for 4 or more laminations) is $E_{mean,r}$, and K_{28} is only applicable to E_{min} . It should be noted that the size factors K_7 and K_{14} for all glulam members, whether horizontally or vertically laminated, should be applied to glulam strengths not to the individual laminate strengths.

It should also be noted that for tension perpendicular to grain and torsional shear, permissible stresses should be calculated in accordance with Clause 2.7 of the code, and factor K_{27} should be disregarded.

Deflection

BS 5268: Part 2 : 1996, Clause 3.5 recommends that in addition to the deflection due to bending, the shear deflection may be significant and should be taken into account. The deflection criteria for flexural members which were covered in detail previously also apply to glulam beams.

As it is easily possible to build-in a *camber* at the time of assembly, the code specifies that members can be *pre-cambered* to offset the deflection under dead or permanent loads. In such cases the deflection under live or intermittent imposed load should not exceed 0.003 of the span.

Table 4.2 Modification factors K27,K28, and K29. For vertically glued laminated members,(Table, BS 5268: Part 2)

Number of laminations	Bending // to grain, $(\sigma_{m,g,//})$, tension // to grain, $(\sigma_{t,g,//})$, and shear // to grain, $(\tau_{g,//})$ K_{27}		Modulus of elasticity, $(E_{min})^{a}$ and compression // to grain, $(\sigma_{c,g, \text{ff}})$ K_{28}		Compresison \perp to grain ^b $(\sigma_{c,g,\perp})$ K_{29}	
	Softwoods	Hardwoods	Softwoods	Hardwoods	Softwoods and hardwoods	
2	1.11	1.06	1.14	1.06		
3	1.16	1.08	1.21	1.08		
4	1.19	1.10	1.24	1.10		
5	1.21	1.11	1.27	1.11	1.10	
6	1.23	1.12	1.29	1.12		
7	1.24	1.12	1.30	1.12		
8 or more	1.25	1.13	1.32	1.13		

^a When applied to the value of the modulus of elasticity, E, K_{28} is applicable to the minimum value of E

^b If no wane is present, K_{29} should have the value 1.33, and, regardless of the grade of timber used, should be applied to the SS or HS grade stress for the species

Design examples

Example 4.1 Design of a single-grade glued laminated beam

A series of glulam beams of 115 mm wide x 560 mm deep (14 laminations) with an effective span of 10.5m is to be used in the construction of the roof of an exhibition hall. The roof comprises exterior tongued and grooved solid softwood decking exposed on the underside and covered on the top with 3 layer felt and chippings. For the design data given below,

(a) check the suitability of the section,

(b) determine the camber required.

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Design data Timber

	service c
Bearing width	90 mm eac
Dead load (long-term):	
Exterior decking	0.75 kN/m
3 layer felt and chippings	0.63 kN/m
Beam self-weight	0.3 kN/m
Imposed load (medium-term)	2.5 kN/m

Imported whitewood in strength class C24 and service class 1 90 mm each end



Figure 4.6 Cross-section of glulam beam (Example 4.1)

Definitions

Force, kNN := newtonLength, m $kN := 10^3 \cdot N$ Cross-sectional dimensions, mmDirection parallel to grain, //Stress, Nmm⁻²Direction perpendicular to grain, pp

Design of Glued Laminated Members

1. Geometrical properties

Effective span, L_e	$L_e := 10.5 \cdot \mathrm{m}$
Bearing width, bw	$bw := 90 \cdot mm$
Beam dimensions:	
Breadth of section, b	$b := 115 \cdot \mathbf{mm}$
Depth of section, h	$h := 560 \cdot \mathrm{mm}$
Cross-sectional area, A	$A := b \cdot h$
	$A = 6.44 \times 10^4 \circ \mathrm{mm}^2$
Second moment of area, I_{xx}	$I_{xx} := \frac{1}{12} \cdot b \cdot h^3$
	$I_{xx} = 1.68 \times 10^9 \circ \mathrm{mm}^4$

2. Loading

Dead load:	
Self-weight of beam (kN/m), Swt	$Swt := 0.3 \cdot kN \cdot m^{-1}$
t and g decking, (kN/m), tg	$tg := 0.75 \cdot kN \cdot m^{-1}$
3 layer felt and chippings (kN/m), <i>fchip</i>	$fchip := 0.63 \cdot kN \cdot m^{-1}$
Total dead load, DL	$DL := (Swt + tg + fchip) \cdot L_e$
	$DL = 17.64 \circ kN$
Imposed load:	
Prescribed imposed load, (kN/m), pil	$pil := 2.5 \cdot kN \cdot m^{-1}$
Total imposed load, IL	$IL := pil \cdot L_e$
•	$IL = 26.25 \circ kN$
Total load (kN):	
Long-term, W _{long}	$W_{long} := DL$
	$W_{long} = 17.64 \circ kN$
Medium-term, W _{med}	$W_{med} := DL + IL$
	$W_{med} = 43.89 \circ kN$

3. K-factors

Service class 1 (K2, Table 13) Medium-term loading (K₃, Table 14)

$$K_2 := 1$$

 $K_3 := 1.25$

	÷
Bearing: 90 mm (K_4 , Table 15) (bearing position not defined)	$K_4 := 1.0$
Notched end effect $(K_5, Clause 2.10.4)$	$K_5 := 1$
Form factor (K_{ℓ} , Clause 2.10.5)	$K_6 := 1$
Depth factor (K_7 , Clause 2.10.6) to provide a dimensionless	
value for K_7 in Mathcad, let:	$h_1 := h \cdot \mathrm{mm}^{-1}$
for $h > 300 \mathrm{mm}$	$K_7 := 0.81 \cdot \frac{h_1^2 + 92300}{h_1^2 + 56800}$
	$K_7 = 0.89$
No load sharing (K_8 , Clause 2.9)	$K_8 := 1$
Glulam modification factors (Table	21):
Single-grade, 15 laminates in C24 timber	
Bending // to grain, K_{15}	$K_{15} := 1.48$
Compression perpendicular to grain, K_{18}	$K_{18} := 1.55$
Shear // to grain, K_{19}	$K_{19} := 2.34$
Modulus of elasticity, K_{20}	$K_{20} := 1.07$

Design of Glued Laminated Members

4. Grade stresses

BS 5268 : Part 2, Table 7	
Timber strength class, C24	
Bending // to grain	$\sigma_{m \cdot g \cdot / /} := 7.5 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Compression perpendicular	$\sigma_{c.g.pp} := 1.9 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
to grain	
Shear // to grain	$\tau_{g./\!/} := 0.71 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$
Mean modulus of elasticity	$E_{mean} := 10800 \cdot \mathrm{N} \cdot \mathrm{mm}^{-2}$

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5. Bending stress

Applied bending moment

Section modulus

Applied bending stress

Permissible bending stress

6. Lateral stability

Clause 2.10.8 and Table 16

Maximum depth-to-breadth ratio, h/b

$$\frac{h}{b} = 4.87$$

 $F_v := \frac{W_{med}}{2}$

Ends should be held in position and compression edges held in line by direct connection of t & g decking to the beams

7. Shear stress

Applied shear force

Applied shear stress

Permissible shear stress

$$F_{v} = 21.95 \circ \text{kN}$$

$$\tau_{a.\parallel} := \frac{3}{2} \cdot \left(\frac{F_{v}}{b \cdot h}\right)$$

$$\tau_{a.\parallel} = 0.51 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\tau_{adm.\parallel} := \tau_{g.\parallel} \cdot K_{2} \cdot K_{3} \cdot K_{5} \cdot K_{8} \cdot K_{19}$$

$$\tau_{adm.\parallel} = 2.08 \circ \text{N} \cdot \text{mm}^{-2}$$

Shear stress satisfactory

$$M_{med} := \frac{W_{med} \cdot L_e}{8}$$

$$M_{med} = 57.61 \circ \text{kN} \cdot \text{m}$$

$$Z_{xx} := \frac{b \cdot h^2}{6}$$

$$Z_{xx} = 6.01 \times 10^6 \circ \text{mm}^3$$

$$\sigma_{m.a.//} := \frac{M_{med}}{Z_{xx}}$$

$$\sigma_{m.a.//} := 9.58 \circ \text{N} \cdot \text{mm}^{-2}$$

$$\sigma_{m.adm.//} := \sigma_{m.g.//} \cdot K_2 \cdot K_3 \cdot K_6 \cdot K_7 \cdot K_8 \cdot K_{15}$$

$$\sigma_{m.adm.//} = 12.32 \circ \text{N} \cdot \text{mm}^{-2}$$
Bending stress satisfactory

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8. Bearing stress

Applied load

Applied bearing stress, (bearing area = $b \cdot bw$)

Permissible bearing stress

9. Deflection

Modulus of elasticity of glulam, E	$E := E_{mean} \cdot K_{20}$ $E = 1.16 \times 10^4 \circ \text{N} \cdot \text{mm}^{-2}$
Deflection due to bending	$\Delta_m := \frac{5 \cdot W_{med} \cdot L_e^3}{384 \cdot E \cdot I_{xx}}$
	$\Delta_m = 34.02 \circ \mathrm{mm}$
Deflection due to shear	$\Delta_s := \frac{19.2 \cdot M_{med}}{(b \cdot h) \cdot E}$
	$\Delta_s = 1.49 \circ \mathrm{mm}$
Total deflection	$\Delta_{total} := \Delta_m + \Delta_s$
	$\Delta_{total} = 35.5 \circ \mathrm{mm}$
Permissible deflection	$\Delta_{adm}:=0.003\cdot L_e$
	$\Delta_{adm} = 31.5 \circ \mathrm{mm}$

Camber required = deflection under permanent (long-term) loading:

	$\Delta_{long} := rac{5 \cdot W_{long} \cdot L_e^3}{384 \cdot E \cdot I_{xx}}$
	$\Delta_{long} = 13.67 \circ \mathrm{mm}$
	Provide 15 mm camber
ad	$\Delta_{\mathit{live}} := \Delta_{\mathit{total}} - \Delta_{\mathit{long}}$
	$\Delta_{live} = 21.83 \circ \text{mm}$
	Deflection satisfactory

 $F := \frac{W_{med}}{2}$

 $F = 21.95 \circ kN$

 $\sigma_{c.a.pp} := \frac{F}{b \cdot bw}$

 $\sigma_{c.a.pp} = 2.12 \circ N \cdot mm^{-2}$

Bearing stress satisfactory

 $\sigma_{c.adm.pp} := \sigma_{c.g.pp} \cdot K_2 \cdot K_3 \cdot K_4 \cdot K_8 \cdot K_{18}$ $\sigma_{c.adm.pp} = 3.68 \circ \mathrm{N} \cdot \mathrm{mm}^{-2}$

Deflection under live load

Therefore $115 \text{ mm} \times 560 \text{ mm}$ glulam sections in single-grade C24 are satisfactory

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Design of timber connections

The common connecting systems in structural timber may be classified as mechanically fastened and glued joints. Mechanical fasteners may also be categorised into two groups; dowel-type fasteners and connectors. The classification is based on how they transfer the forces between the connected members. Examples of fasteners are nails, screws, bolts and dowels, while connectors can be toothedplates, split rings, shear plates and punched metal plates in which load transmission is primarily achieved by a large bearing area at the surface of the members.

Design considerations

Design considerations for timber joints are detailed in section 6 of the code. This section stipulates the basic lateral shear and withdrawal loads for most common dowel type fasteners in Tables 54-74. A summary of the design considerations are given as follows:

- (1) Joints should be designed so that the loads induced in each fastener or timber connector unit by the design loads appropriate to the structure should not exceed the permissible values determined in accordance with Section 6 of the code.
- (2) When more than one nail, screw, bolt, etc is used in a joint, the permissible load is the sum of the permissible loads for the individual units provided that the centroid of the fasteners' group lies on the axis of the member, and that the spacing, edge and end distances are sufficient to develop the full strength of each fixing unit.
- (3) If the line of action of a force in a member does not pass through the centroid of the fasteners' group, account should be taken of the stresses and rotation due to the secondary moments induced by the eccentricity. This will impose higher loads on the fasteners furthest from the centroid of the group.
- (4) If the load on a joint is carried by more than one type of fastener, due account should be taken of the relative stiffnesses. Glue and mechanical fasteners have very different stiffness properties and should not be assumed to act in unison.

- (5) For the purpose of designing joints in glued laminated timber, tabulated values should be taken for the strength class of timber from which the glued laminated timber was made.
- (6) The effective cross-section of a jointed member should be used when calculating its strength. In addition, the shear stress condition shown in Figure 5.1 should be satisfied in the jointed member. Therefore the design requirement is to ensure that:

$$V \le \frac{2}{3} \cdot \tau_{adm} \cdot h_e \cdot t \tag{5.1}$$

where:

V = maximum shear force per member at the joint where

 $(V_1 + V_2 = F\sin\alpha)$

 h_e = distance from the loaded edge to the furthest fastener

 τ_{adm} = permissible shear stress, and

t = member thickness.



Figure 5.1- Shear stresses in a jointed member.

Joint slip

All mechanically connected timber joints, under the action of applied load, will undergo slip which occurs between their component parts. Slip can be a significant factor in the deformation and, in some cases, it can also affect the strength of some components. Clause 6.2 of BS5268: Part 2 provides a method for calculation of slip

in joints, whereby the slip per fastener per shear plane, u, in millimetres, may be obtained from:

$$u = \frac{F}{K_{ser}}$$
(5.2)

Where:

F = applied load per fastener per shear plane, in Newtons (N)

 K_{ser} = slip modulus, in Newtons per millimetre per fastener per shear plane, given in Table 52 of the code (reproduced here as Table 5.1).

For bolted joints it is recommended that an additional 1 mm should be added at each joint to allow for take-up of bolt hole tolerances. It is to be noted that the magnitude of slip is time-dependent and the final slip following medium-term or long-term loading may be considerably greater than the slip calculated as above, particularly in conditions of fluctuating moisture content.

Table 5.1 Fastener slip moduli K_{ser} per fastener per shear plane for service classes 1 and 2 (Table 52, BS 5268: Part 2)

Fastener type	Kser	
	Timber-to-timber or panel-to-timber (N/mm)	Steel-to-timber ^a (N/mm)
Nails (no pre-drilling)	$\frac{1}{25} \rho^{1.5} d^{0.8}$	$\frac{1}{15} \rho^{1.5} d^{0.8}$
Nails (pre-drilled) Screws Bolts and dowels	$\frac{1}{20}\rho^{1.5}d$	$\frac{1}{12}\rho^{1.5}d$

where ρ is the joint member characteristics density given in Table 7 (in kg/m³) d is the fastener diameter (in mm)

^a For the values in this column to apply, the holes in the steel plate should be the minimum clearance diameter and the steel plate should have a thickness of 1.2 mm or 0.3 times the fastener diameter, whichever is the greater
Effective cross-section

The effective cross-section of a jointed member should be used when calculating its strength. This is because several connector units remove part of the cross-sectional area of the timber and hence affecting the load capacity of the timber section. In such cases the effective cross-section should be determined by deducing the net projected area of the connectors from the gross area of the cross-section being considered.

The recommendations of BS 5268: Part 2 for multi-dowel-type joints are as follows:

(1) Nailed and screwed joints. When assessing the effective cross-section of multinailed or multi-screwed joints, all nails or screws of 5 mm in diameter, or larger, that lie within a distance of five nail or screw diameters, measured parallel to the grain from a given cross-section, should be considered as occurring at that cross-section.

No reduction of cross-section needs to be made for screws of less than 5mm diameter, and for nails of the same diameter but driven without pre-drilling.

(2) Bolted and dowelled joints. When assessing the effective cross-section of multibolted or multi-dowelled joints, all bolts or dowels that lie within a distance of two bolt or dowel diameters, measured parallel to the grain from a given cross-section, should be considered as occurring at that cross-section.

Spacing rules

In order to avoid splitting of timber, all dowel-type fasteners must be spaced at suitable distances from each other and from the ends and edges of timber or wood-based material. The various distances involved are illustrated in Figure 5.2.

An end distance is said to be loaded when the load on the fastener has a component towards the end of the timber. Otherwise, it is referred to as an unloaded end distance. Loaded end distances need to be greater than unloaded ones. In a similar way the edge distance may be loaded or unloaded.



Figure 5.2 Spacing of dowel type connectors.

The values of spacings and distances vary from one fastener type to another, as well as between the various material types. The spacings and distances recommended in Section 6 of BS 5268: Part 2 are detailed below:

(1) Nail spacing. The recommended end and edge distances and spacing of nails, to avoid undue splitting, are given in Table 53 of the code (reproduced here as Table 5.2).

Pre-drilling (with a hole diameter up to 0.8 times the nail diameter) reduces the splitting tendency considerably and hence allows much closer spacing of the nails.

All softwoods, except Douglas fir, may have values of spacing (but not edge distances) for timber to timber joints multiplied by 0.8. For nails driven into a glued laminated section at right angles to the glue surface, the spacings (but not edge distances) can be multiplied by a further factor of 0.9.

- (2) Screw spacing. The recommended end and edge distances and spacing of screws, to avoid undue splitting, are given in Table 59 of the code (reproduced here as Table 5.3).
- (3) Bolt and dowel spacings. The recommended end and edge distances and spacing of bolts and dowels are given in Table 75 of the code (reproduced here as Table 5.4).

Spacing ^a	Timber-to-t	imber joints	Steel plate-to- timber joints	Wood-based panel products ^b to-timber joints	
	Without pre-drilled holes	With pre-drilled holes	Without pre-drilled holes	Without pre-drilled holes	
End distance parallel to grain (a_3)	20 <i>d</i>	10 <i>d</i>	14 <i>d</i>	14 <i>d</i>	
Edge distance perpendicular to grain (a_4)	5 <i>d</i>	5 <i>d</i>	5 <i>d</i>	c	
Distance between lines of nails, perpendicular to grain (a_2)	10 <i>d</i>	3 <i>d</i>	7 <i>d</i>	7 <i>d</i>	
Distance between adjacent nails in any one line, parallel to grain (a_1)	20 <i>d</i>	10 <i>d</i>	14 <i>d</i>	14 <i>d</i>	

Table 5.2 Minimum nail spacings (Table 53, BS 5268: Part 2)

where d is the nail diameter

^aSpacings and distances are illustrated in Figure 5.2

^bPlywood, tempered hardboard or particleboard

^c The loaded edge distance should not be less than: in timber 5d, in plywood 3d, in particleboard and tempered hardboard 6d, and in all other cases the edge distance should not be less than 3d

Spacing ^a	With pre-drilled holes
End distance parallel to grain (a_3)	10 <i>d</i>
Edge distance perpendicular to grain (a_4)	5 <i>d</i>
Distance between lines of screws, perpendicular to grain (a_2)	3 <i>d</i>
Distance between adjacent screws in any one line, parallel to grain (a_1)	10 <i>d</i>

Table 5.3 Minimum screw spacings (Table 59, BS5268: Part 2)

where d is the shank diameter of the screw

^aSpacings and distances are illustrated In Figure 5.2

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Table 5.4 Minimum bolt and dowel spacings (Table 75 and clause 6.6.7.3, BS 5268: Part 2)

Direction of loading	End o	listance	Edge	distance	Distance between bolts or rows of bolts		
	Loaded	Unloaded	Loaded	Unloaded	Across the grain	Parallel to grain	
Loading // to grain Loading \perp to grain	7d 4d	4d 4d	1.5d 4d	1.5d 1.5d	4d 4d	4d 5d ^a	

^a Where the member thickness, t, is less than 3 times the bolt diameter, d, the spacing may be taken as the greater of 3d or (2 + t/d)d.

Multiple shear lateral loads

Dowel-type fasteners, such as nails, screws, bolts and dowels, are used to hold two, three or more members together to form a joint. In general, they are designed to carry lateral shear loads, but there are occasions where they might be subjected to axial loads (or withdrawal loads in the case of nailed or screwed joints) or a combination of lateral shear and axial loads.

Dowel-type fasteners, depending on their type and size and the arrangement of the joint components, may be subjected to single, double or multiple lateral shear forces (see Figure 5.3). It is important to note that the dowels in joints (a) and (b), shown in Figure 5.3, are all in single shear (i.e. one shear plane per dowel) and the dowels in joint (c) are in double shear (i.e. two shear planes per dowel).

The basic multiple shear lateral load for each dowel can generally be obtained by multiplying the value for its basic single shear load (given in the relevant tables of the code) by the number of shear planes; provided that the recommended standard thickness for members of the joint and the recommended joint details are met.



Figure 5.3 Single shear (a) and (b) double shear (c) joints.

Nailed joints

Nails are straight slender fasteners, usually pointed and headed, and are available in a variety of lengths and cross-sectional areas. There are also many types and forms of nails; but the most frequently used nails are the bright, smooth, common steel wire nails with circular cross-sectional area with a minimum tensile strength of 600N/mm², which are available in standard sizes of 2.65 mm to 8mm in diameter. Nails can be plain or enamelled, etched, electroplated, galvanised or polymer coated, for example, for protection against corrosion.

Nailed joints, which should normally contain at least two nails, are simple to fabricate and are of considerable economic value. They are suitable for lightly loaded structures and for thin members. They are commonly used in framing, walls, decks, floors and roofs and in nearly every construction that involves light loads and simple elements.

Improved nails

The performance of an ail, both under lateral and withdrawal loading, may be enhanced by mechanically deforming nail shanks to form annular ringed shank or helical threaded shank nails (see Figure 5.4). Such nails provide higher withdrawal resistance than plain shank nails of the same size. Other forms of improved nails are obtained by grooving or twisting of square cross-sectioned nails. The process of twisting not only modifies then ail surface but also work-hardens the steel, increasing its yield strength.

In order to provide for the enhanced performance of improved nails, BS 5268: Part 2, Clause 6.4.4.4 permits a 20% increase in the basic single shear lateral load capacities of the square grooved and square twisted shank nails, and a 50% increase in the basic withdrawal load capacities for the threaded part of annular ringed shank and helical threaded shank nails.

Pre-drilling

Nails driven into dense timbers are likely to induce timber splitting. Various research studies have shown that pre-drilling can prevent timber splitting and also has other advantages of less slip in the joint, increase in lateral load-carrying capacity and permits a more densely nailed joint (i.e. reduced nail spacings and distances).

The recommendations of BS 5268: Part 2 for pre-drilling are as follows:

(1) Hardwoods in strength classes of D30 to D70 will usually require pre-drilling and the code recommends that the diameter of pre-drilled holes should not be greater than 0.8 times the nail diameter.

In general it is usual to provide pre-drilling when the density of the timber is 500 kg/m^3 or greater.

(2) For nails driven into pre-drilled holes in softwood strength classes C14 to C40 and TR26, the code permits a 15% increase in their load capacities.



Figure 5.4 Types of nails.

Basic single shear lateral loads

1. Timber-to-timber nailed joints

The basic single shear lateral loads for single round wire nails with a minimum tensile strength of 600N/mm², driven at right angles to the side grain of timber in service classes 1 and 2, are given in Table 54 of the code (reproduced here as Table 5.5).

For nails driven into the end grain of timber, the values given in Table 54 should be multiplied by the end grain modification factor K_{43} which has a value of 0.7, i.e. $K_{43} = 0.7$.

For nails driven into pre-drilled holes in softwood strength classes C14 to C40 and TR26, the basic single shear lateral load values given in Table 54 may be multiplied by 1.15, i.e. for convenience introduce a modification factor for pre-drilling, as $K_{\text{pre-drill}} = 1.15$.

For square grooved and square twisted shank nails of steel with a yield stress of not less than 375 N/mm², the basic single shear lateral loads given in Table 54 should be multiplied by the improved nail lateral modification, K_{44} , which has a value 1.2, i.e. $K_{44} = 1.2$.

For the basic loads in Table 54 to apply, the nails should fully penetrate the tabulated standard values in both the headside and pointside members. In addition, for joints where nails are subjected to multiple shear lateral loads, the thickness of the inner member should be at least 0.85 times the standard thickness given in Table 54. Otherwise, the basic shear load should be multiplied by the smaller of the following ratios, where relevant:

(1) actual to standard thickness of headside member, or

(2) actual penetration to standard pointside thickness, or

(3) actual inner member to 0.85 times the standard thickness.

It is important to note that no load-carrying capacity should be assumed where the ratios given above are less than 0.66 for softwoods and 1.0 for hardwoods. Where improved nails are used, the ratios may be reduced to 0.50 for softwoods and 0.75 for hardwoods. In addition no increase in basic load is permitted for thicknesses or penetrations greater than the standard values.

Therefore, with reference to Figure 5.5, it is possible to define a modification factor for thickness of members and penetration depth (i.e. bearing lengths) for a nailed connection, $K_{\text{nail-bearing}}$, as:

for a nail in single shear,

$$K_{nail-bearing} = \left(\text{the lesser of } \frac{t_{a,head}}{t_{standard}}, \frac{t_{a,point}}{t_{standard}} \text{ or } 1 \right)$$

for a nail in multiple shear,

$$K_{nail-bearing} = \left(\text{the lesser of } \frac{t_{a,head}}{t_{standard}}, \frac{t_{a,inner}}{0.85 \cdot t_{standard}}, \frac{t_{a,point}}{t_{standard}} \text{ or } 1 \right)$$

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Table 5.5 Basic single shear lateral loads for round wire nails in a timber-to-timber joint (Table 54, BS 5268: Part 2)

Nail		Soft	woods (not pre-d		Ha	rdwoods (pre-drilled)			
diameter (mm)	Standard		Basic single she	ar lateral lo	oad, (N)	Minimum	Basic single shear lateral load, (N)		
	(mm)		Strength class				Strength class		
		C14	C16/18/22	C24	C27/30/35/40		D30/35/40	D50/60/70	
2.65	32	242	251	266	274	21	374	414	
3.00	36	297	308	327	336	24	467	517	
3.35	40	357	369	392	403	27	569	630	
3.75	45	430	444	472	486	30	695	770	
4.00	48	478	494	525	540	32	780	864	
4.50	54	581	600	638	656	36	961	1065	
5.00	60	691	714	759	781	40	1159	1284	
5.60	67	833	861	915	941	45	1417	1569	
6.00	72	934	965	1026	1054 ^b	48	1601	1773	
6.70	80	1120	1158	1230	1265 ^b	54	1945	2154	
8.00	96	1501	1552	1649	1695 ^b	64	2658	2944	

 $^{\rm a}$ These values apply to both the headside thickness and pointside penetration $^{\rm b}$ Holes should be pre-drilled



Figure 5.5 Member thicknesses in nailed joints.

2. Steel plate-to-timber nailed joints

Steel plate-to-timber nailed joints can have a considerably higher load capacity than timber-to-timber joints with similar nailing arrangements. BS 5268 :Part 2, Clause 6.4.5 recommends a 25% higher load capacity for steel plate-to-timber joints provided that steel plates have a minimum thickness equal to the greater of 1.2mm or 0.3 times the nail diameter.

Where a pre-drilled steel plate or component of such thickness is nailed to a timber member, the basic single shear lateral load given in Table 54 should be multiplied by the steel-to-timber modification factor, K_{46} , which has a value of 1.25, i.e. $K_{46} = 1.25$.

It is recommended that the diameter of the hole in the steel plate should not be greater than the diameter of the nail.

3. Plywood-to-timber nailed joints

The basic single shear lateral loads for single nails in a plywood-to-timber joint, where the nails are driven through the plywood at right angles into the side grain of timber in service classes 1 and 2, are given in Table 56 of the code (reproduced here as Table 5.6).

For the basic loads in Table 56 of the code to apply, the plywood should be selected from those given in Section 4 of the code, and the nails should be fully embedded and should have an overall length not less than those tabulated.

Axially loaded nails (withdrawal loads)

Round wire nails are weak when loaded axially and, thus, their strength should not be relied on to any great extent. The best resistance is generally obtained when the nails are driven into side grain (Figure 5.6). No withdrawal load is permitted by the code for nails driven into the end grain of timber.

Some of the main factors that influence the withdrawal resistance of a nail include the density and moisture content of the timber into which the nail driven and the nail type. As mentioned earlier, the type and surface condition of nails enhances their performance, for example, cement or polymer coated, annular ringed shank and helical threaded shank and square twisted nails all perform better under withdrawal loading than round wire nails.

In addition such improved nails are less affected by changes in the moisture content of the timber. The basic withdrawal loads for single nails driven at right angles to the side of timber in service classes 1 and 2 are given in Table 55 of the code (reproduced here as Table 5.7). The values apply to each 1 mm depth of actual pointside penetration achieved; where the pointside penetration of the nail should not be less than 15 mm.

For the threaded part of annular ringed shank and helical threaded shank nails, BS 5268 : Part 2 recommends that the basic withdrawal loads given in Table 55 should be multiplied by the improved nail withdrawal modification factor, K_{45} , which has a value of 1.5, i.e. $K_{45} = 1.5$.

Table 5.6 Basic single shear lateral loads for round wire nails in a plywood-to-timber joint, (Table 56, BS 5268: Part 2)

Plywood	Nail		Basic single shear lateral load, (N)											
Nominal	Diameter (mm)	Length (mm)						Streng	gth class					
(mm)	(IIIII)			Softw	oods (not pre	e-drille	d)		Hardv	voods	(pre-di	rilled)	
			C	4	C16/1	18/22	C	24	C27/30	/35/40	D30/3	35/40	D50/6	50/70
						Plywoo	od grou	up ^a			Ph	wood	group) ^a
			I	п	I	п	I	п	I	п	I	11	I	п
6	2.65	40	207	216	212	221	220	230	224	234	267	281	270	296
	3.00	50	252	262	258	268	268	279	273	284	295	346	295	358
	3.35	50	301	311	308	319	318	332	318	339	318	387	318	387
	3.75	75	344	373	344	382	344	399	344	407	344	419	344	419
	4.00	75	360	414	360	424	360	438	360	438	360	438	360	438
9	2.65	40	218	230	223	241	231	250	235	255	274	301	286	315
	3.00	50	261	280	267	286	277	297	281	302	333	362	347	379
	3.35	50	308	328	315	335	327	348	332	355	397	429	415	450
	3.75	75	366	387	374	396	389	412	395	419	479	514	495	539
	4.00	175	405	427	414	436	430	454	438	463	518	571	518	599
12	2.65	45	240	256	244	268	253	278	256	282	297	330	309	345
12	3.00	50	281	306	287	312	297	324	302	329	353	389	367	406
	3.35	50	320	336	334	353	345	374	351	380	415	454	432	474
	3.75	75	383	410	391	419	406	435	412	443	492	535	513	559
	4.00	75	421	449	430	459	446	477	453	485	545	590	567	616
15	2.65	50	266	285	271	208	280	310	284	314	327	365	330	380
15	3.00	65	308	337	314	343	324	356	329	361	382	424	397	441
	3.35	65	353	383	360	391	372	405	378	412	442	487	459	508
	3.75	75	408	440	416	449	431	466	438	473	518	566	538	590
	4.00	75	445	478	454	488	470	506	477	515	568	619	591	645
18	2.65	50	272	292	283	301	305	314	311	320	361	388	374	409
	3.00	65	339	361	345	369	356	386	361	303	417	461	432	479
	3.35	65	384	417	391	425	404	440	410	446	477	525	495	546
	3.75	75	439	474	447	483	462	500	469	508	552	603	572	627
	4.00	75	476	511	485	522	501	540	509	549	601	655	623	681
21	2 65	50	275	280	288	2C3	306	314	311	320	372	387	301	408
21	3.00	65	352	360	360	369	375	385	382	393	456	479	472	504
	3 35	65	411	432	426	443	440	462	446	471	517	570	535	592
	3.75	75	474	513	483	523	499	541	506	549	591	649	612	674
	4.00	75	510	551	520	562	537	581	545	590	641	701	664	727
20	2.65	50	222	225	242	245	261	262	270	272	272	390	201	300
29	2.05	50	255	255	242	245	201	203	200	200	461	470	192	104
	3.00	65	403	407	422	426	450	456	458	465	557	569	403	507
	3.75	75	509	515	521	527	542	550	552	560	675	690	707	723
	4.00	75	552	572	573	586	603	611	614	623	754	770	780	808
	-1.00	15	202	512	515	200	000		014	545	754	,,,,	109	000

^a Plywood group I comprises:

(a) American construction and industrial plywood

(b) Canadian Douglas fir plywood

(c) Canadian softwood plywood

(d) Finnish conifer plywood

(e) Swedish softwood plywood

Plywood group II comprises:

(a) Finnish birch-faced plywood

(b) Finnish birch plywood



Figure 5.6 Axially loaded nails

Table 5.7Basic withdrawal loads/mm of pointside penetration for smooth round wire nailsdriven at right angles to the grain (Table 55, BS 5268: Part 2)

Nail	Basic withdrawal load ^a , (N/mm) Strength class									
(mm)										
	C14	C16/18/22	C24	C27/30/35/40	D30/35/40	D50/60/70				
2.65	1.28	1.50	1.99	2.42	5.86	9.80				
3.00	1.45	1.69	2.25	2.74	6.64	11.09				
3.35	1.62	1.89	2.52	3.06	7.41	12.39				
3.75	1.81	2.12	2.82	3.42	8.30	13.87				
4.00	1.93	2.26	3.00	3.65	8.85	14.79				
4.50	2.17	2.54	3.38	4.11	9.96	16.64				
5.00	2.41	2.82	3.76	4.56	11.06	18.49				
5.60	2.70	3.16	4.21	5.11	12.39	20.71				
6.00	2.90	3.39	4.51	5.48	13.28	22.19				
6.70	3.23	3.78	5.03	6.12	14.82	24.78				
8.00	3.86	4.51	6.01	7.30	17.70	29.58				

^a The pointside penetration of the nail should not be less than 15 mm

Permissible load for a nailed joint

Clause 6.4.9 of BS 5268: Part 2 states that the permissible load for a nailed joint should be determined as the sum of the permissible loads for each nail in the joint, where each permissible nail load, F_{adm} , should be calculated from the equation:

$$F_{adm} = F \times K_{48} K_{49} K_{50} \tag{5.3}$$

where:

- F is the basic load for a nail under shear loading (Table 54) or withdrawal loading (Table 55) and is taken from Clauses 6.4.4, 6.4.5.2, 6.4.6.2, 6.4.7.2 or 6.4.8.2 of the code which were described earlier. Therefore F is given by:
 - (a) for a nail under single or multiple shear lateral load,

$$F = F_{basic} \text{ (Table 54)} \times n_{shear} K_{pre-drill} K_{nail-bearing} K_{43} K_{44} K_{46} \tag{5.4}$$

(b) for a nail under axial (withdrawal) load with a pointside penetration depth of t_{point},

$$F = F_{basic} \text{ (Table 55)} \times t_{point} K_{45} \tag{5.5}$$

where:

 n_{shear} = number of shear planes per nail

 $K_{pre-drill} = 1.15$, the pre-drilling factor for softwoods in strength classes C14 to C40 and TR26

$$K_{nail-bearing} = \left(\text{the lesser of } \frac{t_{a,head}}{t_{standard}}, \frac{t_{a,point}}{t_{standard}} \text{ or } 1 \right) \text{ for a nail in single shear}$$



 K_{48} is the modification factor for duration of loading

where:

 $K_{48} = 1.00$, for long-term loads

- = 1.25, for tempered hardboard-to-timber joints for mediumterm loads
- = 1.4, for particleboard-to-timber joints for medium-term loads
- = 1.12, for other than tempered hardboard-to-timber and particleboard-to-timber joints for medium-term loads
- = 1.62, for tempered hardboard-to-timber joints for short and very short-term loads
- = 2.10, for particleboard-to-timber joints for short and very shortterm loads
- = 1.25, for other than tempered hardboard-to-timber and particleboard-to-timber joints for short and very short-term loads.
- K_{49} is the modification factor for moisture content

where:

 $K_{49} = 1.00$, for lateral loads in joints in service classes 1 and 2

- = 0.70, for lateral loads in timber-to-timber joints in service class 3
- = 1.00, for lateral loads using annular ringed shank nails and helical threaded shank nails in all service class conditions
- = 1.00, for withdrawal load in all constant service class conditions
- =0.25, for withdrawal loads where cyclic changes in moisture content can occur after nailing.

K_{50}	is the modification factor for the number of nails of the same diameter
	in each line, acting in shear, for an axially loaded member, loaded
	parallel to the line of nails

where:

 $K_{50} = 1.00$, for the number of nails in each line < 10

= 0.9, for the number of nails in each line ≥ 10

= 1.00, for all other loading cases.

Design Examples

Example 5.1 Design of a simple nailed connection

The nailed joint shown in Figure 5.7 is subjected to medium-term tensile loading under service class 2. The joint comprises six 3.00 mm diameter x 60 mm long round wire nails acting in single shear in C18 timber. Determine the load-carrying capacity of the connection.



Figure 5.7 Nailed joint details (Example 5.1)

Definitions

Force, kN	N := newton
Length, m	$kN := 10^3 \cdot N$
Cross-sectional dimensions, mm	Direction parallel to grain, //
Stress, Nmm ⁻²	Direction perpendicular to grain, pp

1. Geometrical details

Nail diameter, d	$d := 3.00 \cdot \mathrm{mm}$
Nail length, <i>l_{nail}</i>	$l_{nail} := 60 \cdot mm$
No. of nails, n_{nail}	$n_{nail} := 6$
Head side penetration, $t_{a,head}$	$t_{a.head} := 33 \cdot mm$
Pointside penetration, $t_{a,point}$	$t_{a.point} := l_{nail} - t_{a.head}$
	$t_{a \text{ point}} = 27 \circ \text{mm}$

2. Basic values

BS 5268 : Part 2, Table 54	
Strength class, C18 timber	
Standard penetration, t _{standard}	$t_{standard} := 36 \cdot mm$
Basic single shear lateral	$F_{basic} := 308 \cdot N$
land E	

load, F_{basic}

3. Modification factors

BS 5268 : Part 2, Clause 6.4 Number of shear planes, n_{shear} No pre-drilling

Nail bearing is the lesser of

$$\begin{split} n_{shear} &:= 1\\ K_{pre.drill} &:= 1\\ K_{nail.bearing.1} &:= \frac{t_{a.head}}{t_{standard}}\\ K_{nail.bearing.1} &= 0.92\\ K_{nail.bearing.2} &:= \frac{t_{a.point}}{t_{standard}}\\ K_{nail.bearing.2} &= 0.75\\ K_{nail.bearing} &:= [K_{nail.bearing.1} K_{nail.bearing.2}]\\ \min(K_{nail.bearing}) &= 0.75 > 0.66 \text{ (Clause 6.4.4.1)}\\ \text{okay} \end{split}$$

and

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Thus	$K_{nail.bearing} := \min(K_{nail.bearing})$
Side grain nailing, K_{43} does not apply	$K_{43} := 1$
Ordinary nails, K_{44} does not apply	$K_{44} := 1$
Timber-to-timber nailing, K_{46} does not apply	$K_{46} := 1$
Load duration, K_{48} , medium-term	$K_{48} := 1.12$
Moisture content, K_{49} , service class 2	$K_{49} := 1$
No. of nails, <10 , in one line, K_{50}	$K_{50} := 1$

4. Permissible load

Permissible shear load per nail	$F_{adm} := F_{basic} \cdot n_{shear} \cdot K_{pre.drill} \cdot K_{nail.bearing}$
	$\cdot K_{43} \cdot K_{44} \cdot K_{46} \cdot K_{48} \cdot K_{49} \cdot K_{50}$
	$F_{adm} = 258.72 \circ \mathbf{N}$
Load capacity of connection,	$P_{capacity} := F_{adm} \cdot n_{nail}$
P _{capacity}	$P_{capacity} = 1.55 \circ kN$

5. Minimum nail spacings

BS 5268 : Part 2, Table 53

For softwoods other than Douglas fir, nail spacing should be multiplied by 0.8 but the edge distance $\geq 5d$.

$a_3 := 0.8 \cdot 20 \cdot d$
$a_3 = 48 \circ \mathrm{mm}$
$a_4 := 5 \cdot d$
$a_4 = 15 \circ \mathrm{mm}$
$a_2 := 0.8 \cdot 10 \cdot d$
$a_2 = 24 \circ \mathrm{mm}$
$a_1 := 0.8 \cdot 20 \cdot d$
$a_1 = 48 \circ \mathrm{mm}$

6. Joint slip

BS 5268 : Part 2, Clause 6.2 and Table 52 C18 timber Characteristic density (Table 7) $\rho := 320 \cdot kg \cdot m^{-3}$ Slip modulus:

Since the equation for slip modulus is an empirical one (i.e. with hidden units in the coefficient part), for Mathcad to produce correct units, multiply it by the units shown inside the parentheses; or alternatively, use the equation without units for ρ and d.

Thus, slip modulus,
$$K_{ser}$$

Slip per nail, u , under
permissible load
$$K_{ser} := \frac{1}{25} \cdot \rho^{1.5} \cdot d^{0.8} \cdot (\text{kg}^{-1.5} \cdot \text{m}^{2.7} \cdot \text{N} \cdot 10^{5.4})$$

$$K_{ser} = 551.42 \circ \text{N} \cdot \text{mm}^{-1}$$

$$u := \frac{F_{adm}}{K_{ser}}$$

$$u = 0.47 \circ \text{mm}$$

Therefore the load capacity of the joint is 1.55 kN with a joint slip of 0.47 mm

Example 5.2 Design of a timber-to-timber connection with nailing into the end grain

The nailed connection shown in Figure 5.8 is subjected to short-term loading under service class 2 conditions. The joint comprises two 3.75 mm diameter x 65 mm long round wire nails acting in single shear in C22 timber. Determine the load capacity of this connection.



Figure 5.8 Nailed joint details (Example 5.2)

Definitions

Force, kN	N := newton
Length, m	$kN := 10^3 \cdot N$
Cross-sectional dimensions, mm	Direction parallel to grain, //
Stress, Nmm ⁻²	Direction perpendicular to grain, pp

1. Geometrical details

Nail diameter, d	$d := 3.75 \cdot \mathrm{mm}$
Nail length, <i>l_{nail}</i>	$l_{nail} := 65 \cdot mm$
No. of nails, <i>n_{nail}</i>	$n_{nail} := 2$
Head side penetration, $t_{a,head}$	$t_{a.head} := 33 \cdot mm$
Pointside penetration, $t_{a,point}$	$t_{a.point} := l_{nail} - t_{a.head}$
	$t_{a.point} = 32 \circ mm$

2. Basic values

BS 5268 : Part 2, Table 54 Strength class, C22 timber Standard penetration, $t_{standard}$ Basic single shear lateral load, F_{basic}

3. Modification factors

BS 5268 : Part 2, Clause 4.6 Number of shear planes, n_{shear} No pre-drilling

Nail bearing is the lesser of

$$\begin{split} n_{shear} &:= 1 \\ K_{pre.drill} &:= 1 \\ K_{nail.bearing.1} &:= \frac{t_{a.head}}{t_{standard}} \\ K_{nail.bearing.1} &= 0.73 \\ K_{nail.bearing.2} &:= \frac{t_{a.point}}{t_{standard}} \\ K_{nail.bearing.2} &:= 0.71 \\ K_{nail.bearing} &:= [K_{nail.bearing.1} K_{nail.bearing.2}] \\ \min(K_{nail.bearing}) &= 0.71 > 0.66 \quad \text{okay} \end{split}$$

 $t_{standard} := 45 \cdot mm$

 $F_{basic} := 444 \cdot N$

and

Thus End grain nailing, K_{43} does not apply $K_{nail.bearing} := \min(K_{nail.bearing})$ $K_{43} := 0.7$

	0
Ordinary nails, K_{44} does not apply	$K_{44} := 1$
Timber-to-timber nailing, K_{46} does not apply	$K_{46} := 1$
Load duration, K_{48} , short-term	$K_{48} := 1.25$
Moisture content, K_{49} , service class 2	$K_{49} := 1$
No. of nails, <10 , in one line, K_{50}	$K_{50} := 1$

4. Permissible load

BS 5268 : Part 2, Clause 4.6.9	
Permissible shear load per nail	$F_{adm} := F_{basic} \cdot n_{shear} \cdot K_{pre.drill} \cdot K_{nail.bearing}$
	$\cdot K_{43} \cdot K_{44} \cdot K_{46} \cdot K_{48} \cdot K_{49} \cdot K_{50}$
	$F_{adm} = 276.27 \circ \mathbf{N}$
Load capacity of connection,	$P_{capacity} := F_{adm} \cdot n_{nail}$
P _{capacity}	$P_{capacity} = 0.55 \circ \mathrm{kN}$

5. Joint slip

BS 5268 : Part 2, Clause 6.2 and	Table 52
C22 timber	
Characteristic density (Table 7)	$ ho := 340 \cdot kg \cdot m^{-3}$

Slip modulus:

Since the equation for slip modulus is an empirical one (i.e. with hidden units in the coefficient part), for Mathcad to produce correct units, multiply it by the units shown inside the parentheses; or alternatively, use the equation without units for ρ and d.

Thus, slip modulus, K _{ser}	$K_{ser} := \frac{1}{25} \cdot \rho^{1.5} \cdot d^{0.8} \cdot (\text{kg}^{-1.5} \cdot \text{m}^{2.7} \cdot \text{N} \cdot 10^{5.4})$ $K_{ser} = 721.94 \circ \text{N} \cdot \text{mm}^{-1}$
Slip per nail, <i>u</i> , under permissible load	$u := \frac{F_{adm}}{K_{ser}}$ $u = 0.38 \circ \text{mm}$

Therefore the load capacity of the joint is 0.55 kN with a joint slip of 0.38 mm