Timber and wood-based products

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Contents

ABSTRACT

Timber has been used as a structural material for many centuries and a renewed interest in (and calls for) a greater use in construction is now observed. This has been mostly the result of an increasingly wider body of literature claiming the superior environmental benefits of timber (especially in terms of embodied greenhouse gas emissions) sourced from sustainably managed forests. The development of innovative manufacturing processes, such as glue lamination and cross lamination, has allowed in the last decades to produce a variety on wooden-based products, which maintain the main advantages of timber for structural use (e.g., high strength to weight ratio) while at the same time reducing the inherent limitations of solid timber in terms of maximum sizes and natural defects. An overview of the mechanical properties of structural timber and related products is given in this chapter, along with relevant information on how these properties are affected by environmental conditions, and how these aspects are dealt with during design. The main reference standard for the design of timber structures is the Eurocode 5 (BS EN 1995) and it represents the basis of this chapter. An introduction and explanation of rules for the design of common structural elements such as beams and columns, is therefore provided. This is then followed by design rules for timber connections and rules for the design of composite systems such as thin webbed sections and thin flanges sections. The Concluding section recapitulates the main topics touched in this chapter.

1. Introduction

The current framework for design and analysis of timber and wood-based products in UK is based on the Eurocode 5 (EC5), specifically:

— BS EN 1995-1-1 Design of timber structures – Part 1-1: General – Common rules and rules for buildings [1].
— BS EN 1995-1-2 Design of timber structures – Part 1-2: General – Structural fire design [2].
— BS EN 1995-2 Design of timber structures – Part 2: Bridges [3].

The three documents cover all aspects of timber design related to strength and serviceability, fire performance and durability, whereas design aspects involving sound insulation performances are covered in other standards.

Most of the content covered in this chapter is related to the design of timber and wood-based products as covered in *BS EN 1995-1-1*. As for all Eurocodes, design rules given in *BS EN 1995-1-1* are based on a probabilistic limit states approach, hence it is assumed that the reader is somehow familiar with the concept. Specifically, two limit states are defined in the Eurocode framework:

— Ultimate limit states (SLS): related to safety aspects such as structural failure or collapse.
— Serviceability limit states (SLS): related to serviceability performance such as deformations and vibrations.

In addition to rules and parameters set out in the main documents, countries adopting and incorporating the EC5 standard also issue so called national annexes (NA). These annexes provide nationally determined parameters (NDP) that may differ from the original values stated in the main document they are referred to. These NDPs will overrule the original parameters as set out in the original document, and indication of this is provided in this chapter whenever relevant.

Before delving into the design requirements for timber structures and wood-based products and understanding of material properties and processes involved in their production is certainly required, especially regarding the sensitivity and variability of mechanical properties when working in different environmental conditions. All these aspects are covered in the following sections (2,3 and 4), whereas other aspects more strictly related to structural design rules are covered in section 4 to section 8 for most common products and systems.

2. Timber for structural use

Structural sawn timber is produced by milling tree trunks into rectangular shapes of various sizes and lengths. In some cases, the trunk is peeled into thin sheets of veneer, that is used as the base material for production of plywood. Timber for structural use is classified in two main categories, namely: *softwoods* and *hardwoods*. Such terminology relates primarily to the botanical categorisation of timber species rather than to their mechanical properties. Softwoods are mostly from evergreen trees such as conifers, they tend to have a relatively quick rate of growth, and hence a lower density and strength compared to hardwood species, although this is not always the case. Commonly used softwoods for structural timber include spruce, Scot's pine and Douglas fir, whereas typical UK grown hardwoods are oak, beech and birch. Timber sourced from hardwood species is generally characterised by a slow growth rate, which results in higher density and strength, as well as better resistance to decay and attacks from insects. On the other hand, the longer rotation period required between tree harvests (up to 100 years for some species) makes hardwoods more expensive to produce (and hence to buy) compared to softwoods.

2.1 Natural characteristics

Unlike many other structural materials, timber is the result of a natural growing process, which leads to the presence of defects within the material such as knots and deviation of grain orientation. These natural defects are mostly responsible for the great variability of strength that is observable among samples from a population harvest, or even between samples taken (at different locations) from the same timber specimen.

2.2 Knots and slope of grain

Knots are the result of branches stemming from the tree trunk during growth. The presence of knots is responsible for localised distortions of wood fibres (grain) from the main orientation, which corresponds to the longitudinal direction of the tree log. The material discontinuities caused by knots generate concentration of stresses and non-uniform stress distributions, which can greatly affect the overall resistance of the timber member. Bigger knots tend to have a greater impact on strength, however their location and frequency withing the specimen also play a role in determining the final strength grade (with fewer knots resulting in higher timber grades).

Wood grain is the term referring to the orientation of wood fibres, and this corresponds the longitudinal direction of the structural member or tree log. In practice however, the grain orientation in never perfectly aligned to the longitudinal axis, one reason being the sawing of timber from bent or tapered logs. Small deviations of grain from the main (longitudinal) direction have little to no effect on mechanical properties. However, reductions in bending strength up to half the original value can occur for slopes of grain bigger than 11° [4]. The term *cross grain* is used to indicate sawn timber with a very steep slope of grain, as shown in Figure 1.

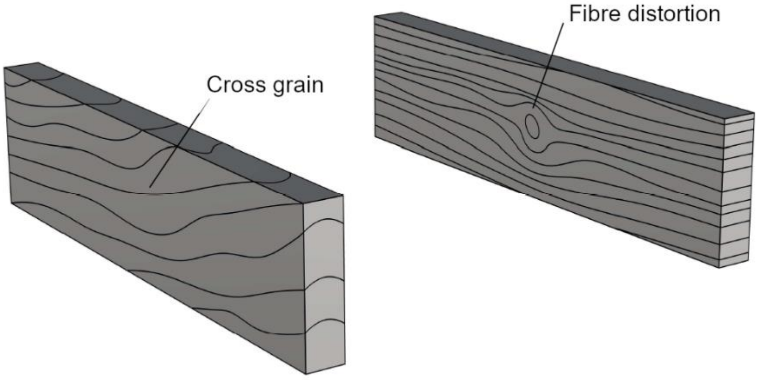


Figure 1: Natural defects such as the presence of knots and cross grain have a detrimental effect on strength of timber specimens.

2.3 Moisture content and service classes

Following the harvesting and milling phases, the moisture content internal to sawn timber drops to about 27% and remains to this level unless further moisture is forcibly removed (e.g., through seasoning process). Such a percentage in moisture content roughly corresponds to the *fibre saturation point*, that is, a state of equilibrium between internal moisture and humidity and temperature of surrounding air. Sawn timber usually undergoes a seasoning process before being graded, at the end of which the internal moisture content is usually below the saturation point (between 12% and 25% depending on the seasoning method e.g., air-dried or kiln-dried). Shrinkage occurs during seasoning as a result of reduction in moisture content. This can lead to distortions of the sawn member (e.g., twisting, cupping and bowing). Unacceptably pronounced defects may arise from the seasoning process, which can prevent some specimens from passing the required grade.

Swelling-induced structural displacements, due to increases in relative humidity, are mainly relevant with cross-sectional dimensions. For softwoods, a 1% increase (decrease) in depth and width can be assumed to occur for a 4% increase (decrease) in moisture content. Deformations along the grain direction can be ignored in practice as being only ~1/40 of cross-sectional deformations.

The level of moisture content also affects the strength and stiffness, both following a quasi-linear decreasing trend when moisture content increases. This relationship however only holds for moisture content values up to the fibre saturation point (~27%) above which no significant strength/stiffness deteriorations occur.

Since moisture content will increase/decrease according to variations in humidity levels of the working environment, a system of service classes is adopted in EC5, 2.3.1.3. Each class corresponds to different levels of exposure (both frequency and intensity) to a humid environment:

- Service class 1: relative humidity of the surrounding air only exceeding 65% for a few weeks per year. → average moisture content in most softwoods will not exceed 12%.

- Service class 2: relative humidity of the surrounding air only exceeding 85% for a few weeks per year. → average moisture content in most softwoods will not exceed 20%.

- Service class 3: environmental conditions are such that none of the above clauses applies. → the average moisture content in most softwoods exceeds 20%.

To help designers correctly classify their timber structures the UK National Annex to EC5 [5] provides a range of examples covering most design situations. These are summarised in here in Table 1.

Table 1: Service classes according to some types of building construction. Source: UK National Annex to EC5 [5].

Construction type	**Service class**
Cold roofs	2
Warm roofs	1
Intermediate floors	1
Ground floors	2
Timber framed walls – internal walls	1
Timber framed walls – external walls	2
External uses – where member is protected from direct wetting	2
External uses – fully exposed	3

2.4 Load duration classes

Due to its internal structure, made of tubular cells (fibres) timber strength slowly decreases when subject to long-term loading. The reason can be traced to accumulation of damages at the cells' walls, whose tubular structure buckles under the effect of compressive stress parallel to the grain direction. On the other hand, timber shows greater strength if subject to instantaneous loads such as from a blast explosion. This time-dependent behaviour is illustrated here in Figure 2, from Booth and Reece [6]. Here it can be seen how the maximum stress that can be supported in one-minute time decreases roughly to half its original value in 50-year time.

A system of load duration classes is provided in the UK National Annex to EC5 to facilitate the allocation of types of loadings to corresponding classes based on the time duration expected for that loading. The classes are reproduced here in Table 2.

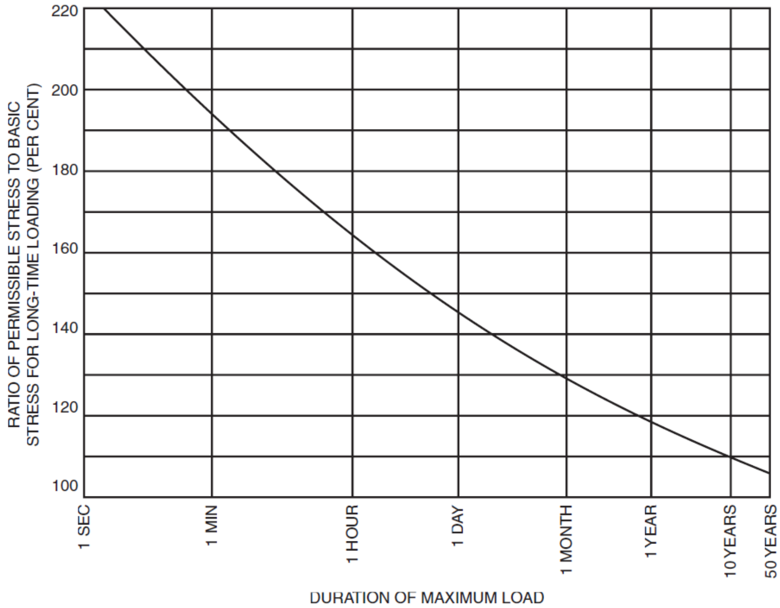


Figure 2: Relationship between strength to duration of load. Figure source: Booth and Reece [6].

Table 2: Definitions of load duration classes. Adapted from the UK National Annex to BS EN 1995-1-1 (EC5) [5].

Load-duration class	**Duration**	**Examples of loading**
Permanent	More than 10 years	Self-weight
Long-term	6 months to 10 years	Storage loading (including in lofts), water tanks
Medium-term	1 week to 6 months	Imposed floor loading
Short-term	Less than 1 week	Snow, maintenance or man loading on roofs, residual structure after accidental event
Instantaneous		Wind, impact loading, explosion

3. Solid timber and engineered wood products

3.1 Solid Timber

Geometric tolerances for structural sawn timber are prescribed in *BS EN336: 2013 — Structural timber. Sizes, permitted deviations*. The upper size limit of 300mm for cross-sections of solid structural timber has been removed in the latest update (2013) of the standard [7]. Permitted deviations are given for two different tolerance classes in *BS EN336* and are reported in Table 3 in here according to the corresponding target size. The more stringent tolerance class 2 applies for machine planed sections (either thickness and/or width) whereas tolerance class 1 is limited to sawn timber.

It is important to note that target sizes are assumed at reference moisture content of 20%. Corrections to target sizes, to account for changes in moisture content from the 20% reference value, are given as follows:

— Softwoods: thickness and width increase (decrease) by 0.25% for a 1% increase (decrease) in moisture content.
— Hardwoods: thickness and width increase (decrease) by 0.35% for a 1% increase (decrease) in moisture content.

Table 3: allowed tolerances for sawn structural timber according to *BS EN336: 2013* [7].

Tolerance Class	**Target size**	**Allowed deviation**
TC1	Thicknesses and widths ≤ 100mm	(-1 +3) mm
TC1	100mm < Thicknesses and widths ≤ 300mm	(-2 +4) mm
TC1	Thicknesses and widths > 300mm	(-3 +5) mm
TC2	Thicknesses and widths ≤ 100mm	(-1 +1) mm
TC2	100mm < Thicknesses and widths ≤ 300mm	(-1.5 +1.5) mm
TC2	Thicknesses and widths > 300mm	(-2 +2) mm

Commonly available (target) sizes in the UK for solid timber sections are listed in the National Annex to the 2003 version of BS EN336 [8] and reported in Table 4 here. Different standard sizes are usually found for sawm timber produced in North America. Commonly available dimensions for American Lumber Sizes (ALS) and Canadian Limber Sizes (CLS) are 38mm x 89 or 140mm.

Deviations from specified lengths are only permitted for positive values, however if over-length may be a problem (e.g., for handling on site) tolerance limits can be specified during procurement.

Table 4: Commonly available cross-section sizes for sawn timber in UK. Adapted from [8].

Sawn thickness (to tolerance class 1) [mm]	**Sawn width (to tolerance class 1) [mm]**								
	75	100	125	150	175	200	225	250	300
22		x							
38		x		x	x	x	x		
47	x	x	x	x	x	x	x	x	x
63				x	x	x	x		
75		x		x	x	x	x	x	x
100		x		x		x	x	x	x
150				x					x
300									x

3.2 Glue Laminated and Cross Laminated Timber

The maximum available length for solid timber (up to 5.4 m for customarily UK lengths) is limited in practice by the tree dimension and, most importantly, by natural defects such for instance the presence, size and frequency of knots. Since mechanical properties are affected by these natural defects, a way to eliminate them is via finger joining technique.

Finger joining provides a way to connect shorter lengths together to form arbitrarily long timber members. The finger joining process involves a series of successive steps as schematically shown in Figure 3. These involve the identification and removal of the defected portion of the specimen, as well as milling, gluing and pressing together the end-faces resulting from cutting off the defected part. Although finger joint technology has been available in UK since the 1960s, the described process is now mostly carried out in a fully automated fashion. Relevant information on performance requirements for finger joints are given in *BS EN 14080:2013 — Timber structures. Glued laminated timber and glued solid timber — Requirements.* [9].

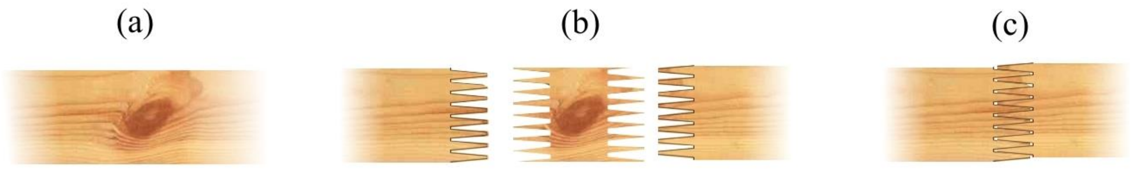


Figure 3: Finger joining techniques. Material imperfections such as knots are identified (a) and removed (b); the resulting timber ends are milled and glued together (c).

Although finger joining technique allows the production of arbitrarily long structural timber members, any increase in cross-sectional size above the limits dictated by the raw material (i.e., the log diameter) can only be achieved by glue-lamination technique.

Glue-laminated timber (also termed Glulam) is a fabrication method in which solid timber boards (laminates) are bonded together using adhesives, so to form structural members having one dimension much bigger than the other two (e.g., as for beams and columns). The longitudinal axis of glulam members is parallel to the grain's orientation of timber. The individual laminates are usually 20 mm to 50 mm thick and 1.5 m to 5.0 m long. If the final glulam member is longer than the individual laminates, these are end-jointed with finger joints at staggered locations. After applying glue on both sides of each laminate, the whole stack is placed into mechanical or hydraulic jigs, by which the required pressure for curing the glue is applied. It is also quite common to use glue-lamination to produce curved or tapered members. Thinner laminates (12 mm to 33 mm) are usually used for curved glulam beams, so that any residual stress arising from bending these laminates into shape is minimized.

The mechanical performances of glulam elements are mostly dependent on the strength and stiffness of the individual laminates (and type of glue being used). Two alternative options are usually available for the lay-up of the laminates:

— *Homogeneous glulam:* same-grade timber is used for all laminates.
— *Combined glulam:* higher-grade timber is used for the outer laminates compared to the inner layers.

Combined glue-lamination allow for a more efficient use of the material strength since the outer laminates experience higher stress levels under bending action.

There is no general limit on the number of laminates that can be laid-up and glued together — and hence no specific limitation exists on cross-sectional depth of glulam members. The maximum width of laminates is however 250 mm, therefore any additional increase in width for the glulam member requires alternative layup arrangements usually involving more than one laminate per layer (e.g., as shown in Figure 4). The reference standard for the design and manufacture of glulam is the *BS EN 14080:2013: Timber structures. Glued laminated timber and glued solid timber. Requirements.* [9]

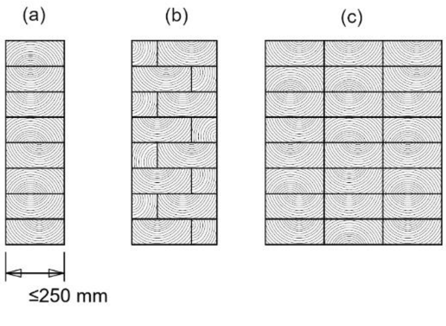


Figure 4: Cross-sectional layouts of glue laminated timber. Widths up to 250 mm can be achieved using one laminate per layer (a). Typical layout arrangements for widths > 250 mm are shown in (b) and (c).

Similar to glulam, cross-laminated timber (CLT) is fabricated by gluing timber laminates together. Unlike glulam, laminates are arranged as to make planar structural elements such as panels, slabs and plates. The laminates in two adjacent layers are oriented perpendicular to each other (see Figure 5). The reason for such an arrangement has to do with the peculiar nature of timber as a structural material. Timber is an anisotropic material (see section 4.1) therefore it shows great differences in terms of strength and stiffness depending on the orientation of loading (or bending axis) with respect to the grain direction. By alternating the orientation of laminates at 90° angles for adjacent layers, a two-way spanning component is provided, which enables for more uniform mechanical properties regardless of loading orientation (and bending axis). The timber grade used for CLT manufacturing is usually at least C24 (see section 4.2). Cross-sectional properties along the panel's main axes are calculated only considering the cross-sections of laminates parallel to the axis under consideration. Main structural applications for CLT are floor slabs and load-bearing walls. CLT shows a similar behavior to solid timber with regard to moisture-dependent deformations and stiffness. Similarity with solid timber is also shown in the way CLT behaves when dowel-type fasteners such as screws or bolts are used for connections.

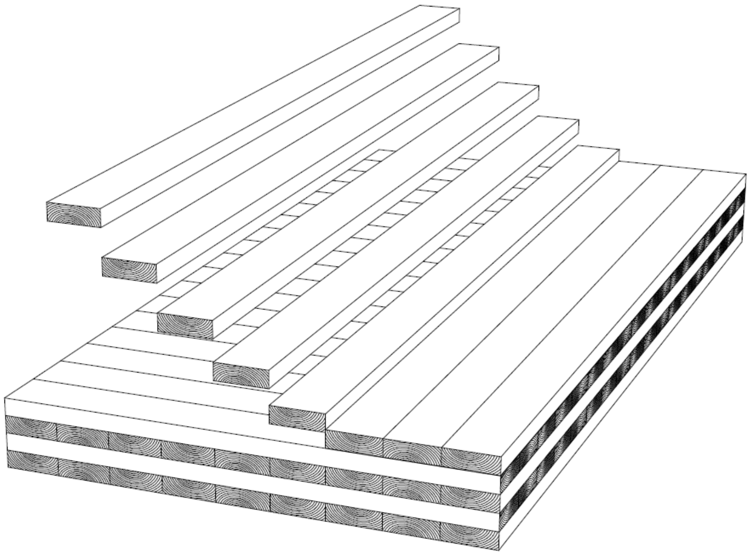


Figure 5: Arrangement of timber laminates in cross-laminated timber panels.

Maximum sizes for CLT panels usually depend on the manufacturer's own capacity. Panels up to 16 m in length and 3 m hight are currently available on the market, however access and transportation may be the critical issue for size rather than production limits. Rules for design and manufacture of cross-laminated timber structures are set in a recently released standard *BS EN 16351:2021. Timber structures. Cross laminated timber. Requirements* [10].

3.3 Other engineered wood products

Plywood is one of the most widely used engineered wood products (EWP) and one of the first to be invented. Plywood panels are made by bonding a series of veneer layers together using adhesive and then applying pressure during the curing phase. Veneers are very thin layers of wood (2 mm to 4 mm thick) obtained by rotary-peeling of wooden logs. These are preliminary debarked and then steamed (or soaked into hot water) to facilitate the peeling process. As for CLT, adjacent layers are oriented at 90° angles with respect to the grain direction. An odd number of layers are usually laid up to form plywood panels. This is to assures same grain orientation for the two external layers (also referred to as faces). A minimum of three layers are usually adopted for plywood panels. The external layers are always made of veneer whereas the internal layer (also called core) can either be veneer or sliced solid wood. Available maximum sizes for plywood panels are 1200 mm x 2400 mm or 1220 mm x 2440 mm.

Laminated Veneer Lumber (LVL) is another engineering wood product made from glued veneer laminates. Unlike plywood, all laminates are laid up with their grain orientation following the same direction, therefore obtaining structural members with orthotropic mechanical properties similar to that of solid timber. Strength capacity is however substantially higher compared to this latter, thanks to the lamination process which greatly reduces the size and distribution of natural defects within LVL members. For certain grade types of LVL some of the veneer layers are laid perpendicular to the longitudinal axis as this can further increase strength.

Oriented Strand Board (OSB) is an engineered wood product that comes into panels of different sizes (up to 2400 mm wide x 4800 mmm long) and thicknesses ranging from 8 mm to 25 mm. Similar to plywood, OSB is a composite material, it is made by gluing together thin wooden strands using water resistant adhesives and applying heat and pressure during the curing process. OSB is often adopted as a more cost-effective alternative to plywood in a range of applications, from roof decking to sheathing in timber framed walls, as well as for use in other composite products such as Structural Insulated Panels (SIPs) and I-joists. Characteristic values of material properties for OSB are set out in *BS EN 12369-1:2001 Wood-based panels. Characteristic values for structural design. OSB, particleboards and fireboards* [11].

4. Mechanical properties of timber and engineered wood products

4.1 Orthotropic and directionally isotropic models

Timber is an anisotropic material, meaning that its shows different mechanical properties depending on the orientation of the applied load. Specifically, the mechanical behavior of timber is best characterized by an orthotropic model, that is a particular kind of anisotropy where symmetry of behavior exists along mutually orthogonal axes. With the help of Figure 6, these three axes are so defined: the longitudinal axis L, corresponding to the grain (fiber) direction; the tangential axis T, corresponding to the tangent direction to the wood's annular rings; and the radial direction R, orthogonal to both L and T. To note how the orientation of both radial and tangential axes change as we move along the annular ring. To fully characterize such an orthotropic model, three moduli of elasticity (Young moduli) are required for each direction: E_L, E_T and E_R, as well as three shear moduli, one for each of the three reference planes: G_{LR}, G_{LT}, G_{RT}.

In general, these six material stiffness constants will vary among species and within samples, nonetheless some general relationships exist, expressed in terms of ratios between constants. These relationships are given by Bodig and Jayne [12] as follows:

$$\begin{gathered} E_L \div E_R \div E_T \approx 20 \div 1.6 \div 1 \\ G_{LR} \div G_{LT} \div G_{RT} \approx 10 \div 9.4 \div 1 \\ E_L \div G_{LR} \approx 14.1 \end{gathered} \tag{1}$$

Eqs. (1) show that tangential and radial Young moduli are approximately one order of magnitude smaller that E_L (the Young modulus parallel to the grain direction). Similarly, the shear modulus in the cross-sectional plane RT is one order of magnitude smaller than the two shear moduli along the LR and LT reference planes.

A further model approximation can therefore be made on the number of independent stiffness terms required. Specifically, one term to describe the elastic modulus parallel to the grain (E_0) and only one term to describe the elastic modulus for any direction orthogonal to the grain (E_{90}) which essentially correspond to all directions lying in the cross-sectional plane RT:

$$E_0 = E_L \qquad (2)$$
$$E_{90} \approx E_R \approx E_L$$

Similarly, one value of transverse shear modulus ($G_{transverse}$) can be used to represent the equivalent shear stiffness for any plane passing through the longitudinal direction L, as well as one value of rolling shear modulus ($G_{rolling}$) to indicate the shear stiffness term in the cross-sectional plane RT:

$$G_{transverse} \approx G_{LR} \approx G_{LT} \qquad (3)$$
$$G_{rolling} = G_{RT}$$

The orthotropic material model so simplified (with only two elastic moduli and two shear moduli) corresponds to a particular case of anisotropy also called *transverse isotropy*, which is the standard model underpinning design and analysis of timber structures, as further described in the next section.

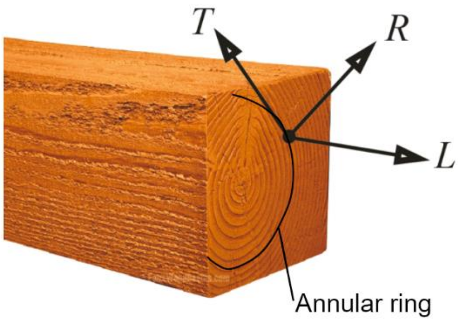


Figure 6: Orthotropic axes of timber: L = longitudinal axis; R = radial axis; T = tangential axis.

4.2 Strength, stiffness and density

In order for timber and its derivative products to be used for structural applications, its mechanical properties must be determined first. The characterization of mechanical properties of timber is termed *strength grading*, and it consists to evaluate the statistical distribution of strength properties within a population of size-specific sawn timber. Characteristic values of strength distributions (e.g., mean or 5-percentile) as well as their correlation with other properties vary among species and within species, depending on a range of specific factors, including climatic conditions. Rules for strength grading of timber are set out in the multi document *BS EN 14081 - Timber structures. Strength graded structural timber with rectangular cross section* [13].

To facilitate the design calculations and market availability of structural timber with certified properties, the concept of sorting strength grades into classes was first introduced in the UK in 1984. The main advantage of having a set of classes corresponding to predefined strength levels and other statistically correlated properties (e.g., stiffness and density) is to provide a consistent specification framework for both designers and suppliers. Engineers can specify the material properties in their design without having to preliminary check for availability of particular species. Similarly, producers/suppliers can source timber from different species, thus allocating it to the required strength class, as long as the material properties required for that class are met.

The standard *BS EN 338:2016 Structural timber — Strength classes* [14] provides strength, stiffness and density values for various strength classes of full-size solid timber. Strength classes for softwoods (C) and hardwoods (D) are reproduced here respectively in Table 5 and Table 6, based on values from *BS EN 338:2016.* Among the various strength, stiffness and density values reported in these two Tables, the bending strength, f_m, is empirically assessed using destructive bending tests on a statistically representative population sample. This means in practice that other strength properties (e.g., tensile/compressive strengths) for the same specimen can only be derived indirectly, conservatively based on the existing correlation between strengths, density and stiffnesses.

Table 5: Strength classes for solid timber softwoods according to BS EN 338:2016 [14].

	Class:	**C14**	**C16**	**C18**	**C20**	**C22**	**C24**	**C27**	**C30**	**C35**	**C40**	**C45**	**C50**
Strength properties [N/mm^2]													
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	45	50
Tension parallel	$f_{t,0,k}$	7.2	8.5	10	11.5	13	14.5	16.5	19	22.5	26	30	33.5
Tension perpendicular	$f_{t,90,k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Compression parallel	$f_{c,0,k}$	16	17	18	19	20	21	22	24	25	27	29	30
Compression perpendicular	$f_{c,90,k}$	2.0	2.2	2.2	2.3	2.4	2.5	2.5	2.7	2.7	2.8	2.9	3.0
Shear	$f_{v,k}$	3.0	3.2	3.4	3.6	3.8	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Stiffness properties [kN/mm^2]													
Mean modulus of elasticity parallel bending	$E_{m,0,mean}$	7.0	8.0	9.0	9.5	10.0	11.0	11.5	12.0	13.0	14.0	15.0	16.0
5-percentile modulus of elasticity parallel bending	$E_{m,0,k}$	4.7	5.4	6.0	6.4	6.7	7.4	7.7	8.0	8.7	9.4	10.1	10.7
Mean modulus of elasticity perpendicular	$E_{m,90,mean}$	0.23	0.27	0.30	0.32	0.33	0.37	0.38	0.40	0.43	0.47	0.50	0.53
Mean shear modulus	G_{mean}	0.44	0.50	0.56	0.59	0.63	0.69	0.72	0.75	0.81	0.88	0.94	1.00
Density [kg/m^3]													
5-percentile	ρ_k	290	310	320	330	340	350	360	380	390	400	410	430
Mean	ρ_{mean}	350	370	380	400	410	420	430	460	470	480	490	520

Table 6: Strength classes for solid timber hardwoods according to BS EN 338:2016 [14].

	Class:	**D18**	**D24**	**D27**	**D30**	**D35**	**D40**	**D45**	**D50**	**D55**	**D60**	**D65**	**D70**	**D75**	**D80**
Strength properties [N/mm^2]															
$f_{m,k}$		18	24	27	30	35	40	45	50	55	60	65	70	75	80
$f_{t,0,k}$		11	14	16	18	21	24	27	30	33	36	39	42	45	48
$f_{t,90,k}$		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
$f_{c,0,k}$		18	21	22	24	25	27	29	30	32	33	35	36	37	38
$f_{c,90,k}$		4.8	4.9	5.1	5.3	5.4	5.5	5.8	6.2	6.6	10.5	11.3	12.0	12.8	13.5
$f_{v,k}$		3.5	3.7	3.8	3.9	4.1	4.2	4.4	4.5	4.7	4.8	5.0	5.0	5.0	5.0
Stiffness properties [kN/mm^2]															
$E_{m,0,mean}$		9.5	10.0	10.5	11.0	12.0	13.0	13.5	14.0	15.5	17.0	18.5	20.0	22.0	24.0
$E_{m,0,k}$		8.0	8.4	8.8	9.2	10.1	10.9	11.3	11.8	13.0	14.3	15.5	16.8	18.5	20.2
$E_{m,90,mean}$		0.63	0.67	0.70	0.73	0.80	0.87	0.90	0.93	1.03	1.13	1.23	1.33	1.47	1.6
G_{mean}		0.59	0.63	0.66	0.69	0.75	0.81	0.84	0.88	0.97	1.06	1.16	1.25	1.38	1.50
Density [kg/m^3]															
ρ_k		475	485	510	530	540	550	580	620	660	700	750	800	850	900
ρ_{mean}		570	580	610	640	650	660	700	740	790	840	900	960	1020	1080

An additional strength class Table has been introduced in the latest version (2016) of *BS EN 338* and reproduced in Table 7 here. The table refers to tension strength classes (T) and is primarily aimed at solid timber graded for use in glulam making. For this case, tensile forces are more predominant than bending at the level of the individual laminate, therefore tensile strength values $f_{t,90}$ in Table 7 are derived empirically using destructive tensile tests, whereas bending strength is the indirectly estimated quantity, based on conservative modeling of the underlying correlation between variables.

Strength values in Table 5 to Table 7 are indicated with the symbol "*f*" followed by up to three subscripts. The first subscript indicates the type of strength component, namely: bending (*m*), pure tension (*t*), pure compression (*c*) and transverse shear (*v*). When three subscripts appear, the second subscript indicates the direction of stress. This can be either parallel to the grain direction (0) or perpendicular to the grain direction (90). All the strength values are expressed in terms of the 5-percetile of the statistical distribution, and this is indicated by the subscript *k*.

Similarly, the value upper bound value with 5-percentile frequency of occurrence is also provided for the elastic modulus parallel to the grain ($E_{0,k}$) as well as mean values of elastic modulus parallel and perpendicular to the grain ($E_{0,mean}$ and $E_{90,mean}$ respectively) and mean value of transverse shear modulus, G_{mean}. Densities are also expressed in terms of 5-percentile and mean value (ρ_k and ρ_{mean} respectively).

Table 7: Strength classes for solid timber softwoods (based on tension tests) according to BS EN 338:2016 [14].

	Class:	**T8**	**T9**	**T10**	**T11**	**T12**	**T13**	**T14**	**T14.5**	**T15**	**T16**	**T18**	**T21**	**T22**	**T24**	**T26**	**T27**	**T28**	**T30**
Strength properties [N/mm²]																			
Bending	$f_{m,k}$	13.5	14.5	16	17	18	19.5	20.5	21	22	23	25.5	29	30.5	33	35	36.5	37.5	40
Tension parallel	$f_{t,0,k}$	8	9	10	11	12	13	14	14.5	15	16	18	21	22	24	26	27	28	30
Tension perpendicular	$f_{t,90,k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Compression parallel	$f_{c,0,k}$	16	17	17	18	19	20	21	21	21	22	23	25	26	27	28	29	29	30
Compression perpendicular	$f_{c,90,k}$	2.0	2.1	2.2	2.2	2.3	2.4	2.5	2.5	2.5	2.6	2.7	2.7	2.7	2.8	2.9	2.9	2.9	3.0
Shear	$f_{v,k}$	2.8	3.0	3.2	3.4	3.6	3.8	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Stiffness properties [kN/mm²]																			
Mean modulus of elasticity parallel tension	$E_{t,0,mean}$	7.0	7.5	8.0	9.0	9.5	10.0	11.0	11.0	11.5	11.5	12.0	13.0	13.0	13.5	14.0	15.0	15.0	15.5
5-percentile modulus of elasticity parallel tension	$E_{t,0,k}$	4.7	5.0	5.4	6.0	6.4	6.7	7.4	7.4	7.7	7.7	8.0	8.7	8.7	9.0	9.4	10.1	10.1	10.4
Mean modulus of elasticity perpendicular tension	$E_{t,90,mean}$	0.23	0.25	0.27	0.30	0.32	0.33	0.37	0.37	0.38	0.38	0.40	0.43	0.43	0.45	0.47	0.50	0.50	0.52
Mean shear modulus	G_{mean}	0.44	0.47	0.50	0.56	0.59	0.63	0.69	0.69	0.72	0.72	0.75	0.81	0.81	0.84	0.88	0.94	0.94	0.97
Density [kg/m³]																			
5-percentile	ρ_k	290	300	310	320	330	340	350	350	360	370	380	390	390	400	410	410	420	430
Mean	ρ_{mean}	350	360	370	380	400	410	420	420	430	440	460	470	470	480	490	490	500	520

4.3 Engineered wood products

For characteristic material properties of engineered wood products, reference in general has to be made to manufacturer data. Characteristic values for structural plywood panels can be found in *BS EN 12369-1:2001 Part 2* [15], whereas *BS EN 12369-1:2011 Part 1* [11] provides characteristic values for structural OSB panels that are compliant with specifications set out in *BS EN 300:2006* [16]. In this latter, OSB panels are classified into four types as follows:

- OSB type 1: general purpose boards, not suitable for structural use.
- OSB type 2: load-bearing boards, suitable only for use in dry conditions (service class 1)
- OSB type 3: load-bearing boards, suitable for use in humid conditions (service classes 1 and 2)
- OSB type 4: heavy duty load-bearing boards, suitable for use in humid conditions (service classes 1 and 2)

An extract from *BS EN 12369-1:2011 Part 1* is shown in Table 8 here for structural OSB type 3 panels, thus reporting characteristic (5-percentile) strengths and density values and (mean) stiffnesses for different thickness ranges. According to the standard, 5-percentile stiffness values can be conservatively taken as equal to 85% of the mean values given in the Table.

Wooden strands in the external layers of OSB panels tend to be oriented parallel to the long side of the panel, therefore OSB panels show anisotropic behavior (similar to solid timber) when loaded in different directions. This is evident from Table 8 which provides strength and stiffness values along the direction parallel to the grain (0) as well as orthogonal to the grain (90, corresponds to the panel's short side). Furthermore, two distinct sets of terms are provided for shear, specifically: *panel shear* terms (f_v *and* G_v) referring to the shear plane parallel to the panel; and rolling shear terms f_r and G_r (also referred to as *planar shear* terms) which relates to the shear plane cutting perpendicularly through the panel (i.e., the panel's cross-section).

Table 8: Characteristic values of material properties for OSB type 3 panels according to BS EN 12369-1:2011 Part 1 [11].

Thickness [mm]	**Characteristic density [kg/m³] and strength [N/mm²] values**								
	Density	**Bending**		**Tension**		**Compression**		**Panel shear**	**Planar shear**
t_{nom}	ρ	f_m		f_t		f_c		f_v	f_r
		0	**90**	**0**	**90**	**0**	**90**		
$6 < t_{nom} \leq 10$	550	18.0	9.0	9.9	7.2	15.9	12.9	6.8	1.0
$10 < t_{nom} \leq 18$	550	16.4	8.2	9.4	7.0	15.4	12.7	6.8	1.0
$18 < t_{nom} \leq 25$	550	14.8	7.4	9.0	6.8	14.8	12.4	6.8	1.0

Thickness [mm]	**Mean stiffness values [N/mm²]**								
		Bending		**Tension**		**Compression**		**Panel shear**	**Planar shear**
t_{nom}		E_m		E_t		E_c		G_v	G_r
		0	**90**	**0**	**90**	**0**	**90**		
$6 < t_{nom} \leq 10$		4930	1980	3800	3000	3800	3000	1080	50
$10 < t_{nom} \leq 18$		4930	1980	3800	3000	3800	3000	1080	50
$18 < t_{nom} \leq 25$		4930	1980	3800	3000	3800	3000	1080	50

5. Design rules for ultimate limit states

The design values of a material or product property, X_d, is derived from the corresponding characteristic values, X_k, using the following relationship, adapted from BS EN 1990 [17]:

$$X_d = k_{mod}\frac{X_k}{\gamma_M} \tag{4}$$

with the partial safety factor γ_M accounting for any unfavorable deviation of the material property from the underlying statistical distribution, whereas the modification factor k_{mod} is introduced to account for load duration and moisture content; two factors these that have great influence on strength and stiffness (see sections 2.3 and 2.4). The EC5 [1] provides a value of $\gamma_M = 1.3$ for solid timber and particleboards, whereas a value of 1.25 is recommended for glulam. A value of $\gamma_M = 1.2$ is given instead for LVL, plywood and OSB. Same values are also recommended in the UK NA to EC5 [5] except a further distinction is made between solid timber individually marked with a grade stamp ($\gamma_M = 1.3$) and solid timber where the grade is stamped on the package ($\gamma_M = 2.0$). Recommended values in EC5 (BS EN 1995-1-1) for the modification factor k_{mod} as a function of service class and load-duration are reproduced here in Table 9 for solid timber and some other EWPs.

It is important to note that the *characteristic* value of material property to insert in Eq. (4) will be the *5-percentile* value when dealing with strength properties, whereas the *mean* value should be used to derive design values of the member stiffness as function of the material stiffness (E and G).

For design verifications involving the properties of two different materials (e.g., checking for the strength of a nailed connection between OSB and sawn timber) an equivalent modification factor is taken as follows:

$$k_{mod} = \sqrt{k_{mod,1} + k_{mod,2}} \tag{5}$$

Table 9: Values for the modification factor for load duration and service class (k_{mod}) based on values from BS EN 1995-1-1 [1].

Material	**Standard**	**Service class**	**Load-duration class**				
			Permanent	**Long term**	**Medium term**	**Short term**	**Instantaneous**
Solid timber	EN 14081-1	1	0.60	0.70	0.80	0.90	1.10

		2 3	0.60 0.50	0.70 0.55	0.80 0.65	0.90 0.70	1.10 0.90
Glue laminated timber	EN 14080	1 2 3	0.60 0.60 0.50	0.70 0.70 0.55	0.80 0.80 0.65	0.90 0.90 0.70	1.10 1.10 0.90
LVL	EN 14374 and EN 14279	1 2 3	0.60 0.60 0.50	0.70 0.70 0.55	0.80 0.80 0.65	0.90 0.90 0.70	1.10 1.10 0.90
Plywood	(Type 1) EN 636-1	1	0.60	0.70	0.80	0.90	1.10
	(Type 2) EN 636-2	2	0.60	0.70	0.80	0.90	1.10
	(Type 3) EN 636-3	3	0.50	0.55	0.65	0.70	0.90
OSB	(Type 2) EN 300	1	0.30	0.45	0.65	0.85	1.10
	(Type 3, Type 4) EN 300	1	0.40	0.50	0.70	0.90	1.10
	(Type 3, Type 4) EN 300	2	0.30	0.40	0.55	0.70	0.90
Particleboard	(Type P4, Type P5) EN 312	1	0.30	0.45	0.65	0.85	1.10
	(Type P5) EN 312	2	0.20	0.30	0.45	0.60	0.8
	(Type P6, Type P7) EN 312	1	0.40	0.50	0.70	0.90	1.10
	(Type P7) EN 312	2	0.30	0.40	0.55	0.70	0.90

After deriving the design value of material properties based on Eq. (4), further reductions (or enhancement) of the design value may be required to account for additional (and case-specific) effects. These partial factors are introduced in the following sub-section for the relevant design cases.

5.1 Bending

For design cases involving bending only around the *major* cross-sectional axis, *y*, two distinct cases may arise:

- Relative slenderness for bending ≤ 0.75: there is no risk of instability. The critical condition is dictated by the material strength.
- Relative slenderness for bending > 0.75: Instability may occur. The beam can fail via lateral torsional buckling (LTB) mode.

In the first case, the following relation must be verified:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} \leq 1 \tag{6}$$

where $\sigma_{m,y,d}$ is the design bending stress, i.e., the maximum normal stress at the fibers furthest away from the neutral axis = y.

The same equation applies for checking the member's resistance for bending around the *minor* cross-sectional axis, *z*, by replacing the corresponding subscripts. For rectangular cross-sections like the one shown in Figure 7, LTB is only relevant for bending action around the major (*y*) axis, meaning that the relative slenderness for bending will always be ≤ 0.75 around the minor (*z*) axis.

In the second case, that is, when the relative slenderness for bending is bigger than 0.75, LTB can be the determining criteria dictating the bending moment resistance of the member. Therefore, a modification factor, k_{crit}, is introduced to account for the reduction in bending strength induced by lateral torsional instability:

$$\frac{\sigma_{m,d}}{k_{crit} f_{m,d}} \leq 1 \tag{7}$$

Where:

$$k_{crit} = \begin{cases} 1.56 - 0.75\lambda_{rel,m} & (For\ \ 0.75 < \lambda_{rel,m} \leq 1.4) \\ 1/\lambda_{rel,m}^2 & (For\ \ 1.4 < \lambda_{rel,m}) \end{cases} \tag{8}$$

For $\lambda_{rel,m} \leq 0.75$ the modification factor k_{crit} is effectively se to unity. The relative slenderness for bending term, $\lambda_{rel,m}$, is obtained from the following EC5 Equation:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{9}$$

with $f_{m,k}$ being the characteristic (5-percentile) bending strength and $\sigma_{m,crit}$ the critical bending stress. This latter can be derived as follows for solid wood rectangular beams:

$$\sigma_{m,crit} = \frac{0.78b^2}{hl_{ef}} E_{0.05} \tag{10}$$

whereas the following applies to hardwoods, glulam and LVL beams with rectangular sections:

$$\sigma_{m,crit} = \frac{\pi b^2}{hl_{ef}} \sqrt{E_{0.05} G_{0.05} \left(1 - 0.63\frac{b}{h}\right)} \tag{11}$$

The terms $E_{0.05}$ and $G_{0.05}$ represent the 5-percentile elastic modulus and shear modulus, respectively. The effective length term, l_{ef}, will be function of the beam's end conditions and distribution of applied loads. Values for l_{ef} for both simply supported and cantilevered beams are given in EC5 (expressed as a ratio of the span) and reproduced here in

Table 10. It is important to note that these l_{ef} values are derived based on the assumption that the beam is fully restrained against lateral (torsional) movements at the supports as shown in Figure 8.

Table 10: Effective length as ratio of the span, l. Reproduced from BS EN 1995-1-1 [1].

End-conditions	**Loading type**	$(\boldsymbol{l_{ef}/l})^{\boldsymbol{a}}$
Simply supported	Constant moment	1.0
	Uniformly distributed load	0.9
	Point load at mid-span	0.8
Cantilever	Uniformly distributed load	0.5
	Point load at free end	0.8
$^{\boldsymbol{a}}$ The ratio between effective length $\boldsymbol{l_{ef}}$ and the span $\boldsymbol{l}$ is valid for a beam with torsionally restrained supports and loaded at the center of gravity. If the load is applied at the compression edge of the beam, the tabulated value of $\boldsymbol{l_{ef}}$ must be increased by 2*h* and may be decreased by 0.5*h* for a load at the tension edge of the beam.		

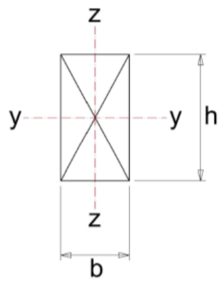


Figure 7: Cross-section reference axes of a rectangular beam.

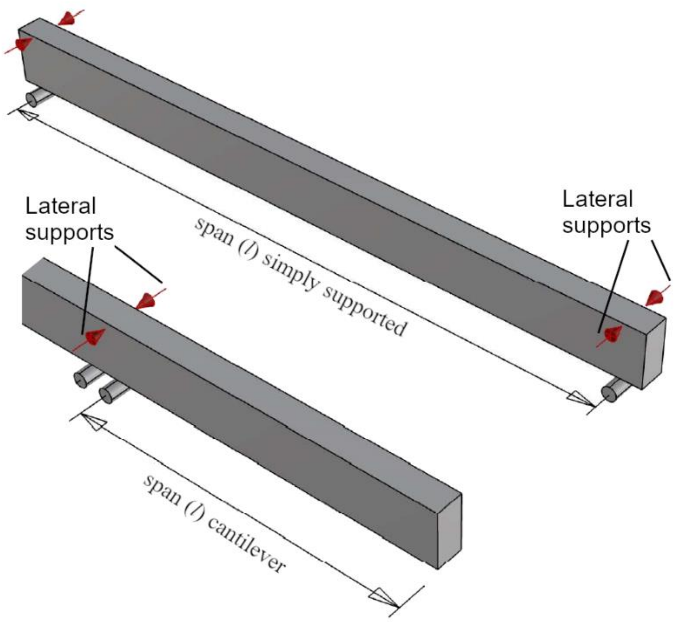


Figure 8: Simply supported beam and cantilever beam are assumed as fully restrained against lateral movement by lateral supports.

The design bending stress term in Eqs. (6) and (7) is obtained from the ratio between design bending moment, M_d, and associated section modulus W:

$$\sigma_{m,d} = \frac{M_d}{W} \tag{12}$$

For rectangular sections subject to bending around the strong axis, we have $W_y = (bh^2)/6$.

The design bending strength term $f_{m,d}$ in Eqs. (6) and (7) is obtained according to Eq (4), however two additional modification factors (k_{sys} and k_h) may apply when converting the characteristic value of bending strength into a design value:

$$f_{m,d} = k_{mod} k_{sys} k_h \frac{f_{m,k}}{\gamma_M} \tag{13}$$

The term k_{sys} is taken equal to 1.1 and it applies for the design of flooring, when there are at least four beams equally spaced at a distance of no more than 610 mm (centre-to-centre). In this case, the stiffness of floor panels (or planks) connected to the timber beams enables for the applied load to be partially redistributed to the surrounding beams. The factor is taken equal to unity for all other cases. The other factor, k_h, is introduced to account for the weakening effect (due to natural defects) of increasing the cross-section. Specifically, for rectangular solid timber in bending around the *y* axis and a depth (*h*) bigger than 150 mm: $k_h = 1.0$, whereas for *h* < 150 mm the following applies:

$$k_h = min\begin{cases}(150/h)^{0.2} \\ 1.3\end{cases} \tag{14}$$

Eq. (15) applies instead for glulam beams of rectangular section and a depth *h* < 600 mm:

$$k_h = min\begin{cases}(600/h)^{0.1} \\ 1.1\end{cases} \tag{15}$$

And Eq. (16) is for LVL beams with a depth *h* < 300 mm, where the exponent *s* is derived from tests according to requirements from *BS EN 14374* [18].

$$k_h = min\begin{cases}(300/h)^{s} \\ 1.2\end{cases} \tag{16}$$

For glulam members with a depth bigger than 600 mm (300 mm for LVL) the factor for effect size k_h is set to 1.0.

When dealing with design cases involving by-axial bending, and there is no risk of lateral torsional buckling (i.e., $\lambda_{rel,m} \leq 0.75$) the verification must account for the combined effect of bending stresses around both *y* and *z* axes:

$$\begin{cases}\dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \\ k_m \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1\end{cases} \tag{17}$$

The factor k_m in Eq. (17) accounts for the effect of stress redistribution on bending strength, as well as for the effect of location of imperfections within the cross-section. Values for k_m are set equal to unity in EC5 except for solid timber, glulam and LVL with a rectangular section. In such cases $k_m =$ 0.7. For situations involving axial compression and combined bending, all three components of normal stress σ shall be included in the verification:

$$\begin{cases}\left(\dfrac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \\ \left(\dfrac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1\end{cases} \tag{18}$$

The terms $\sigma_{c,0,d}$ and $f_{c,0,d}$ are respectively the design stress and design strength parallel to the grain.

5.2 Shear

The EC5 sets the following inequality criteria when dealing with shear verifications at Ultimate Limit States:

$$\tau_d \leq f_{v,d} \tag{19}$$

Eq. (19) applies to shear stress components, τ, acting either in the longitudinal plane (i.e., with one component parallel to the grain) or in the cross-sectional plane (rolling shear). For beams with a rectangular cross-section as shown in Figure 7, the design shear stress is derived from the vertical design shear force V_d:

$$\tau_d = \frac{3V_d}{2b_{ef}h} \tag{20}$$

where the effective width term b_{ef} (introduced to account for the detrimental effect of cracks on shear strength) is derived from the nominal width using the factor k_{cr}:

$$b_{ef} = k_{cr} b \tag{21}$$

Numerical values for k_{cr} are set to 0.67 for Solid timber and glulam, whereas a value of 1.0 is used for LVL and wood-based panels according to the UK NA to EC5.

For simply-supported beams, the maximum value of design shear force V_d will be near the supports, therefore if a notch is present (see Figure 9) the reduction in cross-sectional depth must be accounted for, hence the term h in Eq. (20) is to be replaced by the effective depth term h_{ef}:

$$\tau_d = \frac{3V_d}{2b_{ef}h_{ef}} \tag{22}$$

Furthermore, if the notch is on the same side as the support (e.g., as shown in Figure 9-(a)) a reduction factor k_v is introduced in Eq. (19) to account for concentrations of stress generated by the notch which can lead to propagation of cracks along the beam:

$$\tau_d \leq k_v f_{v,d} \tag{23}$$

The factor k_v is given by the following empirically-based formula:

$$k_{\mathrm{v}} = min \begin{cases} 1.0 \\ \dfrac{k_{\mathrm{n}}\left(1 + \dfrac{1.1 i^{1.5}}{\sqrt{h}}\right)}{\sqrt{h}\left[\sqrt{\alpha(1-\alpha)} + 0.8\dfrac{x}{h}\sqrt{\dfrac{1}{\alpha} - \alpha^2}\right]} \end{cases} \tag{24}$$

Where the term $i = b/(h - h_{ef})$ represents the notch inclination as shown in Figure 9-(a) (e.g., $i = 0$ for a 90° notch) whereas the term x is the distance (in mm) from the centroid support to the corner of the notch, and k_{v} is a factor that is equal to 4.5 for LVL, 5.0 for solid timber and 6.5 for glulam. Lastly, $\alpha = h_{ef}/h$.

The design shear strength term in Eqs (19) and (23) is derived from the characteristic (5-percentile) shear strength, $f_{v,k}$, as follows:

$$f_{\mathrm{v,d}} = k_{mod} k_{sys} \frac{f_{v,k}}{\gamma_M} \tag{25}$$

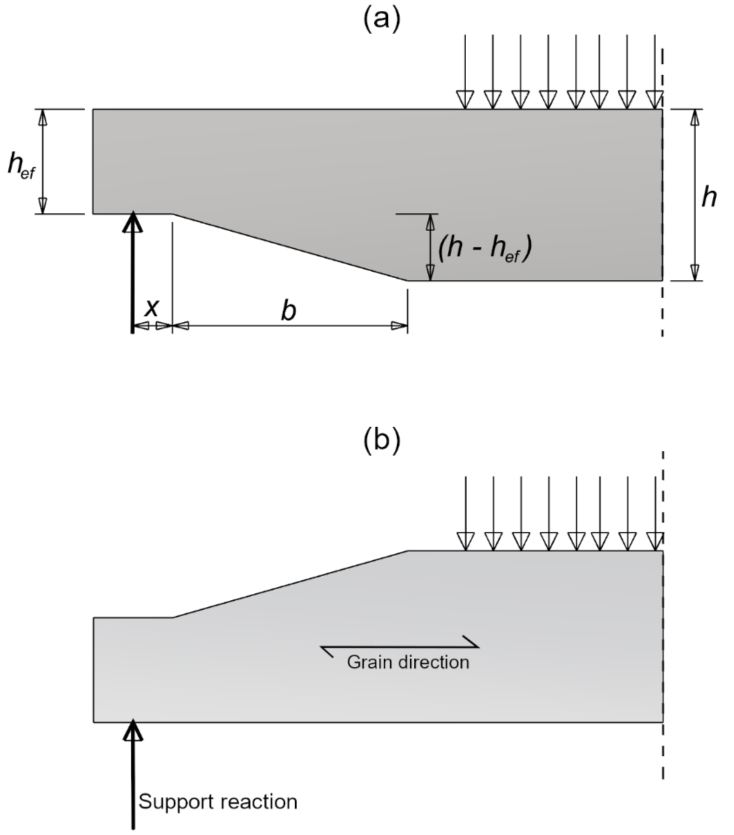


Figure 9: Beams with end notches: (a) notch on the same side as the support; (b) notch on the opposite side of the support.

5.3 Compression perpendicular to the grain

When timber is loaded orthogonally to the grain direction, the compressive strain ($\varepsilon_{c,90}$) will increase linearly with the increase in compressive stress ($\sigma_{c,90}$) according to the young modulus relationship $E_{c,90} = \sigma_{c,90}/\varepsilon_{c,90}$.

This linear relationship will hold up to the point at which the cylindrical walls of the individual wooden fibres will start to collapse and hence a drop in stiffness with large strain deformations will take place (see Figure 10).

If, after such a drop in stiffness, the applied compressive load is further increased, the wall cells will eventually be fully squeezed and compacted together, therefore they will be able to withstand increasingly higher loads, with strain deformations exceeding 30% at this point. To control for such inelastic deformations, the value of characteristic compressive strength orthogonal to the grain ($f_{c,90,k}$) as given in *BS EN 338* (e.g., see Table 5 in here) is determined using a 1% strain limit. Design values for compressive strength orthogonal to the grain may be increased if larger strains do not represent a critical issue. This is accounted for by introducing the partial factor $k_{c,90}$ when verifying the stress-to-strength limit ratio:

$$\frac{\sigma_{c,90,d}}{k_{c,90} f_{c,90,d}} \leq 1 \tag{26}$$

where:

$$f_{c,90,d} = k_{mod} k_{sys} \frac{f_{c,90,k}}{\gamma_M} \tag{27}$$

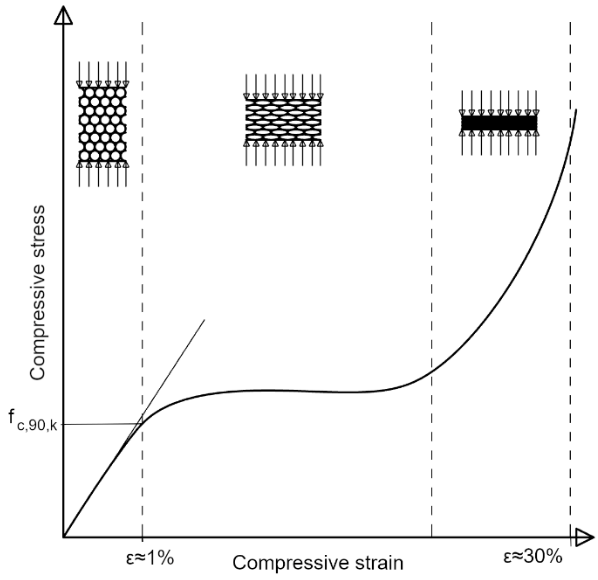


Figure 10: Stress-strain relationship for timber loaded in compression orthogonal to the grain direction.

The design compressive stress orthogonal to the grain, $\sigma_{c,90,d}$, is obtained from the ratio between the design bearing load, V_d, and the effective bearing area A_{ef}:

$$\sigma_{c,90,d} = \frac{V_d}{A_{ef}} = \frac{V_d}{b\ell_{ef}} \tag{28}$$

where b and ℓ_{ef} are the bearing width and the effective bearing length (i.e., parallel to the grain), respectively. With reference to Figure 11, the effective bearing length (ℓ_{ef}) should be taken based on the actual bearing length (ℓ) as follows:

$$\ell_{ef} = \ell + 2 \cdot min \begin{cases} 300\ mm \\ a \\ \ell \\ \ell_1/2 \end{cases} \tag{29}$$

For instance, for bearing load near the edge the effective bearing length is taken equal to the actual bearing length when a = 0.

The factor $k_{c,90}$ in Eq. (26) is usually set equal to unity unless certain conditions apply. These conditions are illustrated as follows.

— For members lying on a *continuous* support and where the distance between bearing points (i.e., ℓ_1 in Figure 11-(a)) is bigger than 2h, then $k_{c,90} = 1.25$ and 1.5 for solid softwood and glulam softwood, respectively.
— For members lying on *discrete* supports where the distance from the support to the nearest bearing point (ℓ_1 in Figure 11-(b)) is bigger than 2h, then $k_{c,90} = 1.5$ and 1.75 for solid softwood and glulam softwood, respectively.

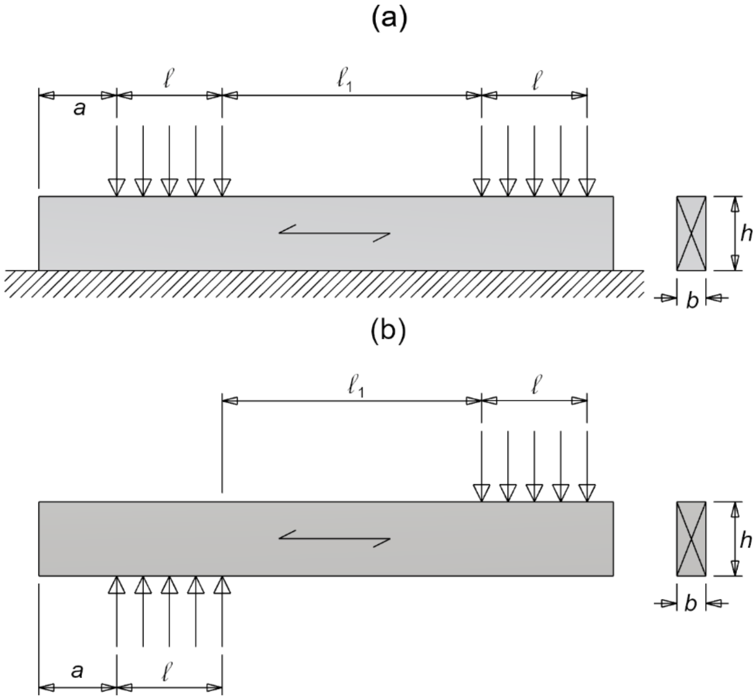


Figure 11: Timber members subject to compression orthogonal to the grain direction. (a): member lying on continuous supports. (b) member lying on discrete supports.

5.4 Compression parallel to the grain

For timber members that are axially loaded in compression, flexural buckling may be the critical parameter to consider when assessing the resistance for axial compression. A preliminary step is therefore required to evaluate the member's slenderness ratio, λ, which is taken as the ratio between the effective length, L_e, and the radius of gyration of the cross-section, *i*:

$$\lambda = \frac{L_e}{i} \tag{30}$$

Eq. (30) applies for both principal axes (*y* and *z*) of any given cross-section. Specifically for rectangular sections like the one shown in Figure 7, the two slenderness ratios λ_y and λ_z are obtained as follows:

$$\lambda_y = \frac{L_{e,y}}{h\sqrt{12}} \quad ; \quad \lambda_z = \frac{L_{e,z}}{b\sqrt{12}} \tag{31}$$

From Euler's theory of elastic buckling, the effective length represents the distance (along the member's length) between two adjacent points of contra-flexure, that is, the points along the buckled member where bending moment is null. The effective length will therefore be function of the actual member's length, *L*, and support conditions. For instance, for a column pinned at both ends: $L_e = L$; whereas for a column with positional restraints (pinned) as well as rotational restrains (clamped) at both ends: $L_e = 0.7L$.

The approach set out in EC5 (*BS EN 1995*) to assess the risk of buckling failure is to first calculate the relative slenderness ratio around both principal axes:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \quad ; \quad \lambda_{rel,z} = \frac{\lambda_z}{\pi}\sqrt{\frac{f_{c,0,k}}{E_{0.05}}} \tag{32}$$

and then check which of the following two cases applies:

— $\lambda_{rel,y}$ and $\lambda_{rel,z}$ are both ≤ 0.3
— $\lambda_{rel,y}$ and (or) $\lambda_{rel,z}$ are (is) > 0.3

In the first case, the member is stocky enough for buckling not to occur. The critical condition is therefore dictated by the strength of material; hence the following inequality must be verified:

$$\sigma_{c,0,d} \leq f_{c,0,d} \tag{33}$$

where the design compressive stress, $\sigma_{c,0,d}$, is taken as the ratio between compressive design load, N_d, and cross-sectional area. The design compressive strength, $f_{c,0,d}$, is instead obtained from the characteristic value of strength parallel to the grain:

$$f_{c,0,d} = k_{mod} k_{sys} \frac{f_{c,0,k}}{\gamma_M} \tag{34}$$

Noting that the k_{sys} factor in Eq. (34) only applies when the compression member is part of a load-sharing system. Conversely, $k_{sys} = 1.0$ when dealing with single columns. Furthermore, Eq. (18) should be used instead of Eq. (33) if the member is subject to axial compressive load and combined bending.

In the second case, that is when either $\lambda_{rel,y}$ or $\lambda_{rel,z}$ (or both) is bigger than 0.3, a strength reduction factor, k_c, is introduced to account for the member's instability under compression:

$$\sigma_{c,0,d} \leq k_{c,y} f_{c,0,d} \quad ; \quad \sigma_{c,0,d} \leq k_{c,z} f_{c,0,d} \tag{35}$$

The following equations apply when deriving instability factors for solid timber, glulam and LVL:

$$\begin{aligned} k_{c,y} &= \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}} \\ k_{c,z} &= \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}} \end{aligned} \tag{36}$$

where:

$$\begin{aligned} k_y &= 0.5\left[1 + \beta_c\left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right] \\ k_z &= 0.5\left[1 + \beta_c\left(\lambda_{rel,z} - 0.3\right) + \lambda_{rel,z}^2\right] \end{aligned} \tag{37}$$

with the factor β_c introduced to account for the effect of out-of-straightness imperfections. For members compliant with straightness limits given in *BS EN 1995* (Section 10), β_c is taken equal to 0.2 for solid timber and equal to 0.1 for glulam and LVL. Lastly, if the member is subject to axial compressive load and combined bending (and $\lambda_{rel,y}$ or $\lambda_{rel,z}$ > 0.3) the relevant expression from Eqs. (38) should be used instead of Eqs. (35):

$$\begin{cases} \dfrac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \\ \dfrac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \dfrac{\sigma_{m,y,d}}{f_{m,y,d}} + \dfrac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \end{cases} \quad (38)$$

5.5 Tension

Members subject to tensile loads along the main axis will experience tensile stress parallel to the grain direction. Therefore, the design criterion is to check that the design tensile stress, $\sigma_{t,0,d}$, is lower than the design tensile strength $f_{t,0,d}$:

$$\sigma_{t,0,d} \leq f_{t,0,d} \quad (39)$$

where:

$$\sigma_{t,0,d} = \frac{N_d}{A_{net}} \quad ; \quad f_{t,0,d} = k_{mod} k_{sys} k_h k_\ell \frac{f_{t,0,k}}{\gamma_M} \quad (40)$$

with N_d being the design tensile load while A_{net} is the cross-sectional area accounting for any area reduction at the connections (i.e., due to holes). The factor k_{sys} only applies when the tensile member is part of a load-sharing system ($k_{sys} = 1$ otherwise). Also, for the size factor k_h in Eq. (40) the same values as given in Eqs. (14) to (16) apply, taking *h* as the largest sectional dimension. The length factor k_ℓ only applies for LVL members ($k_\ell = 1$ for solid timber and glulam) and it is taken as follows:

$$k_\ell = min \begin{cases} (3000/\ell)^{s/2} \\ 1.1 \end{cases} \quad (41)$$

5.6 Torsion

The design condition for verifications at ultimate limit states of members subject to torsion is given as follows:

$$\tau_{tor,d} \leq k_{shape} f_{v,d} \quad (42)$$

The maximum design shear stress due to torsion, $\tau_{tor,d}$ is obtained from linear elasticity theory:

$$\tau_{tor,d} = \frac{2T_d}{\pi r^3} \quad ; \quad \tau_{tor,d} = \frac{T_d}{k_2 h b^2} \quad (43)$$

where the first of Eqs. (43) applies to circular sections whereas the second of Eqs. (43) applies to rectangular sections. T_d is the design torque, while r, h and b are (respectively) the cross-sectional radius, depth and width. Numerical values for the factor k_2 are given in Table 11 for different ratios *h/b*.

The design shear strength is derived based on the characteristic (5-percentile) shear strength $f_{v,k}$:

$$f_{v,d} = k_{mod} \frac{f_{v,k}}{\gamma_M} \quad (44)$$

whereas the shape factor k_{shape} is taken equal to 1.2 for circular sections. The following expression applies instead for rectangular sections:

$$k_{shape} = min\begin{cases} 1 + 0.15\left(\frac{h}{b}\right) \\ 2.0 \end{cases} \tag{45}$$

Table 11: values for the factor k_2 as function of *h/b*.

h/b	k_2
1.0	0.208
1.2	0.219
1.5	0.231
2.0	0.246
3.0	0.267
5.0	0.291
∞	0.333

6. Design rules for serviceability limit states

The EC5 requirement for serviceability limit states (SLS) is to make sure that the structural system behaves in a satisfactory way with regard to its normal use. This effectively translates in assuring adequate performances in terms of deflections and vibrations.

When dealing with SLS, the partial safety factors for loadings (γ_G and γ_Q) and material properties (γ_M) are taken equal to unity.

6.1 Deflections

When dealing with deformations of structural timber members, there are several components to account for. Specifically for floor systems and beams in general, the critical serviceability aspects will be about their vertical deflection. The deflection of timber beams is made of several components, specifically:

— w_c = in-built camber (if applicable)
— w_{inst} = instantaneous deflection (i.e., the elastic deflection occurring immediately after design loads are being applied)
— w_{creep} = long-term deflection (i.e., the non-reversible deflection arising as a result of loadings sustained over the long term)
— w_{fin} = final deflection (combination of both instantaneous and long-term deflection)
— $w_{net,fin}$ = net final deflection (combination of both instantaneous and long-term deflection excluding any in-built camber)

With the help of Figure 12, the net final deflection is given as follows:

$$w_{net,fin} = w_{inst} + w_{creep} - w_c \tag{46}$$

Guidelines on limiting values for the net final deflections are given in the UK NA to EC5 [5] and reported here in Table 12.

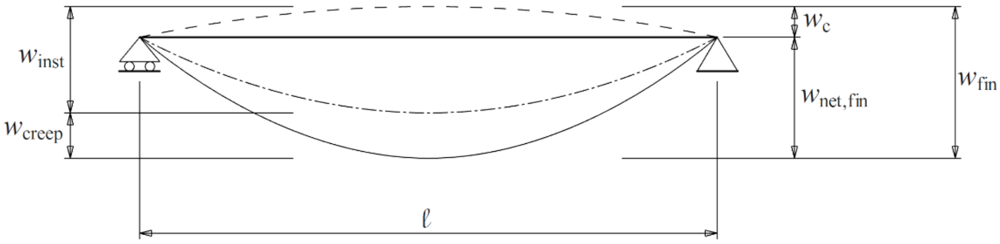


Figure 12: Components of deflection for a simply-supported beam.

Table 12: Limiting values of deflection for beams according to the UK NA to EC5 [5].

Type of member	**Limiting value for net final deflections, $w_{net,fin}$**	
	Beam of length ℓ spanning between two supports	Cantilever of length ℓ
Roof or floor members with a plastered or plasterboard ceiling	ℓ/250	ℓ/125
Roof or floor members without a plastered or plasterboard ceiling	ℓ/150	ℓ/75

The design loading to consider when calculating the instantaneous deflection w_{inst} should be based on the characteristic combination of actions at SLS as given in Eq. (6.14b) of *BS EN 1990* [17], and mean values for the modulus of elasticity and shear modulus (respectively: $E_{0,mean}$ and G_{mean}) should be used. The net final deflection $w_{net,fin}$ is obtained using Eq. (46) with w_{creep} taken as follows:

$$w_{creep} = k_{def} w_{inst} \tag{47}$$

Values for the deformation factor k_{def} are given in EC5 and reported in here in Table 13 for various timber materials and service classes.

Table 13: values for the deformation factor k_{def} based on *BS EN 1995* [1].

Material	**Values for k_{def}**		
	Service class 1	Service class 2	Service class 3
Solid timber	0.6	0.8	2.0
Glulam	0.6	0.8	2.0
LVL	0.6	0.8	2.0
Plywood (EN 636-1)	0.8	-	-
Plywood (EN 636-2)	0.8	1.0	-
Plywood (EN 636-3)	0.8	1.0	2.5
OSB (type 2)	2.25	-	-
OSB (type 3 and type 4)	1.50	2.25	-

6.2 Vibrations

Excessive vibrations, e.g., from floors, can have an impact on serviceability requirements. Specifically, two main source of vibrations that should be considered are:

- Machine-induced vibrations
- Footstep-induced vibrations

Reference limits to assure for human comfort, when exposed to machine-induced vibrations, are set out in *ISO 2631:2* [19] along with guidelines concerning the evaluation methods.

Discomfort induced by footstep-induced vibrations is covered in EC5 with regard to residential floors. Specifically, the fundamental frequency of the floor is required to be above 8 Hz. A formula is also provided for an approximate evaluation of the fundamental frequency, f_1:

$$f_1 = \frac{\pi}{2\ell^2}\sqrt{\frac{(EI)_\ell}{m}} \tag{48}$$

Eq. (48) only applies to rectangular floors simply supported on all four sides and with timber beams (or joists) spanning in the ℓ direction. The terms are:

- ℓ: the beams' design span in meters.
- $(EI)_\ell$: the *equivalent plate* flexural stiffness of the floor (in Nm^2/m). For traditional timber floor construction with floorboard panels or planks supported by beams or joists $(EI)_\ell$ can be derived based on the flexural stiffness of the individual timber beam/joist and their spacings. Any contribution in flexural stiffness deriving form composite action between timber beams and floorboards, should be accounted for only when glue-based connections are used.
- m: mass per unit area of the floor in kg/m^2 (excluding partition walls and variable loadings).

Specific requirements for the UK regarding footstep-induced vibrations are set out in the UK NA to EC5 [5] for joist-type floors. An equivalent static method is adopted in there, by setting a limit on the maximum displacement (vertical deflection) of the floor when subject to a point load of 1 kN applied at the centre of the floor. The corresponding deflection at the point, *a*, must verify the following deflection limit (in mm):

$$\begin{aligned} a &\leq 1.8 \text{ for } \ell \leq 4000 \text{ mm} \\ a &\leq \frac{16500}{\ell^{1.1}} \text{ for } \ell > 4000 \text{ mm} \end{aligned} \tag{49}$$

where ℓ is the design span of the joists in mm.

7. Connections

Two broad categories can be defined when dealing with timber connections: glue-based connections and mechanical connections. These latter can be further differentiated in two groups: dowel-type fasteners, which essentially rely on the shear resistance of the dowel (screws, nails, bolts) to transfer the load and bearing connections (e.g., punched metal plates, split-rings) where the load is also transferred by bearing directly onto the timer.

7.1 Dowel-type connections

Requirements for minimum spacing between holes for dowel-type connectors, and distances from the timber edges, are set out in EC5 [1]. These minimum spacing and distances are introduced to reduce the risk of brittle failure for the connection, e.g., as a result of timber splitting in tension orthogonally to the grain. Requirements for spacing and distances for nails are reproduced in here in Table 14 whereas minimum values for bolts are given in Table 15. The various terms in both Tables are illustrated here in Figure 13. The standard also requires holes for nails to be pre-drilled when the timber members have a thickness which is smaller than the value of *t* given by Eq. (50):

$$t = max\begin{cases} 7d \\ (13d - 30)\dfrac{\rho_k}{400} \end{cases} \tag{50}$$

where d is the nail diameter and ρ_k is the characteristic value of density for timber.

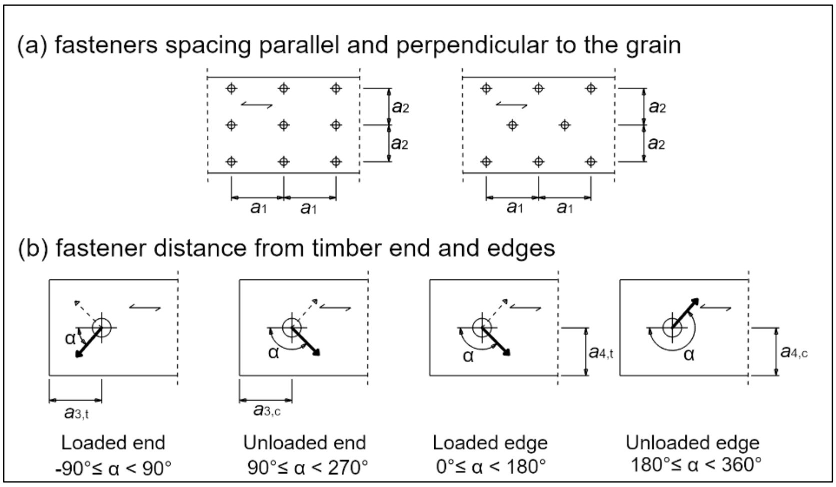


Figure 13: spacing and distances of dowel-type fasteners from the timber ends and edges.

Table 14: Minimum spacing and edge and end distances for nails, based on *BS EN 1995* [1].

Spacing or distance (as from Figure 13)	**Angle α**	**Minimum spacing and end/edge distance**		
		Without predrilled holes		**With predrilled holes**
		$\rho_k^* \leq 420$ **kg/m³**	$420 < \rho_k \leq 420$ **kg/m³**	
a_1	$0^\circ \leq \alpha \leq 360^\circ$	$d^* < 5mm$: $(5 + 5\,\lvert\cos\alpha\rvert)d$ $d \geq 5mm$: $(5 + 7\,\lvert\cos\alpha\rvert)d$	$(7 + 8\,\lvert\cos\alpha\rvert)d$	$(4 + \lvert\cos\alpha\rvert)d$
a_2	$0^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$(3 + \lvert\sin\alpha\rvert)d$
$a_{3,t}$	$-90^\circ \leq \alpha \leq 90^\circ$	$(10 + 5\cos\alpha)d$	$(15 + 5\cos\alpha)d$	$(7 + 5\cos\alpha)d$
$a_{3,c}$	$90^\circ \leq \alpha \leq 270^\circ$	$10d$	$15d$	$7d$
$a_{4,t}$	$0^\circ \leq \alpha \leq 180^\circ$	$d < 5mm$: $(5 + 2\sin\alpha)d$ $d \geq 5mm$: $(5 + 5\sin\alpha)d$	$d < 5mm$: $(7 + 2\sin\alpha)d$ $d \geq 5mm$: $(7 + 5\sin\alpha)d$	$d < 5mm$: $(3 + 2\sin\alpha)d$ $d \geq 5mm$: $(3 + 4\sin\alpha)d$
$a_{4,c}$	$180^\circ \leq \alpha \leq 360^\circ$	$5d$	$7d$	$3d$

* d = nail diameter; ρ_k^* = characteristic timber density

Table 15: Minimum spacing and edge and end distances for bolts, based on *BS EN 1995* [1].

Spacing or distance (as from Figure 13)	**Angle α**	**Minimum spacing and end/edge distance**
a_1	$0^\circ \leq \alpha \leq 360^\circ$	$(4 + \lvert\cos\alpha\rvert)d$
a_2	$0^\circ \leq \alpha \leq 360^\circ$	$4d$
$a_{3,t}$	$-90^\circ \leq \alpha \leq 90^\circ$	$max(7d; 80mm)$
$a_{3,c}$	$90^\circ \leq \alpha \leq 150^\circ$	$(1 + 6\sin\alpha)d$
	$150^\circ \leq \alpha \leq 210^\circ$	$4d$
	$210^\circ \leq \alpha \leq 270^\circ$	$(1 + 6\lvert\sin\alpha\rvert)d$
$a_{4,t}$	$0^\circ \leq \alpha \leq 180^\circ$	$\max\,[(2 + 2\sin\alpha)d; 3d]$
$a_{4,c}$	$180^\circ \leq \alpha \leq 360^\circ$	$3d$

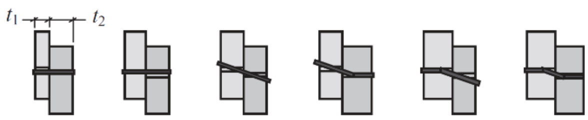
EC5 also provides a method to calculate the resistance of dowelled timber connections for ultimate limit states. The method is based on developments from plastic theory of analysis developed by Johansen [20]. The standard differentiates between non-symmetric connections, i.e., with a single shear plane, and symmetric connections with two shear planes. The resistance of the connection will be function of three main parameters, namely: the connection geometry, the embedment strength of timber (or wood-based material) and bending strength of the fastener. For timber-to-timber connections with one shear plane, the possible modes of failure are shown in Figure 14-(a) and the corresponding values of characteristic load carrying capacity, $F_{v,Rk}$, is to be taken as the minimum among those values, as shown in Eq. (51).

$$F_{v,Rk} = min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d \\ f_{h,2,k} \cdot t_2 \cdot d \\ \dfrac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_2}{t_1}+\left(\dfrac{t_2}{t_1}\right)^2\right]+\beta^3\left(\dfrac{t_2}{t_1}\right)^2} - \beta\left(1+\dfrac{t_2}{t_1}\right)\right] + \dfrac{F_{ax,Rk}}{4} \\ 1.05\dfrac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} \\ 1.05\dfrac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_2^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} \\ 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{51}$$

Similarly, the range of failure modes for symmetric timber-to-timber dowelled connections are shown in Figure 14-(b) and equations to calculate the corresponding resistances are given in Eq. (52).

$$F_{v,Rk} = min \begin{cases} f_{h,1,k} \cdot t_1 \cdot d \\ 0.5 f_{h,2,k} \cdot t_2 \cdot d \\ 1.05\dfrac{f_{h,1,k} \cdot t_2 \cdot d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,Rk}}{f_{h,1,k} \cdot t_2^2 \cdot d}} - \beta\right] + \dfrac{F_{ax,Rk}}{4} \\ 1.15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{52}$$

(a)



(b)

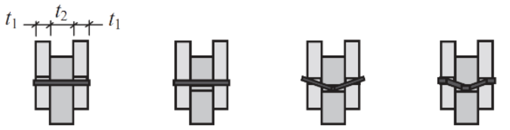


Figure 14: Plastic failure modes of (a): non-symmetric timber-to-timber connections; (b) symmetric timber-to-timber connections. Adapted from *BS EN 1995* [1].

The modes of failure for timber-to-steel connections with one shear plane are shown in Figure 15-(a) whereas Figure 15-(b) shows the possible mode of failures for timber-to-steel connections that are symmetric (i.e., with two shear planes).

For timber-to-steel connections a distinction is made between connections with a *thin* steel plate and a *thick* steel plate. The steel plate is classified as *thin* when its thickness is smaller than 0.5 times the dowel's diameter (*thick* otherwise).

The characteristic load carrying capacity of timber-to-steel connections with one shear plane and with a *thin* plate (i.e., first two modes in Figure 15-(a)) is given as follows:

$$F_{v,Rk} = min \begin{cases} 0.4 f_{h,1,k} t_1 d \\ 1.15\sqrt{2M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{53}$$

whereas for timber-to-steel connections with one shear plane and with a *thick* plate (i.e., last three modes in Figure 15-(a)) the following expressions applies to derive $F_{v,Rk}$:

$$F_{v,Rk} = min \begin{cases} f_{h,1,k} t_1 d \\ f_{h,1,k} t_1 d \left[\sqrt{2 + \dfrac{4M_{y,Rk}}{f_{h,1,k} d t_1^2}} - 1\right] + \dfrac{F_{ax,Rk}}{4} \\ 2.3\sqrt{M_{y,Rk} f_{h,1,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{54}$$

Eq. (54) also applies to symmetric timber-to-steel connections with two shear planes if the central member is the steel plate (either *thin* or *thick*) and the corresponding modes of failure are the first three in Figure 15-(b). If the central member is timber instead and the outer members are steel plates (i.e., last two in Figure 15-(b)) the following expression applies for *thin* steel plates:

$$F_{v,Rk} = min \begin{cases} 0.5 f_{h,2,k} t_2 d \\ 1.15\sqrt{2M_{y,Rk} f_{h,2,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{55}$$

whereas for *thick* steel plates as outer members and timber for the central member Eq. (56) applies instead:

$$F_{v,Rk} = min \begin{cases} 0.5 f_{h,2,k} t_2 d \\ 2.3\sqrt{M_{y,Rk} f_{h,2,k} d} + \dfrac{F_{ax,Rk}}{4} \end{cases} \tag{56}$$

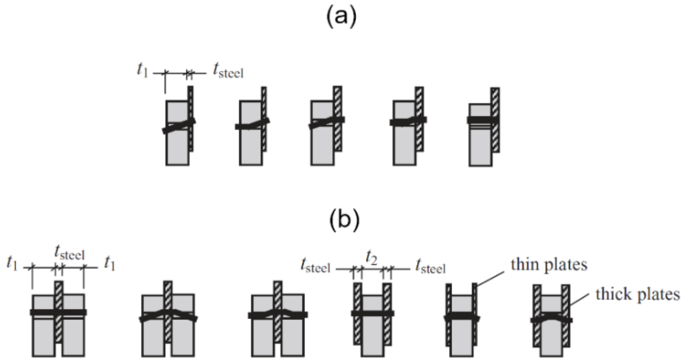


Figure 15: Plastic failure modes of (a): non-symmetric timber-to-steel connections; (b) symmetric timber-to-steel connections. Adapted from *BS EN 1995* [1].

It is important to note that the values of $F_{v,Rk}$ for symmetric connections refers to the resistance of the single dowel (bolt, nail or screw) per single shear plane. Given that these symmetric connections have two shear planes, the actual connection resistance (per dowel) will be twice the value obtained from Eq. (52) and Eqs. (54) to (56).

The terms used in Eqs. (51) to (56) are defined as follows:

— $f_{h,k}$ = characteristic embedment strength
— t_1 = thickness of the thinner timber member (for connections with single shear plane) or that of the side members (for connections with two shear planes)
— t_2 = thickness of the thicker timber member (for connections with single shear plane) or that of the mid member (for connections with two shear planes)
— d = fastener's diameter
— $M_{y,Rk}$ = characteristic yield moment of the fastener
— $F_{ax,Rk}$ = characteristic withdrawal capacity of the fastener
— $\beta = f_{h,2,k}/f_{h,1,k}$

The embedment strength ($f_{h,k}$) equates to the average bearing strength of the timber area in contact with the compression side of the dowel. Although dependent on the bearing strength of timber, the embedment strength is in fact also function of other parameters such as the type and size of dowel and load orientation. Empirical relations have been derived to estimate the characteristic embedment strength and are given in *BS EN 1995*, thus summarized here, differentiating between nails, bolts and screws.

For timber and LVL using nails up to 8 mm in diameter:

$$\begin{aligned} f_{h,k} &= 0.082\rho_k d^{-0.3} \quad \text{(without predrilled holes)} \\ f_{h,k} &= (1-0.01d)\rho_k \quad \text{(with predrilled holes)} \end{aligned} \tag{57}$$

For panel-to-timber connections with nails with a head diameter $\geq 2d$:

$$\begin{aligned} f_{h,k} &= 0.11\rho_k d^{-0.3} \quad \text{(plywood panels)} \\ f_{h,k} &= 65 d^{-0.7} {t_{panel}}^{0.1} \quad \text{(OSB panels)} \end{aligned} \tag{58}$$

For timber and LVL using nails or bolts with a diameter greater than 8 mm, the orientation angle of load with respect to the grain orientation needs to be accounted for when deriving the embedment strength. For dowels loaded parallel to the grain, the following equation applies:

$$f_{h,0,k} = 0.082(1-0.01d)\rho_k \tag{59}$$

If the load is at an angle α to the grain:

$$f_{h,\alpha,k} = \frac{f_{h,0,k}}{k_{90}\sin\alpha^2 + \cos\alpha^2} \tag{60}$$

where $k_{90} = (1.35 + 0.015d)$ for softwoods; $(1.35 + 0.01d)$ for LVL and $(0.9 + 0.15d)$ for hardwoods.

When using screws with a diameter ≤ 6 mm, the same equations provided for nails apply to derive the embedment strength, while the equations for bolts are used for screws with a diameter > 6 mm.

A value in N/mm^2 is obtained for the embedment strength when applying Eqs. (57) to (60), however these empirically-based equations take a value in kg/m^3 for the characteristic density ρ_k, whereas nail diameter (d) and panel thickness (t_{panel}) are to be inputted in mm.

When using screws or nails, the length of their point-side penetration will likely be shorter than the member thickness. This must be accounted for when calculating the connection resistance by setting the thickness term (either t_1 or t_2, whichever relevant) equal to the point-side penetration of the nail or screw.

Reference values for the characteristic yield moment of the fastener ($M_{y,Rk}$) are also given in *BS EN 1995* for common fastener types:

$$\begin{aligned} M_{y,Rk} &= 0.3 f_u d^{2.6} \quad \text{(smooth round nails)} \\ M_{y,Rk} &= 0.45 f_u d^{2.6} \quad \text{(square nails)} \\ M_{y,Rk} &= 0.3 f_{u,k} d^{2.6} \quad \text{(bolts)} \end{aligned} \tag{61}$$

where f_u is the nail's tensile strength and $f_{u,k}$ is the bolt's characteristic tensile strength (both terms in N/mm^2). A value in Nmm will be obtained from Eqs. (61). When using screws with a diameter ≤ 6 mm, the first of Eqs. (61) apply to derive $M_{y,Rk}$, while the equation for bolts (third of Eqs. (61)) is used to calculate the characteristic yield moment of screws with a diameter > 6 mm.

The withdrawal capacity of the fastener, $F_{ax,Rk}$, is a term that was not considered in the original version of Johansen theory. It has been introduced to account for a further increase in the connection resistance, as a result of frictional forces arising when the failure mode involved plastic bending of the dowel. For smooth wire nails:

$$F_{ax,Rk} = min\begin{cases} f_{ax,k} \cdot d \cdot t_{pen} \\ f_{ax,k} \cdot d \cdot t + f_{head,k} \cdot d_h^2 \end{cases} \tag{62}$$

For nails other than smooth (as per *EN 14592* specifications):

$$F_{ax,Rk} = min\begin{cases} f_{ax,k} \cdot d \cdot t_{pen} \\ f_{head,k} \cdot d_h^2 \end{cases} \tag{63}$$

where t_{pen} is the penetration length of the nail in the point-side member, whereas t is the thickness of the head-side member. The diameter terms d and d_h refer to the nail shank and nail's head, respectively. $f_{ax,k}$ and $f_{head,k}$ are the characteristic point-side withdrawal strength and the head-side pull-through strength, respectively. For $t_{pen} > 12d$, the following relations can be used as alternative to empirical testing to derive both strengths (in N/mm^2):

$$\begin{aligned} f_{ax,k} &= 20 \times 10^{-6} \rho_k^2 \\ f_{head,k} &= 70 \times 10^{-6} \rho_k^2 \end{aligned} \tag{64}$$

with ρ_k being the characteristic (5-percentile) density of timber.

When bolts are used, the withdrawal capacity, $F_{ax,Rk}$, will depend on the tensile strength of the bolt and bearing strength of the timber area under the washers. The *BS EN 1995* also sets a limit on the additional withdrawal capacity term ($F_{ax,Rk}/4$) that can be considered in Eqs. (51) to (56). Specifically, the value of ($F_{ax,Rk}/4$) is capped as a percentage of the first terms in the corresponding equation according to the type of fastener being used: 15% for smooth round nails; 50% for nails other than smooth; 100% for screws and 25% for bolts.

Additional requirements to prevent brittle types of failure apply to design situation where the fastener generates a force at an angle to the timber grain, e.g., when dealing with truss systems, as shown in Figure 16. I this case, the concentration of tensile stress orthogonal to the grain around the dowel may cause timber to split. To avoid this the following check must be verified:

$$F_{v,Ed} \leq F_{90,Rd} \tag{65}$$

where $F_{v,Ed}$ is taken as the higher value between the two design shear forces on each side of the connection (i.e., $F_{v,Ed,1}$ and $F_{v,Ed,2}$ as shown in Figure 16) whereas the design splitting capacity, $F_{90,Rd}$, is derived as follows (in N):

$$F_{90,Rd} = k_{mod}\frac{F_{90,Rk}}{\gamma_M} \quad ; \quad F_{90,Rk} = 14bw\sqrt{\frac{h_e}{\left[1-\left(\frac{h_e}{h}\right)\right]}} \tag{66}$$

with the term $w = max\left\{\left(w_{pl}/100\right)^{0.35};1\right\}$ for punched metal plates (w_{pl} = width of the punched metal plate) and $w = 1$ for all other dowel-type fasteners. The terms h and b are the member depth and width (respectively) of the member subject to the splitting force. With reference to Figure 16, h_e is the distance (in mm) from the timber edge to the fastener furthest away.

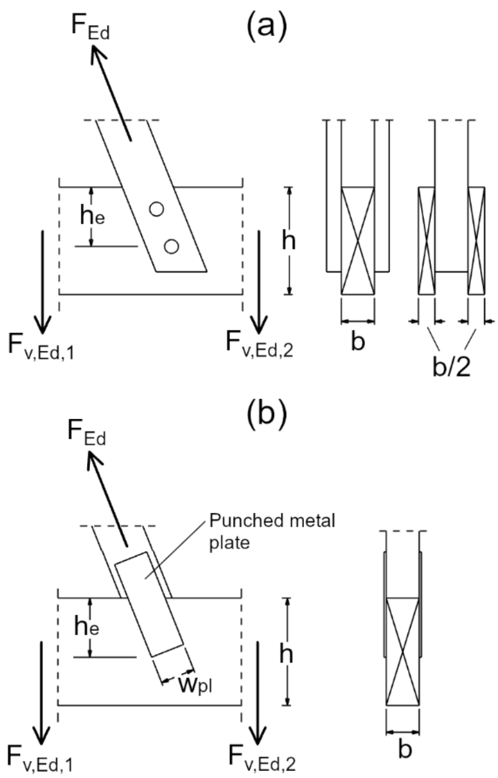


Figure 16: (a) dowelled connection; (b) punched metal plate connection.

7.2 Punched metal plates

When using punched metal plates, the assessment of the connection strength must rely on the manufacturer information as no specific methods of assessment are provided in the EC5 framework. If nail plates are used instead (e.g., tie straps, joist hangers, etc.) the design rules for nailed steel-to-timber connections apply.

7.3 Connectors

Connectors provide additional capacity to dowel-type fasteners by embedding into to timber adjacent the fastener, therefore greatly increasing the bearing surface area of the fastener. There are several types of connector designs available, from toothed plates to discs or rings, and their use is mostly relevant for laterally loaded connections, where the shear force in the fastener is converted into a bearing force between the timber and the connectors' surface. Material and design specifications for connectors are given in BS EN 912 [21]. Connectors such as toothed plates can be either single-sided or double-sided. In the first case the load is transferred from the timber directly to the bolt holding the connector in place, whereas in the latter case the bolt is only required to hold the connector in place, thus enabling the load to be directly transferred (via bearing onto the teeth) from one timber member to the other. Minimum requirements for member thicknesses when using toothed plate connectors are as follows:

$$\begin{aligned} t_1 &\geq 2.25h_e \\ t_2 &\geq 3.75h_e \end{aligned} \tag{67}$$

where t_1 and t_2 are the thicknesses of outer timber member and inner member respectively, while h_e is the embedment depth of the teeth.

8. Glued composite sections

8.1 Thin webbed beams

Composite beams such as I-joists or box beams (see Figure 17) are engineered wooden products usually made using solid timber or LVL for the flanges and OSB or plywood for the web element. Due to the different materials being used, to analyse these sections an equivalent section approach (e.g., similar to reinforced concrete design) is adopted to account for the difference in elastic modulus. Verifications for bending strength of glued thin webbed beams must be carried for the bending stress at the outer fibres of the flange as well as for the axial tensile and compressive stresses at the centroid of the flanges. These latter stresses relate to the compressive/tensile strength of the flanges. Furthermore, strength verifications will also apply to:

— Axial buckling of the compression flange.
— Axial, shear, and bending strengths of the web. This latter should also be checked for buckling if its depth, h_w, is bigger than 70 times its thickness, b_w.
— Strength of the glued connection between the web and flanges.

The connection strength at the interface between web and flanges will usually fail as a result of rolling shear stress exceeding the design rolling shear strength, $f_{v,90,d}$, of the web material (usually OSB) rather than by failure of the glue. Therefore, the following inequality must be satisfied:

$$\tau_{mean,d} \leq \begin{cases} f_{v,90,d} & (\text{for } \mathrm{h_f} \leq 4\mathrm{b_{ef}}) \\ f_{v,90,d}\left(\frac{4b_{ef}}{h_f}\right)^{0.8} & (\text{for } \mathrm{h_f} > 4\mathrm{b_{ef}}) \end{cases} \tag{68}$$

where:

$$b_{ef} = \begin{cases} b_w & \text{(for box beams)} \\ \dfrac{b_w}{2} & \text{(for I} - \text{beams)} \end{cases} \quad (69)$$

The design rolling shear stress $\tau_{mean,d}$ is taken as follows:

$$\tau_{mean,d} = \frac{V_d S_f}{I_{ef} n h_f} \quad (70)$$

where V_d is the design shear force at the location of maximum value; S_f is the first moment of area of the flange around the neutral axis (excluding the web area); I_{ef} is the second moment of area of the equivalent section; n is the number of glue interfaces and h_f is the flange height (see Figure 17).

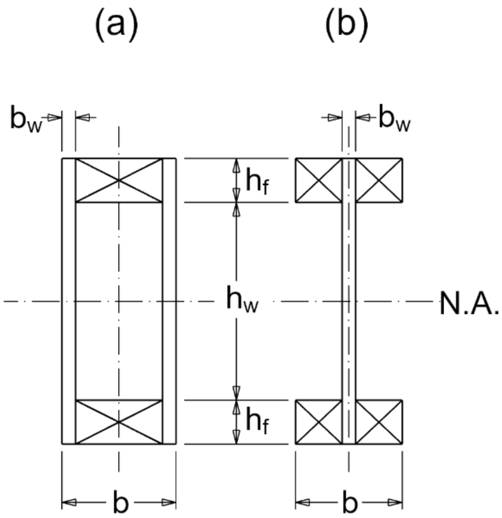


Figure 17: Glued thin webbed sections. (a): box section; (b): I-section.

8.2 Thin flanged beams

Thin flanged beams (also known as *stressed skin panels*) are composite products comprising of solid wood members, used for the webs, and plywood panels for the flanges. As shown in Figure 18, these panels are analysed as individual beams, either as having an I-section (for internal beams) or as C-sections (for edge beams). There is also an option to have the flange panels only on one side. Although mechanical fasteners (e.g., nails or screws) can be used for connecting the flange panels to the solid timber webs, most often these connections are glued, therefore achieving greater bending stiffness by taking full advantage of composite action. As for thin flanged beams, the equivalent section approach is adopted in the analysis to account for the difference in elastic modulus between the two materials. To account for shear lag effects, and effective width, b_{ef}, is assumed for the webs when calculating the section properties. With reference to Figure 18:

— For internal I-sections:

$$b_{ef} = b_{c,ef} + b_w \quad or \quad b_{ef} = b_{t,ef} + b_w \quad (71)$$

— For edge C-sections:

$$b_{ef} = 0.5b_{c,ef} + b_w \quad or \quad b_{ef} = 0.5b_{t,ef} + b_w \quad (72)$$

Values for the effective flange width in compression and in tension ($b_{c,ef}$ and $b_{t,ef}$ respectively) must be equal or smaller than the values given in the third column of Table 16. Furthermore, to avoid the possibility of buckling, the effective width of the compression flange, $b_{c,ef}$, must also comply with the limits set out in the second column of Table 16 (the term ℓ in the Table refers to the beam span).

Table 16: limit values for the effective flange widths, based on *BS EN 1995*.

Flange material	**Plate buckling**	**Shear lag**
OSB	$25h_f$	0.15ℓ
Plywood (outer ply parallel to webs)	$20h_f$	0.10ℓ
Plywood (outer ply perpendicular to webs)	$25h_f$	0.10ℓ
Particleboards or fibreboards	$30h_f$	0.20ℓ

As for thin webbed sections, in addition to design checks for bending stress in the flanges and shear stress in the web, thin flanged webs must also be checked for horizontal shear stress in the glued joints between webs and flanges. Again, the glued bond will be stronger than the rolling shear strength of the web panels ($f_{v,90,d}$) therefore the requirement is for this latter to be greater than the design rolling shear stress $\tau_{mean,d}$. For internal I-shaped sections:

$$\tau_{mean,d} \leq \begin{cases} f_{v,90,d} & (\text{for } \mathrm{b_w} \leq 8\mathrm{h_f}) \\ f_{v,90,d}\left(\frac{8h_f}{b_w}\right)^{0.8} & (\text{for } \mathrm{b_w} > 8\mathrm{h_f}) \end{cases} \tag{73}$$

Eq. (73) applies also to edge C-sections, but replacing the term $8h_f$ with $4h_f$.

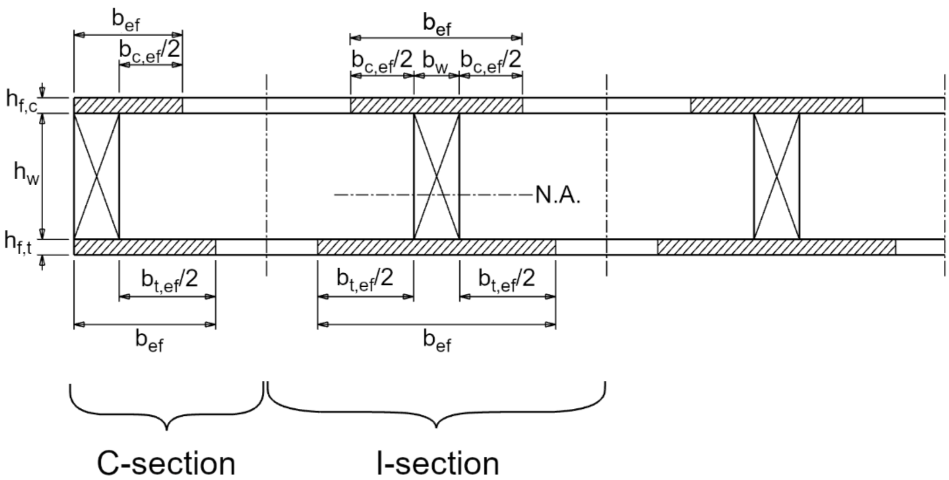


Figure 18: Thin flanged beams.

9. Conclusions

In this chapter, the main set of reference standards for the design of timber and wooded-based products for structural use in UK were described in detail. An overview of the natural characteristic of timber was first introduced and considerations on how these affect the mechanical performances were also provided. Relevant information on the various types of timber-based materials and products, their range of applicability and mechanical performances, were also discussed. The chapter then moved on illustrating the anisotropic nature of wood and how this is accounted for when estimating strength and stiffness properties for structural design applications. The discussion then focused on illustrating the rules and methods, as given in *BS EN 1995* and other relevant standards, for the design of structural members such as beams and columns made from solid timber and glulam, as well as for composite systems such as thin webbed and thin flanged sections. A description of relevant rules and methods for the design of various connection systems and products was also included.

Such a comprehensive overview of normative reference knowledge for timber engineering design in the UK context, will be useful and relevant both for readers that are new to the subject, as well as for practitioners and students of structural design with an interest in the use of timber and wooden-based products as a structural material.

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