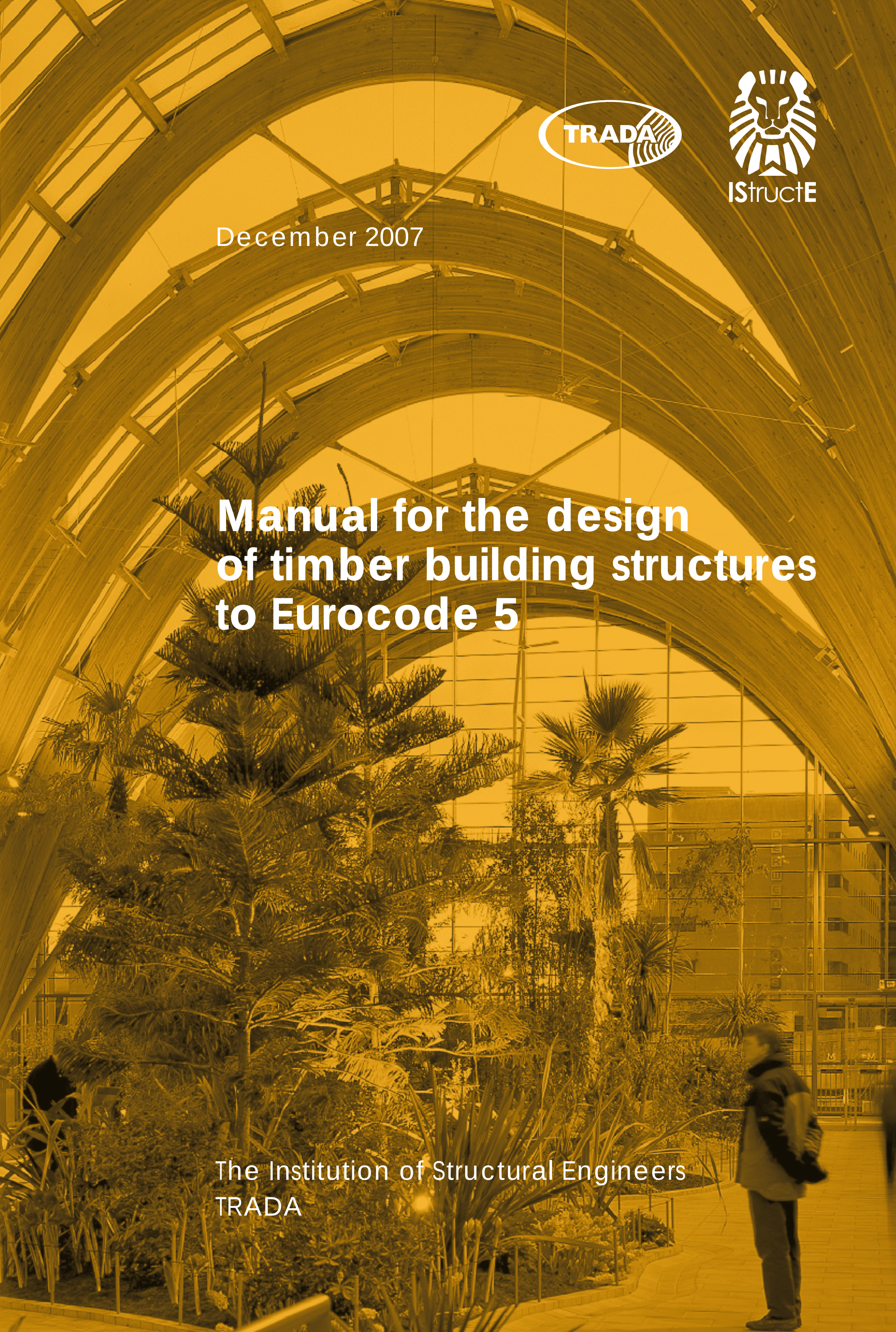

December 2007

Manual for the design of timber building structures to Eurocode 5

The Institution of Structural Engineers
TRADA

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Constitution of Task Group

R J L Harris BSc CEng FIStructE FICE AIWSc (*Chairman*)
R Hairstans BEng PhD
I Janes CEng MIStructE
A C Lawrence MA(Cantab) CEng MIStructE MICE
J P Marcroft BSc CEng MICE
M W Milner MSc BSc CEng MIStructE MICE
P J Steer BSc(Eng) CEng MIStructE
D Trujillo MSc DIC Civil Eng

Consultant and technical writer
A V Page BSc(Eng) BD AIWSc

Secretary to the Task Group
B H G Cresswell Riol BEng

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Chiltern House, Stocking Lane, Hughenden Valley, High Wycombe, Bucks HP14 4ND, UK
T: +44 (0) 1494 569600, W: www.trada.co.uk

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Glossary

Accidental actions	Actions, usually treated as of instantaneous duration but significant magnitude, which are unlikely to occur within the design life of the structure, e.g. exceptional snow drifting, impact, fire, explosion or earthquake.
Accidental design situation	A design situation involving an accidental action.
Action	Either a force or load applied to a structure ('direct action'), or else an imposed deformation ('indirect action') such as temperature effects or settlement.
Arris	The line of intersection between two adjacent sides of a piece of timber.
Assembly	A substructure consisting of several members, e.g. a roof truss or floor diaphragm.
Balloon frame construction	2- or 3-storey height timber framed and sheathed wall panels which act as vertical diaphragms and support roofs and floors acting as horizontal diaphragms.
Block shear failure	The tearing out of a block of material, usually as the result of a force applied by a group of fasteners which produce a combination of tensile and shear failure around the perimeter of the fastener group.
Buckling length	The distance between the points of contraflexure in the fully buckled mode of a compression member or flange. In BS 5268-23 terms this is the effective length.
Building element	A principal part of a building, e.g. roof, wall, floor.
Characteristic value	The characteristic value of an action or material property is its appropriate representative test value, before combination or safety factors are applied to it.
Chipboard	See 'Particleboard'.
Combination factor	Usually referred to as a 'psi factor', a combination factor adjusts the values of variable actions to obtain an appropriate representative design value for the combination.
Combined glulam	Glulam in which the outer laminations are made of timber which has a higher strength than the inner ones.
Compartment floor, compartment wall	A floor or wall which subdivides a large building into compartments of a specified size or volume for fire safety. The use of the term in National Building Regulations is not always consistent.
Component	A member made up of various parts (e.g. a glued thin-webbed beam), often manufactured as a product, or part of a force (e.g. the vertical component).

Connector	See timber connectors.
Creep	The time- and moisture-dependent deformation of a loaded timber member or a laterally loaded mechanically fastened timber connection which occurs in addition to its instantaneous deformation.
Cripple stud	An additional stud which stands on the bottom rail of a timber frame wall panel and supports a sill or the end of a lintel.
Design value	The design value of an action or group of actions or material property is the appropriate characteristic value or values modified as necessary by the relevant combination and safety factors.
Deformation	The deflection or displacement of a member, component or assembly, or the slip in a connection.
Dowelled connection	A connection made with nails, screws, bolts or round steel dowels.
Element	A single part of a connection, component, or structural model.
Engineer	In this *Manual* the Engineer is the person who has overall responsibility for ensuring that the strength, stability and structural serviceability of a building and its elements meet the requirements of the client and the relevant Building Regulations.
Engineered timber joists	This term principally refers to: • prefabricated timber floor joists: most commonly I-joists with flanges made of solid timber or a structural timber composite, and webs made of OSB • metal open-web timber joists with flanges made of solid timber and punched metal plate webs in a zigzag shape.
Engineered wood products	This term normally covers structural timber composites and engineered timber joists.
ENV edition	The preliminary edition of a Eurocode published for trial use.
European Technical Approval (ETA)	European Technical Approval (ETA) for a construction product is a favourable technical assessment of its fitness for an intended use. An ETA is used when a relevant European harmonized standard for the product does not exist and is not likely to exist in the near future.
Execution	The act of constructing the works. For timber structures this includes both fabrication and erection.
Favourable effect	A structural effect which, in combination with other effects, makes the overall effect less unfavourable.
Fibre saturation point	The moisture content of timber when all the free water contained by the cells has drained out but all the water bound within the cell walls is still present.
Final deformation	The total deformation in a structural member, component or assembly produced by an action or group of actions at the end of its design life, including creep.

Flitch beam	A vertically laminated beam consisting of alternate laminates of steel and either solid timber or a structural timber composite. Most flitch beams consist of a single steel plate bolted between two solid timber sections.
Flitch plate	A plate, usually of steel, sandwiched between two timber members and connected to them with bolts or dowels so that the members act as one. It may also be used to join members end-to-end or at a node.
Frame	An assembly of members capable of carrying actions.
Fundamental action	A quantifiable persistent (permanent) or transient (variable) action which is likely to occur with significant magnitude within the design life of the structure.
Fundamental design situation	A design situation which involves only fundamental actions.
Girder truss	A multiple trussed rafter (see Section 7.3.1.2) that: • supports a roof width greater than 2.5 times the normal truss spacing in a roof, or • directly supports other trusses, or • supports another girder truss.
Glulam	Glued laminated timber. Consists of lengths of planed timber glued together with a structural adhesive to form larger members with mechanical properties that are better than those of the original timber. It can be made in almost any length and shape, but is generally manufactured in standardised sizes.
Gusset plate	A plate, usually of steel or plywood, used to join or reinforce principal members in the same plane.
Hardboard	A panel product made from lignocellulosic fibres combined with an adhesive and bonded under heat and pressure. It is denser than MDF or particleboard.
Hardwood	Solid timber from a broadleaf species, widely used for beams and columns with medium or high loading. The less dense hardwoods can also be made into high strength glulam, and the more durable species are useful externally and in wet environments. Their relatively fine grain makes them more suitable than softwoods for high quality joinery.
Homogenous glulam	Glulam made with a single structural grade of timber.
Instantaneous deformation	The deformation produced by an action or group of actions at the moment of application, i.e. without any component of creep.
Irreversible serviceability limit state	A serviceability limit state in which the effects of exceeding it remain when the actions causing it are removed.
Kiln dried	Timber which has been dried in a kiln under controlled temperature and humidity. Structural graded timber is usually dried to a moisture content of 18% to 20% before grading and is stamped ‘KD’.

Limit states	States beyond which the structure no longer satisfies the design performance requirements.
Load case	An action or a combination of simultaneous actions producing a structural effect on a member, component or assembly (see Section 3.2.1.2.).
Laminated strand lumber (LSL)	An alternative to large-section solid timber, LSL consists of strands of aspen up to 300mm long and up to 30mm wide. These are coated with a polyurethane adhesive, orientated parallel to the finished length, then bonded under heat and pressure to form a material superior in strength and stiffness to the wood from which it was made.
Laminated veneer lumber (LVL)	An alternative to large-section solid timber, LVL consists of veneers of timber around 3mm thick glued together with a structural adhesive under pressure to form a material superior in strength and stiffness to the wood from which it was made.
M.C.	See 'Moisture content'.
Medium density fibreboard (MDF)	This is a panel product made from wood or other lignocellulosic fibres combined with an adhesive and bonded under heat and pressure. It is not commonly used as a structural material in the UK.
Medium-rise	For platform timber frame buildings this term is used to mean 4 to 7 storeys.
Member	A beam or column within a structure or assembly.
Moisture content (m.c.)	A measure of the percentage of water in timber, calculated as 100 $\times$ (weight of water) / (oven dry weight).
Multiple member	A multiple lintel, joist, beam, truss or trussed rafter consists of two or more single members of one kind joined in parallel so that they act together.
Nailing plate	A metal plate with holes in it to receive nails - used to join together adjacent timber members in the same plane. Sometimes called a 'cam plate'.
Net projected area	The total area occupied by drilled holes within the plane of a given cross-section. In connection design holes lying within a designated distance of a given cross-section are considered to occur within that cross-section.
Notified Body	An organisation which has been nominated by a government within the European Union and notified by the European Commission to provide services for assessing the conformity of products to the requirements of the relevant European Directives.
Open frame construction	Statically determinate beams and columns stabilised by bracing and/or vertical and horizontal diaphragms, or frameworks with rigid joints such as portal frames, or a combination of the above.

Oriented strand board (OSB)	This is a panel product made from flakes or large chips of wood at least twice as long as they are wide, bonded together with an adhesive under heat and pressure in layers. The directions of the fibres within each layer are generally in the same direction, but in some cases the direction alternates between layers. Its most common structural use is in timber frame walls and prefabricated I-joists.
Parallel strand lumber (PSL)	An alternative to large-section solid timber, PSL consists of strands of timber 2mm to 3mm thick and up to 2400mm long cut from peeled log veneers. They are orientated parallel to the finished length, then glued together with a structural adhesive under pressure to form a material superior in strength and stiffness to the wood from which it was made.
Particleboard	A panel product made from small wood particles and a synthetic resin bonded under heat and pressure. Boards are available in thicknesses from 3 to 50mm. They may be of uniform construction through their thickness, of graded density or of a distinct 3- or 5-layer construction. Commonly called ‘chipboard’, its most common structural use is in flooring.
Party floor, party wall	Generic terms for floors and walls which separate dwellings or areas designated for some other purpose from other dwellings or areas designated for some other different purpose within the same building, or which are required to subdivide a large building for purposes of fire resistance. Party floors are described in the National Building Regulations as compartment or separating floors depending on the function being considered.
Permanent actions	Dead loads, such as the self-weight of the structure or fittings, ancillaries and fixed equipment.
Platform frame construction	A building method based on storey height timber framed and sheathed wall panels which act as vertical load-bearing diaphragms and support roofs and floors acting as horizontal diaphragms.
Plug shear failure	A form of block shear failure in which the block does not extend through the full thickness of the ruptured member.
Plywood	A panel product made by gluing together veneers of timber, usually with alternating fibre directions.
Portal frame	A frame in the form of a 2- or 3-pinned arch, designed to be stable in its plane. It is generally used to resist horizontal loads where other bracing methods are not permitted and a large, open space is required. Timber portals are usually designed as 2-pinned arches, with moment resisting site joints for ease of transportation.
Punched metal plate fastener (PMPF)	A metal plate with integral projections punched out in one direction perpendicular to the base of the plate, used to join together adjacent softwood members in the same plane. Also called a ‘connector plate’.
PVAc	A polyvinyl acetate (PVA) adhesive with improved properties, often used to provide additional fixity to floor decking.

Racking	The effect caused by horizontal actions in the plane of a wall. (The racking strength of a wall is the load it can resist applied in the plane of the wall.)
Raised tie truss	Sometimes called a ‘collar truss’, this is a triangulated roof truss in which the ceiling tie is attached to the rafters at a level above the eaves, normally to provide more headroom.
Resistance	The strength of a member in a particular mode of failure.
Reversible serviceability limit state	A serviceability limit state in which the effects of exceeding it disappear when the actions causing it are removed.
Rim beam	A beam positioned at the outer edge of a floor which provides full or partial support across an opening in the case of its accidental removal. In platform timber frame design a rim beam is usually a solid timber, LVL or prefabricated timber I-joist which is nailed into the ends of joists or placed parallel to the edge joists and transfers part of the vertical load from a wall to the wall or foundation beneath it.
Rope effect	A contribution made to the lateral load capacity of a dowel-type connection by the resistance to axial withdrawal of the fastener when the mode of failure includes bending of the fastener.
Sarked roof	A roof made with sarking.
Sarking	Wood based panels fastened to the upper side of rafters. Sarking may be designed and used as a structural diaphragm.
Separating floor, separating wall	A floor or wall which separates dwellings or areas designated for some other purpose from other dwellings or areas designated for some other different purpose within the same building. (The use of the term in National Building Regulations is not always consistent.)
Serviceability limit states	Limit states beyond which specified service criteria are no longer met.
Service class	Refer to Section 2.15.
Shear plane	A plane between two connected members which are loaded in different directions. A dowel connecting 5 members loaded in alternate directions would have 4 shear planes.
Shear plate	A timber connector consisting of a circular plate flanged on one side and with a central bolt hole, which is fitted into a recess in the face of a connected timber member.
Structural Insulated Panel System (SIPS)	A SIPS panel consists of two wood-based panels with a rigid insulating foam plastic-based core between them. They are laminated together to produce a one-piece structural building system which can be used to form whole wall, roof or floor components.
SLS	Abbreviation for Serviceability limit state.

Softwood	Solid coniferous timber, widely used for beams and columns with low or medium loading, in the manufacture of glulam, LVL, OSB, plywood and particleboard, for cladding and joinery, and for the flanges of some types of prefabricated timber joist. Softwoods generally require preservative treatment for external structural use.
Slip	The relative movement between two loaded members within the area of a mechanically fastened connection between them.
Slip modulus	A property used to calculate the relative movement between two connected members of a structure, expressed in N/mm. For a laterally loaded dowelled connection the slip modulus allows for deformation in the fastener and the members on both sides of the shear plane.
Space structures	Domes, grid shells, etc.
Spandrel	The triangular area of a gable wall between the eaves and the ridge.
Splice plate	A plate, usually of steel or plywood, fastened to two or more timber members to join them end-to-end or at a node.
Split ring	A timber connector consisting of a steel ring which is fitted into circular grooves cut into adjacent faces of connected timber members.
Stressed skin panel	Termed a 'thin-flanged beam' in EC5, a timber stressed skin panel can be described as a set of parallel I-joists of which the flanges are made of a wood-based panel product and the webs of solid timber or possibly an STC, with the flange material continuous across the joists. The connection between the flanges and webs may be made by means of mechanical fasteners (generally nails or screws) or adhesive bonding. When mechanical fasteners are used, joint slip must be allowed for in calculating the composite strength and stiffness of the panel.
Structural timber composites	Also known as 'engineered wood products', glulam, LSL, LVL, PSL are efficient replacements for large sections of solid timber.
STCs	See 'Structural timber composites'.
Stud, wall stud	A vertically orientated solid timber or engineered wood product member which resists vertical loads and wind loads within a timber wall. In structurally sheathed wall panels a full height wall stud also transfers panel shear forces to the foundation.
System strength	The capacity of a timber structure consisting of several equally spaced similar members, components or assemblies acting together to resist load. Since wood based members even of the same grade vary in their properties, it may be assumed that when several of them act together they do not all have the minimum characteristic strength properties which would have to be used if they acted alone. The weaker members are less stiff and therefore attract less load than the stronger members. Their combined strength is therefore greater than the sum of their individual strengths, and this is allowed for by a system factor in the Code.

T & G	See 'Tongue and groove'.
Target size	The specified size (breadth and depth or length) of timber at a reference moisture content, to which permissible deviations are related. Target sizes are used for engineering design calculations, even though the actual size depends on the moisture content (see Section 3.3.2.3).
Timber connectors	Toothed plates, split rings, shear plates or similar devices used in conjunction with a bolt to transfer shear loads between adjacent timber members.
Timber frame construction	A building method or a building in which the load-bearing walls are made of a rectangular timber framework sheathed with a wood-based panel product or gypsum plasterboard. The external walls are invariably sheathed on the outside with a wood-based panel product, most commonly OSB. The two principal types of timber frame construction are balloon frame and platform frame, the latter type being nearly always used in the UK.
Tolerance class	A set of permitted dimensional deviations from the target size at the reference moisture content.
Tongue and groove	The edge of a solid timber board or wood-based board profiled in such a way that adjacent boards can be locked together. A tongue in one board fits into a groove in its neighbour. This keeps adjacent boards in line and enables them to share loads.
Toothed plate	A round or square timber connector made of sheet steel with triangular teeth projecting at right angles around its circumference on one or both sides. The teeth are embedded into the connected timber member or members, normally by the use of a high tensile steel bolt and nut.
Trussed rafter	A structural assembly of timber members of the same thickness, fastened together in one plane by metal plate fasteners or plywood gussets for the support of roofs and ceilings. Trussed rafters are used at spacings of 400mm to 600mm and are generally made with softwood and punched metal plate fasteners.
Variable actions	Variable actions are actions which vary in magnitude with time. They mainly comprise imposed loads, snow loads and wind loads.
ULS	Abbreviation for Ultimate limit state.
Ultimate limit states	Limit states associated with collapse or other forms of structural failure that may endanger the safety of people.
Unfavourable effect	An undesirable structural effect on a structural member, component or assembly, normally produced by an action.
Wood particleboard	See 'Particleboard'.

Notation

Latin upper case letters

A	Cross-sectional area; Accidental action
A_d	Design value of accidental action
E	The effect of an action – e.g. bending moment, deflection
$E_{0,05}$	Fifth percentile value of modulus of elasticity
$E_{0,mean}$	Mean value of modulus of elasticity parallel to grain (or parallel to surface grain of plywood or OSB)
$E_{90,mean}$	Mean value of modulus of elasticity perpendicular to surface grain of plywood or OSB
$E_{ct,0,mean}$	Mean value of modulus of elasticity in compression and tension parallel to surface grain of plywood or OSB
$E_{ct,90,mean}$	Mean value of modulus of elasticity in compression and tension perpendicular to surface grain of plywood or OSB
E_{mean}	Mean value of modulus of elasticity
$E_{mean,fin}$	Final mean value of modulus of elasticity
F	Force or action; Point load
F_d	Design value of a force or point load
$\mathrm{F}_{f,Rd}$	Design load capacity per fastener in wall diaphragm
$F_{i,t,Ed}$	Design tensile reaction force at end of shear wall
$F_{i,vert,Ed}$	Design vertical load on wall
$F_{i,v,Rd}$	Design racking resistance of wall *i* (in Section 10.8.1.1)
F_t	Tensile force
$F_{v,Ed}$	Design shear force per shear plane of fastener; Horizontal design effect on wall diaphragm
$F_{v,Rd}$	Design load capacity per shear plane per fastener; Design racking load capacity
$F_{v,Rk}$	Characteristic load capacity per shear plane per fastener
G	Permanent action
$G_{k,j}$	Characteristic value of a permanent action, numbered *j*
G_{mean}	Mean value of shear modulus (in panel shear for panel products)
$G_{r,mean}$	Mean planar shear modulus in bending for panel products
$G_{v,mean}$	Mean planar shear modulus in racking for panel products (same as G_{mean})
H	Overall rise of a truss
K_{ser}	Slip modulus
$K_{ser,fin}$	Final slip modulus
K_u	Instantaneous slip modulus for ultimate limit states
M_d	Design moment
$M_{y,Rk}$	Characteristic yield moment of fastener
N	Axial force
Q	Variable action
$Q_{k,1}$	Characteristic value of a leading variable action

$Q_{k,i}$	Characteristic value of a variable action, numbered i
R_d	Design value of a load capacity
R_k	Characteristic load capacity
V	Shear force
W_y	Section modulus about axis y
X_d	Design value of a strength property
X_k	Characteristic value of a strength property

Latin lower case letters

a	Distance
a_1	Spacing, parallel to grain, of fasteners within one row
a_2	Spacing, perpendicular to grain, between rows of fasteners
$a_{3,c}$	Distance, parallel to grain, between fastener and unloaded end
$a_{3,t}$	Distance, parallel to grain, between fastener and loaded end
$a_{4,c}$	Distance, perpendicular to grain, between fastener and unloaded edge
$a_{4,t}$	Distance, perpendicular to grain, between fastener and loaded edge
b	Width
b_i	Width of wall i (in Section 10.8.1.1)
b_{net}	Clear distance between studs
d	Diameter
d_0	Charring depth parameter
$d_{char,n}$	Notional charring depth
d_{ef}	Effective diameter; Effective charring depth
$f_{c,0,d}$	Design compressive strength parallel to grain (or surface grain of plywood or OSB)
$f_{c,0,k}$	Characteristic compressive strength parallel to grain (or surface grain of plywood or OSB)
$f_{c,90,k}$	Characteristic compressive strength perpendicular to grain (or surface grain of plywood or OSB)
$f_{h,k}$	Characteristic embedment strength
f_1	Fundamental frequency
$f_{m,k}$	Characteristic bending strength
$f_{m,0,k}$	Characteristic bending strength parallel to surface grain of plywood or OSB
$f_{m,90,k}$	Characteristic bending strength perpendicular to surface grain of plywood or OSB
$f_{m,y,d}$	Design bending strength about the major y-axis
$f_{m,z,d}$	Design bending strength about the minor z-axis
$f_{t,0,d}$	Design tensile strength parallel to grain (or surface grain of plywood or OSB)
$f_{t,0,k}$	Characteristic tensile strength parallel to grain (or surface grain of plywood or OSB)
$f_{t,90,d}$	Design tensile strength perpendicular to grain (or surface grain of plywood or OSB)
$f_{t,90,k}$	Characteristic tensile strength perpendicular to grain (or surface grain of plywood or OSB)
$f_{v,0,k}$	Characteristic transverse shear strength in bending parallel to surface grain of plywood or OSB
$f_{v,90,k}$	Characteristic transverse shear strength in bending perpendicular to the surface grain of plywood or OSB

$f_{v,d}$	Design shear strength (panel shear in panel products)
$f_{v,k}$	Characteristic shear strength (panel shear in panel products)
$f_{v,r,k}$	Characteristic rolling shear strength in plywood
g	Distributed permanent load
h	Depth of member; Height of wall
h_e	Distance between loaded edge and centre of most distant fastener
h_{ef}	Effective depth
k_0	Charring coefficient
k_{bond}	Glued bonding pressure factor
$k_{c,y}$ or $k_{c,z}$	Instability factor
k_{crit}	Factor used for lateral buckling
k_d	Dimension factor for panel
k_{def}	Deformation factor, dependent on duration of load and moisture content
k_h	Depth or width factor
$k_{c,90}$	Bearing strength modification factor
k_{dist}	Proportion of point load acting on a single joist
k_{joint}	Deformation factor for connections
k_m	Factor to allow for the re-distribution of bending stresses in a cross-section
$k_{masonry}$	Factor to allow for the wind shielding effect of masonry walls
k_{mc}	Glued bond moisture effect factor
k_{mod}	Strength modification factor for duration of load and moisture content
k_n	Material factor for notched beams
k_{shear}	Amplification factor to account for shear deflections in vibration calculations
k_{sys}	System strength factor
k_v	Reduction factor for notched beams
l	Span
l_{ef}	Effective length
m	Mass per unit area
n_{ef}	Effective number of fasteners in a row parallel to grain
p	Distributed load
q	Distributed variable load
q_i	Equivalent uniformly distributed load
s	Spacing; Size effect parameter
s_0	Basic fastener spacing
t	Thickness
t_{pen}	Penetration depth
u	Deformation – slip or horizontal deflection
u_{creep}	Creep deformation
u_{fin}	Final deformation
u_{inst}	Instantaneous deformation
$u_{inst,j}$	Instantaneous deformation for a permanent action G_j
$u_{inst,Q,1}$	Instantaneous deformation for the leading variable action Q_1
$u_{inst,Q,i}$	Instantaneous deformation for accompanying variable actions Q_i

w_{creep} Creep deflection
w_{fin} Final deflection
$w_{inst,G}$ Instantaneous deflection for a permanent action, *G*
$w_{inst,Q}$ Instantaneous deflection for a variable action, *Q*
$w_{net,fin}$ Net final deflection

Greek lower case letters

α Angle ('alpha')
γ Partial factor ('gamma')
γ_G Partial factor for permanent actions
$\gamma_{G,j}$ Partial factor for permanent action *j*
γ_M Partial factor for material properties, also accounting for model uncertainties and dimensional variations
γ_Q Partial factor for variable actions
$\gamma_{Q,i}$ Partial factor for variable action *i*
υ Poisson's ratio ('nu')
λ_y Slenderness ratio corresponding to bending about the *y*-axis ('lambda')
λ_z Slenderness ratio corresponding to bending about the *z*-axis
$\lambda_{rel,y}$ Relative slenderness ratio corresponding to bending about the *y*-axis
$\lambda_{rel,z}$ Relative slenderness ratio corresponding to bending about the *z*-axis
ρ_k Characteristic density (normally a fifth percentile of the density) ('rho')
ρ_{mean} Mean density
$\sigma_{c,0,d}$ Design compressive stress parallel to grain ('sigma')
$\sigma_{m,crit}$ Critical bending stress
$\sigma_{m,y,d}$ Design bending stress about the major *y*-axis
$\sigma_{m,z,d}$ Design bending stress about the minor *z*-axis
$\sigma_{t,0,d}$ Design tensile stress parallel to grain
$\sigma_{t,90,d}$ Design tensile stress perpendicular to grain
τ_d Design shear stress ('tau')
ξ Reduction factor ('xi')
ϕ Diameter ('phi')
ψ_0 Factor for combination value of a variable action ('psi')
ψ_1 Factor for frequent value of a variable action
ψ_2 Factor for quasi-permanent value of a variable action
ζ Modal damping ratio ('zeta')

Note that '*f*' denotes a material property and 'σ' or 'τ' a stress.

Foreword

The Eurocode for the Design of Timber Structures (EC5) comprising BS EN 1995-1-1: *General: Common rules and rules for buildings* was published in December 2004. The UK National Annex (NA) setting out the Nationally Determined Parameters (NDPs) has also been published. These documents, together with previously published documents BS EN 1990: *Basis of Structural Design* and BS EN 1991: *Actions on Structures* and their respective NAs, provide a suite of information for the design of most types of timber building structures in the UK. After a period of co-existence, the current National Standards will be withdrawn and replaced by the Eurocodes.

The Institution of Structural Engineers has not previously published a manual for the design of timber structures. This *Manual* follows the basic format of manuals published by the Institution for other structural materials. It provides guidance on the design of structures of single-storey and medium-rise multi-storey buildings using common forms of structural timberwork. Structures designed in accordance with this *Manual* will normally comply with EC5. However it is not intended to be a substitute for the greater potential range of EC5. The NDPs from the UK NA have been taken into account in the design formulae that are presented.

Timber is a relatively complex structural material therefore a manual for the design of timber structures is bound to be more extensive than that for other materials. Despite its length, designers should find this *Manual* concise and useful in practical design. It is laid out for hand calculation, but the procedures are equally suitable for spread sheet and/or computer application. An example is in the design of connections; EC5 requires the solution of a series of expressions, a process that is not practicable in hand calculations and so tabulated values are provided in the *Manual*. The accompanying CD provides connection design software and more extensive material properties.

The Timber Engineering community in the UK is small but through those directly associated with the *Manual's* preparation we were able to draw on a wealth of knowledge. EC5 is a design code on the European model; it contains many design rules but little practical advice. Throughout the preparation of the *Manual*, we have been conscious of the need to capture practical knowledge and set it down. We have worked hard to interpret the intent of the Eurocode and where appropriate due to Eurocode limitations we have used alternative methods.

Special thanks are due to Arnold Page who researched and drafted the *Manual*. Through his participation, we were fortunate to have been able to draw on information from both TRADA's knowledge base, built up over 70 years, as well as Arnold's own deep understanding gained over half a lifetime in timber engineering. The *Manual* was more demanding and time consuming than originally envisaged and particular thanks are due to TRADA for recognising the importance of the *Manual* to the construction industry and continuing to fund Arnold's time to enable a proper completion.

Special thanks are also due to all of the members of the Task Group and to their organisations, who have given their time voluntarily. I am also grateful to Ben Cresswell Riol for acting as secretary to the Group and for having undertaken this with patience and skill. During the review process, members of the Institution provided a substantial response, both in quantity and quality on the draft *Manual*, which has contributed to its improvement and completion.

I join with all of the other members of the Task Group in commending this *Manual* to the construction industry.

R J L Harris
Chairman

1 Introduction

1.1 Aims of the *Manual*

This *Manual* and the accompanying CD provides qualified Structural Engineers with guidance on the structural design of single-storey and medium-rise multi-storey buildings using common forms of structural timberwork. Structures designed in accordance with the *Manual* will normally comply with BS EN 1995-1-1: *Eurocode 5: Design of timber structures – Part 1-1: General: Common rules and rules for buildings* (EC5)[1], together with its supporting codes and standards. The *Manual* is primarily intended for carrying out simple calculations, and is not necessarily relevant to the design of complex buildings requiring more sophisticated analysis. However it is good practice to check the output of complex analyses using simplified methods such as those provided.

For simplicity reference to clauses in BS EN 1995-1-1 will be in the form 'EC5 4.2(1)' Reference to clauses in the *Manual* will be by section, e.g. 'Section 2.1.1'.

1.2 The Eurocode system

1.2.1 Origin and purpose

The structural Eurocodes are produced by the European Committee for Standardisation (CEN), its members being the national standards bodies of the EU and EFTA countries, e.g. BSI.

1.2.2 List of Eurocodes

The complete set of Eurocodes consists of the following:

BS EN 1990: Eurocode:		Basis of structural design (EC0)
BS EN 1991: Eurocode 1:		Actions on structures (EC1)
	Part 1-1:	General actions – Densities, self-weight and imposed loads
	Part 1-2:	General actions on structures exposed to fire
	Part 1-3:	General actions – Snow loads
	Part 1-4:	General actions – Wind loads
	Part 1-5:	General actions – Thermal actions
	Part 1-6:	Actions during execution
	Part 1-7:	Accidental actions from impact and explosions
	Part 2:	Traffic loads on bridges
	Part 3:	Actions induced by cranes and machinery
	Part 4:	Actions in silos and tanks
BS EN 1992: Eurocode 2:		Design of concrete structures (EC2)
BS EN 1993: Eurocode 3:		Design of steel structures (EC3)
BS EN 1994: Eurocode 4:		Design of composite steel and concrete structures (EC4)
BS EN 1995: Eurocode 5:		Design of timber structures
	Part 1-1:	General – Common rules and rules for building (EC5)
	Part 1-2:	General – Structural fire design (EC5-1-2)
	Part 2:	Bridges (EC5-2)

BS EN 1996: Eurocode 6: Design of masonry structures
BS EN 1997: Eurocode 7: Geotechnical design
BS EN 1998: Eurocode 8: Design of structures for earthquake resistance
BS EN 1999: Eurocode 9: Design of aluminium structures

Eurocodes 1 to 9 all comprise several parts, but only EC1 and EC5 have been listed in full.

1.2.3 Principles and Application Rules

All the Eurocodes contain 'Principles' and 'Application Rules'.

Principles are general statements, definitions, design rules or analytical models for which no alternative is permitted, for example EC5 8.2.3(2)P "The strength of the steel plate shall be checked." Clauses which comprise a principle are identified by the letter 'P'.

Application Rules are generally recognised rules which comply with and satisfy the Principles. Alternative design rules may be used instead, provided that they can be demonstrated to comply with the Principles and to produce similar levels of safety, serviceability and durability to the Application Rules.

1.2.4 National Annexes

Every National Standards body may produce its own National Annex (NA) for each part of each Eurocode. An NA provides values or decisions related to 'Nationally Determined Parameters' (NDPs) which allow for differences in such matters as climatic conditions, standards of workmanship, and perceptions of acceptability in deflections. UK NDPs are identified by **bold type** in the *Manual*.

1.2.5 Non contradictory complementary information

The Eurocode system also permits reference in NAs to sources of 'non contradictory complementary information' (NCCI) which help designers to use the associated Eurocodes. In the UK the principal source for EC5 is BS PD 6693[2].

This *Manual* includes a number of NCCI items which will not be found in EC5 itself.

1.2.6 Eurocode design basis

The Eurocode common basis of design for all structural materials is based on limit states and partial safety factors. For structural timber design in the UK this represents a major change from BS 5268-2[3], in which all the safety factors are incorporated in the permissible stresses. A limit state is simply a state beyond which a structure no longer satisfies its performance requirements. Ultimate limit states are associated with collapse or similar forms of structural failure that may endanger the safety of people, and generally involve the consideration of strength and stability. Serviceability limit states are associated with user discomfort or dissatisfaction or a lack of functionality, and generally involve the consideration of deformation (i.e. the deflections of members or slip in connections). Partial safety factors are used to increase the values of loads and to decrease the material strength values (also to adjust stiffness properties for second order linear elastic analysis – see EC5 2.2.2(1) Note 2). In each case the values of the factors are specified and are applied to the characteristic values of the loads or material properties, so the approach to safety is known and transparent.

1.3 Scope of the *Manual*

1.3.1 National scope

The *Manual* is intended primarily for the design of buildings within the United Kingdom. Where values and design methods specified in UK National Annexes are quoted the information given may not be applicable elsewhere.

1.3.2 Structures covered

For the majority of design situations and materials involving timber the information required has been provided in this *Manual* or in the accompanying CD.

Two principal types of timber structure are covered:

- open frame buildings, i.e.
  - statically determinate beams and columns stabilised by bracing and/or vertical and horizontal diaphragms
  - frameworks with rigid joints such as portal frames
  - a combination of the above
- timber platform frame buildings with a maximum height of 18 metres to the finished floor level of the top storey.

1.3.3 Principal subjects covered

- roofs, floors and walls
- flexural, tension and compression members
- diaphragms, flitch beams
- mechanically fastened and glued connections
- load duration, service class, creep, durability and fire resistance.

1.3.4 Subjects not covered

- foundations and geotechnical design (see BS EN 1997: *Eurocode 7: Geotechnical design* (EC7)[4])
- seismic design (see BS EN 1998: *Eurocode 8: Design of structures for earthquake resistance* (EC8)[5])
- the following detailed design issues:
  - analysis of frame structures – EC5 5.4.2
  - analysis of trusses with punched metal plate fasteners – EC5 5.4.3
  - glued thin-webbed beams – EC5 9.1.1
  - glued thin-flanged beams – EC5 9.1.2
  - mechanically jointed beams – EC5 9.1.3
  - mechanically jointed and glued columns – EC5 9.1.4
  - trusses – EC5 9.2.1 and 9.2.2.

1.3.5 Additional information contained in the CD

- material properties of solid timber, glulam, wood-based panel products and structural timber composites
- nail, screw, bolt and dowel connection spreadsheets
- links to manufacturers web sites.

For a more detailed list see the Contents.

1.3.6 Sources of additional information

For timber-related subjects which are not covered by the *Manual*, EC5 or its supporting standards should be consulted.

Other useful publications are:

- *STEP Timber Engineering*, Volumes 1 and 2[6]
- TRADA's EC5 Guidance Documents and EC5 Design Examples[7]
- TRADA's Software Toolbox[8] (includes the design of connections to EC5 and will include domestic timber members in the near future)
- Panel Guide Partnership's *PanelGuide*[9]
- Building Research Establishment published material.

It is also intended to publish other manuals in this series on EC0 and EC1. Further sources of information are given in the References.

1.4 Contents of the *Manual*

The *Manual* is set out in the sequence normally followed in design.

Sections 2, 3	Principles of structural timber design
Section 4	Initial building design process
Sections 5, 6	Design of individual members and connections
Sections 7-10	Design of roofs, floors and two principal types of building

There are two additional sections.

Section 11	Checking and specification guidance
Section 12	Workmanship, installation, control and maintenance

1.5 Definitions

1.5.1 Technical terms

In order to rationalise the meanings of various technical terms for easy translation, some of the terms used in the past have been redefined more precisely in the Eurocodes. Those of particular importance are listed, together with other timber related terms which may not be familiar to engineers who are more accustomed to other materials, in the glossary.

1.5.2 Axis nomenclature

The use of traditional axis nomenclature in the UK has been altered to match a consistent European approach throughout the Eurocodes, as shown in Figure 1.1. The *x*-*x* axis lies along the length of the member, *y*-*y* is the principal or major axis, and *z* -*z* is the minor axis.

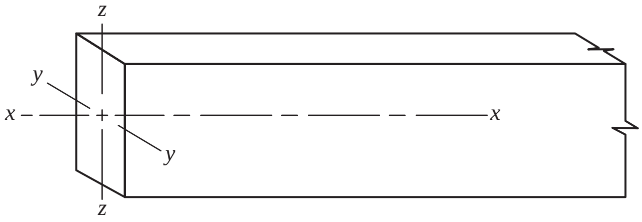


Fig 1.1 Nomenclature of axes

1.6 Notation

The Latin and Greek characters that apply to designs to EC5 are listed under Notation in the preliminary pages of this *Manual*.

2.1 Basis of design

2.1.1 Basic requirements

BS EN 1990: *Eurocode: Basis of Structural design* (EC0)[10] requires structures to be designed so that, within specified limits, they are safe, serviceable, robust and durable. It is a legal requirement that building designs conform to the requirements of the applicable Building Regulations in force at the time. House-building guarantors may impose additional requirements on structural design.

2.1.2 Design codes

2.1.2.1 Structural Eurocodes

Structural designs carried out in accordance with EC5 must be based on the design principles set out in EC0, and on the imposed loads specified in the various parts of BS EN 1991: *Eurocode 1: Actions on Structures* (EC1). Unless more accurate values are known, the material weights specified in BS EN 1991-1-1: *Actions on Structures. General actions. Densities, self-weight, imposed loads for buildings*. (EC1-1-1)[11] should be used. Reference should be made to the National Annexes to these codes for the values of Nationally Determined Parameters, and to supporting European standards for material properties, dimensions, tolerances and other information which are required to complete a structural design. EC0 5.2 also allows design assisted by testing in cases where reliable modelling is difficult, large numbers of similar components are to be made, or to verify design assumptions.

2.1.2.2 Differences between BS 5268 and EC5

- The two codes have different definitions of duration of loads. BS 5268 has long term, medium term, short term and very short term; whereas EC5 has permanent, long term, medium term, short term and instantaneous. It follows that a particular action will fall into different load duration categories in the two codes. For example snow loading is medium-term in BS 5268 but it is short-term in EC5 (see Table 2.2).
- BS 5268 grade values of strength properties are safe for all load durations, but the values used with EC5 are characteristic test values which must be reduced for the appropriate load duration and by a safety factor.
- The stiffness moduli for solid timber strength classes given in BS 5268 differ from those in BS EN 338[12].
- EC5 fastener load capacities derived from calculations or tables yield Eurocode design values which are significantly higher than the permissible values obtained from the tables in BS 5268, so they must not be used in conjunction with the unfactored loads.
- EC5 requires creep effects to be considered in calculating deflections, and some of the recommended limits on deflection, e.g. in the National Annex, refer to final not initial deflections.

Where EC5 does not provide necessary information it is acceptable to use information from other design codes provided that it is compatible with the Eurocode safety format and does not conflict with EC5.

2.1.3 Structural materials compliance

All products specified for structural use in Europe must satisfy the requirements of the Construction Products Directive.

This means that solid timber should be structurally graded to a standard listed in BS EN 14081-1[13] and stamped accordingly by a certified timber grading agency.

In all but the three European States listed below, glulam, wood-based panel products and other proprietary wood-based products may be specified only if they have been certified by a Notified Body or declared as fit for purpose by a manufacturer, in accordance with the relevant harmonised European standard or European Technical Approval. Such products will have associated literature detailing the relevant technical specification which they meet, and they may in addition carry a ‘CE’ mark with a summary of this information. The certification literature will include the characteristic material properties and any associated modification factors which should be used for structural calculations.

In the UK, Ireland and Sweden alternative methods of demonstrating compliance with the Directive are permitted for these products via third party certification from notified bodies. Products certified by such alternative methods may be specified in designs to EC5 but they may not be used in any structures which will be erected in European countries other than the three mentioned above. However they should not be specified unless the certification literature includes either characteristic material properties derived in accordance with standard European test methods or BS 5268-2[3] grade values in the case of wood-based panel products (for which a conversion procedure is available (see Section 3.3.3)).

For further information on certification, contact the Timber Trades Federation for solid timber, the Glued Laminated Timber Association for glulam, or the Wood Panel Industries Federation for wood-based panel products, contact details for whom may be found in Appendix B.

2.2 Responsibility for design

The Engineer has overall responsibility for ensuring that the strength, stability and structural serviceability of a building and its elements will, if properly constructed, maintained and used for its intended purpose, meet the requirements of the client and the relevant Building Regulations.

The Engineer also has a duty of care concerning durability. This is principally a matter of suitable architectural detailing but it may sometimes require the specification of preservative or protective treatments for timber materials and metal fastenings.

While the Engineer’s work is concerned primarily with the adequacy of load-bearing members, components and assemblies, it might also include non load-bearing items where the integrity of their fixing has safety implications, e.g. the attachment of external cladding.

2.3 Building use and location

The designated use and geographical location of the building should be specified. This will determine the variable actions, requirements for resistance to disproportionate collapse, requirements for the corrosion protection of metal fasteners, and in certain limited areas the necessity for the protective treatment of roof timbers against house longhorn beetle.

2.4 Design life

A design life for the building should be specified (EC0 2.3). A properly designed and maintained timber building can last for centuries, but EC0 suggests a design life of 50 years for building structures, and that has been assumed in this *Manual*. For temporary structures EC0 suggests a 10 year design life, and where there is no risk that this period may be exceeded the assumption of 10 years will permit the use of higher strength properties and lower creep factors.

2.5 Design situations

The building must be designed to have adequate strength, stability and structural serviceability in the following situations:

- during construction (the execution phase)
- in designated use throughout its design life (see Section 2.4)
- in accidental design events.

If any significant seismic actions on the building are likely to occur within the design life then EC8 should be consulted.

2.6 Stability

Overall stability checks are particularly important for timber structures because they are relatively light in weight. The structure should resist uplift, sliding and overturning forces produced by the wind, both during the execution phase and in the finished structure. Particular care should be taken when the height:breadth ratio of a timber building exceeds 2:1.

For individual members and assemblies EC5 9.2.5.1 states three principles.

- Structures which are not otherwise stiff must be braced to prevent instability and excessive deflection.
- Stresses caused by geometrical and structural imperfections and by induced deflections, e.g. from joint slip, must be taken into account.
- The required bracing forces should be determined on the basis of the most unfavourable combination of structural imperfections and induced deflections.

For methods of providing stability see Section 5.7.

2.7 Construction

The resistance of a timber building to overturning or sliding normally takes into account the weight of the roof, but this will not be present during the construction phases. Similarly the resistance of timber frame buildings to horizontal wind forces may depend on the connection of the roof to an adjoining terrace of houses, or on the shielding effect of brick cladding, neither of which will be present during construction. BS EN 1991-1-4: *Eurocode 1: Actions on structures: General actions. Wind loads* (EC1-1-4)[14] allows a reduction in wind loads for the execution phase, but the structural adequacy during this period must nevertheless be verified. Where necessary consideration should be given to the need for temporary bracing during the execution phase.

To comply with the Construction (Design and Management) Regulations[15] the Engineer should consider and where necessary provide safe procedures or instructions for:

- manual handling
- the method of erection
- the type of craneage
- lifting points.

The Engineer should ensure that this information is recorded in the construction phase plan and the health and safety file.

Where design assumptions dictate certain methods and sequences of erection, full information concerning them should be included in the health and safety plan as well as being shown on the design drawings.

2.8 Movement

2.8.1 Moisture movement

The moisture content of timber varies with temperature and humidity. While its dimensions are relatively stable along the grain, across the grain timber swells or shrinks as its moisture content increases or decreases, changing by about 1% for every 5% change in moisture content. Kiln dried structural graded softwood can have a moisture content of 18% to 20% on delivery, and will dry out in a heated environment to 10% to 15%, so its dimensions across the grain may decrease by 1% to 2%. Engineered wood products have either less timber content (e.g. metal web joists) or are delivered to site at a much lower moisture content and are therefore less susceptible to shrinkage. It is also possible to specify 'super-dried' solid timber which is delivered at a moisture content of 14% or less and has a moisture resistant coating, significantly reducing cross grain movement particularly in timber floor joists. Some structural timber composites are also manufactured with water resistant coatings, and if necessary such coatings may be specified both for structural timber composites and for glulam. However none of these materials will retain its dimensions if it is exposed on site to the weather for an extensive period. Particular care is needed in the case of large diaphragms made with wood based panel products (see Table 12.1 and Section 4.5 of Panel Guide Partnership's *PanelGuide*[9]). Where large timber buildings such as sports halls are likely to take a long time to erect it may be advisable to consider a temporary roofed area for stored materials.

2.8.2 Thermal movement

It is rarely necessary to consider thermal movement in timber structures. The linear coefficient of thermal expansion of timber along the grain (about 3.5×10^{-6} per ºC) is smaller than that of steel or concrete, and its greater elasticity enables it to accommodate movement more easily. Across the grain the coefficient ranges from 26×10^{-6} to 35×10^{-6} per °C, but any increase in dimension due to temperature is more than counteracted by shrinkage due to the reduced moisture content. Also timber's thermal conductivity is much lower than that of steel, so its temperature is less responsive to ambient changes in temperature.

2.8.3 Differential movement

Differential movement in timber frame buildings is covered in Section 10.11.

2.8.4 Movement joints

Movement joints to accommodate thermal and moisture movements in a timber structure are rarely required. Movement joints in claddings are required as in non-timber buildings. For the spacing of expansion gaps in wood based horizontal diaphragms refer to Table 12.1.

2.9 Creep

For SLS calculations creep in the deflection of loaded members and creep in the slip between mechanically connected members should be included whenever final values are relevant. For ULS design situations creep in the deflection of members and in the slip of connections should be allowed for in the analysis of structures if such deformations cause a significant redistribution of forces and moments.

2.10 Robustness and disproportionate collapse

2.10.1 Robust construction

Timber structures, like those made of other materials, must be sufficiently robust in accidental situations to ensure that the loss of individual members or components does not cause the disproportionate collapse of the entire structure or large parts of it (EC0 2.1(4)P). Any damage due, for example, to explosion, impact or the consequences of human error should not be disproportionate to the original cause. Procedures for meeting these requirements are given in Section 5.11.

Designing a robust structure also involves:

- consideration of how it will be used and the possible consequences – for example flour mills and chemical plants may pose specific problems with regard to explosion and pressure venting
- consideration of construction procedures – for example the ability to make the designed connections properly
- provision of access for routine maintenance and inspection.

2.10.2 Accidental actions

Where relevant the magnitude of an accidental action for a specific occurrence may be agreed between the client and the designer. The design value of accidental actions, including exceptional snow drifts, is calculated in accordance with Section 3.2.1.3.

2.11 Fire resistance

The design should satisfy requirements in the relevant Building Regulations for the particular building type.

Depending on their function, members, components and assemblies exposed to fire may be required to provide one or more of the following for a specified period:

- mechanical resistance – they should remain sufficiently strong and stiff to permit a safe exit from the building
- integrity – they should form an effective barrier to smoke and flame
- insulation – they should limit the transfer of heat by conduction from an area on fire to another area.

In addition there may be requirements to:

- limit the thermal radiation from the side unexposed to fire
- ensure that the surface spread of flame properties do not unduly endanger the occupants of the building or neighbouring buildings.

Adequate mechanical resistance of structural assemblies and members in fire can be achieved by:

- insulating them from heat
- the use of sacrificial timber.

Structural elements including mechanical fasteners can be insulated from heat by covering them with one or more layers of insulating material of a specified thickness. The most common material for this is gypsum plasterboard, but cork, plywood or other wood-based materials may be used instead.

The use of sacrificial timber involves checking that the dimensions of the residual sections of timber members after charring are sufficiently large to support the design loads with acceptable deflections within the specified period of fire resistance. Where necessary the section size can be increased to provide the necessary period of fire resistance. This approach could be used for exposed columns or beams for example, and is applicable to solid timber, glulam, LVL and other structural timber composites, but not to engineered timber joists.

In structures designed to EC5 the requirements for fire resistance should be achieved in accordance with the design rules given in EC5-1-2.

2.12 Acoustic, thermal and air tightness requirements

The design should satisfy the requirements of the relevant Building Regulations and house building guarantors relating to acoustic insulation, thermal insulation and air tightness. Although they may impinge on design, these factors are not essential aspects of structural design and are not addressed in any detail in this *Manual*.

2.13 Durability

The building should be designed so that it continues to satisfy all the relevant limit state requirements for its specified design life (see Section 2.4). This is achieved by design details which ensure that all the timber remains dry and well ventilated, and where this cannot be ensured by specifying durable species or suitable protection treatments for its structural members (see Section 3.4).

2.14 Maintenance

The Construction (Design and Maintenance) Regulations[15] require the Designer to consider risks to the health and safety of persons engaged in the maintenance of the structure, to provide safe means of access to elements needing maintenance, and to provide this information for recording in the health and safety file.

2.15 Service class

2.15.1 Effect of moisture on strength and stiffness

An increase in moisture content reduces the strength and stiffness properties of timber and wood-based materials and increases the amount of creep that occurs. Designs to EC5 should allow for the effects of moisture on strength and creep, but not on the instantaneous stiffness properties (see Section 2.17).

2.15.2 Definitions of service classes

Moisture content depends on both temperature and atmospheric humidity. For simplicity the moisture content is related to one of three 'service classes' which are defined in Table 2.1.

Table 2.1 Definition of service classes and examples

Service class	Temperature	Approximate maximum humidity	EMC[a] (%)	Examples from the EC5 NA
1	20°C	65%	12%	Warm roofs, intermediate floors, timber-frame walls – internal and party walls
2	20°C	85%	20%	Cold roofs, ground floors, timber-frame walls – external walls, external uses where member is protected from direct wetting
3	Conditions leading to higher moisture contents than service class 2.		> 20%	External uses – fully exposed

Note

a EMC = Maximum equilibrium moisture content for most softwoods.

2.16 Load duration

2.16.1 Effect of load duration on strength and stiffness

An increase in load duration reduces the loads which timber can resist and increases the creep but not the instantaneous elastic stiffness. Designs to EC5 should allow for these effects (see Section 2.17).

2.16.2 Definitions of load duration classes

Load duration classes are defined according to the approximate accumulated duration of characteristic load, as shown in Table 2.2.

Table 2.2 Load duration classes

Class	Definition	Examples from the EC5 NA and EC0
Permanent	More than 10 years	Self-weight
Long-term	6 months to 10 years	Temporary structures Storage loading including loading in lofts Water tanks
Medium-term	1 week to 6 months	Imposed floor loading
Short-term	Less than one week	Snow Maintenance or man loading on roofs Residual structure after accidental event
Instantaneous	Instantaneous	Wind Impact loading Explosion

2.17 Factors to allow for effects of moisture and load duration

2.17.1 Strength modification factor, k_{mod}

Table 2.3 shows values of k_{mod} for the most common wood-based materials for the service classes in which they may be used. For structural fibreboards see EC5 Table 3.1. For PSL, LSL and engineered wood joists consult the manufacturers. For timber which is installed at or near its fibre saturation point, e.g. large timbers with minimum cross-sectional dimension > 100mm and not specially dried, use the value of k_{mod} for service class 3.

2.17.2 Deformation modification factor, k_{def}

Table 2.4 shows value of k_{def} for the most common wood-based materials. For structural fibreboards see EC5 Table 3.2. For PSL, LSL and engineered wood joists consult the manufacturers. For timber which is installed at or near its fibre saturation point, e.g. large timbers with minimum cross-sectional dimension > 100mm and not specially dried, and which is likely to dry out under load, the value of k_{def} should be increased by 1.00.

Table 2.3 Values of k_{mod} (from EC5 Table 3.1)

Material	Standards	Service class	Load duration class				
			Perm -anent	Long term	Medium term	Short term	Instant -aneous
Solid timber	BS EN 14081-1[13]	1,2	0.60	0.70	0.80	0.90	1.10
Glulam	BS EN 14080[16]						
LVL	BS EN 14374[17] BS EN 14279[18]	3	0.5	0.55	0.65	0.70	0.90
Plywood	BS EN 636[19]						
	Parts 1, 2 and 3	1	0.60	0.70	0.80	0.90	1.10
	Parts 2 and 3	2	0.60	0.70	0.80	0.90	1.10
	Part 3	3	0.50	0.55	0.65	0.70	0.90
OSB	BS EN 300[20]						
	OSB/2	1	0.30	0.45	0.65	0.85	1.10
	OSB/3, OSB/4	1	0.40	0.50	0.70	0.90	1.10
	OSB/3, OSB/4	2	0.30	0.40	0.55	0.70	0.90
Particle-board	BS EN 312[21]						
	Parts 4 and 5	1	0.30	0.45	0.65	0.85	1.10
	Part 5	2	0.20	0.30	0.45	0.60	0.80
	Parts 6 and 7	1	0.40	0.50	0.70	0.90	1.10
	Part 7	2	0.30	0.40	0.55	0.70	0.90

Table 2.4 Values of k_{def} (from EC5 Table 3.2)

Material	Standards	Service class		
		1	2	3
Solid timber	BS EN 14081-1[13]	0.60	0.80	2.00[a]
Glulam	BS EN 14080[16]			
LVL	BS EN 14374[17] BS EN 14279[18]			
Plywood	BS EN 636[19]			
	Part 1	0.80	–	–
	Part 2	0.80	1.00	–
	Part 3	0.80	1.00	2.50[a]
OSB	BS EN 300[20]			
	OSB/2	2.25	–	–
	OSB/3, OSB/4	1.50	2.25	–
Particle-board	BS EN 312[21]			
	Part 4	2.25	–	–
	Part 5	2.25	3.00	–
	Part 6	1.50	–	–
	Part 7	1.50	2.25	–

Note

a Consult manufacturer on suitability for use in service class 3. Preservative treatment may be necessary.

2.17.3 Large sections of solid timber

If both the thickness and breadth of a solid timber section exceed 100mm it is usually supplied at a moisture content above 20%, as sections of this size are difficult to dry without special arrangements. Therefore, for such sections of solid timber, the Engineer should either specify that it must be specially dried to below 20% moisture content before installation, or use a value of k_{mod} corresponding to service class 3. In a heated environment large sections may dry out sufficiently to fall within service class 2 conditions so if necessary two separate verifications for strength may be carried out, one with no action assigned a duration longer than medium-term in service class 3, and the other with actions assigned their full term in service class 2. Allowance should be made for movement which will occur during drying out, particularly in respect of green oak[22]. Also connection detailing may require special attention.

3.1 Actions

3.1.1 Types of action

EC0 distinguishes between permanent actions (dead weight of structure, finishes, permanent fixtures and fixed partitions) and variable actions (every other type of action).

3.1.2 Characteristic values of actions

For permanent actions it is normally acceptable to use an average weight as the characteristic value or, if a range of weights is specified, the highest value in the range. For variable actions an upper characteristic value appropriate to the design life (e.g. once in 50 years) is normally used. However, in ULS load cases where actions can have a favourable effect (e.g. tile weight resisting overturning), the lowest value should be used for permanent loads if a range of weights is specified. Other adjustments for favourable effects are made by means of the partial load factors (see Table 3.1).

The values to be used in calculations are:

- Characteristic dead load, EC1-1-1
- Characteristic imposed load, EC1-1-1
- Characteristic snow load, BS EN 1991-1-3: *Eurocode 1. Actions on structures. General actions. Snow loads* (EC1-1-3)[23]
- Characteristic wind load, EC1-1-4.

3.1.3 Design values of actions

For fundamental ULS the characteristic values of actions are converted to design values by the partial load factors shown in Table 3.1. Additional factors applicable to snow and wind loads are given in the relevant codes. For accidental ULS the load factors for permanent and variable actions are set at 1.0. For SLS the load factor is 1.0, unless the load is favourable (e.g. an imposed load on the second span of a two-span joist) in which case a value of 0.0 should be used.

3.1.4 Action combinations

Where more than one variable action acts simultaneously, a leading variable action is chosen, and the others are reduced by a specified combination factor. Where it is not obvious which should be the leading variable action, each action should be checked in turn to determine the combination which produces the worst effect. Other factors are used to determine the design value of accidental load combinations and the average value of variable loads in the calculation of creep deformation. All these factors, normally referred to as ‘psi’ factors, are shown in Table 3.2. For examples of their use, see Section 3.2.

Table 3.1 Partial load factors for ULS[a]
(based on EC0 NA Tables NA.A1.2(A) and NA.A1.2(B))

	Permanent actions, γ_G		Variable actions, γ_Q	
	Unfavourable	**Favourable**	**Unfavourable**	**Favourable**
Strength checks	**1.35**	**1.00**	**1.50**	**0.00**
Equilibrium checks	**1.10**	**0.90**	**1.50**	**0.00**
Combined equilibrium and strength checks[b]	**1.35**	**1.15**	**1.50**	**0.00**

Notes

a Values for unfavourable effects are for main values. Values for favourable effects are for actions which relieve some of the load on a member or tend to stabilise a member or structure. For the design of elements in a foundation, e.g. timber piles, see EC0 A1.3.1(5), **Approach 1.**

b The combined check is an optional alternative to separate calculations for equilibrium and strength verifications when both have to be carried out. However if it is employed then it must also be verified that setting γ_G to 1.00 for both the favourable and unfavourable parts of the permanent load does not produce a less favourable effect.

Table 3.2 Factors for the representative values of variable actions for the buildings covered by this *Manual*
(from EC0 NA Table A1.1)

Action	**Factors for the representative value of an action**		
	ψ_0	ψ_1	ψ_2
Permanent actions – weights of materials and permanent fixtures	N/A	N/A	1.0
Imposed loads in buildings – category from EC1-1-1			
A: domestic and residential areas	**0.7**	**0.5**	**0.3**
B: office areas	**0.7**	**0.5**	**0.3**
C: congregation areas	**0.7**	**0.7**	**0.6**
D: shopping areas	**0.7**	**0.7**	**0.6**
E: storage areas (including lofts)	**1.0**	**0.9**	**0.8**
H: roofs[a]	**0.7**	**0.0**	**0.0**
Snow loads on buildings – see EC1-1-3			
Sites up to 1000m above sea level	**0.5**	**0.2**	**0.0**
Wind loads on buildings – see EC1-1-4	**0.5**	**0.2**	**0.0**

Note

a The roof imposed load should not be applied at the same time as wind or snow – see EC1-1-1 3.3.2(1).

EC1-1-1 3.3.1(P) states that when imposed loads act simultaneously with other variable actions (e.g. wind or snow) the total imposed loads on the floors of multi-storey buildings shall be considered as one variable action. For timber structures this requirement is applicable principally to the instantaneous load case (wind + snow + floor imposed) and to the short-term load case (snow + floor imposed). In addition, EC1-1-1 6.3.1.2(11) states than for building "*categories A to D, for columns and walls, the total imposed loads from several storeys may be multiplied by a reduction factor* α_n." α_n is applicable to buildings with 4 or more storeys, and its values should be taken from the National Annex to EC1-1-1. For timber the medium-term load case (floor imposed) may be critical, and since this does not involve other variable actions the design value of the individual imposed loads from two or more floors may be calculated as $Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i}$. This value may be reduced by α_n if and where it is applicable.

3.2 Limit states

This *Manual* adopts the limit state principle and the partial factor format common to all Eurocodes and defined in EC0.

3.2.1 Ultimate limit states (ULS)

3.2.1.1 Definitions

ULS are breached by loss of equilibrium or material failure. All relevant states should be checked.

- Equilibrium:
  - uplift of the whole or part (e.g. roof) of the structure
  - overturning
  - sliding.

  The stability of the foundations should be checked in accordance with EC7.

- Strength:
  - material rupture in bending, tension, compression or shear
  - stability failure (e.g. buckling of columns, lateral torsional buckling of beams, stresses induced by sway in portals, diagonal bracing in roofs or walls)
  - connection failure – yield in the fasteners or metal-work, or timber failure by embedment, shear, splitting, fastener pull-out or head pull-through.

Where time-dependent deformations (deflections of members or slip in connections) cause a significant redistribution of effects in an assembly or framework, the strength of the members should be considered both before and after creep deformation has occurred.

Fundamental design situations occur during the construction phase and in normal use. Accidental design situations occur at or following an exceptional event such as fire, exceptional snowdrift, impact or explosion. All relevant design situations should be considered.

3.2.1.2 Load cases

The load which timber can resist depends on the duration of the load. Therefore a duration should be assigned to every action (see Table 2.2). When several actions occur simultaneously, a separate load case should be considered for each load duration represented, unless it is obvious that it will not govern the design. The duration of each load case should be taken as the shortest load duration of any load in the load case.

For example, a permanent + medium-term + short-term load which can act simultaneously on a member produce three load cases:

- permanent load only (permanent duration load case)
- permanent load + medium-term load (medium-term load case)
- permanent load + medium-term load + short-term load (short-term load case).

3.2.1.3 Design values of action combinations

The design value of the actions combined in any particular load case is calculated as follows. It is usually necessary to try each variable load, Q_k, in turn as $Q_{k,1}$ in order to find the worst case.

i) Fundamental design situations (strength or equilibrium)

$$\sum_{i>1}\gamma_{G,j}G_{k,j} + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}$$ – EC0 expression (6.10)

ii) Accidental design situations (fire, impact or explosion)

$$\sum_{i>1}G_{k,j} + (A_d) + \psi_{1,1}Q_{k,1} + \sum_{i>1}\psi_{2,i}Q_{k,i}$$ – EC0 expression (6.11b)

iii) Exceptional snowdrifts (see EC1-1-3, NA.2.4 and NA 2.5)

$$\sum_{i>1}G_{k,j} + Q_{k,1} + \sum_{i>1}\psi_{2,i}Q_{k,i}$$ – EC0 expression (6.11b)

(It is assumed that the snowdrift load is the leading variable action, so $Q_{k,1}$ represents the snowdrift.)

In expression ii), A_d is in parenthesis because it should be included only when considering the direct effects on a structure of an impact or explosion. It would not be included when considering the situation after such an event or following a fire.

For final designs it may be beneficial to use expression 6.10b from EC0 6.4.3.2(3) instead of expression 6.10.

iv) $$\sum\xi\gamma_{G,j}G_{k,j} + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,I}$$ – EC0 expression (6.10b)

where ξ = **0.925**

This may be used only:

- for fundamental (not accidental) design situations
- for strength (not equilibrium) verifications
- when the leading variable action is wind or snow and it is unfavourable and its characteristic value exceeds 13.5% of the characteristic value of the permanent actions, or when the leading action is any other type of variable action except storage and its characteristic value exceeds 22.5% of the characteristic value of the permanent actions.

3.2.1.4 Design value of effects of actions

In practice it is often convenient to use the above expressions to combine the effects of actions (e.g. bending moments, shear forces) rather than the actions themselves (see EC0 6.3.2). Thus EC0 expression (6.10) becomes:

$$\sum \gamma_{G,j} E_{G,k,j} + \gamma_{Q,1} E_{Q,k,1} + \sum_{i>1} \gamma_{Q,i} \psi_{0,i} E_{Q,k,i}$$

For example, consider a rafter supporting four loads.

1	A full length permanent UDL	– e.g. roof weight
2	A partial permanent UDL	– e.g. solar panel weight
3	A short-term UDL	– e.g. snow
4	An instantaneous UDL	– e.g. wind

These will produce three load cases with design bending moments calculated as shown in Table 3.3.

Table 3.3 Design values of bending moments – domestic floor beam example

Load case	Loads	Duration of load case	Design value of bending moment using EC0 (6.10) (see Section 3.2.1.3(i))
A	1+2	Permanent	$M_d = \gamma_G (M_{k,1} + M_{k,2}) = 1.35\,(M_{k,1} + M_{k,2})$
B	1+2+3	Short	$M_d = \gamma_G (M_{k,1} + M_{k,2}) + \gamma_Q M_{k,3}$ $= 1.35\,(M_{k,1} + M_{k,2}) + 1.5 M_{k,3}$
C	1+2+3+4	Instantaneous	M_d = maximum of $\gamma_G (M_{k,1} + M_{k,2}) + \gamma_Q M_{k,3} + \gamma_Q \psi_{0,4} M_{k,4}$ and $\gamma_G (M_{k,1} + M_{k,2}) + \gamma_Q M_{k,4} + \gamma_Q \psi_{0,3} M_{k,3}$ = maximum of $1.35\,(M_{k,1} + M_{k,2}) + 1.5 M_{k,3} + 1.5 \times 0.7 \times M_{k,4}$ and $1.35\,(M_{k,1} + M_{k,2}) + 1.5 M_{k,4} + 1.5 \times 0.7 \times M_{k,3}$

For each of the three load cases, the resulting bending stress should be compared with the bending strength of the joist calculated for the corresponding load duration.

3.2.2 Serviceability limit states (SLS)

3.2.2.1 Definitions

SLS are limit states beyond which specified service criteria are no longer met. In practice these comprise excessive deflection in bending members, joint slip producing excessive deflection, and unacceptable vibration in floors. An irreversible serviceability limit state is a serviceability limit state in which the effects of exceeding it remain when the actions causing it are removed. A reversible serviceability limit state is one in which the effects of exceeding it disappear when the actions causing it are removed. All relevant SLS should be checked.

- Typical irreversible SLS:
  - damage to finishes – e.g. cracking of plasterboard, glass or wall tiles
  - ponding in flat roofs (which may lead to premature failure)
  - damage to masonry produced by excessive deflection of attached structural timberwork
  - deflections of members onto other members not designed to support them.
- Typical reversible SLS:
  - malfunction – e.g. windows jamming beneath lintels, gaps beneath partitions
  - unacceptable appearance – e.g. gaps beneath partitions, visually unacceptable deformations in walls, ceilings or exposed beams
  - unacceptable vibration or movement in floors
  - horizontal deflection in upper storeys of medium-rise buildings (provided that this does not cause instability).

The term 'deformation' covers the deflection of bending members and slip in connections. Slip in connections is the relative lateral movement between the connected members or relative rotation in moment-resisting connections. In frameworks it is usually necessary to consider the effects of slip on the overall deflections of the members.

For deformation checks it may be necessary to check either instantaneous deformation or final deformation, or both. Instantaneous deformation is the deformation produced by an action or group of actions at the moment of application, i.e. without any component of creep. Final deformation is the total deformation in a structural member, component or assembly produced by an action or group of actions at the end of its design life, i.e. instantaneous deformation + creep.

3.2.2.2 Load cases – SLS

When calculating the design values of actions or the effect of actions for SLS, all partial load factors are set to 1.0, or 0.0 if the loading is favourable.

EC0 requires as a principle that "a distinction shall be made between reversible and irreversible serviceability limit states" (EC0 3.4(2)P) and it gives separate expressions for combining actions for the two limit states. However EC5 2.2.3(2) states that the "characteristic combination", which is normally used only for irreversible limit states, should be used for all calculations of instantaneous deformation.

i) Instantaneous deformation

$$\sum_{i>1} G_{k,j} + Q_{k,1} + \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad \text{– EC0 expression (6.14b)}$$

Creep deformation is related to the average value of the deforming actions over the design life, which is calculated using the 'quasi-permanent' combination.

ii) Creep deformation

$$\sum_{i>0} G_{k,j} + \sum_{i>0} \psi_{2,i} Q_{k,i} \qquad \text{– EC0 expression (6.16b)}$$

The resulting deformation is factored by k_{def} to produce the creep deformation.

3.2.2.3 Design value of deformations

In practice it is generally necessary to use the expressions in Section 3.2.2.2 to combine the deformations produced by the actions rather than the actions themselves (see EC0 6.3.2).

i) Instantaneous deformation

$$u_{inst} = \sum_{i>1} u_{inst,Gj} + u_{inst,Q,1} + \sum_{i>1} \psi_{0,i}\, u_{inst,Q,}$$

ii) Creep deformation

$$u_{creep} = k_{def} \sum u_{inst,Gj} + k_{def} \sum_{i>0} \psi_{2,i}\, u_{inst,Q,i}$$

iii) Final deformation

$$u_{fin} = u_{inst} + u_{creep}$$

The expressions in Sections 3.2.2.2 and 3.2.2.3 produce the same final deformations as the method given in EC5 2.2.3(5).

As a simpler alternative to ii) and iii) above, u_{fin} may be calculated in the same way as u_{inst} but using final stiffness moduli as given in Section 3.2.3 (see EC5 2.3.2.2(1)). This method gives a conservative solution and is equivalent to omitting the ψ_2 factors in expression ii).

3.2.2.4 Components of deflection

Figure 3.1 shows the components of deflection, w, in a simple beam.

Key

Using EC5 symbols: w_{inst} Instantaneous deflection due to permanent and variable loads

w_{creep} Creep deflection due to permanent and variable loads

w_{fin} Total final deflection due to permanent and variable loads

Fig 3.1 Components of deflection in a beam without precamber (extract from EC5 Figure 7.1)

Precambering a beam will reduce w_{fin}. Large glulam beams can be manufactured with a camber, but precambering is not appropriate for solid timber beams.

3.2.2.5 Recommended deflection limits

In Eurocode design none of these is mandatory: the serviceability criteria and limits should be specified for each project and agreed with the client (see EC0 A1.4.2(2)). Tables 3.4 and 3.5 show some suggested limits on deflections.

Table 3.4 Recommended vertical deflection limits based on span, l
(from NA Table NA.4 and BS 5268-4.1 Clause 5.1.1[24])

Example of use	**Limit state**	**Instantaneous/ Final**	**Recommended limits for beams spanning between two supports[a]**
Cracking of plasterboard, glass, ceramics, etc, in roofs, ceilings or floors. Also recommended for lintels	Irreversible	Final	$w_{fin} \leqslant \frac{l}{250}$ where w_{fin} = deflection due to permanent + imposed loads + creep. In certain circumstances a finite limit may be more appropriate, e.g. around glazing
Appearance of roofs and ceilings with no attached brittle finishes	Reversible	Final	$w_{fin} \leqslant \frac{l}{150}$ where w_{fin} = deflection due to permanent + imposed loads + creep
In fire, at end of required period of fire resistance, where protection depends on attached plasterboard, unless proved by test	Accidental	Instantaneous	$w_{inst} \leqslant \frac{l}{20}$ where w_{inst} = instantaneous deflection due to permanent + imposed loads

Note
a For cantilevers, the limit based on the cantilever span of l should be doubled.

Table 3.5 Recommended horizontal deflection limits

Example of use	Limit state	Instantaneous/ Final	Recommended limits
Portal frames with masonry or large areas of glass within the plane of the frame	Irreversible	Instantaneous	$w_{\text{inst,Q}} \leqslant \dfrac{h_e}{300}$ where $w_{\text{inst,Q}}$ = instantaneous horizontal deflection at top of column or storey caused by wind. h_e = height of column or storey
Portal frames without masonry or large areas of glass within the plane of the frame	Reversible	Instantaneous	$w_{\text{inst,Q}} \leqslant \dfrac{h_e}{200}$ where $w_{\text{inst,Q}}$ = instantaneous horizontal deflection at top of column or storey caused by wind. h_e = height of column or storey
Timber frame dwellings with masonry	Irreversible	Instantaneous	The design method automatically limits the instantaneous deflection to $h_e/333$. Tighter limits, e.g. $h_e/500$, may be approximately obtained by increasing the racking strength proportionately. h_e = storey height
	Reversible	Instantaneous	
Masonry walls in dwellings – horizontal deflection at each end of a raised tie truss	Irreversible	Instantaneous	$w_{\text{inst}} \leqslant$ 6mm where w_{inst} = instantaneous horizontal deflection at each eaves caused by splaying of collar truss under dead + imposed load
Masonry walls in dwellings – horizontal deflection at each end of a raised tie truss	Irreversible	Final	$w_{\text{inst}} + w_{\text{creep}} \leqslant h_e/650$ where w_{inst} is defined as above and w_{creep} is creep deformation under all loads h_e = height of building to eaves

3.2.3 Creep effects in assemblies

In an assembly consisting of timber components with different creep properties or a mixture of members made of timber and other materials, the distribution of moments and forces will change over time. This also occurs in assemblies with mechanically fastened connections, because EC5 states that the value of k_{def} should be doubled for mechanically fastened connections (see Section 6.13.4), giving them different time-dependent properties from the main timber members. The final stresses and deformations may be calculated using the appropriate design loads from Section 3.2.1.3 i) to iv) for ULS or Section 3.2.2.2 i) for SLS, in conjunction with reduced values of the stiffness moduli of each member and connection. These are calculated as follows:

$$E_{mean,fin} = \frac{E_{mean}}{(1 + k_{def})} \qquad G_{mean,fin} = \frac{G_{mean}}{(1 + k_{def})} \qquad K_{ser,fin} = \frac{K_{ser}}{(1 + k_{def,joint})}$$

using the appropriate values for each member or connection. For the values of K_{ser} and $k_{def,joint}$ see Sections 6.13.2 and 6.13.4 respectively.

It is recommended that these expressions be used for both ultimate and serviceability limit states.

3.2.4 Use of frame analysis programs

For solid timber, glulam and LVL use $E_{0,mean}$ and G_{mean}. If values for stiffness moduli perpendicular to the grain are required, use $E_{0,mean}/30$ for softwoods and softwood glulam, $E_{0,mean}/15$ for hardwoods and $G_{mean}/16$ for both.

For plywood and OSB use $E_{ct,0}$ or $E_{ct,90}$ as appropriate, and G_{mean}, the shear modulus for panel shear.

For plywood use the value of E_{ct} appropriate to the direction of load to the grain direction of the surface veneer.

For LVL see Table 3.16 or the manufacturer's literature. (See Tables 3.14 to 3.18 for values.)

For initial designs ignore slip in connections. For final, more accurate designs, if the program allows a spring stiffness to be entered at connections, use the appropriate value of the slip modulus (see Section 3.2.3). Otherwise insert a short fictitious element between each member and its adjacent connections, continuous with the member and with a stiffness equivalent to the connection – e.g. a 1mm long element with a 1mm² cross-section and a modulus of elasticity of $E = nK_{ser}$ N/mm² or $E_{fin} = nK_{ser,fin}$ N/mm², as appropriate, where n is the number of fasteners. (With bolted joints an additional 1mm lack of fit should be entered.)

To analyse an assembly after creep occurs use the approximate method given in Section 3.2.3 or refer to TRADA's European Guidance Document GD5[25].

Some frame analysis programs require values of Poisson's ratio, ν, which are used to calculate shear moduli via the formula $G = 0.5E/(1 + \nu)$. This formula is not applicable to timber and wood based composites, so where Poisson's ratio is required an equivalent value ν' should be inserted, where $\nu' = (E - 2G)/2G$.

3.2.5 Floor vibration

Limiting floor vibration is a serviceability limit state which is covered in Section 8.4.3.2.

3.3 Timber materials

3.3.1 Structural timber materials

A summary of the most common types of structural timber materials is shown in Table 3.6. Definitions are provided in the Glossary.

Benefits of using timber

- Timber has a relatively high strength:weight ratio compared with other materials, producing light-weight structures which require lighter foundations and make them particularly suitable for brownfield developments.
- Timber materials are relatively resistant to both acids and alkalis, and are therefore a good choice for corrosive environments such as salt barns and swimming pools.
- Most structural timber originates from sustainable plantations (and may be specified as such) so in general it may be regarded as a renewable material.
- Timber's low thermal conductivity and low thermal mass help to ensure good heat retention and reduce the heat required to warm a building up.
- Producing structural timber materials requires relatively little energy compared with other structural materials, resulting in less damage to the environment.
- Sources of information on the procurement of timber from sustainable sources are listed on the website of CPET, the Central Point of Expertise on Timber Procurement.

3.3.2 Dimensions and tolerances

For further information see BS EN 336[26], BS EN 1313-2[27] and BS EN 390[28].

3.3.2.1 Length

Solid timber for construction is normally available in lengths from 1.8m to 5.4m or up to 7.2m on order, in steps of 0.3m. European oak can be obtained in lengths up to 8m, or a little more by special arrangement, and tropical hardwoods up to 9m to 12m, depending on the species. The maximum length of LVL that is produced is 23m and the maximum length of glulam is limited only by transport considerations, however maximum stock lengths of LVL and glulam are generally 12m and 15m respectively.

3.3.2.2 Thickness and width

Timber is initially sawn to size: the resulting dimensions are commonly called 'sawn sizes'. It may then be machined or planed to a more exact size as follows:

- timber machined on the two narrower faces is particularly suitable for joists for which an accurate depth is required, this is commonly called 'regularised timber' but should be called 'machined on the width'
- timber machined on all four faces is commonly referred to as 'PAR' or planed all round. It is used in trussed rafters and wall panels and for exposed timbers where appearance is important
- 'CLS/ALS' timber (Canadian Lumber Sizes or American Lumber Sizes) is machined all round and is imported from North America and some Scandinavian countries. It has rounded arrises and is usually lightly waxed.

Standard cross-sectional dimensions are shown in Section 3.3.2.4.

Table 3.6 Structural timber materials

Material	Uses	Comments
Solid softwood	Joists, light beams, wall framing, light columns, roofing, bracing members, concrete formwork. May be used externally with preservative treatment	Relatively cheap and readily available from managed plantations. Less strong than other timber alternatives
Solid hardwood	Beams and columns, exposed beams and columns, portal frames, timber bridges, large roof members, harbours and groynes	Stronger and stiffer than softwood, and many species are available in larger sections. Some species are more durable than softwoods and are more resistant to fire. Care needed if required from a sustainable source
Glulam, LVL, PSL, LSL	Beams, columns, roofing, arches, domes and portal frames, timber bridges. May be used externally with preservative treatment and good detailing	Large members, more stable dimensionally than solid timber. STCs are stiffer and stronger than solid softwood of the same species. Best protected from the weather if used externally
Plywood	Floor and roof decking, wall sheathing, box beam and I-joist webs, truss gussets	Exterior grades may be preferable to OSB for more humid environments or situations where wetting may occur or under long term loading, e.g. in flat roofs. Birch plywood is stronger than OSB and particleboard
OSB	Floor and roof decking, wall sheathing, box beam and I-joist webs	The preferred choice on cost grounds for timber frame wall panels and the webs of I-joists. Also used for floor decking, particularly the structural decking in party floors. High creep factor under long term loading
Particle-board (also known as chipboard)	Floor decking	Its mass and price make it very suitable for floor decking in a dry environment. Must not be exposed to the weather on building sites, and is more brittle than plywood. High creep factor under long term loading
Hardboard	Structural sheathing and box beam and I-joist webs in dry environments	Tempered hardboard is stronger and more resistant to water absorption than other types of fibre building board. Its properties are not covered in this *Manual*

<table>
<tr><th colspan="4">Table 3.7 Dimensional tolerances for solid timber, glulam and LVL</th></tr>
<tr><td colspan="4">Solid timber (BS EN 336[26])</td></tr>
<tr><td>Tolerance class</td><td colspan="2">For dimensions ⩽ 100mm</td><td>For dimensions > 100mm</td></tr>
<tr><td>T1 – sawn dimensions</td><td colspan="2">+3/-1mm</td><td>+4/-2mm</td></tr>
<tr><td>T2 – machined dimensions</td><td colspan="2">±1mm</td><td>±1.5mm</td></tr>
<tr><td>Length</td><td colspan="3">For supply: not less than the length specified
For construction: at the designer's discretion</td></tr>
<tr><td colspan="4">Glulam (BS EN 390[28])</td></tr>
<tr><td>Thickness, b</td><td colspan="3">±2mm</td></tr>
<tr><td rowspan="2">Depth, h</td><td colspan="2">For $h \leqslant$ 400mm</td><td>For h > 400mm</td></tr>
<tr><td colspan="2">+4/-2mm</td><td>+1/-0.5mm per 100mm</td></tr>
<tr><td>Squareness of cross-section</td><td colspan="3">Maximum deviation from a right angle:
2mm per 100mm, or ± 1.15°</td></tr>
<tr><td rowspan="2">Length, l</td><td>For $l \leqslant$ 2m</td><td>For 2m < $l \leqslant$ 20m</td><td>For l > 20m</td></tr>
<tr><td>± 2mm</td><td>± 1mm per metre</td><td>± 20mm</td></tr>
<tr><td colspan="4">LVL (BS EN 14374[17])</td></tr>
<tr><td>Thickness, b</td><td colspan="3">+(0.8+0.03b)/-(0.4+0.03b) mm</td></tr>
<tr><td rowspan="2">Width, h</td><td colspan="2">For h < 400mm</td><td>For $h \geqslant$ 400mm</td></tr>
<tr><td colspan="2">± 2mm</td><td>± 0.5 %</td></tr>
<tr><td>Squareness of cross-section</td><td colspan="3">Maximum deviation from a right angle:
2mm per 100mm, or ± 1.15°</td></tr>
<tr><td>Length, l</td><td colspan="3">± 5mm</td></tr>
</table>

3.3.2.3 Target sizes and tolerances

Thickness and width are specified as target sizes which refer to the dimensions at 20% moisture content for solid timber and at 12% moisture content for glulam and structural timber composites. Engineering calculations should be based on target sizes, even though at other moisture contents the thickness and width will change a little (see Section 2.8.1). This is because reductions in cross-section are compensated for by increases in strength and stiffness, and vice versa. The permitted tolerances on target sizes at the reference moisture content are shown in Table 3.7.

3.3.2.4 Standard sizes

Tables 3.8 to 3.10 show the most commonly available target sizes for softwood and hardwood sections used in UK construction. Tables 3.11 to 3.13 show the most commonly available glulam and LVL section sizes.

Table 3.8 Preferred softwood sizes (BS EN 336[26])

Thickness (mm)		Finish	Width (mm)									
Finish		Sawn	75	100	120	150	175	200	225	250	275	300
Sawn	Machined	Machined	72	97	120	145	170	195	220	245	270	295
22	19		□	▣	▣	▣	▣	▣	▣	□	□	□
25	22		▣	▣	▣	▣	▣	▣	▣	□	□	□
38	35		■	■	■	■	■	■	■	□	□	□
47	44		■	■	■	■	■	■	■	■	□	■
63	60		□	▣	▣	▣	▣	▣	▣	□	□	□
75	72		■	■	▣	■	■	■	■	▣	▣	▣
100	97		□	■	□	▣	□	■	■	▣	□	▣
150	145		□	□	□	▣	□	▣	□	□	□	▣

Key: ■ = most commonly available
▣ = generally available
□ = not generally available

Table 3.9 CLS/ALS (planed all round) softwood sizes (BS EN 336[26])[a]

Thickness (mm)	Width (mm)						
	63	89	114	140	184	235	285
38	▣	■	▣	■	■	■	▣

Key: ■ = most commonly available
▣ = generally available

Note

a These sizes are commonly used for wall studs in platform timber frame buildings.

Table 3.10 Preferred hardwood sizes (BS EN 1313-2[27])

Thickness (mm)		Finish	Width (mm)							
Finish		Sawn	50	60	70	80	90	100	120	Continue in 20mm steps up to 300mm depending on species
Sawn	Machined	Machined	47	57	67	77	87	97	115	
19	16		▣	▣	▣	▣	▣	▣	▣	▣
26	24		▣	▣	▣	▣	▣	▣	▣	▣
38	35		▣	▣	▣	▣	▣	▣	▣	▣
52	49		▣	▣	▣	▣	▣	▣	▣	▣
63	60		▣	▣	▣	▣	▣	▣	▣	▣
75	72		▣	▣	▣	▣	▣	▣	▣	▣

Key: ▣ = preferred sizes from BS EN 1313-2 (consult supplier for availability)

Table 3.11 Typical glulam sections – based on Imperial sizes (more common)

Breadth (mm)	Depth (mm)										
	225	270	315	360	405	450	495	540	585	630	675
65	■	■	■	■							
90	■	■	■	■	■	■	□				
115	■	■	■	■	■	■	■	□		■	
140	■	■	■	■	■	■	■	□	□	■	□
165						□	□	□	□	□	□
190								□	□	□	□

Key: ■ = typical sizes available off the shelf
□ = typical sizes

Note
a Square and round sections are also available for columns.

Table 3.12 Typical glulam sections – Metric sizes

Breadth (mm)	Depth (mm)						
	240	280	320	360	400	480	520
60	□	□	■				
80	□	□	■	□	■		
100	□	□	□	□	■	■	
120	□	□	□	□	□	■	□
140	□	□	□	■	□	□	□
160	□	□		■	□	□	□
180	□	□	□	■	□		
200	□	□	□	■	□		

Key: ■ = typical sizes available off the shelf
□ = typical sizes

Note
a Square and round sections are also available for columns.

Table 3.13 Typical LVL sections

Breadth (mm)	Depth (mm)									
	200	260	300	360	400	450	500	600	900	1800
27	■	■	■	■	■	■	■	■	■	■
33	■	■	■	■	■	■	■	■	■	■
39	■	■	■	■	■	■	■	■	■	■
45	■	■	■	■	■	■	■	■	■	■
51	■	■	■	■	■	■	■	■	■	■
57	■	■	■	■	■	■	■	■	■	■
63	■	■	■	■	■	■	■	■	■	■
75	■	■	■	■	■	■	■	■	■	■

Key: ■ = typical sizes

Glulam can be manufactured in very large sections and lengths, but lengths above 15m may be difficult to transport. One large manufacturer quotes data for sizes from 90mm wide × 180mm deep to 240mm wide × 2050mm deep, and up to 31m in length. The Glued Laminated Timber Association (GLTA) quotes data for sizes from 65mm × 180mm to 215mm × 1035mm. Some manufacturers also stock glulam machined to a round cross-section. Glulam is usually manufactured from laminations 33mm or 45mm thick: 33mm for curved members and 45mm for straight or nearly straight members. Some straight glulam made from pressure-treated material may also use 33mm laminates. The depth is therefore generally a multiple of 33mm or 45mm. For more information consult a supplier or the GLTA.

Before completing a design it is advisable to check with a stockist that the proposed sizes are available. Prefabricated timber joists are made in standard sizes which should be selected from the catalogue of the chosen manufacturer. There are also specialist manufacturers of timber engineering hardware whose catalogues should be consulted for both sizes and prices. A comprehensive on-line list of stockists of timber and timber-related products can be accessed at www.trada.co.uk.

3.3.2.5 Strength grading and strength classes

All timber for load-bearing use in construction must be strength graded by an approved grading body, either visually according to standardised rules or by means of a grading machine supplemented by visual rules. (For further information see references 13, 22, 29, 30 and 31.) The visual grades used in the UK are GS (General structural) and SS (Special structural) for softwoods and HS (Hardwood structural) for tropical hardwoods. There are special grades for the home grown temperate hardwoods oak and sweet chestnut, and for unseasoned oak.

To simplify design, combinations of species and grade are grouped into sets of strength classes, each of which has a corresponding set of characteristic material properties which are safe for all the species/grade combinations in the class. The strength classes for softwoods, poplar and tropical hardwoods are designated by a letter and a number. The letter ‘C’ (coniferous) generally indicates softwoods or poplar and ‘D’ (deciduous) generally indicates hardwoods, while the number refers to the characteristic bending strength of the class in N/mm^2.

Glulam is manufactured from graded timber in one of two forms: homogenous, in which every laminate is of the same grade, or combined, in which higher grades are used in the outer laminates. The resulting lay-ups are assigned to special glulam strength classes, each with its own set of characteristic material properties.

Every length of graded solid timber or glulam must be stamped with the grade, species or strength class, and the registered number of the grading agency.

Solid timber and glulam are normally specified by strength class. Where there are particular requirements, e.g. appearance or local source, a species and strength grade may be specified instead. For solid hardwoods, it is usually necessary to specify ‘HS grade’ and a particular species.

3.3.3 Characteristic values

Tables 3.14 to 3.18 give characteristic values for some common timber materials. For more comprehensive tables see the CD.

The bending strength and stiffness classes for plywood given in BS EN 12369-2[32] appear to be of little practical use. Either seek characteristic values directly from the manufacturers, or convert the grade values tabulated in BS 5268-2[3] for particular types of plywood to safe characteristic values as follows:

$$X_k = 2.7X_{grade}$$
$$E_k = 1.8E_{grade}$$

Where X_k = a characteristic strength property
X_{grade} = the corresponding grade strength property from BS 5268-2 Tables 40-56
E_k = a characteristic stiffness property
E_{grade} = the corresponding grade stiffness property from BS 5268-2 Tables 40-56

(The factor for stiffness properties is required because the E values tabulated for plywood in BS 5268-2 are for final stiffness, i.e. they are, on average, $E_{inst}/1.8$.)

Table 3.14 Characteristic properties for some common strength classes of solid softwood (from BS EN 338[12])[a]

Property	Symbol	Strength class		
		C16	C24	C27
Strength properties (N/mm^2)				
Bending parallel to grain	$f_{m,k}$	16	24	27
Tension parallel to grain	$f_{t,0,k}$	10	14	16
Tension perpendicular to grain	$f_{t,90,k}$	0.5	0.5	0.6
Compression parallel to grain	$f_{c,0,k}$	17	21	22
Compression perpendicular to grain	$f_{c,90,k}$	2.2	2.5	2.6
Shear parallel to grain	$f_{v,k}$	1.8	2.5	2.8
Stiffness properties (N/mm^2)				
Mean MOE parallel to grain	$E_{0,mean}$	8000	11000	11500
5th percentile MOE parallel to grain	$E_{0,05}$	5400	7400	7700
Mean MOE perpendicular to grain	$E_{90,mean}$	270	370	380
Mean shear modulus	G_{mean}	500	690	720
Density (kg/m^3)				
Mean	ρ_{mean}[b]	370	420	450
Minimum	ρ_k[c]	310	350	370

Notes

a A full table can be found on the CD.

b Used for calculating the weight.

c Used for calculating the strength of mechanically fastened connections.

Table 3.15 Characteristic values for some common strength classes of softwood glulam complying with BS EN 14080[16]
(values from BS EN 1194[33])

Property	Symbol	Glulam strength class (see Section 3.3.2.5)					
		Combined glulam			Homogenous glulam		
		GL24c	GL28c	GL32c	GL24h	GL28h	GL32h
Strength properties (N/mm²)							
Bending parallel to grain	$f_{m,k}$	24	28	32	24	28	32
Tension parallel to grain	$f_{t,0,k}$	14	16.5	19.5	16.5	19.5	22.5
Tension perp. to grain	$f_{t,90,k}$	0.35	0.4	0.45	0.4	0.45	0.5
Compression parallel to grain	$f_{c,0,k}$	21	24	26.5	24	26.5	29
Compression perp. to grain	$f_{c,90,k}$	2.4	2.7	3.0	2.7	3.0	3.3
Shear parallel to grain	$f_{v,k}$	2.2	2.7	3.2	2.7	3.2	3.8
Stiffness properties (N/mm²)							
Mean MOE parallel to grain	$E_{0,mean}$	11600	12600	13700	11600	12600	13700
5th percentile MOE parallel to grain	$E_{0,05}$	9400	10200	11100	9400	10200	11100
Mean MOE perp. to grain	$E_{90,mean}$	320	390	420	390	420	460
Mean shear modulus	G_{mean}	590	720	780	720	780	850
Density (kg/m³)							
Mean	ρ_{mean} [a]	395	430	460	420	450	475
Minimum	ρ_k	350	380	410	380	410	430

Notes

a Calculated as 1.1 ρ_k for homogenous glulam, 1.125 ρ_k for combined glulam.

Table 3.16 Characteristic values for some common types of LVL
(from VTT Certificate no. 184/03 March 2004[34])

Property	Symbol	Product[a]		
		LVL-S[b] Thickness 21 – 90mm	LVL-Q[c] Thickness 21 – 24mm	LVL-Q[c] Thickness 27 – 29mm
Strength properties (N/mm²)				
Bending strength				
Edgewise	$f_{m,0,edge,k}$	44.0	28.0	32.0
Size effect parameter	s	0.12	0.12	0.12
Flatwise	$f_{m,0,flat,k}$	50.0	32.0	36.0
Tensile strength				
Parallel to grain	$f_{t,0,k}$	35.0	19.0	26.0
Perpendicular to grain, edgewise	$f_{t,90,edge,k}$	0.8	6.0	6.0
Perpendicular to grain, flatwise	$f_{t,90,flat,k}$	–	–	–
Compressive strength				
Parallel to grain	$f_{c,0,k}$	35.0	19.0	26.0
Perpendicular to grain, edgewise	$f_{c,90,edge,k}$	3.4	9.0	9.0
Perpendicular to grain, flatwise	$f_{c,90,flat,k}$	1.7	1.7	1.7
Shear strength				
Edgewise	$f_{v,0,edge,k}$	5.7	5.7	5.7
Flatwise	$f_{v,0,flat,k}$	4.4	1.3	1.3
Stiffness properties (N/mm²)				
Modulus of elasticity				
Mean parallel to grain	$E_{0,mean}$	13500	10000	10500
Mean perpendicular to grain	$E_{90,mean}$	–	–	–
5th percentile parallel to grain	$E_{0,k}$	11600	8300	8800
5th percentile perpendicular to grain	$E_{90,k}$	–	–	–
Shear modulus				
Mean, edgewise	$G_{0,edge,mean}$	600	600	600
Mean, flatwise	$G_{0,flat,mean}$	600	–	–
5th percentile, edgewise	$G_{0,edge,k}$	400	400	400
5th percentile, flatwise	$G_{0,flat,k}$	400	-	-
Density (kg/m³)				
Mean	ρ_{mean}	510	510	510
Minimum	ρ_k	480	480	480

Notes
a For precise values refer to the manufacturers' literature.
b LVL-S: the grain direction of all the veneers is the same.
c LVL-Q: some veneers are cross-grained to increase its dimensional stability in large panels.

Table 3.17 Ranges of characteristic values for common structural plywoods

These are for general guidance only. For design purposes use the manufacturer's values corresponding to the selected plywood type or convert from BS 5268-2[3] grade values as described previously in this Section.

Property	Symbol[a]	Characteristic values
Strength properties (N/mm²)		
Bending strength		
Parallel to surface grain	$f_{m,0,k}$	20 - 49
Perpendicular to surface grain	$f_{m,90,k}$	15 - 36
Tensile strength		
Parallel to surface grain	$f_{t,0,k}$	10 - 51
Perpendicular to surface grain	$f_{t,90,k}$	10 - 44
Compressive strength		
Parallel to surface grain	$f_{c,0,k}$	24
Perpendicular to surface grain	$f_{c,90,k}$	24 - 27
Compression on face	$f_{c,k}$	7 - 11
Shear strength		
Panel, used to calculate racking strength	$f_{v,k}$	4.1 - 13
Rolling, in plane of board	$f_{v,r,k}$	1.2 – 3.3
Transverse, in bending parallel to surface grain	$f_{v,0,k}$	1.5 – 3.6
Transverse, in bending perpendicular to surface grain	$f_{v,90,k}$	1.4 – 3.1
Stiffness properties (N/mm²)		
Modulus of elasticity		
Mean, in bending parallel to surface grain	$E_{0,mean}$	8100 - 8600
Mean, in bending perpendicular to surface grain	$E_{90,mean}$	3000 - 5800
Mean, in tension and compression parallel to surface grain	$E_{ct,0,mean}$	6200 - 7600
Mean, tension and compression perpendicular to surface grain	$E_{ct,90,mean}$	6500 - 6800
Shear modulus		
Panel, for racking	G_{mean}	450 - 580
Density		
Mean	ρ_{mean} [b]	430 - 690
Minimum	ρ_k [c]	390 - 620

Notes

a '0' and '90' refer to the direction of the grain in the surface veneers. In most plywoods this is parallel to the longer sides of the sheet, but in Finnish plywoods with birch in them (birch, combi, mirror and twin) it is parallel to the shorter sides.

b Calculated from the approximate masses per unit area quoted in BS 5268-2.

c Calculated as $0.9 \times \rho_{mean}$.

Table 3.18 Characteristic values for OSB/3[a] (from BS EN 12369-1[35])[b]				
Property	**Symbol**	**Thickness range (mm)**		
		6+ to 10	**10+ to 18**	**18+ to 25**
Strength properties (N/mm²)				
Bending strength				
Parallel to span	$f_{m,0,k}$	18.0	16.4	14.8
Perpendicular to span	$f_{m,90,k}$	9.0	8.2	7.4
Tensile strength				
Parallel to span	$f_{t,0,k}$	9.9	9.4	9.0
Perpendicular to span	$f_{t,90,k}$	7.2	7.0	6.8
Compressive strength				
Parallel to span	$f_{c,0,k}$	15.9	15.4	14.8
Perpendicular to span	$f_{c,90,k}$	12.9	12.7	12.4
Shear strength				
Panel (as in a racking panel)	$f_{v,k}$	6.8	6.8	6.8
Planar (as in floor decking)	$f_{v,r,k}$	1.0	1.0	1.0
Stiffness properties				
Modulus of elasticity				
Mean, in bending parallel to span	$E_{0,mean}$	4930	4930	4930
Mean, in bending perp. to span	$E_{90,mean}$	1980	1980	1980
Mean, in tension and compression parallel to span	$E_{ct,0,mean}$	3800	3800	3800
Mean, in tension and compression perpendicular to span	$E_{ct,90,mean}$	3000	3000	3000
Shear modulus				
Panel (as in a racking panel)	$G_{v,mean}$	1080	1080	1080
Planar (as in floor decking)	$G_{r,mean}$	50	50	50
Density				
Mean	ρ_{mean} [c]	650	650	650
Minimum	ρ_k	550	550	550

Notes

a OSB/3 to BS EN 300[20].

b BS EN 12369-1 gives higher values for heavy duty boards, which may also be found in the CD. Compression on the face of the board is not listed in the standards. Boards certified for use in flooring have passed tests which demonstrate their ability to resist point loads.

c Calculated as 1/0.85 times the characteristic density.

3.3.4 Design values of strength properties

The design value of a strength property is calculated as

$$X_d = k_{mod} \Phi \frac{X_k}{\gamma_M}$$

Where: k_{mod} = modification factor for service class and load duration (see Table 2.3)

Φ = multiple of relevant member and system modification factors (see Table 3.20)

X_k = characteristic value of strength property (see Tables 3.14 to 3.18)

γ_M = material safety factor (see Table 3.19)

Table 3.20 references the clauses in EC5 from which all the modification factors may be calculated if required. For compression strength at an angle to the grain, see EC5 6.2.2.

Table 3.19 Partial factors for material properties and connections

Fundamental combinations	γ_M
Solid timber, untreated or treated with preservatives	**1.3**
Glulam	**1.25**
LVL, plywood and OSB	**1.2**
Punched metal plate fasteners – anchorage strength	**1.3**
Punched metal plate fasteners – strength of steel plate	**1.15**
Other mechanically fastened connections	**1.3**
Accidental combinations	
All materials and connections	**1.0**

Table 3.20 Principal member and system modification factors

Symbol and EC5 reference	Factor	Property modified	Value	Comment
k_{mod} 3.1.3	Effect of service class and load duration on strength	All strength properties	See Table 2.3, or for prefabricated timber joists consult manufacturer's literature	
k_{def} 3.1.4	Effect of service class and time on deformation of loaded members	E G k_{ser} k_u	See Table 2.4, or for prefabricated timber joists consult manufacturer's literature	Not normally relevant to ULS but can be if creep affects the stress distribution

Table 3.20 (continued)

Symbol and EC5 reference	Factor	Property modified	Value	Comment
k_h 3.2(3)	Bending and tension strength for solid timber	$f_{m,k}$ $f_{t,0,k}$	$k_h = \min.\begin{Bmatrix}\left(\frac{150}{h}\right)^{0.2}\\1.3\end{Bmatrix}$ for h < 150mm	h = depth in bending or max. cross-section dimension in tension, in mm. Valid for $\rho_k \leqslant 700\text{kg/m}^3$ Not applicable for $h \geqslant 150\text{mm}$
k_h 3.3(3)	Bending and tension strength for glulam	$f_{m,k}$ $f_{t,0,k}$	$k_h = \min.\begin{Bmatrix}\left(\frac{600}{h}\right)^{0.1}\\1.1\end{Bmatrix}$	h = depth in bending or maximum cross-section dimension in tension, in mm
k_h 3.4(3)	Bending strength for LVL	$f_{m,k}$	$k_h = \min.\begin{Bmatrix}\left(\frac{300}{h}\right)^{s}\\1.2\end{Bmatrix}$	h = depth in bending in mm s = size effect parameter declared by manufacturer (see Table 3.15)
k_h 3.4(4)	Tension strength for LVL	$f_{t,0,k}$	$k_l = \min\begin{Bmatrix}\left(\frac{3000}{l}\right)^{\frac{s}{2}}\\1.1\end{Bmatrix}$	l = length of member in mm s = size effect exponent declared by manufacturer (see Table 3.15)
k_{shear}	Shear strength for deep sections of solid and glued laminated timber and LVL	$f_{v,k}$	A reduction in the shear capacity of deep timber sections is to be introduced to EC5 to allow for drying splits and glue-line failure. Meanwhile it is suggested that for sections with a depth > 100mm only 75% of the shear capacity should be used	
k_{crit} 6.3.3(3) and 6.3.3(4)	Bending strength for members subject to lateral torsional buckling[a]	$f_{m,d}$	Table 3.21 and Figures 3.2 – 3.4 (see also Table 4.2.)	Reduces bending strength when relative slenderness ratio in bending, $\gamma_{rel,m} > 0.75$[b]

Table 3.20 (continued)

Symbol and EC5 reference	Factor	Property modified	Value	Comment
k_m 6.1.6	Secondary bending stress in biaxial bending (with or without axial tension or compression)	$\sigma_{m,d}$ when it is the secondary stress	k_m = 0.7 for rectangular or square beams made of solid timber, glulam or LVL. Otherwise k_m = 1.0	Allows for the redistribution of stress when the neutral axis is skewed under biaxial bending in a rectangular member
$k_{c,y}$, $k_{c,z}$ 6.3.2(3)	Compression strength parallel to the grain for members subject to lateral buckling	$f_{c,0,d}$	Table 3.22 and Figures 3.5 and 3.6	Allows for possibility of buckling
$k_{c,90}$ 6.1.5	Compression strength perpendicular to the grain for solid timber, glulam and LVL	$f_{c,90,d}$	Figures 3.7 and 3.8	EC5 6.1.5 has more comprehensive design rules if these are required. It is anticipated that the method for calculating $k_{c,90}$ will be simplified in the near future
k_{sys} 6.6	System strength	All affected strength properties	1.1	Applicable when several[c] equally spaced, similar members, components or assemblies are connected in such a way that load can be transferred effectively between them[d,e,f]

Table 3.20 (continued)

Notes

a k_{crit} may be taken as 1.0 if lateral displacement of the compressive edge is prevented throughout the length and torsional rotation is prevented at the supports, provided that $h/b \leqslant 5$ (see Table 4.3).

b $\lambda_{rel,m}$ is defined in EC5 6.3.3(2). It is used to calculate k_{crit}, which can be obtained directly from Figures 3.2 to 3.4.

c The word ‘several’ is ambiguous, but EC5 6.6(2) states that k_{sys} is applicable if load can be transferred from a member to “the neighbouring members” which suggests more than two. It is recommended that k_{sys} should be applied when there are at least four load-sharing members such as rafters, joists, trusses or wall studs with an intermediate load transfer system, or when at least two members are fastened securely together as in trimmer joists and lintels. This follows traditional UK guidance in BS 5268-2[3].

d k_{sys} may be applied to roof trusses spaced no more than 1200mm apart if they are adequately connected by tiling battens, purlins or panels spanning continuously over at least three trusses.

e Caution is required when applying k_{sys} to load-sharing members made of LVL or other composite materials which do not have the variability of solid timber. In such cases refer to the manufacturer’s literature.

f EC5 6.6(3) states that short-term load duration should be used to verify the strength of a load-distribution system. This refers to the members and connections which distribute the load between the roof trusses or joists, etc. which they connect together. In many cases, as with the roof trusses described above, it may be assumed that the strength of the load distribution system is adequate.

Figures 3.2 to 3.4 require a knowledge of the effective length, l_{ef}, of a member in bending. This can be found in Table 3.21, or from the distance between adjacent points of zero bending moment between which the member is in single curvature.

Table 3.21 Effective lengths in bending, l_{ef} [a]			
End restraint against rotation in plan	**End restraint against torsional rotation**	**Section**	**Effective length l_{ef}**
Single span – uniformly distributed load			
None	Partial		$1.20l$
None	Full		$1.00l$
Full at one end	Full		$0.85l$
Partial at both ends	Full		$0.85l$
Full at both ends	Full		$0.70l$
Single span – concentrated load at mid-span [b]			
None	Full		$0.90l$
Multiple span – uniformly distributed load			
None	Full	End span	$0.90l$
None	Full	Internal span	$0.85l$

Notes

a The values in this table assume that the beam is loaded on its compression edge. This is a worse case than a beam loaded at its centre of gravity, as assumed in EC5 Table 6.1.

b For other restraint conditions the tabulated values may be used conservatively with a concentrated load at mid-span.

In Figures 3.2 to 3.8 it is possible to interpolate between lines for the exact value of the required property on the 'y' axis. The values shown in Figures 3.2 and 3.3 are based on the grade which has the minimum value of $E_{0,05}/f_{m,k}$, i.e. C30 in Figure 3.2 and D40 in Figure 3.3. For other grades the values of k_{crit} will be conservative.

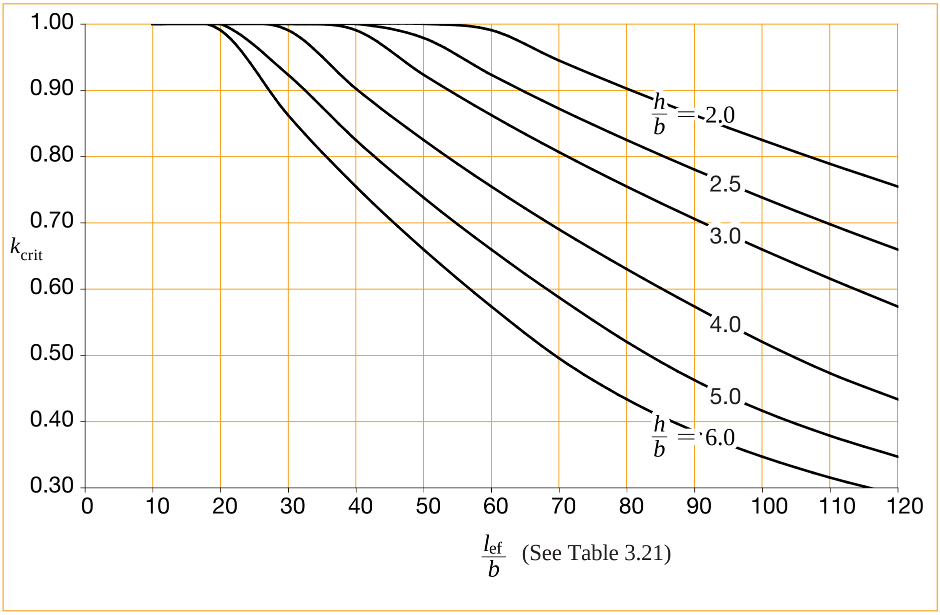


Fig 3.2 Values of k_{crit} for solid softwood strength classes C14 to C30 (derived from EC5 6.3.3(4))

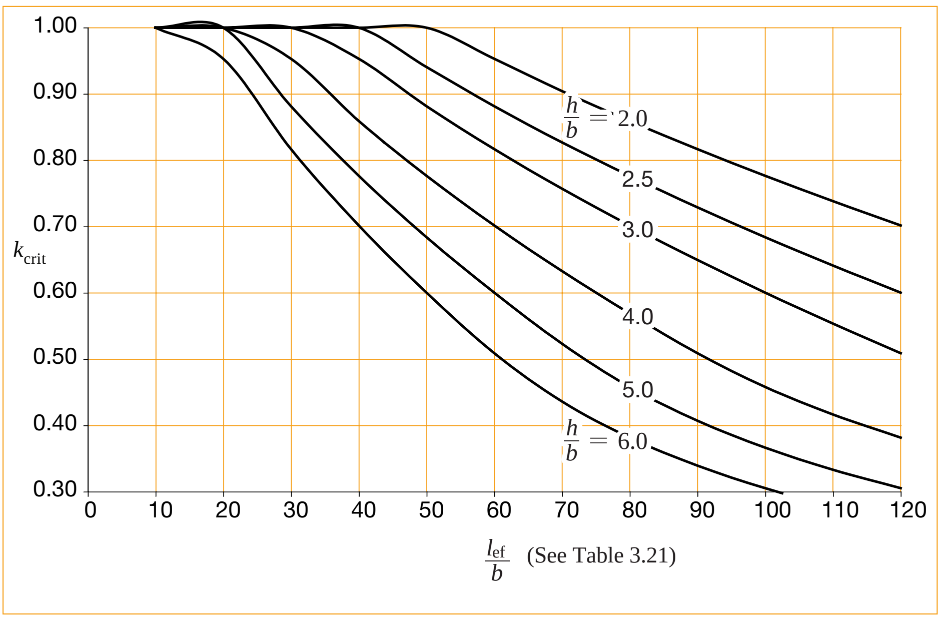


Fig 3.3 Values of k_{crit} for solid hardwood strength classes D30 to D70 (derived from EC5 6.3.3(4))

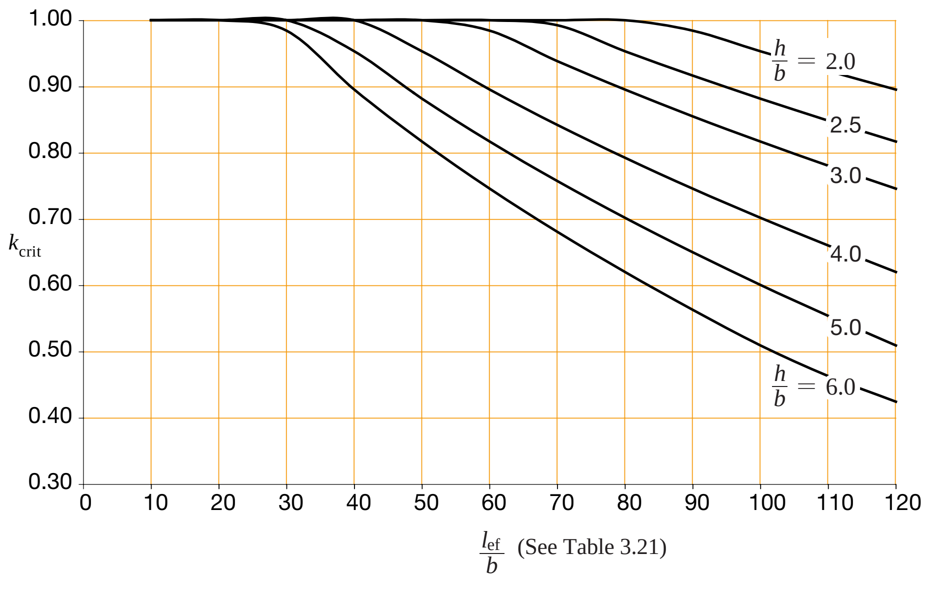


Fig 3.4 Values of k_{crit} for glulam strength classes GL24h and GL24c (derived from EC5 6.3.3(4)). For glulam classes see Table 3.15.

Figures 3.5 and 3.6 require a knowledge of the effective length, l_{ef}, of a member in axial compression. Standard values can be found in Table 3.22, where l is the length of the member. These values assume that the member is unbraced along its length. A timber frame wall stud nailed at 300mm centres or less on one side to structural sheathing may be considered to have an effective length of $0.85l$.

Table 3.22 Effective lengths in compression, l_{ef}

Restraints applied at each end		Effective length l_{ef}
End 1	**End 2**	
Position, direction	Position, direction	$0.70l$
Position, direction	Position	$0.85l$
Position	Position	$1.00l$
Position, direction	(Direction)	$1.50l$
Position, direction		$2.00l$

Notes

a 'Position' means that the end is restrained against lateral displacement in the direction in which buckling resistance is being checked.

b 'Direction' means that the end is restrained against rotation in the direction in which buckling resistance is being checked.

c '(Direction)' indicates partial restraint against rotation in the direction in which buckling resistance is being checked.

d l is the length of the member between points of intersection with supports or supporting members.

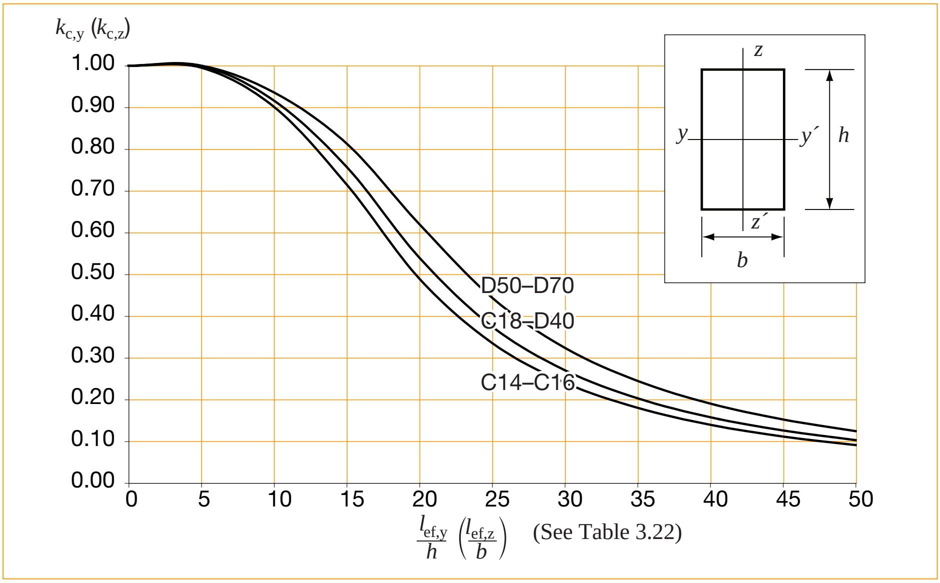


Fig 3.5 Values of $k_{c,y}$ and $k_{c,z}$ for columns: solid timber (derived from EC5 6.3.2(3))

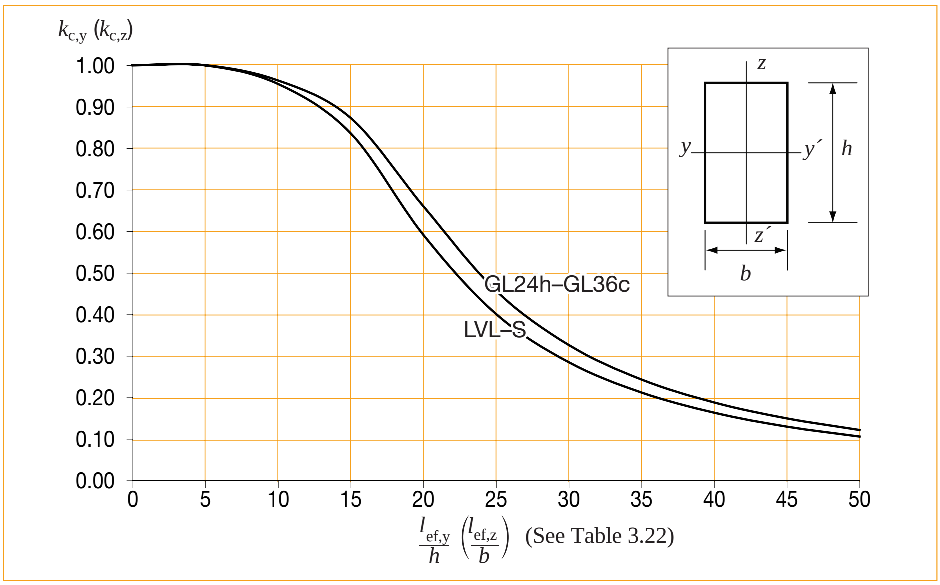
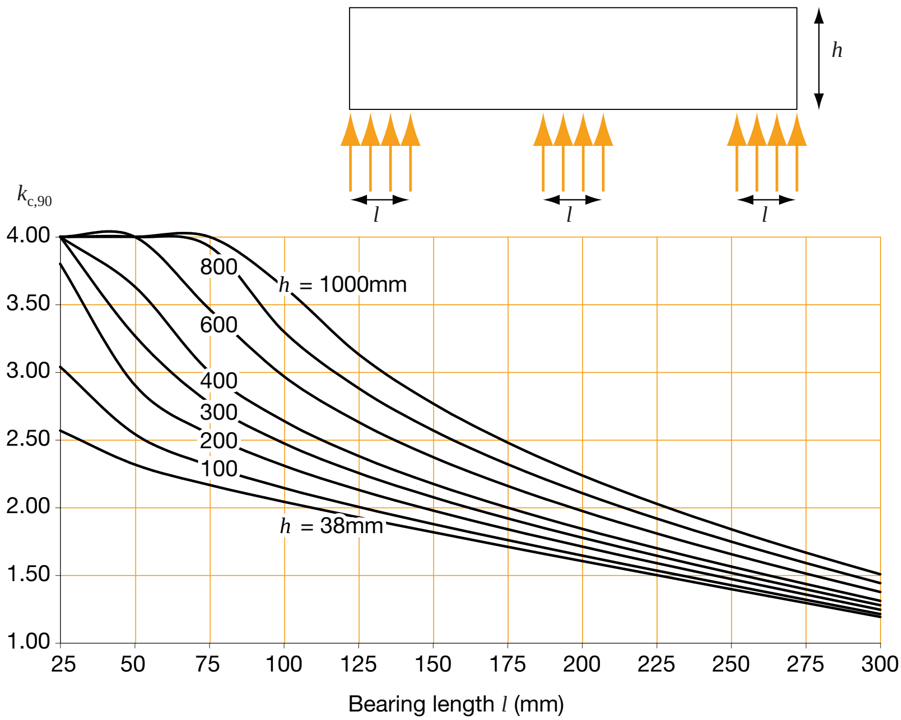


Fig 3.6 Values of $k_{c,y}$ and $k_{c,z}$ for columns: glulam and LVL (derived from EC5 6.3.2(3))

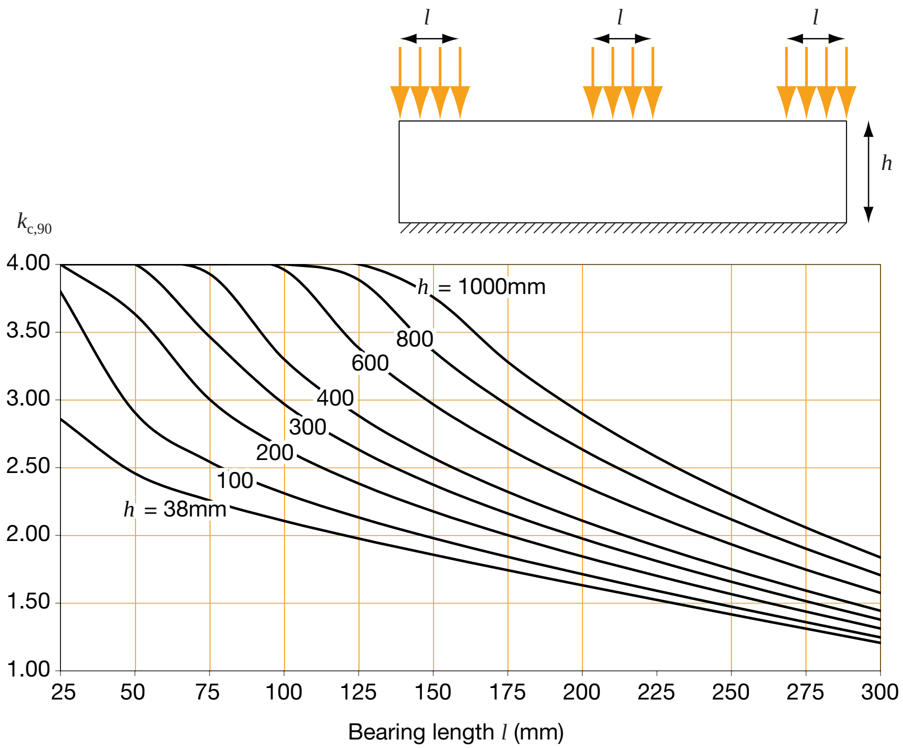
Note: k_c values for LVL, Figure 3.6 relate to a commonly available type. For final designs, k_c values should be checked against the manufacturer's published material properties.



Note

a These values are up to 20% conservative for bearings away from the end of a beam. See EC5 6.1.5 if exact values are required.

Fig 3.7 $k_{c,90}$, bearing strength modification factor for a timber member on discrete supports[a] (derived from EC5 6.1.5(4))



Note

a These values are up to 25% conservative for bearings away from the end of a bearer. See EC5 6.1.5 if exact values are required.

Fig 3.8 $k_{c,90}$, bearing strength modification factor for a continuously supported member subject to discrete loads[a] (derived from EC5 6.1.5(4))

3.3.5 Design values of stiffness properties

To calculate instantaneous deflections in flexural members E_{mean} and G_{mean} should be used for ULS and SLS design situations including the analysis of frames where the distribution of forces and moments depends on the member stiffness, except for the stability of beams and columns when the minimum value of the elastic modulus, $E_{0,05}$, should be used (see EC5 2.2.2 and 6.3). To calculate the instantaneous slip between mechanically fastened elements the serviceability slip modulus, K_{ser}, is used for SLS and the ULS slip modulus, K_u, for ULS (see Section 6.13.1). However the current edition of EC5 specifies the use of K_{ser} for both SLS and ULS limit states in the analysis of assemblies involving members or components with different time-dependent properties (see EC5 2.3.2.2).

Final values of E_{mean}, G_{mean} and K_{ser} may be calculated to determine final deformations after creep has occurred (see Section 3.2.3). For stability calculations the initial value of $E_{0,05}$ should be used. This is because the stability depends on the instantaneous elasticity which does not change over time.

3.4 Protective treatments

Consideration should be given to the possibility of timber decay or insect attack, and to the possibility of corrosion in steel fasteners and need for flamespread treatment.

3.4.1 Decay

Timber decay does not normally occur if the moisture content of the timber is below 20% for most of the year. This applies to all interior situations in the UK, including unheated areas such as wall cavities or cold roof spaces. Even timber exposed to the weather should not suffer from decay if it is adequately drained and ventilated. It is therefore important to design timber structures in such a way that the timber parts do not get wet, or if they do to ensure that water will drain away and that the members themselves will dry out quickly with the help of adequate ventilation. Structural posts and columns should be supported at least 150mm off the ground on concrete plinths or in steel shoes. Horizontal surfaces exposed to the weather should be sloped so that water can drain off them and if possible covered, e.g. by the use of metal coverings over glulam beams or arches. Particular care should be taken to ensure that water and moisture cannot collect within joints by providing drainage and ventilation.

For external structural timberwork it is generally advisable to specify a durable timber or a preservative treated softwood. Where a durable timber is chosen the specification should include the words ‘no sapwood’, because no sapwood is durable. For guidance on the durability of timber species see *Timbers: their properties and uses*[36].

3.4.2 Insect attack

The insects most likely to attack timber in normal buildings are furniture beetles (commonly known as woodworms), death watch beetles and the house longhorn beetle, although fortunately such attacks are uncommon in the British Isles. However it is a requirement of the Building Regulations for England and Wales[37] that softwood timbers within a roof space in designated parts of Hampshire and Surrey be protected against attack by house longhorn beetles by means of suitable preservative treatment (see Section 3.4.3).

3.4.3 Preservative treatment

Except as specified in Sections 3.4.1 to 3.4.2, treatment against decay is unnecessary in service class 1, but is normally required for softwoods at least in structural members exposed to service class 3. For service class 2 preservative treatment is normally unnecessary in properly designed internal cold areas, but is often specified for structural softwoods as an added safeguard and as an insurance against the accidental ingress of water. Currently European standards give no advice on the necessity of treatment, but BS 5268-5[38] recommends that a decision be based on the risk of decay, the natural durability of the heartwood of the timber chosen, and the cost of remedial work if decay or insect damage occurs. The factors considered are shown in Table 3.23. Table 3.24 shows some recommendations for the preservative treatment of non-durable species based on a combination of the above classifications. For risk classes 1 to 3 preservative treatment is generally unnecessary if a species classified as 'moderately durable' or better is specified and all sapwood is excluded. For risk class 4 a species classified as 'durable or better' may similarly be used.

For further guidance on the specification of preservative treatment see *Preservative treatment for timber – a guide to specification*[39], or *Industrial wood preservation – specification and practice*[40]. Do not use BS 5268-5[38] to select specific treatments, since some of those listed are now illegal.

3.4.4 Surface finishes

A suitable water-resistant coating may be specified for external timberwork (see *Finishes for external timber*[41]). Advice on seals for timber floors, although not normally part of the Engineer's remit, may be found in TRADA's *Seals for timber floors: a specification guide*[42].

Table 3.23 Factors for determining the necessity of preservative treatment
(based on 5268-5 Tables 2 and 3[38])

Risk of fungal decay			Implications of remedial work		
Risk class	**Level**	**Treatment**	**Class**	**Implication**	**Treatment**
1	Negligible	Unnecessary	A	Negligible	Unnecessary
2	Low	Optional	B	Repair is simple, so preservation is seen as insurance against expense	Optional
3	High	Desirable	C	Repair expensive	Desirable
4	Unacceptable	Essential	D	Could endanger persons or property	Essential
4	Marine	Essential			

Note
a For examples of interpretation of this table see Table 3.14.

Table 3.24 Recommendations for preservative treatment of non-durable timbers *(based on BS 5268-5 Table 4[34])*

Building element	Classification[a]	Treatment
Internal timber partitions	1B	Unnecessary
First and upper floor joists	1B	Unnecessary
Stairs	1B	Unnecessary
Ground floor joists with adequate ventilation and proper oversite treatment	2B	Optional
Dry pitched roofs – fungal attack	1C	Optional
Dry pitched roofs – common furniture beetle	2C	Optional
Dry pitched roofs – house longhorn beetle[b]	4C	Essential
Warm flat roofs (exposed joists or beams)	2C	Optional
Pitched roofs with a risk of wetting (main members and wall plates)	3C	Desirable
Pitched roofs with a risk of wetting (tiling battens)	2B	Optional
Cold flat roofs (enclosed joists or beams)	3C	Desirable
Cold flat roofs, high humidity (enclosed joists or beams)	4C	Essential
External timber frame wall cavity	2C	Optional
External timber cladding and battens	3B	Desirable
Timber sole plates above DPC	3C	Desirable
Timber below DPC or set in concrete[c]	4C/D	Essential
Load-bearing external joinery (balconies, etc.)	3C/D	Essential

Notes

a See Table 3.23.

b In geographical areas specified in the England and Wales Building Regulations[37].

c This situation should be avoided wherever possible.

3.4.5 Corrosion of metal parts

Steel parts in timber connections may be affected by corrosive environments such as those found in swimming pools or marine locations, or by tannins in the timber, particularly in oak, which attack mild steel. In such cases the use of stainless steel or other corrosion protection should be specified. Table 3.25 gives recommended specifications for the protection of common types of fastener in different service classes (see Section 2.15).

Table 3.25 Examples of minimum specifications for material protection against corrosion of fasteners and steel plates
(EC5 Table 4.1 with some modifications)

Fastener	Service class		
	1	2	3
Nails and screws with $d \leqslant$ 5mm	None	Fe/Zn 12c	Fe/Zn 25c
Bolts, dowels, nails and screws with $d >$ 5mm	None	None[a]	Z350
Staples	Fe/Zn 12c	Fe/Zn 12c	Stainless steel
Punched metal plate fasteners and steel plates with $t \leqslant$3mm	Fe/Zn 12c	Fe/Zn 12c	Stainless steel
Steel plates with 3mm < $t \leqslant$ 5mm	None	Fe/Zn 12c	Fe/Zn 25c
Steel plates with $t >$ 5mm	None	None	Z350

Notes

a BS 5268-3[43] recommends that all steel nails and bolts in trussed rafter roofs should be protected with a minimum of Fe/Zn 12c electroplating.

b Fe/Zn 12c and Fe/Zn 25c are electroplating specifications to BS EN 12329[44].

c Fe/Zn 12c may be replaced by a Z275 hot dip zinc coating to BS EN 10326[45].

d Z350 is a hot dip zinc coating specification to BS EN 10326.

e According to the draft of BS EN 14592[46], stainless steel should be austenitic stainless steel to BS EN 10088-1[47]: molybdenum-chrome-nickel alloy, preferred grade 1.4436 or chrome-nickel alloy, preferred grades 1.4301 or 1.4310.

f Consideration should be given to the use of heavier coatings or stainless steel in especially corrosive conditions such as marine environments, or for metalwork in contact with corrosive timbers such as Western Red Cedar, Douglas Fir and Oak in service class 3 or in large sections (100mm minimum dimension) which may not have fully dried out.

g For steel plates more than 3mm thick an independently certified corrosion protection paint system may be applied instead.

3.4.6 Spread of flame

Where Building Regulations require a class 0 or class 1 reaction to surface spread of flame, this can usually be achieved by specifying the application of a flame-retardant treatment, or by covering with gypsum plasterboard. The correct treatment specification is particularly important in swimming pools or other environments such as conservatories where free water or high humidity may cause leaching of unsuitable treatments. Some flame retardants may react corrosively with metallic fittings or adversely affect the strength of timber elements[48].

3.5 Material specifications

3.5.1 Overview

A full material specification will normally comprise:

- the dimensions
- the material
- optionally a preservative or flame retardant treatment
- optionally a surface finish, e.g. a stain or water repellent
- where necessary a protective treatment for metal parts.

3.5.2 Dimensions

3.5.2.1 Solid timber

For material specifications and drawings state the cross-section, total length and tolerances, e.g. '47(T1) × 195(T2) × 3000(minimum)', together with a note, "The abbreviations 'T1' and 'T2' refer respectively to tolerance class T1 (+3/-1mm for thicknesses and widths $\leqslant$ 100mm and +4/-2mm thereafter) and tolerance class T2 (±1mm for thicknesses and widths $\leqslant$ 100mm and ±1.5mm thereafter). All measurements in mm."

For engineering drawings the tolerance on length may be set at the designer's discretion.

3.5.2.2 Glulam

Specify the cross-section, length, and if appropriate the curvature, taper or shape of each member. Specify 'Tolerances to BS EN 390'[28] or in a more detailed format as recommended for solid timber.

3.5.2.3 LVL

Specify the cross-section and length of each member. Specify 'Tolerances to BS EN 14374'[17], or in a more detailed format as recommended for solid timber.

3.5.2.4 Panel products

For dimensions it is sufficient to specify the thickness and the breadth and width of each board. For some plywoods the number of veneers is also required. Tolerances on thickness and squareness are included in the relevant product standards. Standard panel sizes are 600 × 1200mm, 600 × 2400mm and 1200 × 2400mm. Surface grades often do not have any signficant effect upon the structural performance of a panel but may need to be specified for architectural purposes.

3.5.2.5 Mechanical fasteners

Nail, wood screw, coach screw, bolt and dowel dimensions comprise the diameter and the length. Wood screw diameters are still often catalogued under screw gauges, which are explained in Table 3.26. Many modern wood screw threads have relatively small root diameters, and since the nominal diameter in mm or screw gauge relates to the outer diameter of the thread the bending strength of the screw can be much less than that of a conventional wood screw with the same nominal diameter. To be safe specify a 'minimum root diameter', d_{min}, and base the load capacity on an effective diameter d_{ef} of $1.1d_{min}$. Metric bolt diameters are specified as 'M10', 'M12' etc.

Table 3.26 Outer thread diameters corresponding to standard wood screw gauges

S.G.	3	4	5	6	7	8	9	10	12	14	16	18
ϕ mm	2.39	2.74	3.10	3.45	3.81	4.17	4.52	4.88	5.59	6.30	7.01	7.72

3.5.3 Materials

Specifications for the most common timber materials are shown in Table 3.27. For more detailed specifications of panel products and solid timber and complete sample specifications for a wall panel and floor, see TRADA's *Specifying wood-based panels for structural use*[49].

Panel products for structural use should normally conform to BS EN 13986[50], and the standards cited therein which relate to the particular use for which it is intended. This standard specifies all the requirements which a panel product must satisfy to qualify for CE-marking. Alternative methods of compliance with the Construction Products Directive are given in Section 2.1.3.

The physical and mechanical properties of plywood depend on many factors so it is necessary to specify a particular type in addition to the general specification related to service class. Some common plywoods suitable for structural use are shown in Table 3.28, for which characteristic strength and stiffness properties may be obtained from the manufacturers or be calculated as explained in Section 3.3.3.

Mechanical fasteners should be specified in accordance with Table 3.29.

Proprietary products may be specified by manufacturer, product name, type reference and dimensions, provided that they satisfy the certification requirements set out in Section 2.1.3.

For pool-hall environments only special stainless steel grades are appropriate. Nickel content should be < 6% duplex or > 13%. Stainless steel fixings to Grade 304/316 are not suitable. Stainless steel fixings with nickel contents within the acceptable limits are not generally available (batch production to special order). Pre-galvanized components, i.e. joist hangers have limited life expectancy in pool halls without additional protection, i.e. powder coating or over-painting.

3.5.4 Protective treatments

See Section 3.4.

3.5.5 Marking

All materials for structural use should be marked either on the product itself or on its packaging, indicating the relevant standard or standards to which it conforms and any other information required by such standards in the marking.

Table 3.27 Some basic specifications for timber and plasterboard materials

Product	Description	Service classes	Specification (Examples are shown <*thus*>)
Softwood	See 1.5.1	1,2,3[a]	'Structural softwood graded <*C24*> in accordance with BS EN 14081[13]'
Hardwood	See 1.5.1	1,2,3[a]	'<*Iroko*> hardwood in accordance with BS EN 14081 and graded HS to BS 5756[32]' OR 'Structural hardwood graded <*D35*> in accordance with BS EN 14081'
Homogenous glulam	See 1.5.1	1,2,3[ab]	'Homogenous glulam graded <*GL28h*> in accordance with BS EN 14080[16]'
Combined glulam	See 1.5.1	1,2,3[ab]	'Combined glulam graded <*GL28*[c]> in accordance with BS EN 14080'
LVL	See 1.5.1	1,2,3[a]	LVL is normally specified as a particular product from a particular manufacturer. It should be certified to BS EN 14374[17] for structural use in the service class for which it is required.
Plywood[cd]	Plywood for use in dry conditions	1	'EN 636-1 structural plywood to BS EN 636[19] in accordance with BS EN 13986[50]'
	Plywood for use in humid conditions	1,2	'EN 636-2 structural plywood to BS EN 636 in accordance with BS EN 13986'
	Plywood for use in exterior conditions	1,2,3	'EN 636-3 structural plywood to BS EN 636 in accordance with BS EN 13986'
OSB[cd]	Load bearing boards for use in humid conditions	1,2	'OSB/3 in accordance with BS EN 300[20] and BS 13986'
	Heavy-duty load bearing boards for use in humid conditions	1,2	'OSB/4 in accordance with BS EN 300 and BS 13986'
Particle-board[cd]	Load bearing boards for use in humid conditions	1, 2	'Particleboard Type P5 in accordance with BS EN 312[21]'
	Heavy-duty load bearing boards for use in humid conditions	1, 2	'Particleboard Type P7 in accordance with BS EN 312'

Table 3.27 Continued

Product	Description	Service classes	Specification (Examples are shown <*thus*>)
Plasterboard	Gypsum plasterboard	1	'Gypsum plasterboard to BS EN 520[51]'. Specify the thickness (generally 12.5mm or 15mm) and type required. BS EN 520 Type A is equivalent to BS 1230-1[52] Type 1, Type F is equivalent to Type 5, and Type P is equivalent to Type 6. Consult the manufacturer for a precise specification depending on the performance requirements.

Notes

a See Section 3.4 for necessity of preservative treatment.

b BS EN 14080[16] may have special requirements for glulam when it is to be used in service class 3.

c For flooring it is recommended that particleboard and OSB suitable for use in service class 2 should be specified in case it gets wet on site.

d For use in floors, walls or roofs the specification for panel products should additionally include the words 'certified for use in flooring/walls/roofing in accordance with BS EN 12871'[53], or for use in floating floors 'certified for use in floating floors in accordance with BS EN 13810-1'[54]. For alternative ways to demonstrate compliance with the Construction Products Directive see Section 2.1.3.

Table 3.28 Material specifications for some common structural plywoods[a]

Product standard	Plywood type	Bending strength[b]
Canadian softwood plywood manufactured in accordance with CSA 0151 2004[55] marked 'CANPLY'	Select tight face, select and sheathing grades: unsanded	Low
Canadian Douglas fir plywood manufactured in accordance with CSA 0121-M 1978[56] marked 'CANPLY'	Select tight face, select and sheathing grades: unsanded	Medium
	Good two sides and good one side grades: sanded	Low
Finnish birch plywood manufactured in accordance with SFS 2417[57] and SFS 2413[58] marked 'VTT'	Birch plywood 1.4mm veneer: sanded	High
	Mirror plywood 1.4mm veneer: sanded	
	Twin plywood 1.4mm veneer: sanded	
Finnish birch plywood manufactured in accordance with SFS 4091[59] marked 'VTT'	Combi plywood 1.4mm veneer: sanded	High
	Combi plywood thick veneer: sanded	

Table 3.28 Continued

Product standard	**Plywood type**	**Bending strength**[b]
Finnish conifer plywood manufactured in accordance with SFS 4092[60] marked 'VTT'	Conifer plywood 1.4mm veneer: sanded	Medium
	Conifer plywood thick veneer: sanded	Low

Notes

a All structural plywoods must be certified as suitable for their intended use as stated in Table 3.27. The plywoods listed have adhesives suitable for exterior use but the wood itself may require preservative treatment for use in service class 3 (see Section 3.4.).

b For general guidance only.

Table 3.29 Basic specifications for mechanical fasteners

Fastener type	**Specification**[a]
Nails	Round steel wire nails to BS EN 14592[46] OR Square twisted shank flat head nails to BS EN 14592 or BS 1202-1[61] OR Annular ringed shank flat head nails to BS EN 14592 or BS 1202-1
Wood screws[b] d ⩽ 6mm	Steel <slotted countersunk>/<slotted raised countersunk>/<recessed countersunk> /<recessed round> head wood screws to BS EN 14592 with a minimum tensile strength of 540N/mm²
Coach screws[c]	Steel hexagon head wood screws to BS EN 14592 with a minimum tensile strength of 400N/mm²
Washers for coach screws[d]	<Round>/<Square> mild steel washers with a minimum tensile strength of 400N/mm² to <BS EN ISO 7091[62]>/< BS 4320[63]>. Minimum <side length>/<diameter> <(*Calculate 3 screw diameters*)>, minimum thickness <(*Calculate 0.3 screw diameters*)>
Bolts[e]	1 Metric steel bolts to BS EN 14592, steel grade 4.6 to BS EN ISO 4016[64] OR 2 Metric steel bolts to BS EN 14592, steel grade 8.8 to BS EN ISO 4014[65] OR 3 Stainless steel bolts to BS EN 14592, property class 50 to BS EN ISO 3506-1[66], austenitic stainless steel (molybdenum/chrome/nickel alloys) grade 1.4436 to BS EN 10088-1[47] or austenitic stainless steel (chrome/nickel alloys) grade 1.4301 or 1.4310 to BS EN 10088-1 OR 4 Stainless steel bolts to BS EN 14592, property class 80 to BS EN ISO 3506-1 (+ *stainless steel specification as above*)
Nuts[e]	1 Mild steel nuts steel grade 4.6 to BS EN ISO 4034[67] OR 2 Mild steel nuts steel grade 8.8 to BS EN ISO 4032[68] OR 3 Stainless steel nuts to BS EN ISO 3506-2[69] property class 50 (+ *stainless steel specification as above*) OR 4 Stainless steel nuts to BS EN ISO 3506-2 property class 80 (+ *stainless steel specification as above*)

Table 3.29 Continued

Fastener type	**Specification**[a]
Washers for bolts[e]	1 <Round>/<Square> mild steel washers with a minimum tensile strength of 400N/mm² to <BS EN ISO 7091>/< BS 4320> OR 2 <Round>/<Square> mild steel washers with a minimum tensile strength of 800N/mm² to BS EN ISO 7089[70] OR 3 <Round>/<Square> mild steel washers to BS EN ISO 7089, (+ *stainless steel specification as opposite*). Minimum <side length>/<diameter> <(Calculate 3 bolt diameters)>, minimum thickness <(*Calculate 0.3 bolt diameters*)>
Dowels[e,f]	1 Steel dowels to BS EN 14592 <with 1mm chamfer on each end (*optional*)>, made of steel grade S235 to BS EN 10025-1[71] OR 2 Steel dowels to BS EN 14592 <with 1mm chamfer on each end (*optional*)> minimum tensile strength 800 N/mm² to BS EN 10149-1[72] OR 3 Stainless steel dowels to BS EN 14592 and BS EN ISO 3506-1 property class 50 <with 1mm chamfer on each end (*optional*)> OR 4 Stainless steel dowels to BS EN 14592 and BS EN ISO 3506-1 property class 80 <with 1mm chamfer on each end (*optional*)>
Shear plates, split rings and toothed plates	Timber connectors should conform to BS EN 14545[73] and BS EN 912[74]. The specification should include the type, shape and size (see Table 6.34). The material is specified in BS EN 912, but connectors may also be made of stainless steel, for which a similar specification to bolts may be appropriate

Notes

a **Specification**. The principal dimensions should also be specified, e.g. 'M12' for 12mm diameter bolts and for the matching nuts and washers. For mild steel corrosion protection in accordance with Table 3.25 should also be specified. Stainless steel property classes 50 and 80 are equivalent to grades 4.6 and 8.8 respectively.

b **Wood screws**. These are sized in accordance with the outer diameter of the thread, but this should not be used to assess their load capacity – see Section 6.4.1. A recessed head may also be known as a 'Phillip's head', 'Posidrive', etc. 540N/mm² is the tensile strength assumed in BS 5268-2 for wood screws with diameters up to and including 7.01mm and is the strength assumed in the tables of screw load capacities in this *Manual*. Screws with a diameter > 6mm made from mild steel may have a lower strength.

c **Coach screws**. 400N/mm² is the tensile strength assumed in the tables of coach screw load capacities in this *Manual*. While higher steel strengths could be specified for coach screws and used in conjunction with the tables in the CD, such screws would have to be manufactured specially.

d **Washers for coach screws**. A washer is required if the head of the coach screw is tightened onto timber or a wood-based material.

e **Bolts, nuts, washers and dowels**. The material of the bolt, nut and washers should match.

f **Dowels**. Steel grade S235 is the minimum steel strength permitted for dowels in BS EN 14592 and it corresponds to that of a grade 4.6 bolt.

4.1 Scope

This section provides:

- advice on the general principles to be applied when preparing an initial scheme for a structure
- a table showing the weights of commonly used materials and assemblies
- some quick methods for the sizing of common timber members
- advice on matters which can be left for the final design
- guidance on completing the final design.

Foundation design and ground floor slabs and footings are outside the scope of this *Manual*. Reference should be made to BS EN 1992: *Eurocode 2: Design of concrete structures* (EC2)[75], EC7, and other relevant sources such as TRADA's *Timber Frame Construction*[76].

The quick sizing methods should not be used in the final design except for checking that the final design solutions are of the right order.

4.2 Principles of initial design

4.2.1 General principles

At the initial stages of the structural design of buildings it is necessary, often at short notice, to produce alternative schemes that can be assessed for architectural and functional suitability and cost. It follows that initial design methods should be simple, quick, reliable and moderately conservative. Lengthy analytical methods should be avoided. Standardised members, components and assemblies will usually be cheaper and more readily available than purpose-made items.

For the initial design it is generally sufficient to consider only the finished structure, provided that buildability is properly addressed. Throughout the design process the Engineer should seek to eliminate or mitigate hazards.

4.2.2 Vertical load transfer

Direct vertical loads are normally resisted by simple posts, portal stanchions or timber frame wall panels; floor and ceiling loads by solid or prefabricated timber joists; roof loads by solid or glued-laminated beams, trusses or portal frames. In general factory-made items such as trussed rafters, glulam roof or portal members, timber frame wall panels and mass-produced engineered timber joists are more economical than site-built assemblies and components such as large cut roofs, spaced columns or nailed box beams. For economy, limit solid timber spans to 4m to 5m. For longer spans use engineered timber joists, LVL or glulam.

4.2.3 Horizontal load transfer

Horizontal forces may be transferred via roof and floor diaphragms and either shear walls or braced frames. Rigid frames and moment connections can be produced in timber but generally require many fasteners and are economical only when deep sections are required for other reasons. Curved laminated portal frames provide an aesthetic but relatively costly solution.

4.2.4 Load cases

In principle every load case should be checked (see Section 3.2.1.2). In practice the critical load case for strength can often be identified by inspection. If this is not possible, it can usually be identified by dividing the total factored load for each load case by the value of k_{mod} corresponding to the shortest load duration involved in that load case. The highest resulting value identifies the critical load case. The values of k_{mod} to use for some common timber materials in services classes 1 and 2 are shown in Table 4.1. (Table 2.3 gives a complete list.) Note that for many timber members, especially floor joists, serviceability considerations will govern the size rather than strength.

4.2.5 Sizing

Structural members should be sized for strength and serviceability and to accommodate connections, which often govern the size of a timber member (see Table 4.2). Other determining factors may be standard available sizes, thermal requirements (which may, for example, dictate the depth of wall studs), standardisation within the building, interaction with other structural members or components, connection details and aesthetics.

Take care that the members are large enough to accommodate fasteners and minimum edge and end distances. For a member loaded parallel to the grain the minimum width of member required is 6 times the diameter of bolt used, e.g. an M12 bolt needs a minimum timber thickness of 72mm.

For initial design, tables produced in accordance with BS 5268 will provide approximate sizes. Span tables to BS 5268 are available from TRADA for solid timber members in dwellings[77], from the GLTA[78] for glulam floor and roof beams, and from the manufacturers of LVL and engineered timber joists for their products.

Table 4.1 Values of k_{mod} to use in service classes 1 and 2 for solid timber, glulam, LVL and plywood

Shortest load duration in load case	Principal type of action[a]	Value of k_{mod}
Instantaneous	Wind	1.1
Short-term	Roof imposed load	0.9
Medium-term	Floor imposed load	0.8
Long-term	Storage and water tanks	0.7
Permanent	Dead weights	0.6

Note
a From the National Annex to EC0.

Table 4.2 Approximate strengths of timber connections in terms of unjointed member strength[a]

Connection type	Axial tension/ compression	Bending/ shear stress
Dowel type ⩽ 6mm	90%	75% [b]
Dowel type > 6mm	75%	50% [b]
Finger joints	90% [c]	90%
Glued in rods	75%	75%
Nail plates	100%	80%

Notes

a Where members meet at an angle the connection area is limited and may reduce the capacity further.

b Bending stresses are dependent on depth.

c The strength of finger jointed timber grade C24 and below can be 100% of the unjointed strength.

If span tables are unavailable the following formulae may be used to obtain the required depth of joists and beams in some common materials for initial design purposes, assuming that they are suitably restrained against lateral torsional buckling (see Table 4.3). The formulae are based on a dead load of 0.5 kN/m^2 and an imposed load of 1.5 kN/m^2. They may also be used for other floor weights and categories of floor (see Table 3.2), but will not be quite so accurate. The formulae for longer spans include a 12mm limit for the instantaneous deflection under dead + imposed loading to address vibration requirements in a simplified manner.

Serviceability

For solid timber floor joists and beams the minimum required depth h may be calculated approximately as follows.

For $l \leqslant$ 2.31m without plasterboard and for $l \leqslant$ 3.86m with plasterboard:

$$h = k_p k_E \times \sqrt[3]{\frac{F_k l^2}{b}} \text{ mm}$$

Where k_P = 190 for members with attached plasterboard
160 for members without attached plasterboard

k_E = 1.0 for C16 grade timber
0.9 for C24 grade timber
0.88 for GL24h grade glulam
0.86 for GL28h grade glulam
0.87 for LVL

l = structural span (m) (assumed to be simply supported single span)
b = total breadth of member (mm)
F_k = total unfactored UDL (kN) on a beam, or

= $\left(\sum G_{k,j} + Q_{k,1}\right) sl$ kN on a joist

Where $\sum G_{k,j}$ = total characteristic dead load on floor (kN/m²)
$Q_{k,1}$ = characteristic imposed load on floor (kN/m²)
s = joist spacing (m)

For l greater than the above:

$$h = 120k_E \times \sqrt[3]{\frac{F_k l^3}{b}} \text{ mm}$$

with factors defined previously.

Manufacturers of engineered timber joists may publish a bending stiffness (EI) value. For floor joists and beams supporting domestic and other residential floors with a dead weight of 0.5kN/m² the minimum required bending stiffness for engineered timber joists may be calculated as follows:

for $l \leqslant 2.31$m without plasterboard $EI = 2.88F_k l^2$ kNm²
for $l \leqslant 3.86$m with plasterboard $EI = 4.80F_k l^2$ kNm²
for l greater than the above $EI = 1.25F_k l^3$ kNm²

Strength

For solid members the required depth in shear may be calculated approximately as:

$$h = \frac{1000k_G k_S k_T F_k}{b} \text{ mm}$$

Where k_G = 1.00 for C16 grade timber
0.72 for C24 grade timber
0.67 for GL24h grade timber
0.56 for GL28h grade timber
0.82 for LVL
k_S = 1.00 for a single member
0.91 for two or more members fastened securely together
k_T = 1.17 for storage loading
1.00 for floor imposed loading
0.88 for roof imposed loading

with other factors defined as above.

For solid members the required depth in bending may be calculated approximately as:

$$h = 340k_G k_S k_T \times \sqrt{\frac{F_k l}{b}} \text{ mm}$$

Where	k_G	=	1.00	for C16 grade timber
			0.82	for C24 grade timber
			0.78	for GL24h grade timber
			0.76	for GL28h grade timber
			0.91	for LVL
	k_S	=	0.95	for joists when $s \leqslant 0.6$m or for beams consisting of two or more members fastened securely together
			1.00	for all other bending members
	k_T	=	1.07	for storage loading
			1.00	for floor imposed loading
			0.94	for roof imposed loading

with other factors defined as above.

For simplicity all loads have been factored by 1.5.

Manufacturers of engineered timber joists may publish a ‘moment capacity’. If this is published as a design value for the load duration in question then it simply has to be matched against the design bending moment, which for a UDL on a simply supported beam may be taken as $1.5F_k l/8$ kNm for an initial design. If it is published as a characteristic value then the characteristic value required is $1.2 \times 1.5F_k l/8k_{mod} = 0.225F_k l/k_{mod}$ kNm, with the appropriate value of k_{mod} for the materials and load duration in question taken from Table 2.3. In the case of joists made from wood-based materials with different values of k_{mod}, the lower value should be used.

The degree of restraint required to avoid lateral torsional buckling in solid timber joists and beams is indicated in Table 4.3. When these conditions are met k_{crit} will normally be 1.0 (see Table 3.20).

4.2.6 Outline of initial design process

The initial design process usually involves a certain amount of consultation with the client or architect, and some iterations in the development of a viable and economic scheme. Figure 4.1 outlines the process for a typical timber structure.

Table 4.3 Maximum depth to breadth ratios of solid timber beams to avoid lateral torsional buckling

Lateral support	Maximum depth to breadth ratio
None	2:1
Ends restrained against rotation	3:1
Ends restrained against rotation and member held in line as by purlins or tie rods at centres not more than 30 times the breadth of the member	4:1
Ends restrained against rotation and compression edge held in line, as by direct connection of sheathing, deck or joists	5:1
Ends restrained against rotation 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 of the member	6:1
Ends restrained against rotation and both edges held firmly in line	7:1

4.3 Fire resistance

4.3.1 Introduction

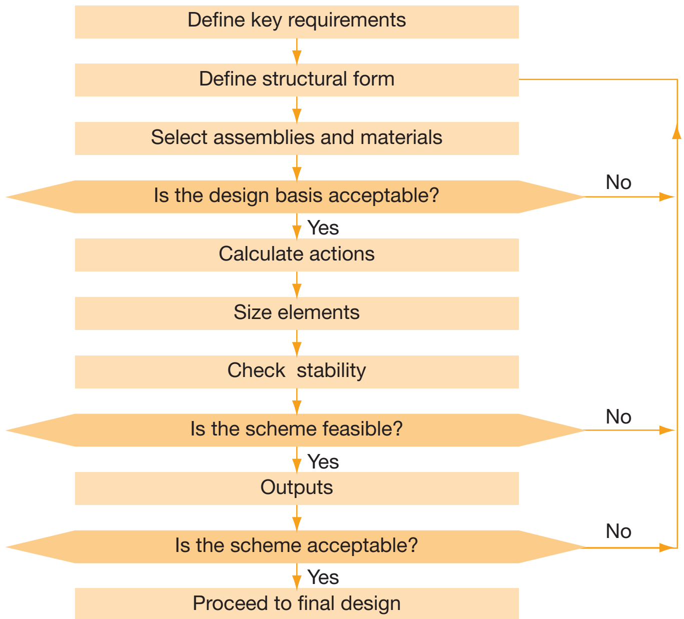
Where fire resistance is required it needs to be considered at the initial design stage because it may affect the size of members, the weight of the structure, the type of connections used and, in the case of party walls and floors, the type of construction. Design methods for meeting requirements for fire resistance are set out in Sections 5.10 (general methods), 8.5 (floors) and 10.10 (walls). Reference should be made to these sections for the final design.

The maintenance of adequate strength and stiffness in timber members is accomplished either by insulating them from heat or by providing additional ‘sacrificial’ timber to a section so that the part exposed to fire can protect the inner material from fire damage while the outer part itself chars at a slow, predictable rate. For an initial design the following simple solutions may be assumed.

4.3.2 Insulation method

Thirty minutes of fire resistance can normally be provided for roof, floor and wall members made of solid timber, glulam or LVL by means of one 12.5mm thick layer of Type A gypsum plasterboard to BS EN 520[51]. However the required thickness depends on the density, and some manufacturers recommend 15mm, so the manufacturer’s literature should be consulted and where necessary a particular product should be specified. For floors made with prefabricated timber I-joists or metal open-web timber joists 15mm thick plasterboard is always required.

For 60 minutes’ resistance (typically required in compartment floors) a plasterboard thickness of 25mm is recommended. However, greater thickness is often required to satisfy acoustic requirements.



Define key requirements

Actions

Serviceability requirements

Environment and service classes

Durability provisions

Fire requirements

Define structural form

Structural forms (see Table 4.6)

Load paths to foundations

- vertical and horizontal loads
- joist directions

Bracing, vertical and horizontal

Serviceability requirements

Connection types

Approach to robustness

Select assemblies and materials

Select assembly types where not already specified

Select materials, considering sustainability issues and end-of-life disposal

Is the design basis acceptable?

Obtain agreement from client and architect to proceed on the proposed design basis

Calculate actions

Identify major load-bearing elements and assemblies

Identify critical load cases

Calculate corresponding actions

Size elementsa

Size elements for ULS, SLS and connections using:

- approximate or quick methods
- guidance documents
- existing similar structures
- non-structural constraints[a]

Check stability

Check overall stability

Check any key connections on which stability depends

Is the scheme feasible?

Assess the structural and economic feasibility of the scheme

Consider the construction logistics

- workshop/site
- delivery/erection
- manufacturing/erection costs

Outputs

Scheme drawings

Schedule of:

- materials
- connections

Information to enable costs estimates to be prepared

Is the scheme acceptable?

Obtain agreement from client and architect to the proposed scheme

Proceed to final design

Check all information

Prepare list of design data

Continue with Sections 5-10

Note

[a] For example standard available sizes and thermal requirements may dictate the depth of wall studs, allowance for distribution of building services within floors or walls, standardisation within the building, matching or re-using existing components, appearance).

Fig 4.1 Initial design flowchart

4.3.3 Sacrificial timber method

For initial design the ‘notional’ charring rates given in EC5-1-2 Table 3.1 should be used. These allow for increased charring at the corners of members exposed to fire but will be conservative for larger members. They are summarised in Table 4.4. The charring rates shown apply to each face exposed to fire (i.e. to each face not directly attached to a fire-resistant material with a resistance to fire at least equal to the required period). So both the breadth and the depth of a softwood column exposed on all four sides to 30 minutes of fire would require $2 \times 30 \times 0.8 = 48$mm of sacrificial timber. For details of this method see Section 5.10.3. For larger members in shorter fire durations little or no additional timber may be required.

Table 4.4 Notional charring rates (based on EC5 1-2 Table 3.1)

Material	Charring rate per exposed face (mm/min)
Softwood timber	0.8
Softwood glulam, and LVL to BS EN 14374[17]	0.7
Hardwood timber and hardwood glulam	0.55
Panel products	See EC5-1-2

4.4 Movement

Movement, particularly in timber frame structures, can have serviceability implications which may affect the choice of materials or even the design at the initial design phase (see Sections 2.8 and 10.11).

4.5 Durability

Durability needs to be considered in the context of design life and service environment because it may affect the choice of timber species or wood-based products, connection details, and fastener materials (all with cost implications). Sections 2.4 and 2.13 set out general principles related to durability and Section 3.4 discusses methods for meeting the requirements.

4.6 Acoustic, thermal insulation and air tightness requirements

4.6.1 General

Requirements for acoustic, thermal insulation and air tightness are primarily the responsibility of the architect, but the Engineer should check that these have no serviceability implications.

4.6.2 Acoustic

Requirements for limiting sound transmission in party floors, party walls and internal walls in certain rooms (e.g. bathrooms) and building types will affect their design and weight (see Sections 8.7.1 and 10.12.1).

4.6.3 Thermal insulation

Requirements for thermal insulation in external timber frame walls will determine their design and possibly the depth of the studs (see Section 10.12.2).

4.7 Densities and weights

Table 4.5 shows the densities and weights of some common materials and assemblies.

Table 4.5 Densities and weights

Material	Description	Mean density (kg/m^3)
Timber	C16 grade softwood	370
	C24 grade softwood	420
	GL24c glulam	400
	GL28c glulam	440
	GL24h glulam	420
	GL28h glulam	460
	Structural hardwoods	550 – 1000
	Structural timber composites, e.g. LVL	550 – 600
Sheet materials	Sheathing/flooring grade plywoods	430 – 630
	Particleboard P5	550 – 620
	Oriented strandboard OSB/2 and OSB/3	550
	Plasterboard type A	800
	Glass	2500
Insulation	Mineral wool, glass	12
	Mineral wool, rock	20
	Mineral wool as resilient layer	60 – 100
Gypsum plaster		1200
Cement render		2500

Material	Description	Mass (kg/m^2)	Weight (kN/m^2)
Roofing felt	3 layers of felt and chippings	37	0.36
Tile battens		3.4	0.033
Concrete paving slabs	50mm thick	117	1.15
Ballast	50mm thick	80	0.78
Roofing	Concrete interlocking tiles	42 – 58	0.41 – 0.57
	Concrete plain tiles	80	0.78
	Machine-made clay tiles	64	0.63
	Hand-made clay tiles	71	0.7
	Slates	25 – 78	0.25 – 0.77
	Insulated metal deck roofing	20 – 30	0.20 – 0.29

Table 4.5 Continued		
Assembly	**Example**	**Weight (kN/m²)**
Dead weight on rafters	Felt + tiling battens + 38 x 140 C27 rafters @ 400mm centres	0.10
Dead weight on rafters	As above + 51kg/m² concrete interlocking tiles	0.60
Dead weight on flat roof joists	3 layers of felt, chippings, 9.0mm OSB, 12.5mm plasterboard, 200mm thick mineral wool + 75 x 220 C24 joists @ 600mm centres	0.65
Trussed rafter roof	51kg/m² concrete interlocking tiles, felt, battens, roof trusses, 200mm mineral wool, 12.5mm plasterboard	0.92
Ceiling, unboarded	200mm mineral wool, 12.5mm plasterboard, 47 x 220 C24 joists @ 400mm centres	0.21
Ceiling, boarded	As above + 18mm particleboard	0.30
Timber suspended floor	22mm particleboard, joists, 100mm mineral wool + 75 x 220 C24 joists @ 400mm centres	0.32
Timber intermediate floor	22mm particleboard, 12.5mm plasterboard + 75 x 220 C24 joists @ 400mm centres	0.38
Timber party floor	Various	Up to about 1.25
External timber frame wall	9mm OSB, 12.5mm plasterboard, 38mm x 97mm C24 framing, 100mm mineral wool	0.24
Internal timber frame partition	38mm x 72mm C16 framing at 600mm spacing, 2 x 12.5mm plasterboard (minimum specification) 38mm x 89mm C24 framing at 400mm spacing, 2 x 12.5mm plasterboard, mid-height noggings, 75mm quilt insulation 10kg/m³	0.24 0.26
Trussed rafter roof		See Section 4.8.2

Table 4.5 Continued			
Quick conversion formulae			
Material	**Density units**	**Weight**	**Symbols**
Beam and column materials	ρ (kg/m³)	$9.81\rho bh \times 10^{-9}$ kN/m	b = breadth mm h = depth mm
Sheet materials	ρ (kg/m³)	$9.81\rho t \times 10^{-6}$ kN/m²	t = thickness mm
Sheet materials	ρ (kg/m³)	$9.81\rho \times 10^{-3}$ kN/m³	

4.8 Roofs

4.8.1 Introduction

Section 7 describes the structural functions of roofs and the principal types of timber roof. Table 4.6 shows some typical timber roof solutions with feasible spans, typical proportions and suitable materials. Figure 4.2 shows the most common forms of trussed rafter configuration.

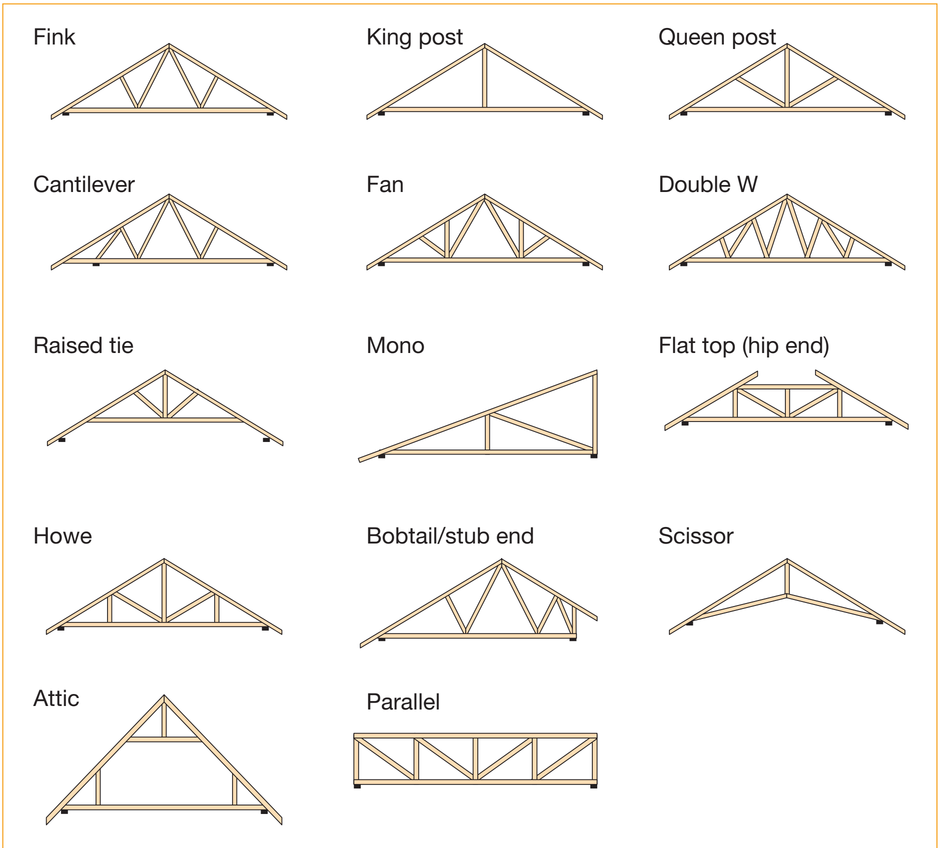
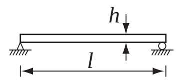
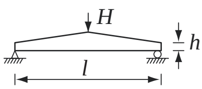
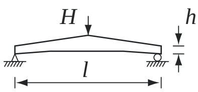
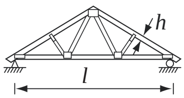
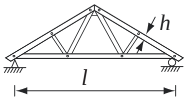
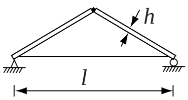
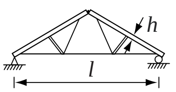
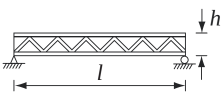
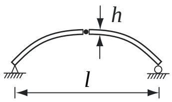
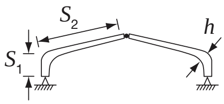
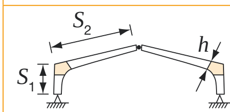
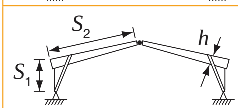


Fig 4.2 Trussed rafter configurations

Table 4.6 Structural forms for timber roofs

Structural form	Description and materials	Pitch (º)	Span range (m)	Approximate proportions
	Straight beam, simply supported Solid timber, glulam, LVL	<3º	Solid <5 LVL <20 Glulam <30	$h \approx \frac{l}{17}$
	Tapered beam (single or double taper), simply supported Glulam, LVL	3–10º	10 – 30	$h \approx \frac{l}{17}$ $H \approx \frac{l}{16}$
	Pitched cambered beam simply supported Glulam	3–15º	10 – 20	$h \approx \frac{l}{17}$ $H \approx \frac{l}{16}$
	Trussed rafter Solid timber and punched metal plate fasteners	15–40º	3 – 15	$h = l/75$ to $h = l/100$
	Bolted or connectored timber truss with purlins and intermediate common rafters and ceiling ties	15–45º	5 - 20	$h = l/60$ to $h = l/80$
	Pitched tied frame Solid timber, glulam, LVL, steel tension members	⩾ 14º	15 – 50	$H \approx \frac{l}{16}$
	Truss Solid timber glulam, LVL, steel tension members	⩾ 14º	20 – 80	$h \approx \frac{l}{40}$
	Trussed girder, Warren type etc. Solid timber, glulam, LVL	< 3º	30 – 80	$h = l/10$ to $h = l/15$
	Arch – optionally tied with steel, glulam or LVL – glulam top chords	Rise/span ≈0.14	20 – 80	$h \approx \frac{l}{40}$
	Portal Curved laminated glulam haunch	⩾ 14º	15 – 40	$h \approx \frac{S_1 + S_2}{15}$
	Portal with large finger jointed haunch in glulam or LVL	10º – 50º	8–30 at 3–6 frame centres	$h \approx \frac{S_1 + S_2}{13}$
	Portal – strutted and braced haunch Glulam, LVL Struts same, or solid timber	⩾ 14º	10 – 35	$h \approx \frac{S_1 + S_2}{15}$

In an initial design the Engineer should begin by checking that the type of roof specified by the architect can support the imposed loads without undue complications or expense. Deflection limits are normally governed by appearance except where there are brittle finishes (normally plasterboard beneath ceiling joists and sometimes beneath rafters), but in the case of flat roofs the possibility of ponding has to be considered (see Table 3.4).

4.8.2 Trussed rafter roofs

Where there are no other considerations a trussed rafter roof may be the most economical solution for spans up to 15m or more, for which a design can be provided by the trussed rafter manufacturer using specialised software. If this is an option it may be simplest for initial designs to assume a trussed rafter roof and investigate an alternative form if required when the final design is carried out. A trussed rafter manufacturer may then be asked to provide an initial roof design.

The characteristic dead weight on each of the two supporting walls of a duopitch trussed rafter roof can be calculated approximately as $0.5 \times$ span (in metres) in kN/m per wall based on a tile weight of about 45kg/m^2. More accurately the weight can be calculated as:

$$F_{\text{wall,k}} = 0.0066\left[(F_{\text{tile,k}} + 24)l + 60\right] \text{ kN/m per wall}$$

Where $F_{\text{tile,k}}$ = tile weight on slope (kg/m^2)
l = trussed rafter span (m)

The above formula assumes 450mm overhang on each side and includes the weight of tiles, felt and battens, ceiling and insulation. It does not include the ceiling imposed UDL, which is of a different load duration and depends on whether there is a room in the roof.

For tile weights see Table 4.5.

4.8.3 Cut roofs

Traditional duopitch cut roofs for dwellings, made with rafters, purlins, ceiling ties and binders are described in Section 7.4 and Figure 7.1. Table 4.7 provides a guide to some traditional C16 grade roof member sizes which may be used for initial designs only. The notes extend its range to C24 and other sizes, slopes and loads.

4.8.4 Other types of roof

Table 4.6 gives some general guidance on the sizing of timber trusses, portals and arch roofs.

For more detailed sizing either dedicated software or a general frame analysis program will normally provide the fastest solution. Member sizes should be estimated so that an analytical model can be entered with simulated node points on the centre lines of the individual truss members. Where double members are envisaged it may be possible to assume that all the members are single members, choosing a fairly high depth:breadth ratio to allow room for the associated connectors. For the initial design assume a pin-jointed structure. Obtain the axial forces by resolution of forces or simple frame analysis. Continuity of members across a node can be allowed for by assuming a bending moment of $ql^2/9$ at the node, where q is the UDL per unit

Table 4.7 Maximum clear spans in metres for C16 roof members

Rafters	**Imposed load 0.75 kN/m² – Dead load 0.75 kN/m² Roof slope 30° or more**	
Cross-section (mm x mm)	Rafter spacing (mm)	
	400	600
47 x 100	2.47	2.17
47 x 125	3.10	2.71
47 x 150	3.71	3.25
Canted purlins	**Imposed load 0.75 kN/m² – Dead load 0.75 kN/m² Roof slope 30° or more**	
Cross-section (mm x mm)	Purlin spacing (mm)	
	1500	1800
75 x 175	2.76	2.41
75 x 200	3.16	2.75
75 x 225	3.55	3.09
Ceiling joists	**Imposed load 0.25kN/m² - Dead load 0.5 kN/m²**	
Cross-section (mm x mm)	Joist spacing (mm)	
	400	600
47 x 97	1.84	1.74
47 x 145	3.08	2.87
47 x 195	4.41	4.07

Notes

- **a** Members are assumed to be simply supported, single span. The values shown may be used for continuous members but will generally be conservative.
- **b** Other standard section sizes are available (see Tables 3.8 and 3.9).
- **c** For C24 timber increase spans by 10%.
- **d** For 38mm thick rafter and ceiling joists reduce spans by 15%.
- **e** For 63mm thick purlins reduce spans by 7.5%.
- **f** **Rafters and purlins.** For roof slopes between 15° and 30° reduce spans by 0.75% per degree. For dead loads not exceeding 0.5 kN/m² increase spans by 10%. For dead loads not exceeding 1.0 kN/m² decrease spans by 10%.
- **g** **Ceiling joists.** For dead loads not exceeding 0.25 kN/m² increase spans by 5%.
- **h** Rafter lengths may be extended by the use of additional purlins.
- **i** Purlin lengths may be extended by the use of posts.
- **j** Ceiling joist lengths may be extended by the use of additional ceiling binders.

length. For initial design slip at joints may be ignored provided that fairly conservative sections are chosen, but it may then be necessary at the final design stage to increase the stiffness of some members or precamber them in order to satisfy serviceability limits, particularly in bolted trusses (see Section 3.2.2.4 and Table 3.4).

Once moments and forces have been established for each member for each relevant load case they should be sized in accordance with the initial design guidance for compression members given in Section 4.10.1, or if appropriate as tension members by verifying that:

$$\sigma_{t,0,d} \leqslant f_{t,0,d}$$

i.e. by verifying that:

$$\frac{F_{t,d}}{bh} \leqslant \frac{f_{t,0,k}\,k_{mod}}{\gamma_M}$$ with k_{mod} chosen to suit the duration of each load case.

(The tension width factor, k_h, should be included in a final design – see Table 3.20.)

4.9 Floors

4.9.1 Introduction

Section 8 describes the structural functions of floors and the principal types of timber floor.

Single spans which can be achieved economically using simple joists are:

- softwoods – up to 5m
- hardwoods and timber composites – 5m to 7m
- engineered timber joists – 4m to 7m or more.

Unless vibration is of critical importance it is normally unnecessary to check a timber floor's vibrational behaviour in an initial design.

For most conventionally constructed timber floors defined in Section 5.8.1 adequate diaphragm action may be assumed. In other cases it may be necessary to calculate it (see Section 5.8.2).

4.9.2 Joists and beams

Table 4.8 provides a guide to some traditional C16 grade domestic floor joist sizes which may be used for initial designs only. The notes extend its range to C24 and other sizes and floor weights. A rough rule of thumb for domestic timber floor joists is that the required depth in mm of 47mm thick C16 grade joists = 50 × span (in metres) at 400mm spacing and 60 × span (in metres) at 600mm spacing.

Initial sizes for the strength and stiffness of non-domestic joists and for beams may be chosen in accordance with Section 4.2.5. Floor spans can be increased by the use of engineered timber joists, supporting beams or stressed skin panels. Beams can be made of glulam, LVL, or large single or double engineered timber joists.

Steel flitch beams can normally be designed so that the stiffness of the timber is increased by 50% to 200%, depending on the relative thickness of the two materials. With 200mm deep C24 timber and a 175mm steel plate the steel thickness needs to about one

Table 4.8 Maximum clear spans in metres for C16 domestic floor joists[a]

Cross-section (mm x mm)	Imposed load 1.5 kN/m² – Dead load 0.5 kN/m²		
	Joist spacing (mm)		
	400	450	600
47 x 97	1.93	1.82	1.47
47 x 120	2.52	2.42	2.05
47 x 145	3.04	2.92	2.59
47 x 170	3.37	3.42	3.00
47 x 195	3.87	3.71	3.41
47 x 220	4.35	4.17	3.63

Notes

- **a** Values from TRADA's span tables for solid timber members in floors, ceilings and roofs for dwellings calculated to BS 5268-2. Spans above 3.5m have been reduced by 5% to meet EC5 point load deflection requirements. Joists are assumed to be simply supported, single span. The values shown may be used for continuous members but will generally be conservative.
- **b** Other standard section sizes are available (see Tables 3.8 and 3.9).
- **c** For C24 timber increase spans by 10%.
- **d** For 38mm thick timber reduce spans by 10%.
  For 44mm thick timber reduce spans by 3%.
  For 75mm thick timber increase spans by 20%.
- **e** For 0.25 kN/m² dead weight increase spans by 5%.
  For 1.25 kN/m² dead weight reduce spans by 12.5%.

fifteenth of the total timber thickness to increase its stiffness by 100%. The corresponding increase in bending strength may be only 75%. For initial design purposes consider a flitch beam as a solid timber beam with the same depth as the proposed timber members and a thickness of $t_{\text{timber}} + 15t_{\text{steel}}$, where t_{timber} is the total thickness of the timber and t_{steel} is the thickness of the steel. (This assumes the use of C24 grade timber and makes an allowance for the reduced height of the steel plate to allow for shrinkage and workmanship.) This method will not check the bending strength of the steel, which might be critical (see Sections 5.5.3 and 5.5.4).

4.9.3 Decking

Table 4.9 shows approximate spans for common timber floor decking materials under a domestic floor load.

Table 4.9 Guide to maximum spans for floor decking materials for domestic flooring[a]

Material	Thickness (mm)	Span (mm)
Tongued and grooved floor boarding	16.0	500
	19.0	600
Canadian Douglas fir plywood – Select tight face, Select and sheathing grades: unsanded	15.5	600
Canadian softwood plywood – Select tight face, Select and sheathing grades: unsanded	15.5	450
	18.5	600
Finnish birch faces plywood, Finnish conifer plywood: sanded	15.0	450
	18.0	600
Particleboard, flooring grade	18.0	450
	22.0	610
OSB/2, OSB/3	15.0	450
	18.0	610

Note
a For other loading conditions, e.g. offices, refer to the manufacturer's published data.

4.10 Walls

4.10.1 Columns

4.10.1.1 Load capacity

For C16 and C24 solid timber and GL24h glulam Table 4.10 gives the maximum design capacity in axial compression for square columns without an applied bending moment. For rectangular pin-jointed columns take the smaller cross-sectional dimension b as the buckling depth and multiply the design load by h/b where h is the larger cross-sectional dimension. For rectangular columns braced against buckling in the less stiff direction take the larger cross-sectional dimension h as the buckling depth and multiply the design load by $b/$h.

With simultaneous bending applied about one axis only in the direction of buckling, e.g. the y axis, verify that:

$$\frac{F_{0,\mathrm{d}}}{F_{0,\mathrm{d,max}}} + \frac{M_{\mathrm{y,d}}\, k_{\mathrm{mod}}\, f_{\mathrm{m,k}}}{W_{\mathrm{y}}\, \gamma_{\mathrm{M}}} \leqslant 1 \text{ (derived from EC5 expression (6.23))}$$

Where $F_{0,\mathrm{d}}$ = design value of axial compression load
$F_{0,\mathrm{d,max}}$ = load capacity of column in axial compression from Table 4.10
$M_{\mathrm{y,d}}$ = design value of maximum bending moment about y-y'
W_{y} = section modulus about y-y'

Table 4.10 Design total axial load capacities for square columns in solid timber and glulam, with no lateral bending, service classes 1 and 2

Effective length (m)	Buckling dimension (mm)	C16			C24			Gl24h		
		Duration of critical load case								
		Long (kN)	Medium (kN)	Short (kN)	Long (kN)	Medium (kN)	Short (kN)	Long (kN)	Medium (kN)	Short (kN)
2.4	38	0.63	0.72	0.81	0.71	0.81	0.91			
	50	1.9	2.1	2.4	2.1	2.4	2.7			
	75	9	10	12	10	12	13			
	100	27	31	35	30	35	39	45	52	58
	150	110	130	150	120	140	160	170	200	220
	200	250	290	330	260	300	340	350	400	450
	300	640	730	830	650	740	830	820	930	1000
	400							1500	1700	1900
	500							2300	2600	3000
3	50	1.2	1.4	1.5	1.4	1.6	1.7			
	75	5.9	6.8	7.6	6.7	7.6	8.6			
	100	18	21	23	20	23	26	30	34	39
	150	83	95	110	92	100	120	140	160	180
	200	220	250	280	230	260	290	320	370	410
	300	610	700	790	620	710	800	800	920	1000
	400							1500	1700	1900
	500							2300	2600	3000
3.6	75	4.2	4.8	5.4	4.7	5.4	6.1			
	100	13	15	17	15	17	19	21	24	27
	150	61	70	79	68	78	88	100	120	130
	200	170	200	220	190	220	240	280	320	360
	300	570	650	730	590	670	750	780	890	1000
	400							1400	1600	1800
	500							2300	2600	2900
4.2	75	3.1	3.5	4	3.5	4	4.5			
	100	10	11	12	11	12	14	16	18	20
	150	46	53	60	52	59	67	77	88	99
	200	140	160	180	150	170	190	230	260	290
	300	520	590	660	540	620	700	750	860	960
	400							1400	1600	1800
	500							2300	2600	2900
4.8	75	2.4	2.7	3.1	2.7	3.1	3.5			
	100	7.4	8.5	10	8.4	10	11	12	14	16
	150	36	41	47	41	47	52	60	69	77
	200	110	120	140	120	140	160	180	210	230
	300	450	520	580	480	550	620	700	800	900
	400							1400	1600	1800
	500							2200	2600	2900
5.4	75	1.9	2.2	2.4	2.1	2.5	2.8			
	100	5.9	6.8	7.6	6.7	7.6	8.6	10	11	12
	150	29	33	37	33	37	42	48	55	62
	200	88	100	110	99	110	130	150	170	190
	300	390	440	500	420	480	540	630	720	810
	400							1300	1500	1700
	500							2200	2500	2800
6	75	1.5	1.8	2	1.7	2	2.2			
	100	4.8	5.5	6.2	5.4	6.2	7	7.8	9	10
	150	24	27	30	27	31	34	39	45	50
	200	73	83	93	82	93	100	120	140	160
	300	330	380	430	370	420	470	550	630	710
	400							1300	1500	1700
	500							2200	2500	2800

4.10.1.2 Bracing

A diagonal brace of a similar section to the columns can be installed between adjacent columns and this will act in both tension and compression. Alternatively a pair of more slender braces may be installed between two adjacent pairs of columns in opposite directions to act mainly in tension like steel bracing ties. Braces may be bolted into steel shoes bolted to the columns, or else, more neatly, be attached using steel flitch plates and dowels. See Section 5.8 for the design requirements.

Columns may also be braced using plywood or OSB/3 sheathing in a timber framework, utilising the shear strength of the panel product to provide the required racking resistance.

4.10.1.3 Connection of columns to the ground

It is essential to prevent uptake of water into the end-grain of timber so the following rules should be observed when connecting columns to the foundation.

- Keep the lower end of the column at least 150mm above ground level by fitting it into a steel support or shoe or attaching it through a moisture-proof membrane to a concrete plinth. Increase the distance if there is any possibility that debris may accumulate.
- If an enclosed steel shoe is used put drainage holes at the bottom so that if the timber shrinks any water entering the shoe can drain out again.

The bottom of a column should bear directly onto the steel support, so that any bolts in the side of the steel support provide lateral restraint only. Alternatively lateral restraint may be provided by means of glued-in rods with threaded ends which can be attached to the underside of steel shoes by bolts. The rods are glued with a suitable epoxy resin into holes drilled from the base of the column before being transported to the site. For further information see Section 6.8.

4.10.2 Timber frame walls

4.10.2.1 External walls

A typical multi-stud timber frame wall panel is described in Section 10.4.3.2.

Racking resistance

Provided that the overturning forces are resolved by adequate vertical restraints, a 2.4m square panel with neither openings nor vertical load made to the minimum specification in Section 10.4.3.2 will resist a design racking load of 3.3kN per horizontal metre with 9mm thick OSB/3. The resistance increases when panels are fastened together or vertical load is taken into account. Additional racking resistance can be provided by increasing the nail diameter from 3.0mm up to 3.75mm (increases the racking resistance by 24%), reducing the spacings of all the fasteners in the structural board (halving the nail spacing gives an increase in racking resistance of 42%), or by using 11mm thick OSB instead of 9mm thick material (increases the racking resistance by 5%).

Vertical load capacity

For initial design assume 38mm $\times$ 89mm studs unless thermal insulation requirements dictate a greater depth.

The 2.4m long wall panel described in Section 10.4.3.2 with 38mm × 89mm studs at 600mm centres will resist a vertical design load of about 43kN for permanent duration (dead weight), 50kN for long-term (storage), 57kN for medium-term (floor imposed) or 65 kN short-term (snow), based on the compression strength perpendicular to the grain on the bottom rail. However with wind loads the vertical load capacity may be reduced by stability requirements. Additional vertical load can be carried by increasing the cross-sectional area of all the framing members or by reducing the stud centres (these two measures increase the load capacity proportionately) or by specifying C24 timber (which increases the loads by 14%) or both.

4.10.2.2 Internal walls

Section 10.4.3.4 describes the construction of internal timber frame walls. Where they are required to be load-bearing, either to support floor joists or to contribute additional racking load, they may designed like external walls.

4.10.2.3 Party walls

Section 10.4.3.5 describes the construction of timber frame party walls. For initial design the design racking resistance of a 2.4m high party wall with only 15mm thick plasterboard sheathing on one side may be taken as 2.2kN per horizontal metre, assuming a minimum length of 5.0m and no vertical load.

4.10.2.4 Lintels

Tables 4.11 and 4.12 show some suitable sizes of lintel.

4.11 Connections

4.11.1 Introduction

Fabricating timber connections is generally labour-intensive, so the cost of connections can exceed the cost of the timber materials. Choosing an efficient and economical connection method can be the key to a successful project. Figure 4.3 shows some typical connection methods. Where members are joined at an angle the available jointing area diminishes, and the load-carrying capacity per unit area may also diminish.

4.11.2 Choosing a connection method

Use Tables 4.13 and 4.14 to choose a suitable connection method.

Table 4.11 Lintel sizes for 89mm thick external walls						
Span (m)	Load on wall					
	5kN/m			10kN/m		
	Timber	Size (mm x mm)	Governed by	Timber	Size (mm x mm)	Governed by
1.00	C16	2x44x145	Bending / Shear	C16	2x44x145	Shear
1.50	C16	2x44x145	Bending / Shear	C16	2x44x195	Shear
2.00	C16	2x44x195	Bending	C16	2x44x220	Bending
2.50	C16	2x44x195	Bending	LVL	90x220	Deflection
3.00	LVL	90x195	Deflection	LVL	90x240	Deflection
3.50	LVL	90x220	Deflection	LVL	90x300	Deflection

Table 4.12 Lintel sizes for 140mm thick external walls						
Span (m)	Load on wall					
	5kN/m			10kN/m		
	Timber	Size (mm x mm)	Governed by	Timber	Size (mm x mm)	Governed by
1.00	C16	3x44x145	Shear	C16	3x44x145	Shear
1.50	C16	3x44x145	Bending	C16	3x44x195	Shear
2.00	C16	3x44x195	Bending	C16	3x44x220	Bending
2.50	C16	3x44x195	Bending	LVL	(45+90)x195	Deflection
3.00	C16	3x44x220	Bending	LVL	(45+90)x220	Deflection
3.50	LVL	(45+90)x195	Deflection	LVL	(45+90)x240	Deflection

4.12 Estimating

Producing an estimate of the cost of a structure is not primarily the task of the Structural Engineer, but the Engineer should provide sufficient information to determine the following:

- volumes of solid timber, glulam and LVL, categorised by grade and section size
- areas of panel products, categorised by type and thickness
- lengths of proprietary joists and beams, categorised by capacity and depth.

If required the cost of materials and connections hardware can be readily obtained from the comprehensive list of construction materials suppliers provided at www.trada.co.uk. Specialist advice should be sought for the costs of fabricating connections and site erection. The wise designer will always bear in mind that simplicity and speed of fabrication will make large savings in labour costs which may in some cases offset higher material or component costs.

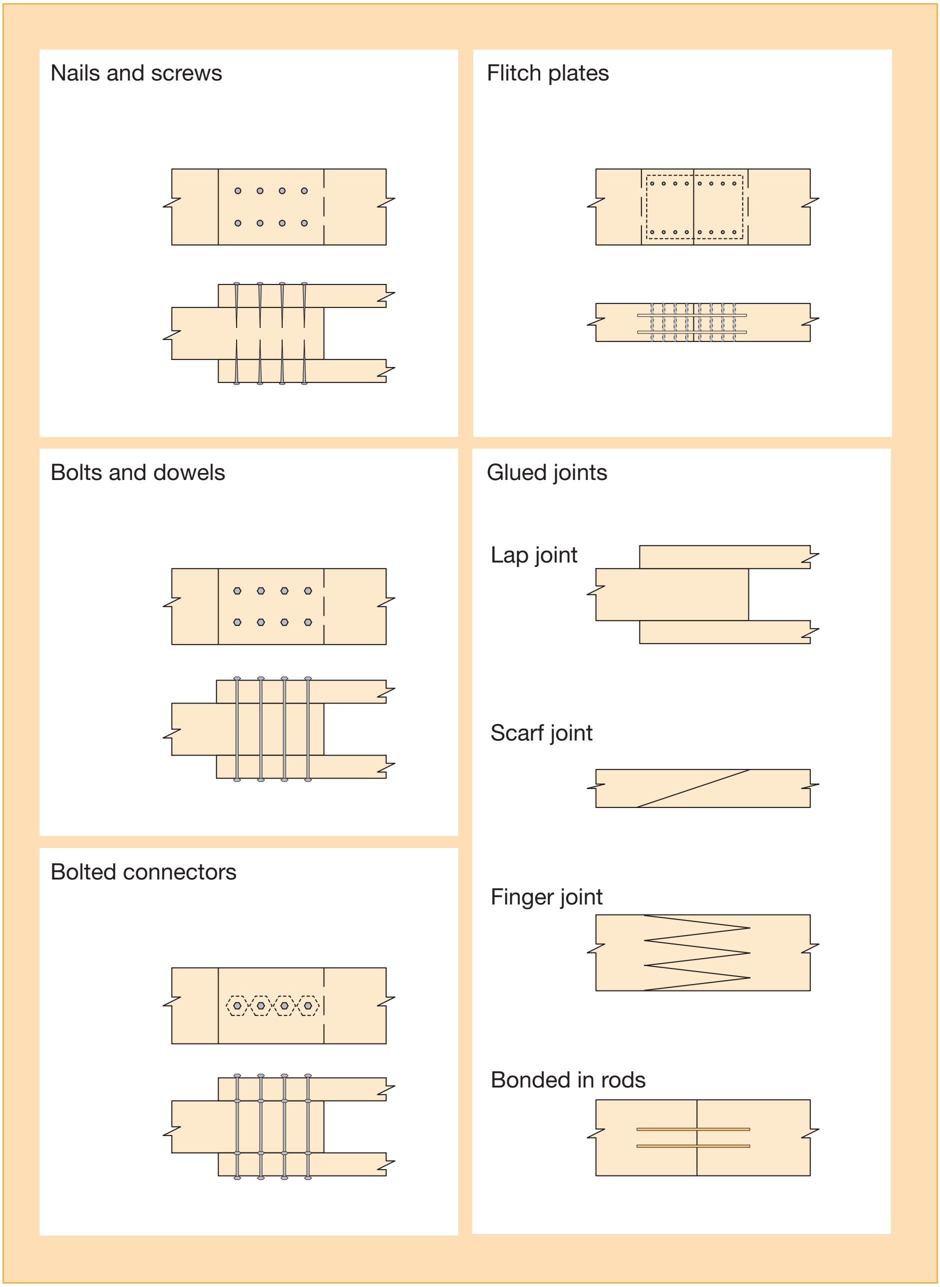


Fig 4.3 Typical connection methods for members in line

Table 4.13 Choice of connection type

No.	Description	Joins members in plane	Fire resistant[a]
1	Nailed or screwed timber-to-timber	✓ but only if stepped	✓ only if covered
2	Coach-screwed or bolted timber-to-timber	✓ but only if stepped	✓ only if recessed and covered
3	Steel dowelled timber-to-timber	✗	✓ only if recessed and plugged
4	Double-sided toothed plate timber-to-timber	✗	✗
5	Split ring or shear plate timber-to-timber	✗	✗
6	Nailing plate	✓	✗
7	Punched metal plate fastener	✓	✗
8	Nailed or screwed plywood-to-timber	✓	✓ only if covered
9	Coach-screwed or bolted external steel plate-to-timber	✓	✗
10	External steel plate with single-sided toothed plate to timber	✓	✗
11	External steel plate with shear plate to timber	✓	✗
12	Bolted steel fin plate(s)-to-timber	✓	✓ only if recessed and covered
13	Steel dowelled internal steel plate(s)-to-timber	✓	✓ only if recessed and plugged
14	Bonded-in steel rods (tension)	✓	✓
15	Glued lap timber-to-timber	✗	✓
16	Glued scarf timber-to-timber	✓	✓
17	Glued plywood-to-timber	✗	✓
18	Glued finger joints	✓	✓
19	Proprietary metal hardware	✓	✗
20	Carpentry connections	✓	✓

Notes

a See Section 6.14 for guidance on making connections fire resistant.

b Where bolt heads and nuts bear on timber special large washers are required (see Section 6.5.1). Square ones are generally not lined up which spoils the appearance.

c Estimated price ranges take into account material and labour costs; £ = low cost, ££ = medium cost, £££ = high cost.

	Neat appearance[b]	Workshop manufacture required	Price range[c]
	✓	✗	£
	✗ but reasonable if round washers are used	✗	£
	✓	✓ unless strict site supervision is provided	££
	✗ but reasonable if round washers are used	✗	£(£)
	✗ but reasonable if round washers are used	✓	£££
	✗	✗	£
	✗	✓	£
	✗	✗	£
	✗	✗	££
	✗	✗	££
	✗	✓	£££
	✗ but reasonable if round washers are used	✗	£££
	✓	✓ unless strict site supervision is provided	£££
	✓	✓ but can be site-made by specialists	£££
	✓	✓	££
	✓	✓	££
	✓	✓ unless strict site supervision is provided and m.c. and temp. controlled	££
	✓	✓	£££
	Depends on type	✗	££(£)
	✓	✓ but can be site-made by specialists	£££

Table 4.14 Commentary to Table 4.13

No.	Description	Notes
1	Nailed or screwed timber-to-timber	Members can be joined in line if they can be stepped but this may reduce the strength due to the reduction in section. Fire resistance can be provided by a timber cap (see Section 6.14).
2	Coach-screwed or bolted timber-to-timber	Members can be joined in line if they can be stepped but this may reduce the strength. Fire resistance can be provided only if the members are wide enough to permit recesses for heads and nuts which can be filled with resin (see Section 6.14). The appearance will be reasonably neat if round washers are specified or fire resistance is provided as described.
3	Steel dowelled timber-to-timber	Members can be joined in line if they can be stepped, but this may reduce the strength. Fire resistance can be provided if the members are wide enough to permit short dowels with plugged ends (see Section 6.14). Dowel holes must be accurately drilled to the exact size of the dowel, so workshop preparation is usually necessary, but the connections can be assembled on site.
4	Double toothed plate timber-to-timber	Toothed plate connections are suitable only for softwoods and the least dense hardwoods. They should be assembled using a high-tensile (8.8 grade) steel bolt. The appearance will be reasonably neat if round washers are specified.
5	Split ring or shear plate timber-to-timber	Recesses for split rings and shear plates would normally be cut in a workshop, but the connections can be assembled on site. The appearance will be reasonably neat if round washers are specified.
6	Nailing plate	Nailing plates are plates with pre-punched holes for nails.
7	Punched metal plate fastener	Punched metal plate fasteners have a similar capacity to nailing plates but have to be applied with a press in a workshop, and are mainly used for prefabricated trussed rafters.
8	Nailed or screwed plywood-to-timber	Plywood-to-timber connections can be made fire resistant with an additional plywood cap (see Section 6.14).

Table 4.14 Continued

No.	Description	Notes
10	External steel plate with single-sided toothed plate to timber	Toothed plate connections are suitable only for softwoods and the least dense hardwoods. They should be assembled using a high-tensile (8.8 grade) steel bolt.
12	Bolted internal steel plate(s)-to-timber	The appearance will be reasonably neat if round washers are specified.
13	Steel dowelled internal steel plate(s)-to-timber	Dowel holes must be accurately drilled to the exact size of the dowel, so workshop preparation is usually necessary. To ensure the correct alignment of steel and timber it is usual to drill them together when already assembled, or to drill the timber using the predrilled steel plates as templates. If final assembly takes place on site plates and timber must be marked to ensure that the plates go into the same members which were drilled with them, the same way round.
14-18	Glued joints	Where possible glued joints should be workshop assembled, but it is possible to join members on site with proper supervision and control of the moisture and temperature (see TRADA's *Adhesively bonded timber connections*[79]).
19	Proprietary metal hardware	Timber engineering hardware varies greatly in appearance, depending on its type, ranging from simple nailed straps to complex interlocking devices for attaching radial members to a post or fully hinged arch bases. Suppliers and catalogues can be found on www.trada.co.uk.
20	Carpentry connections	Carpentry connections generally need to be cut in a workshop to ensure accuracy, but can be assembled on site. These are used only for traditional timber framing, but design guidance is given in *Green Oak in Construction*[22].

4.13 Completing the design

4.13.1 Introduction

Before starting the final design it is good practice to obtain approval of the preliminary drawings from the other members of the design team. When all the comments have been received and the preliminary drawings have been completed, the information should be marshalled into a logical format for use in the final design. This may be carried out in the following sequence:

- checking of all information
- preparation of a list of design data
- amendment of drawings as a basis for final calculations.

4.13.2 Checking of all information

It is now necessary to ensure that the initial design assumptions are still valid, and to take into account the comments and any further information received from the client and the members of the design team, and the results of the ground investigation.

Stability

Ensure that the bracing and shear walls assumed in the original calculations have not been moved or changed and that the calculations are correct.

Loading

Check that the assumptions for dead loads are still correct. Where necessary substitute exact values for approximations or estimations used in the initial design. Consider finishes to floors, ceilings and floors, partitions, services and fixed equipment.

Check snow loading and consider whether drifting or asymmetric loading needs to be allowed for. Make a final check on the design wind loading, and if necessary recalculate wind loads more precisely. Check local effects where necessary such as uplift on overhangs.

Consider whether or not loadings such as earthquake, accidental, constructional or other temporary loadings should be taken into account. In particular consider how to provide adequate strength and stability during construction.

Ensure that all potentially critical load cases have been identified, particularly in the design of roofs and portal frames.

Fire resistance, sound insulation and thermal insulation

Establish with other members of the design team the requirements for fire resistance, sound insulation and thermal insulation in each part of the structure, and ensure that these have been met by appropriate section sizes, protection, insulation or separation. Check the requirements for the provision of cavity wall barriers in timber frame buildings and decide how to communicate these effectively to the construction team. Decide how steelwork in fire resistant connections will be protected and ensure that intumescent or resin sealants will be specified at all joints.

Durability

Establish with other members of the design team the requirements for durability in each part of the structure. Ensure that these have been or will be met by appropriate detailing, choice of materials, preservative treatment, corrosion protection or finishes.

Foundations

Examine the information from the ground investigation and decide on the type of foundation to be used in the final design. Consider especially any existing or future structure adjacent to the structure that may influence not only the location of the foundations but also any possible effect on the superstructure and on adjacent buildings.

Performance criteria

Establish deflection limits, vibration criteria and any other design criteria that are to be used in the final design and obtain agreement to these from the client.

Materials

Confirm the availability of the materials selected in the sizes chosen. If very large items (e.g. prefabricated building modules or long lengths of glulam) are to be transported to the site, check the feasibility of this.

Hazards

Identify any hazards which might result from developing the scheme design. Explore options to mitigate them. (*Construction (Design and Management) Regulations 2007*[15]).

4.13.3 Preparation of a list of design data

The information obtained from the above check and that resulting from any discussions with parties such as the client, design team members, building control and material suppliers should be entered into a design information data list. This list should be sent recorded and agreed before the final design is commenced. Recommendations regarding the information that should be included can be found on the next page.

4.13.4 Amendment of drawings as a basis for final calculations

The preliminary drawings should be bought up to date incorporating any amendments arising out of the final check of information previously accumulated and finally approved.

The following procedure is recommended as an aid to the final calculations.

- Add a three-dimensional grid reference system to the drawings (one unit per floor on the vertical axis), to be coordinated with the rest of the design team where appropriate.
- Give all trusses, walls, columns, beams, lintels and floor areas a unique reference code related if possible to the grid, so that they can be readily identified on the drawings and in the calculations.
- Prepare drawings showing the loads that are to be carried by each member, component and assembly, clearly indicating whether the loads are factored or unfactored.

Design data should include:

- company, contract, job number and date
- client, architect, engineer and checker responsible
- project organisation – details of any design subcontracts
- Building Regulation authority or other and date of submission
- general description of building, intended use, location, and any unusual environmental conditions
- building occupancy class (Table 5.5)
- site constraints
- principal design codes and other important reference documents used
- performance criteria
- design life, deflection limits, etc.
- design methodology including computer programs used
- materials and proprietary systems
- design assumptions
- structural form, stability and robustness provisions
- general loading conditions including environmental and exposure conditions
- fire resistance requirements
- soil and groundwater conditions (including contamination)
- foundation type and design
- drainage
- movement joints
- quality plan
- site supervisor's check list[a] or critical structural information to be included in such a list
- statement of construction method assumed in design
- health and safety including risk management and CDM
- maintenance recommendations
- other relevant data or information

Note

a An eight-page check list for timber frame buildings is provided in Appendix 3 of TRADA's *Timber Frame Construction*[76].

4.13.5 Final design calculations

When the above checks, design data list and preparation of the preliminary drawings have been carried out then the final design calculations for the structure can commence. It is important that these should be carried out in a logical sequence. This is likely to include the following; loads, load cases, roof, structural frame, floors, beams, columns, walls, staircases, foundations, stability, robustness, temporary bracing, detailing, construction requirements. Fire resistance and durability requirements will also need consideration.

5.1 General design procedure for structural members

- Select material and trial member sizes (Section 3.3).
- Obtain the characteristic loads on the structure from EC1. For the weights of timber materials see Section 4.7.
- Convert these to characteristic actions on the component.
- Assign a load duration class to each action (Table 2.2).
- Assign partial factors (γ and ψ to each action (Tables 3.1 and 3.2).
- Determine the service class from Table 2.1. Note that the service class may vary from one part of a building to another.
- Obtain characteristic material properties (Section 3.3.3 or the CD).
- Assign values to k_{mod}, k_{def} and γ_M (Tables 2.3, 2.4 and 3.19).
- Determine any additional strength modification factors for the component (Table 3.20 and relevant sections). Where appropriate include the factor k_{sys} for system strength (see Table 3.20).
- Determine the relevant ULSs (strength, stability, accidental) and SLSs (Section 3.2).
- Determine by agreement with client the SLS deflection limits for the component (Section 3.2.2).
- Verify that no relevant limit state is breached. For ULS verify each relevant load case.

For the definition of bending axes see Section 1.5.2.

For the definitions of the symbols see Notation. Where a strength modification factor is not defined and evaluated it can be found in Table 3.20. In general σ means an applied stress and f means a material strength property.

5.2 Flexural members (solid rectangular sections)

5.2.1 Straight beams

5.2.1.1 Bending strength (ULS) (EC5 6.1.6)

For beams loaded in bending about the y axis only, verify that:

$$\sigma_{m,y,d} \leqslant f_{m,y,d}$$

Where $\sigma_{m,y,d}$ = maximum design bending stress about y axis

$$= \frac{M_{y,d}}{W_y}$$

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

For beams loaded in bending about the z axis only, substitute z for y above.

The factor k_{crit} may be taken as 1.0 for a beam in which lateral displacement of its compressive edge is prevented throughout its length and torsional rotation is prevented at its supports, as by direct connection of sheathing, deck or joints, together with adequate bridging or blocking. For other situations see Table 3.20 and Figures 3.2 to 3.4 for the value of k_{crit}.

For beams loaded in bending about both axes, verify that:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

and $$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

Where k_m = factor to allow for the re-distribution of secondary bending stresses
= 0.7 for rectangular sections in solid timber, glulam and LVL
= 1.0 in all other cases

5.2.1.2 Shear strength (ULS) (EC5 6.1.7)

Verify that $\tau_d \leqslant f_{v,d}$

Where τ_d = maximum design shear stress

$$= \frac{1.5F_{v,d,max}}{bh}$$

and $$f_{v,d} = \frac{k_{sys} k_{mod} f_{v,k}}{\gamma_M}$$

It is anticipated that k_{shear}, a reduction factor for $f_{v,d}$, will be introduced in the near future to allow for splitting (see Table 3.20).

5.2.1.3 Bearing strength (ULS) (EC5 6.1.5)

Bearing stress can occur beneath a beam at its supports or on the top of a beam or horizontal member (see Figures 3.7 and 3.8). Verify that:

$$\sigma_{c,90,d} \leqslant f_{c,90,d}$$

Where $\sigma_{c,90,d}$ = design bearing stress

$$= \frac{F_{c,90,d}}{bl}$$

b = breadth of beam

l = length of bearing

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

For values of $k_{c,90}$ see Figures 3.7 and 3.8.

The strength of the bearing supporting the beam should also be checked. (k_{sys} is not applicable to the strength of bearings which support a multiple member beam or a series of posts such as a timber frame wall panel.)

It is anticipated that the method for calculating $k_{c,90}$ will change in the near future.

5.2.1.4 Notches in the length of a beam (ULS) (from BS 5268-2)

Allowance should be made for the effect of notches or holes on the strength of a beam, the effective depth being taken as the minimum depth of the net section. Notches on the tension side of a beam will induce stress concentrations so it is advisable to prohibit them. However in simply supported floor and roof joists with a depth $h \leqslant$ 250mm and a span l the effect of notches and holes need not be calculated provided that:

- notches not exceeding $0.125h$ are located between $0.07l$ and $0.25l$ from a support
- holes drilled at the neutral axis have a diameter not exceeding $0.25h$ and centres at least 3 diameters apart located between $0.25l$ and $0.4l$ from a support.

For beams deeper than 250mm the same rules will be safe if h is taken to be 250mm.

5.2.1.5 Notches at supports (ULS) (EC5 6.5)

Verify that:

$$\tau_d \leqslant k_v f_{v,d}$$

Where $\tau_d = \dfrac{1.5F_{v,d,max}}{bh_{ef}}$

k_v = strength reduction factor for notched beams defined below

b = breadth of beam

h_{ef} = effective height shown in Figure 5.1

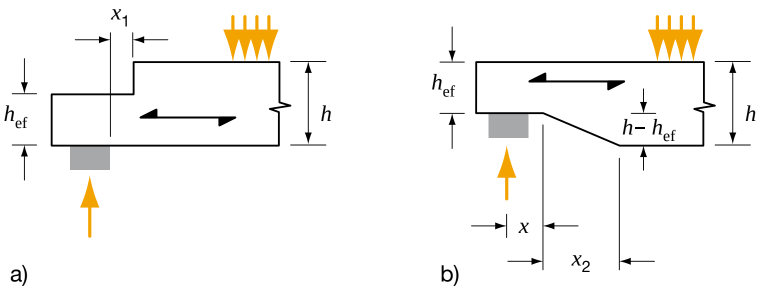


Fig 5.1 Symbols for shear stress at notched ends

For notches in the upper side at supports (Figure 5.1a)

$$k_v = \frac{h\,(h_{ef} - x_1) + x_1 h_{ef}}{h_{ef}^2} \text{ for } x_1 \leqslant h_{ef} \qquad \text{(derived from BS 5268-2[3])}$$

$= 1.0$ for $x_1 > h_{ef}$.

For notches in the underside of supports (Figure 5.1b)

$$k_v = \text{minimum of 1.0 and } \frac{k_n\left[1 + \dfrac{1.1\left(\dfrac{x_2}{h - h_{ef}}\right)^{1.5}}{\sqrt{h}}\right]}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)} + 0.8\dfrac{x}{h}\sqrt{\dfrac{1}{\alpha} - \alpha^2}\right)} \qquad \text{(derived from EC5 (6.62))}$$

Where α = h_{ef}/h

x = distance from the line of action of the support reaction to corner of notch (use the minimum support width which will support the reaction)

x_1, x_2 = refer to Figure 5.1

k_n = 4.5 for LVL

= 5.0 for solid timber

= 6.5 for glulam

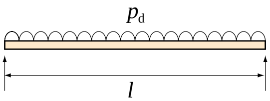
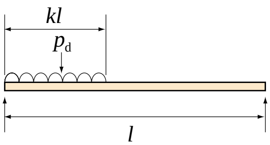
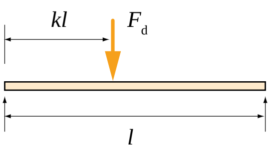
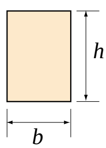
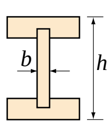
5.2.1.6 Deflection (SLS) (EC5 7.2)

Verify that the initial and/or final deflections do not exceed the serviceability limits (see Section 3.2.2). Deflection calculations for timber members should normally include both bending and shear deflection (see Table 5.1). Notches and holes can generally be ignored in the calculation of deflection. If EC5 Section 9 is used to design a timber I-beam or box beams it is normal to assume that all the bending deflection occurs in the flanges and all the shear deflection in the webs.

5.2.2 Tapered, curved and pitched cambered glulam beams (EC5 6.4.3)

These are illustrated in Figure 5.2. They should be made with the wood fibre direction parallel to the lower edge. The detailed design of these beams is not covered in this *Manual*. For full design rules refer to EC5 6.4.2 and 6.4.3. These beam types are often designed by specialist suppliers.

Table 5.1 Calculation of bending and shear deflection in timber beams

Load type		Instantaneous deflection at mid-span	
Full span UDL 		Bending	$\dfrac{5p_d l^4}{384EI}$
		Shear	$\dfrac{k_{form} p_d l^2}{8Gbh}$
Partial span UDL 	$k \leqslant 0.5$	Bending	$\dfrac{p_d l^4 (12k^2 - 8k^4)}{384EI}$
		Shear	$\dfrac{k_{form} p_d l^2 k^2}{4Gbh}$
	$0.5 < k \leqslant 1$	Bending	$\dfrac{p_d l^4 (1 - 8k + 36k^2 - 32k^3 + 8k^4)}{384EI}$
		Shear	$\dfrac{k_{form} p_d l^2 (-0.5 + 2k - k^2)}{4Gbh}$
Point load 	$k \leqslant 0.5$	Bending	$\dfrac{F_d l^3 (3k - 4k^3)}{48EI}$
		Shear	$\dfrac{k_{form} F_d lk}{2Gbh}$
Key		k_{form}	G
Solid timber and glulam		1.2	See Tables 3.14 and 3.15.
LVL		1.2	See Table 3.16 or manufacturer's certification literature for E and G.
Timber I-joists		1.0	Refer to manufacturer's ETA document or other certification literature for E and G.

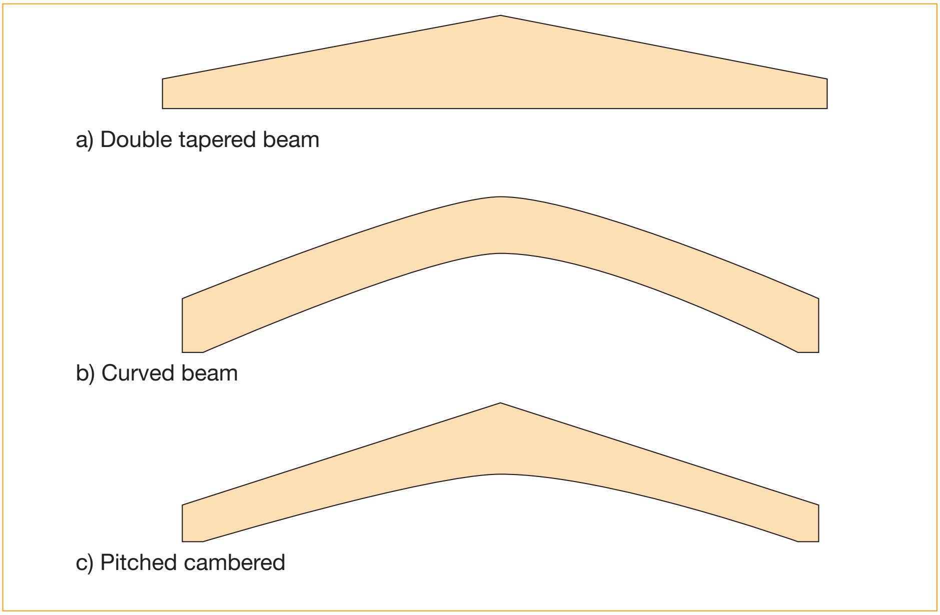


Fig 5.2 Glulam beams

For curved beams (types b) and c)) a reduction in bending strength is necessary unless the inner radius exceeds 10.8m with 45mm thick laminates or 8.0m with 33mm thick laminates.

5.3 Members subject to axial compression (solid rectangular members)

5.3.1 Members subject to axial compression only (EC5 6.3.2)

To prevent buckling about the major *y* axis, verify that

$$\sigma_{c,0,d} \leqslant k_{c,y} \; f_{c,0,d}$$

Where $\sigma_{c,0,d}$ = design axial compression stress

$k_{c,y}$ = instability factor, see Table 3.20 and Figures 3.5 and 3.6

$f_{c,0,d}$ = $\dfrac{k_{sys} \, k_{mod} \, f_{c,0,k}}{\gamma_M}$

To prevent buckling about the minor *z* axis, use $k_{c,z}$ instead of $k_{c,y}$. If the column is adequately restrained against buckling in one direction, k_c in that direction = 1.0.

k_{sys} is applicable only to columns consisting of several members connected together in such a way that they share the loads (see Table 3.20 note **c**).

If the axial load is applied eccentrically, it should be converted to an equivalent axial load and bending moment, and the member should be designed in accordance with Section 5.3.2 or 5.3.3.

5.3.2 Members subject to axial compression and bending about the *y* axis only (EC5 6.2.4, 6.3.2 and 6.3.3)

In this case three verifications are needed.

i) Combined compression and bending stress (strength check).

ii) Column stability (to prevent buckling as a column).

iii) Lateral torsional stability (to prevent torsional instability as in a beam).

Verify that:

i) $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m.y.d}}{f_{m,y,d}} \leqslant 1$$

ii) $$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leqslant 1$$

iii) $$\left(\frac{\sigma_{m,y,d}}{k_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z}\ f_{c,0,d}} \leqslant 1$$

Where $\sigma_{m,y,d}$ = design bending stress about *y* axis

$f_{m,y,d}$ = $\dfrac{k_h k_{sys} k_{mod} f_{m,k}}{\gamma_M}$

and other symbols and notes are as in Section 5.3.1 and Table 3.20. For k_{crit} see Section 5.2.1.1.

For rafters and other beam-columns where the compression stress varies along the axis, it may be necessary to check the combined stresses at more than one position to determine the worst case.

For columns subject to bending about the *z* axis only, substitute *z* for *y* above, but omit iii).

5.3.3 Members subject to axial compression and bending about both axes (EC5 6.2.4, 6.3.2 and 6.3.3)

As in Section 5.3.2 three verifications are needed. Verify that:

i) $$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1 \text{ and } \left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

ii) $$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1 \text{ and } \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

iii) $$\left(\frac{\sigma_{m,y,d}}{k_{crit} f_{m,y,d}}\right)^2 + \frac{\sigma_{c,0,d}}{k_{c,z} f_{c,0,d}} \leqslant 1$$

Where k_m = factor to allow for the re-distribution of bending stresses

= 0.7 for rectangular sections in solid timber, glulam and LVL

= 1.0 in all other cases

and other symbols and notes are as in Sections 5.3.1 and 5.3.2.

5.4 Members subject to axial tension

5.4.1 Members subject to axial tension only (EC5 6.1.2)

Verify that:

$$\sigma_{t,0,d} \leqslant f_{t,0,d}$$

Where $\sigma_{t,0,d}$ = design axial tension stress

$$f_{t,0,d} = \frac{k_h k_{sys} k_{mod} f_{t,0,d}}{\gamma_M}$$

and other symbols are as in Table 3.20.

k_{sys} is applicable only when several members are connected together in such a way that they share the loads (see Table 3.20, note **c**).

5.4.2 Members subject to axial tension and bending about the y axis only (EC5 6.1.2 and 6.2.3)

In this case two verifications are needed.

i) Combined tension and bending stress (strength check).

ii) Lateral torsional stability (to prevent torsional instability as in a beam if the tensile stress is relatively low).

Verify that:

i) $$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} \leqslant 1$$

ii) $$\frac{\sigma_{m,y,d} - \sigma_{t,0,d}}{k_{crit} f_{m,y,d}} \leqslant 1$$

Where $\sigma_{m,y,d}$ = design bending stress about *y* axis

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

and other symbols and notes are as in Section 5.4.1. For k_{crit} see Section 5.2.1.1.

For tension members subject to bending about the *z* axis only, substitute *z* for *y* above, but omit ii).

5.4.3 Members subject to axial tension and bending about both axes (EC5 6.1.2 and 6.2.3)

As in Section 5.4.2 two verifications are needed. Verify that:

i) $$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1 \text{ and } \frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

ii) $$\frac{\sigma_{m,y,d} - \sigma_{t,0,d}}{k_{crit} f_{m,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1$$

with symbols and notes as in Sections 5.4.1 and 5.4.2.

5.5 Flitch beams

5.5.1 Introduction

This section provides guidance on the design of beams in which one or more steel plates are tightly bolted between lengths of timber to form a composite beam, as shown in Figure 5.3. As mild steel is about 20 times as strong and stiff as softwood its thickness should be proportionally less than the combined thickness of the timber members. Most commonly one steel plate and two timber members are used. Typical steel thicknesses range from 8mm to 20mm.

5.5.2 Scope

The design method applies to flitch beams in which:

- the loads and reactions are applied equally to both the timber members
- the depth of the steel plate is 25mm less than depth of timber members to ensure that loads are applied to the timber
- the bending stiffness of the steel plate is between 20% and 80% of the total bending stiffness of the beam
- the steel plate extends the full length of the timber members
- all bolt diameters and positions are in accordance with calculations
- the timber members are orientated so that if there is any cupping or bowing (within grade limitations) the convex side is on the outside.

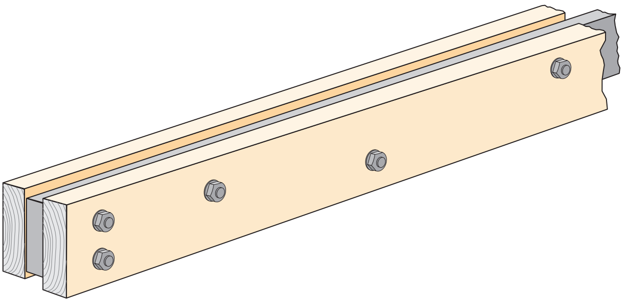


Fig 5.3 Typical flitch beam

5.5.3 Design method

All applied loads except point loads positioned within half the timber depth from a structural support are assumed to be distributed between the timber and the steel in proportion to the relative bending stiffness (EI) of the timber and steel.

Since the modulus of elasticity of the timber effectively diminishes over time as creep occurs, the share of load taken by the steel increases over time. Therefore the strength of the timber has to be checked before creep occurs and the strength of the steel and bolts after creep occurs. A value of E_{mean} in conjunction with a special value of k_{sys} for two members has been found appropriate to check the timber stresses. The final minimum modulus of elasticity, $E_{0,05,fin}$ modified for two timber members is recommended to check the stresses in the steel and the requirements for bolts since this produces higher stresses in them. The forces acting on the steel and bolts have been increased by 10% to allow for the effects on load distribution of shear deflection and slip.

The deflection of the beam can be calculated from the deflection of the timber or the steel, since the timber and steel move together. This may be either the initial or final deflection or both, depending on the serviceability requirements.

The expressions in Table 5.2 are recommended to calculate the share of a load, F_a, which is applied to the timber, F_t, or the steel, F_s. This table also shows the limit state verifications which should be carried out in the design of a flitch beam.

Table 5.2 Load distribution expressions

Limit state	Load
Bending strength of timber Shear strength of timber Bearing strength of timber Instantaneous deflection	$F_t = \dfrac{E_{mean} I_t F_a}{E_{mean} I_t + E_s I_s}$
Bending of steel plate Bolt loads[a]	$F_s = \dfrac{1.1 E_s I_s F_a}{E_{0,05,fin} I_t + E_s}$
Final deflection	$F_t = \dfrac{E_{mean,fin} I_t F_a}{E_{mean,fin} I_t + E_s I_s}$

Note

a It may be necessary to check more than one load case to determine the bolting requirements. $E_{0,05,fin}$ varies with the load case.

In Table 5.2 the value of $E_{0,05,fin}$ may be calculated as:

$$E_{0,05,fin} = k_{sys,2E} E_{0,05} \frac{w_{instG} + w_{instQ}}{(w_{instG} + w_{instQ} + w_{creep})}$$

Where $k_{sys,2E}$ = system modification factor for elastic modulus of two timber members which depends on the variability of the material

= 1.14 for softwoods

	=	1.06 for hardwoods and glulam
	=	1.0 for LVL, LSL and PSL
$E_{0,05}$	=	minimum modulus of elasticity of timber
$w_{inst,G}$	=	instantaneous deflection produced by dead loads without creep
	=	$\sum_{j>0} w_{instG,j}$
$w_{inst,Q}$	=	instantaneous deflection produced by imposed loads
	=	$w_{instQ,1} + \sum_{i>1} \psi_{0,i}\, w_{instQ,i}$
w_{creep}	=	creep deflection calculated using $E_{0,05}$
	=	$k_{def}\left(\sum_{i>0} w_{instG,j} + \sum_{i>0} \psi_{2,i}\, w_{instQ,i}\right)$

$E_{mean,fin}$ (for the calculation of final deflection) may be calculated as:

$$E_{mean,fin} = E_{mean} \frac{w_{instG} + w_{instQ}}{(w_{instG} + w_{instQ} + w_{creep})}$$

For the timber f_m, f_v and $f_{c,90}$ may be increased by k_{sys2}, a system strength modification factor for two connected members:

Where k_{sys2} = 1.04 for solid timber and glulam
= 1.00 for LVL, PSL and LSL

k_{sys2} is not applicable to the bearing strength of any timber supports

For simplicity the combined weight of the steel and timber may be treated as an additional UDL.

5.5.4 Strength checks

For initial design purposes increase the bending stress in the timber and steel by 10% to allow for bolt holes.

For more accurate calculations the factor by which the bending stress in the timber and steel should be increased for a single hole offset from the centre line is:

$$\frac{h^2\left[h^2 - hd + 2dy\right]}{h^4 - h^3 d - hd^3 + d^4 - 12hdy^2}$$

Where h = depth of member (timber or steel)
d = diameter of hole
y = distance between centre of hole and centre line of member

With two holes on opposite sides of the centre line and at an equal distance of y from it the bending stress is increased by a factor of:

$$\frac{h^3}{h^3 - 2d\left(d^2 + 12y^2\right)}$$

In checking the shear strength of the timber, only the shear force in the timber needs to be considered. It is unnecessary to make an allowance for bolt holes at the reactions, since at this point the shear stresses in the timber are mostly converted to compression perpendicular to the grain.

The bearing strength of both the beam and the bearing plate should be checked using R_{max}, the greater of the two reactions. Masonry bearings may require a spreader plate.

The bending strength in the steel must be checked, but it may be assumed that its shear strength and bolt bearing strength are adequate within the design parameters given.

5.5.5 Stability

The requirements in Section 5.6.2 should ensure the stability of the steel plate. The stability of the beam as a whole should be satisfied if the ends are held in position and the compression edge is held in line by the direct connection of sheathing, decking or joists, and if $h \leqslant 3.75b$ where h is the depth of the timber members and b is the total breadth of the timber members.

5.5.6 Bolts for UDLs

For UDLs it is assumed that the load transfer to the steel per unit length is constant if the bolts are spaced equidistantly. So:

$$\text{the load per bolt} = \frac{F_{UDL,s}}{n}$$

Where $F_{UDL,s}$ = total UDL transferred to steel plate
n = total number of bolts to transfer the UDL

When calculating the number of bolts required to resist partial UDLs it is safest to assume that the UDLs extend the full length of the beam.

For stability reasons it is recommended that the bolts along the beam should be staggered, i.e. positioned alternately above and below the centre line with their centres offset from the centre line by $h_t/4$, where h_t is the depth of the timber members. They should not be positioned further from the centre line than this, or the holes will significantly reduce the bending resistance of both the steel and timber members. Two straight rows of bolts may be used along the beam if one staggered row is insufficient to support the UDLs.

To ensure composite action of the timber and steel, the bolts should be equidistantly spaced at no more than 2.5 times the depth of the timber members, and no more than 600mm. They should also be spaced no closer than necessary. Ideally the bolt diameter should be selected so that the required number of bolts can be spaced at the lesser of $2.5h_t$ and 600mm. With staggered bolts the spacing is measured between consecutive bolts in the beam, not between consecutive bolts in the upper row or the lower row.

5.5.7 Bolts for point loads

For a point load the load transfer from the timber to the steel will occur in the vicinity of the load, so a sufficient number of bolts should be provided to transfer a load of $F_{\text{point,s}}$ from the timber to the steel at or near that point. The load duration for an applied point load should be used to calculate the number of bolts required to support it, not the load duration for the load case, since the duration effects on the embedment stress are local.

5.5.8 Bolts at reactions

At each reaction point, sufficient bolts must be specified to transfer the load in the steel plate at that point back into the timber which bears onto the supports. For stability reasons at least two bolts should be specified at each reaction, spaced as near the top and bottom of the steel plate as the minimum edge distances in the steel and timber allow. With large loads two columns of bolts may be used at the supports or beneath a point load, spaced $4d$ apart, where d is the bolt diameter, positioned equidistantly on each side of the line of action.

The bolts at the supports should be positioned so that they do not interfere with any joist or beam hangers which may be used.

5.5.9 Distances, spacings and orientation

The distance between the centre of the bolt holes and the edges of the steel should be at least 1.2 times the hole diameter. The corresponding minimum end and edge distances in the timber should be $4d$. Minimum spacing parallel and perpendicular to the grain should be $4d$.

Where the bolting pattern is not symmetrical about the centre line for any reason, it is essential to ensure that the beam is assembled and installed the right way round, for example by marking the plate or beam in an unambiguous manner.

Bolt holes should be drilled with the plate clamped between the timber members to ensure that they are all precisely in line.

5.6 Providing structural stability

For open frame construction and plane frameworks such as rafters, structural stability is normally provided by triangulation, although timber portal frames are also common. For floors, ceilings and timber frame wall panels it is normally provided by the diaphragm action of the sheet materials. See Section 5.8 for bracing design, Section 5.9 for horizontal diaphragms, Section 9 for portal frames, Section 10.8.1 for timber frame wall panels and Section 10.8.3 for other ways of providing stability in a vertical plane.

5.7 Bracing of compression members and of beam or truss systems

EC5 9.2.5, together with NA.2.10, gives detailed procedures for designing lateral bracing for compression members. For a single compression member the brace must resist a lateral force equal to 2% of the axial load in the member, so for a series of truss compression chords braced by a single brace the total force in the brace will be 2% of the sum of the axial forces in truss chords. In order to prevent buckling the bracing member should also be stiff enough to limit the deflection of the braced system to span/500.

5.8 Horizontal diaphragms

Floors, ceilings and roofs may be used to transfer horizontal forces to the supporting walls.

5.8.1 Simple solution

On the basis of experience it may be assumed that conventional floors, flat roofs, sarked rafters and ceilings, in which a wood-based panel product (or gypsum plasterboard in the case of ceilings) is fastened to timber joists, have adequate strength and stiffness as horizontal diaphragms, subject to the following[80]:

- span: depth ratio does not exceed 2:1 in any wind direction
- span does not exceed 12m between supporting walls
- minimum thickness of decking is 15mm for plywood and OSB, 18mm for particleboard, 12.5mm for plasterboard in roof ceilings attached to roof trusses as below
- all panel edges are fastened either to neighbouring panel edges, via nailing to a common joist, rafter, ceiling tie or batten, or else via a tongue and groove joint glued in accordance with Table 12.1, or else to supports around the wall perimeter
- minimum fastener specification 3.1mm ϕ ringed shank machine-driven nails or 4mm ϕ (No.8) wood screws, length 2.5 times board thickness, at maximum centres of 150mm on the edges of the panel
- maximum fastener centres internally along all crossed joists, rafters and ceiling ties to be 300mm
- supporting battens, if used between panels, to be skew-nailed at each end to a joist or the perimeter framing
- plasterboard in roof ceilings to be fastened to the underside of every truss and truss clips secure every truss to a head binder or wall plate
- characteristic wind pressure $\leqslant$ 1500N/m^2.

5.8.2 Eurocode 5 method

EC5 9.2.3 provides a more general design approach for floors and roofs which consist of wood-based panels fastened with nails or screws to joists and a surrounding frame. In principle the method also applies to plasterboard ceilings made in a similar way. Within the limitations below, the diaphragm may be considered to act like an I-beam on its side (see Figure 5.4). The edges of the floor, roof or ceiling (wall plates, timber-frame wall head binders, header joists or ring beams) act like the flanges of the beam and resist the bending moments, i.e. they are in compression or tension. The decking acts like the web and transfers the shear force to the shear walls. It may be assumed that the shear force is transferred uniformly along the length of the shear walls.

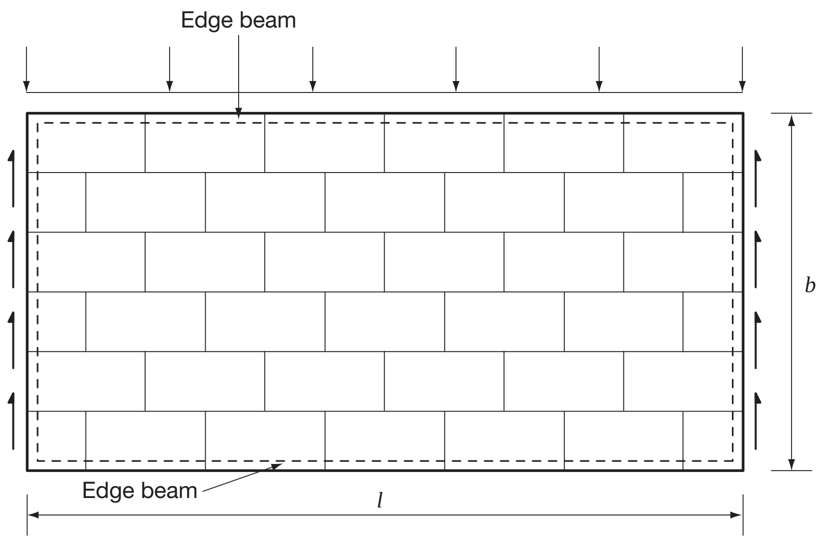


Fig 5.4 Horizontal diaphragm made with panels and joists

Limitations:

- $l \leqslant 6b$ where l and b are shown in Figure 5.4
- all panel edges are fastened either to joists or perimeter framing, or else to neighbouring panels, in accordance with Table 12.1
- supporting battens are double skew-nailed at each end to a joist or the perimeter framing
- a means is provided for the transfer of the tensile force across joints in edge beams.

Three verifications are needed.

i) Shear strength of the 'web'.

ii) Tensile strength of 'flanges' – always less than their compressive strength.

iii) Shear strength of the web connections.

Verify that:

i) $$\frac{F_d}{2tb} \leqslant f_{v,d}$$

Where F_d = total design force on diaphragm (N)

t = thickness of decking (mm)

$f_{v,d}$ = design panel shear strength of decking (N/mm²)

$$= \frac{k_{mod} f_{v,k}}{\gamma_M}$$

ii) $\dfrac{F_d l}{8A_{edge} b} \leqslant f_{t,0,d}$

Where A_{edge} = cross-sectional area of one edge beam (e.g. header joist) (mm^2)

$f_{t,0,d}$ = $\dfrac{k_{mod} f_{t,0,k}}{\gamma_M}$ for edge beam (N/mm^2) (or design tensile strength of other material, e.g. steel)

iii) $F_d \leqslant 2b\left[\dfrac{R_{d,1}}{s_1} + \dfrac{R_{d,2}}{s_2}\right]$

Where b = diaphragm depth as in Figure 5.4 (mm)
R_d = lateral design resistance per fastener (N)
s = fastener spacing (mm)

In expression iii) subscripts 1 and 2 refer to decking and any plasterboard respectively.

Notes

- For wind loads F_d should be taken as the sum of the pressure and suction forces.
- Where wind loads vary with direction, the design must be checked for every direction.
- EC5 9.2.3.1(2) allows calculated values of R_d to be enhanced by a factor of 1.2. Hence for 2.65mm diameter plasterboard nails or screws, the value of R_d may be taken as 1.2×278N = 334N for 12.5mm thick board, or 1.2×296 = 355N for 15mm thick board (see Tables 10.6 to 10.8).
- Decking panels provide greater shear resistance when their longer edges are parallel to the wall which is subject to the greatest force and they are staggered, as in the diagram. Although the provisions above mean that this orientation is not essential it is recommended. Normally this will mean that floor joists span in the shorter direction, which can also improve the vibration performance of a floor.

5.9 Vertical diaphragms

For the design of timber frame walls see Section 10.8.1.

5.10 Fire resistance

5.10.1 Introduction

Structural fire design covers a number of matters, as mentioned in Section 2.11. This section briefly describes how to ensure that structural members and connections retain sufficient mechanical resistance in a fire. For more comprehensive design rules refer to EC5-1-2. For information on surface treatment to prevent the spread of flame, see Section 3.4.6.

Fire is an accidental design situation, for which the following rules apply:

- design values of loads should be calculated in accordance with EC0 (6.11b) (see Section 3.2.1.3)
- partial factors for materials and connections are 1.0 (see Table 3.19)

- twentieth percentile characteristic strength and stiffness properties are used (EC5-1-2 Clause 2.3), increasing them from 5% to 25%
- the NA to EC5-1-2 states that the cross-section properties in fire should be determined using the 'reduced cross-section method', for which $k_{mod} = k_{mod,fi} = 1.0$ (EC5-1-2 Clause 4.2.2(5))
- the effect of the deflection of flexural members on other structural members and on the integrity of protecting elements should be taken into consideration.

It is recommended that the deflection of members which are protected by attached plasterboard should not exceed span/20 (Table 3.4), unless the fire resistance of the assembly for the period required has been demonstrated by test. Deflection calculations should be based on the residual section of the flexural member.

Twentieth percentile characteristic strength and stiffness properties are calculated from the fifth percentile values as follows:

$$f_{d,fire} = k_{fire} f_k$$
$$E_{d,fire} = k_{fire} E_{0,05}$$
$$G_{d,fire} = k_{fire} G_{0,05}$$

Where $f_{d,fire}$ = design value of a strength property
$E_{d,fire}$, $G_{d,fire}$ = design values of elastic and shear modulus, respectively
k_{fire} = factor from Table 5.3

Table 5.3 Values of k_{fire}

Element	k_{fire}
Solid timber	1.25
Glulam and wood based panel products	1.15
LVL	1.10
Connections with fasteners in shear with outer members of wood or wood based panels	1.15
Connections with fasteners in shear with outer members of steel, and connections with axially loaded fasteners	1.05

5.10.2 Protection by insulation

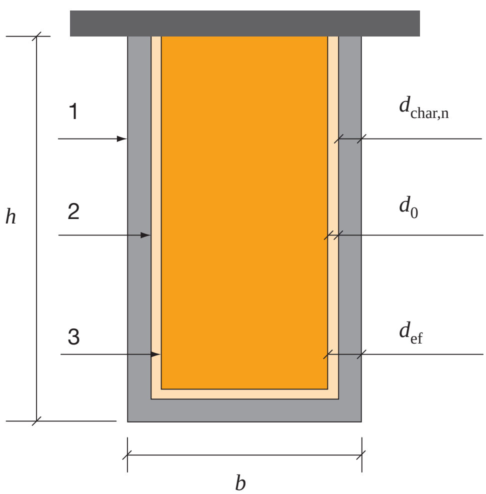
Section 4.3.2 provides common solutions for providing 30 and 60 minutes of fire resistance by means of plasterboard. Exact specifications may be obtained either by calculation in accordance with EC5-1-2 or from the manufacturers based on test data. For party walls and floors reference should normally be made to plasterboard manufacturers for details of constructions which have been proved by test.

In order for plasterboard to protect the underlying timber all joints should be filled with plaster and taped, and the manufacturers' fixing specifications should be conformed to. Where two layers of plasterboard are used the joints should be staggered and the inner layer must be fully fixed to the manufacturer's specifications with its own nails or screws which will be protected from heat by the outer layer.

A ceiling should also have the required surface spread of flame (reaction to fire) performance. This requirement will be met if the ceiling is made of plasterboard.

5.10.3 Calculation of reduced cross-section

Where it is decided to provide fire resistance by the use of sacrificial timber, the reduced cross-section following a period of fire should be determined as follows.



Notes

1	Initial surface of element.
2	Border of residual cross-section.
3	Border of effective residual cross-section.
$d_{char,n}$	Notional charring depth (allows for the detrimental effects of arrises and fissures).
d_0	Depth of layer with assumed zero strength and stiffness.
d_{ef}	Depth to unaffected, full strength material.

Fig 5.5 Definitions of charring dimensions

For structural members exposed to fire on more than one side for more than 20 minutes, calculate the effective charring depth d_{ef} as:

$$d_{ef} = d_{char,n} + d_0$$

Where $d_{char,n} = \beta_n t$ mm

$d_0 = 7$mm

β_n = notional charring rate from Table 5.4

t = time from start of charring in minutes

Charring will occur to this depth on all exposed surfaces as shown in Figure 5.5. Hence in the example shown, where the upper face of a beam is protected by an adequate thickness of plasterboard, the effective residual section will measure $(h - d_{ef}) \times (b - 2d_{ef})$. It may be assumed that this section retains the strength and stiffness properties appropriate to the service class at normal temperatures.

For structural members exposed on only one side to fire, or initially protected from fire, or exposed to fire for less than 20 minutes, refer to EC5-1-2 3.4.2, 3.4.3 or 4.2.2 respectively.

Table 5.4 Notional design charring rate for members exposed on more than one side to fire

Material		Characteristic density ρ_k (kg/m³) (See Tables 3.14 and 3.15)	β_n (mm/min)
Softwood	Glulam Solid timber	$\rho_k \geqslant 290$ $\rho_k \geqslant 290$	0.7 0.8
Hardwood[a]	Solid timber and glulam Solid timber and glulam	$\rho_k \geqslant 290$[b] $\rho_k \geqslant 450$	0.7 0.55
LVL		$\rho_k \geqslant 480$	0.7

Notes

a The charring rate of beech should be taken as for solid softwood.

b Linear interpolation may be used for glulam and solid timber with a characteristic density between 290 and 450kg/m³.

5.10.4 Rules for the analysis of reduced cross-sections

- The effects of fire on compression perpendicular to the grain may be disregarded.
- The effects of fire on the shear strength of solid members may be disregarded. For notched beams verify that the residual cross-section in the vicinity of the notch is at least 60% of the cross-section required for normal temperature design.
- Bracing members made of timber or wood-based panels may be assumed not to fail if their residual cross-sectional area or thickness is at least 60% of its initial value and they are fixed with nails, screws, dowels or bolts.
- Where the bracing of a beam or column is predicted to fail, its stability should be analysed as though without lateral restraint.

5.11 Building a robust structure

General requirements for robustness are set out in Section 2.10. The importance of achieving a robust structure and avoiding disproportionate collapse depends to some extent on the size and purpose of the building. In general robustness can be achieved either by:

i) providing effective horizontal and vertical connections at all floor and roof levels (see Appendix A), or

ii) by ensuring that if any principal column, beam or load-bearing element such as a defined length of timber frame wall is removed then major collapse will not occur, or

iii) by providing critical ‘key’ or ‘protected’ elements, which must be able to resist an accidental pressure of $34kN/m^2$ in any direction.

Where necessary more than one of these methods may be used. Methods i) and ii) are most widely used in timber buildings, but method iii) is sometimes made use of in platform timber frame design, for example by providing an LVL post with steel fixings and no connected panels to support a floor.

Robust design is an accidental design situation, for which the following rules apply:

- design values of loads should be calculated in accordance with EC0 (6.11b) (see Section 3.2.1.3)
- partial factors for materials and connections are 1.0 (see Table 3.19)
- load duration is normally instantaneous
- deflection limits are usually governed by the requirement to maintain a passable exit route from the building.

Table 5.5 provides some recommendations for securing appropriate levels of robustness.

Method ii) (‘notional removal’), although it is offered as an option only for building occupancy class 2 Upper Risk buildings, is sometimes used to justify building occupancy class 2 Lower Risk timber frame buildings on the grounds that it is more severe, especially where tests involving the removal of supporting elements have demonstrated the construction method to be adequately robust. Robustness through Method ii) is normally achieved through providing continuity of beams or deep wall panels and designing them to carry the load from the structure above after the notional removal of the support below. More details on this method for various types of load-bearing element are given in *Multi-storey timber frame buildings – a design guide*[81].

Table 5.5 Methods of achieving a robust construction

(based on BS EN 1991-1-7[82] Table A.1)[a]

Building occupancy class	Number of storeys (n)	Use	Method of achieving compliance
1	Any	Agricultural Buildings into which people rarely go	Acceptable if designed to normal rules and constructed in accordance with normal good practice
	$n \leqslant 4$	Single occupancy houses	
2: Lower Risk Group	$n = 1$	Educational	Acceptable if designed and constructed to normal rules, plus provision of effective horizontal ties (see Appendix A) or effective anchorage of timber floors to walls (as described in Figures 12.1 to 12.3)
	$n \leqslant 2$	Public buildings where the floor area at each storey does not exceed $2000m^2$	
	$n \leqslant 3$	Industrial buildings, and retail premises where the floor area at each storey does not exceed $1000m^2$	
	$n \leqslant 4$	Flats and other residential buildings, hotels and offices	
	$n \leqslant 5$	Single occupancy houses	
2: Upper Risk Group[b]	$1 \leqslant n \leqslant 3$	Hospitals	Acceptable if designed and constructed to normal rules with effective horizontal and vertical ties (see Appendix A) *or* verification that upon the notional removal of each supporting column and each beam supporting one or more columns or any nominal length[c] of load bearing wall (one at a time in each storey of the building) that the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or $70m^2$ whichever is the smaller and does not extend further than the immediate adjacent storeys
	$2 \leqslant n \leqslant 15$	Educational	
	$n \leqslant 2$	Public buildings where the floor area at each storey exceeds $2000m^2$	
	$n \geqslant 3$	Public buildings where the floor area at each level does not exceed $5000m^2$	
	$4 \leqslant n \leqslant 15$	Retail premises not assigned to class 2: lower risk group	
	$5 \leqslant n \leqslant 15$	Flats and other residential buildings, hotels and offices	

Table 5.5 Continued

Building occupancy class	Number of storeys (n)	Use	Method of achieving compliance
3		Class 2 buildings which exceed the specified limits on area or number of storeys	These buildings require specific analysis of the risk and the measures necessary to mitigate it. Buildings in this class are unlikely to be of timber construction, but buildings designed to store hazardous substances might be assigned to this class, and these could be made of timber
		All buildings to which members of the public are admitted in significant numbers	
		Stadia accommodating more than 5000 spectators	

Notes

a Check conformity with applicable Building Regulation requirements.

b Buildings of cellular construction, such as platform frame, are best designed by the notional removal of columns, beams and load-bearing walls, allowing for the catenary action of floor diaphragms and either arching action or rim beam support over removed vertical areas of structure. With relatively small dimensions such as 7.2m spanned by two single 3.6m floor joists, the floor decking, provided that it is continuous, can act as a catenary support on removal of the central support (see Section 8.6). For longer spans it may be necessary to have continuous joists to provide this action or to splice joists together where they meet.

c The ‘nominal length’ of a load-bearing wall should be taken as the distance between lateral supports for external walls and 2.25 x (storey height) for internal walls. Storey height is defined as the vertical distance between the structural floor and the structural ceiling, i.e. the distance between lateral restraints.

6.1 Introduction

Section 6 covers the design of dowel type connections and glued joints. The Task Group took the view that the EC5 design rules for timber connectors were unsafe, so a method based on BS 5268-2[3] for obtaining safe design resistances for connections made with these devices is given instead. The detailed design of punched metal plate fasteners is normally regarded as too specialist for the average designer, but some general advice is given for their use.

While the load capacity of most types of connection can be calculated, more reliable and often higher values can be obtained by testing sample connections in accordance with the appropriate testing standard (BS EN 383[83], BS EN 408[84], BS EN 1075[85], BS EN 1380[86], BS EN 26891[87]. This is particularly advisable in the case of principal connections in large public buildings or mass-produced connections where small economies are multiplied. It is essential in the case of glued finger joints in principal members or in other members acting alone where failure could lead to collapse.

The subject of slip is covered in Section 6.13.

6.2 Dowel type connections

6.2.1 Introduction

This Section covers structural timber dowel type connections made with nails, wood screws, bolts or round steel dowels. With some limitations these fasteners can generally resist either lateral loading or axial loading (withdrawal and pull-through) or both. For staples see EC5 Section 8.4. For materials other than steel use the formulae given in EC5 with appropriate values for the yield moment in bending.

6.2.2 Specification

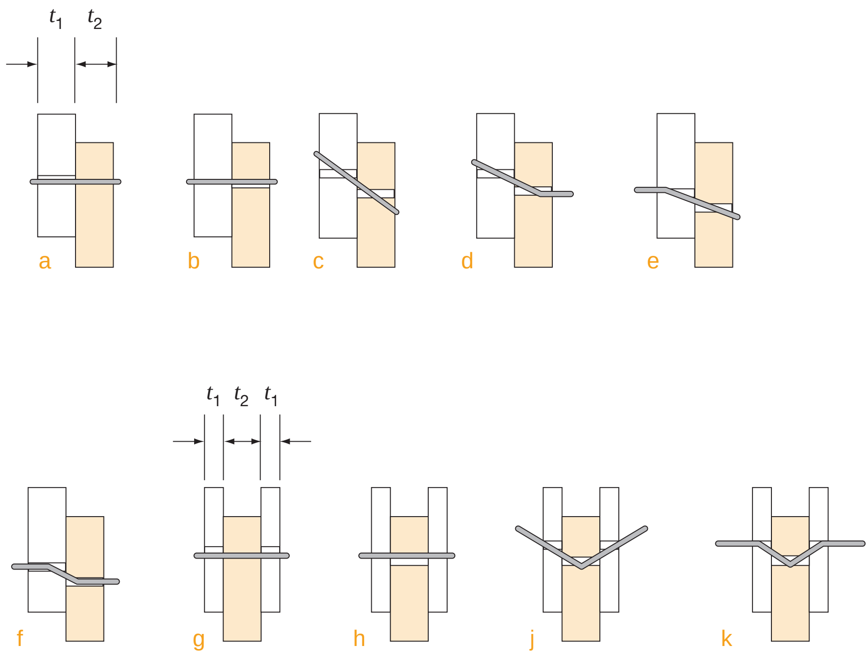
All dowel type fasteners should conform to BS EN 14592[46]. For material specifications see Table 3.29. For corrosion protection specifications, see Table 3.25. For requirements for structural detailing and workmanship see Section 12.3.

6.2.3 Lateral load capacity of a timber connection

6.2.3.1 Failure modes

Laterally loaded dowel-type connections between members made of timber or wood-based panel products can fail in a number of ways, as shown in Figure 6.1. The corresponding failure modes for steel-timber connections are shown in EC5 Figure 8.3. The principal modes of failure are embedment failure in one of the timber members, or yield in bending in the fastener. Expressions given in EC5 8.2 enable the failure load for each of the failure modes shown to be calculated via further expressions for the embedment strength of the timber members and the bending strength of the fasteners. The expressions relating to failure in the fastener include an additional contribution towards the load capacity which depends on the resistance of the fastener to axial withdrawal, known as the ‘rope’ effect (EC5 8.3.2, 8.5.2, 8.7.2). There are rules which limit this contribution, depending on the type of fastener (EC5 8.2.2 (2)). The lowest of the resulting failure loads gives the load capacity of one fastener per shear plane, $F_{v,Rk}$.

Other possible modes of failure are splitting, shear failure in the timber, block shear and plug shear failure, which must all be considered too (see Sections 6.2.4.3 to 6.2.4.6).



Key

a to f	2-member connections
g to k	3-member connections
a, g	embedment failure in member 1
b, h	embedment failure in member 2
c	embedment failure in members 1 and 2
d, j	fastener bending failure in member 2
e	fastener bending failure in member 1
f, k	fastener bending failure in members 1 and 2

Fig 6.1 Principal failure modes for laterally-loaded dowel type connections between timber or wood-based panels

6.2.3.2 Calculation options

The expressions for calculating the load capacities of dowel-type connections are complicated and time-consuming and have therefore been omitted from this *Manual*. Engineers are recommended to use the tables provided below to obtain characteristic load capacities for some common connection configurations, or to obtain design values for laterally loaded fasteners from the tables provided in the accompanying CD, which also cover more materials. Another option is to use the connections software published by TRADA[8].

The tabulated values assume that the fasteners are made of steel with the following ultimate tensile strengths:

Nails	600N/mm^2
Wood screws $d \leqslant$ 6mm	540N/mm^2
Wood screws d > 6mm and coach screws	400N/mm^2
Bolts and steel dowels	400N/mm^2.

For bolts and dowels and for screws with a diameter exceeding 6mm the fastener capacity depends on the angle of load to the grain, so the tables give two sets of values for loads at 0° and 90° to the grain respectively. For intermediate angles the values for 90° may be used conservatively. If the CD is used then more accurate values for intermediate angles may be obtained.

6.2.3.3 Definitions of spacings and distances

The minimum specified distances between fastener centres, and the minimum distances between fastener centres and the edges and ends of timber members must be maintained to avoid the possibility of splitting. These requirements may determine the minimum size of a timber member, e.g. a single row of 12mm bolts with a minimum edge distance of 3d will require a minimum member width of 72mm.

The symbols a_1, a_2, a_3 and a_4 are defined in Figure 6.2.

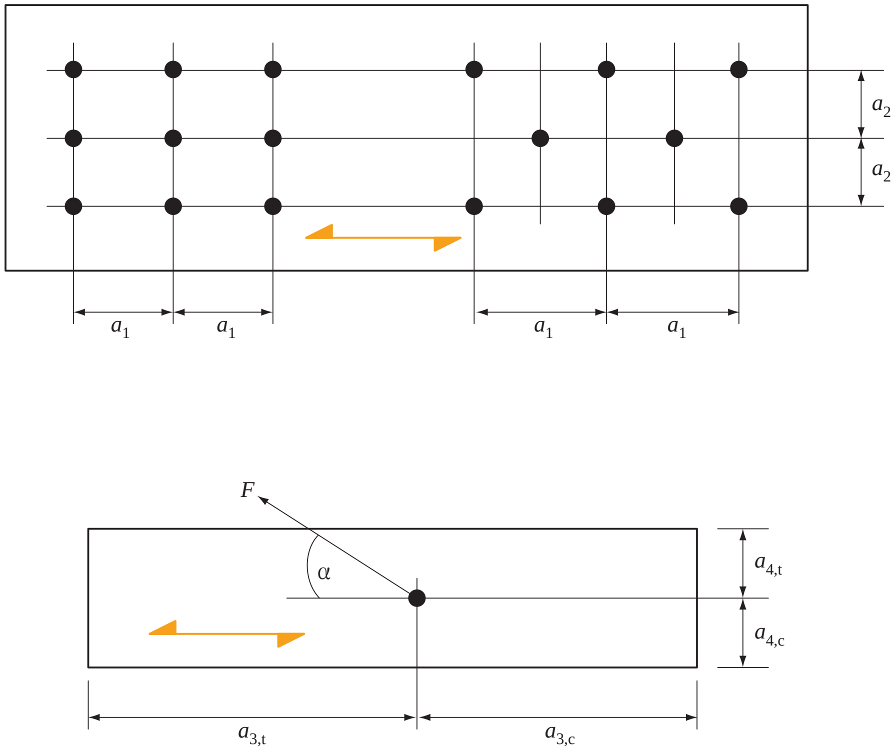
6.2.3.4 Effective number of fasteners (EC5 8.1.2(4) and (5))

The lateral load-carrying capacity of a row of fasteners parallel to the grain depends on their spacing and the number of fasteners, as well as on the capacity of each individual fastener. Allowance for these effects is made via a factor, n_{ef}, the effective number of fasteners per row parallel to the grain. To assist designers, the *Manual* provides tables of pre-calculated values of n_{ef}.

It is recommended that every dowel-type connection should comprise at least two fasteners.

6.2.3.5 Multiple shear planes

The expressions in EC5 give the characteristic load-carrying capacity, $F_{v,Rk}$, per fastener per shear plane. A shear plane is a plane between two connected members which are loaded in different directions. In Figure 6.1 g) to k) the fasteners are loaded in two shear planes, so any calculated values have to be multiplied by 2. There are different expressions for 2- and 3-member connections.



Key

a_1 Spacing parallel to the grain.

a_2 Spacing perpendicular to the grain.

$a_{3,t}$ Loaded end distance (the fastener applies a component of load towards the end).

$a_{3,c}$ Unloaded end distance (no component of load towards the end).

$a_{4,t}$ Loaded edge distance (the fastener applies a component of load towards the edge).

$a_{4,c}$ Unloaded end distance (no component of load towards the edge).

α Angle load to grain ($0° \leqslant \alpha \leqslant 90°$ in this *Manual*).

Fig 6.2 Definition of spacings and distances between fasteners

When a fastener passes through more than 2 shear planes the resistance of each shear plane should be calculated on the assumption that the connection consists of a series of 3-member connections, calculating the resistance per shear plane for each 3-member connection in turn. If the calculated resistances per shear plane differ, either use the lowest value, or adjust the geometry of the connection so that they are constant, or else refer to EC5 8.1.3. The total load-carrying capacity of the fastener is the lowest resistance per shear plane multiplied by the number of shear planes through which it passes.

6.2.4 Design values

6.2.4.1 Connection design

The following procedure covers all the possible modes of failure of a laterally loaded connection involving timber or a wood-based panel product.

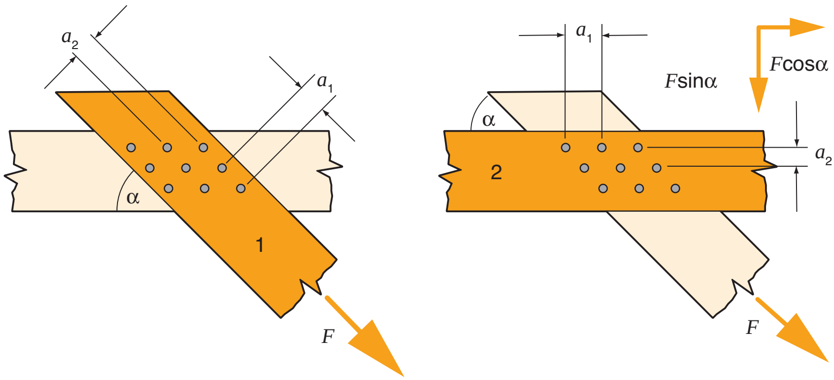
i) Select a suitable fastener diameter.

ii) Estimate the required number of fasteners using the appropriate table of characteristic load-carrying capacities given in Sections 6.3 to 6.6 and the expression in Section 6.2.4.2.

iii) Arrange the fasteners in a grid so that the spacings parallel and perpendicular to the grain in both members (see Figure 6.3) are at least equal to the minimum values tabulated in Sections 6.3 to 6.6. Ensure also that the specified minimum end and edge distances are maintained.

iv) Carry out whichever of the following verifications are relevant:
- total load-carrying capacity (Section 6.2.4.2)
- resistance to splitting parallel to the grain in both members (Section 6.2.4.3)
- resistance to splitting perpendicular to the grain in both members (Section 6.2.4.4)
- resistance of the members to shear failure (Section 6.2.4.5)
- resistance of the timber members to block shear and plug shear failure (Section 6.2.4.6)
- residual strength of timber members loaded in tension where part of the cross-section has been removed to accommodate the fasteners (Section 6.2.4.7)
- tensile, compressive, bending and bearing strength of any steel plates.

Consider the possibility of stress concentrations in thin steel plates bent at an angle.



Key

F	Load applied to Timber 1 parallel to grain of Timber 1
$F\cos\alpha$, $F\sin\alpha$	Components of load parallel and perpendicular to the grain of Timber 2
a_1	Spacing parallel to grain in highlighted member
a_2	Spacing perpendicular to grain in highlighted member

Fig 6.3 Fastener spacings and components of load parallel to the grain

6.2.4.2 Total load capacity

Verify that the total load on a connection does not exceed the capacity of the fasteners to carry load in the direction of the load, i.e. that:

$$F_d \leqslant \frac{n n_{\text{shearplanes}} F_{v,\alpha,Rk} k_{\text{mod}}}{\gamma_M}$$

Where F_d = design value of total load applied to connection

n = total number of fasteners

$n_{\text{shearplanes}}$ = number of shear planes per fastener[a]

$F_{v,\alpha,Rk}$ = characteristic lateral load-carrying capacity per fastener per shear plane at the applied angle of load to the grain[b], α

k_{mod} = service class and duration modification factor[c], see Table 2.3

γ_M = 1.3 – applies to the connection as a whole, see Table 3.19

Notes

a The tables of values for $F_{v,0,Rk}$ and $F_{v,90,Rk}$ for 3-member connections include $n_{\text{shearplanes}}$.

b When using tabulated values for angles of load between 0° and 90° to the grain, use the values for 90°. For more accurate values refer to EC5 or use dedicated software[8].

c When using the EC5 formulae for connections between two materials 1 and 2 with different values of k_{mod}, a value of $k_{\text{mod}} = \sqrt{k_{\text{mod},1} k_{\text{mod},2}}$ should be used.

6.2.4.3 Splitting parallel to the grain

Verify that the parallel-to-the-grain component of the load does not exceed the capacity of the connection to carry load parallel to the grain, i.e. that:

$$F_d \cos\alpha \leqslant \frac{n_{90}\, n_{ef}\, n_{shearplanes}\, F_{v,0,Rk}\, k_{mod}}{\gamma_M}$$

Where α = angle of load to the grain

n_{90} = number of lines of fasteners perpendicular to the grain

n_{ef} = effective number of fasteners in a row parallel to the grain (see Section 6.2.3.4)

$F_{v,0,Rk}$ = value of $F_{v,\alpha,Rk}$ for load at angle of 0° (parallel) to the grain

Other symbols are defined as in Section 6.2.4.2.

All dimensions given in the fastener load tables are the minimum permissible dimensions for the tabulated design loads.

6.2.4.4 Splitting failure perpendicular to the grain

Where a group of fasteners applies a component of force perpendicular to the grain of a timber member, its resistance to splitting should be verified by ensuring that:

$$F_{v,Ed} \leqslant F_{90,Rd} \qquad \text{(EC5 8.1.4(2))}$$

Where $F_{v,Ed}$ = maximum shear force produced in timber member (see Figure 6.4)

$$= \max. \begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases}$$

$F_{90,Rd}$ = design splitting capacity

$$= \frac{14bk_{mod}}{\gamma_M}\sqrt{\frac{h_e}{\left(1 - \frac{h_e}{h}\right)}} \text{ (N) for softwoods (EC5 8.1.4(3))}$$

b = total thickness of timber loaded in shear (mm)

h_e = distance between loaded edge and centre of most distant fastener (mm)

h = depth of timber member(s) loaded in shear (mm)

For hardwoods $F_{90,Rd}$ must be determined by tests; alternatively a conservative enhancement of 20% may be applied to the value of $F_{90,Rd}$ for softwoods[88].

Higher values of $F_{90,Rd}$ may be used for punched metal plate fasteners with a dimension parallel to the grain of more than 100mm (see EC5 8.1.4(3)).

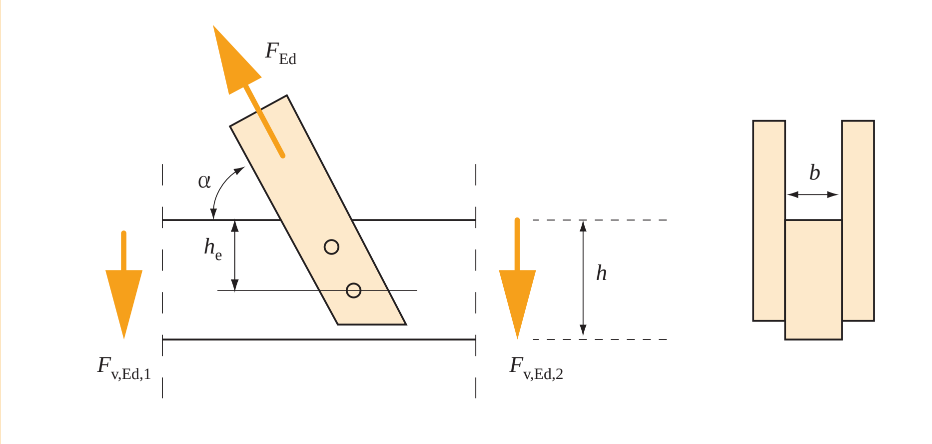


Fig 6.4 Splitting force produced by a connection

6.2.4.5 Shear failure

Although not in the current edition of EC5, the shear capacity of timber loaded by a group of fasteners perpendicular to the grain can be much less than its splitting capacity.

Verify that: $F_{v,Ed} \leqslant F_{90,Rd}$

Where $F_{90,Rd} = \dfrac{2bh_e f_{v,k} k_{mod}}{3\gamma_M}$

h_e = distance between the loaded edge and the far edge of the most distant fastener (mm)

$f_{v,k}$ = characteristic shear strength of timber (N/mm^2)

Other symbols are defined as in the previous section.

6.2.4.6 Block shear and plug shear failure

Where a group of fasteners in a steel plate applies a component of force parallel to the grain of a timber member near the end of the member and towards the end, the possibility of block shear or plug shear failure should be considered. EC5 Annex A gives a design method for this, but the NA states that it should be used only for steel-to-timber connections with dowel-type fasteners when:

$d \leqslant$ 6mm and $n \geqslant$ 10, or

$d >$ 6mm and $n \geqslant$ 5

Where d = fastener diameter in mm

n = number of fasteners in a line parallel to the grain

The implication is that in other cases this mode of failure is unlikely to occur.

6.2.4.7 Effective residual cross-section

The insertion of mechanical fasteners into a member generally reduces its cross-section. This reduction should be taken into account when checking the strength of a member.

For dowel-type fasteners EC5 5.2 gives the following rules:

- reductions in cross-section may be ignored in the case of nails and screws with a diameter of 6mm or less, driven without pre-drilling, and in a compression area if the holes are filled with a material which is stiffer than the member itself
- when assessing the effective residual cross-section of a connection made with multiple fasteners, all holes within a distance from the cross-section of half the minimum permitted fastener spacing measured parallel to the grain should be considered as occurring at that cross-section.

For timber connectors, see Section 6.11.2.3.

A_{res} may then be calculated as:

$$A_{res} = A_{gross} - b \sum_{1}^{n} d_i$$

Where A_{gross} = gross area of cross-section (without holes)

b = breadth of timber member or length of predrilled holes if less

d_i = diameter of each of the n nails, screws, or predrilled holes which are considered as occurring at the cross-section in question

6.3 Nailed connections

6.3.1 Introduction

Nails in structures designed to EC5 can be 'smooth' (which in practice means smooth round nails), 'square' (usually square twisted nails which are frequently used for joist hangers and other light gauge metal-work) or 'threaded' (normally termed 'ringed shank' and favoured for flooring). Smooth nails have less withdrawal resistance than the other 'improved nails', and their calculated resistance to lateral loading may also be less. Where nails have to resist axial loading it is recommended that improved nails be specified. Unfortunately EC5 gives no data from which improved nails can benefit: the axial withdrawal resistance of particular products must be determined by test. It is therefore recommended that in the absence of test data the conservative enhancement factors which were recommended in BS 5268-2[3] be applied to the calculated withdrawal resistance (see Table 6.8).

Table 6.1 gives the most commonly available sizes of galvanized or sheradized nails. Stainless steel nails can also be obtained in a more limited range of sizes.

Smooth nails should not be used in end grain except to resist lateral loads from secondary structures such as fascia boards, nor to resist permanent or long-term axial loading when driven into the side grain. Improved nails should not be used in end grain except to resist lateral loads from secondary structures, or lateral loads from principal structures in service class 1 or 2. For more detailed rules on the use of nails in end grain, see EC5 8.3.1.2(4).

Table 6.1 Commonly available corrosion resistant nail sizes

Hand driven nails, smooth	Hand driven nails, threaded	Machine driven nails (usually but not always threaded)
Diameter x length (mm)	Diameter x length (mm)	Diameter x length (mm)
2.65 x 50	2.80 x 63	2.8 x 51
2.65 x 65	3.35 x 60	2.8 x 57
3.35 x 65	3.35 x 65	2.8 x 63
3.75 x 75	3.76 x 75	3.1 x 50
4.00 x 90		3.1 x 63
4.00 x 100		3.1 x 75
4.00 x 125		3.1 x 90

For square nails the nail diameter *d* should be taken as the side dimension. For round and square nails *d* should be in the range 1.9mm to 8.0mm inclusive (see BS EN 14592[47]). Structural timber composites may have additional limitations on nail diameter and spacing rules: consult the manufacturer's certification literature.

Softwoods need not be predrilled for nails up to 8mm in diameter, but nails in predrilled holes have greater resistance to lateral loads and can be spaced more closely. The values given in this *Manual* assume that nail holes will not be predrilled in softwoods. If values for predrilled holes in softwoods are required see EC5 8.3 or TRADA's connections software[8]. Nail holes must be predrilled if the thickness of a member is less than $7d$ or less than $\rho_k(113d\text{-}30)/400$. For hardwoods EC5 provides spacing rules only for nails in predrilled holes, so in effect hardwoods must be predrilled. For instructions on drilling holes for nails, see Section 12.3.2.

Minimum permitted pointside penetrations in terms of the diameter, *d*, are:

- smooth nails $8d$
- other nails in side grain or in secondary structures in end grain $6d$
- other nails in principal structures in end grain $10d$

For threaded nails pointside penetrations are measured as the threaded length in the pointside member.

There should be at least two nails in a nailed connection, or three in the case of non-smooth nails in end-grain resisting lateral loading from principal structures.

Simplified rules for minimum spacings and distances are shown in Tables 6.2 and 6.3. For more precise rules including nails with predrilled holes see EC5 Table 8.2.

Structural timber composites may have additional limitations on nail diameter and spacing rules, for which the manufacturer's certification literature should be consulted.

For loaded end distances for skew nailing, see EC5 8.3.2(10).

For nails which resist both axial and lateral load, see EC5 8.3.3.

Table 6.2 Minimum spacings and distances for nails and small screws without predrilled holes connecting solid timber or glulam (based on EC5 Table 8.2)

a	d (mm)	C14 – C24, GL24, GL28, GL32c			C 27 – C35, GL32h, GL36		
a_1		$0° \leqslant \alpha < 60°$	$60° \leqslant \alpha < 90°$	90°	$0° \leqslant \alpha < 60°$	$60° < \alpha \leqslant 90°$	90°
	$d < 5$	$10d$	$7.5d$	$5d$	$15d$	$11d$	$7d$
	$5 \leqslant d \leqslant 8$	$12d$	$8.5d$	$5d$			
a_2		$5d$			$7d$		
$a_{3,t}$		$0° \leqslant \alpha < 60°$	$60° \leqslant \alpha < 90°$	90°	$0° \leqslant \alpha < 60°$	$60° \leqslant \alpha < 90°$	90°
		$15d$	$12.5d$	$10d$	$20d$	$17.5d$	$15d$
$a_{3,c}$		$10d$			$15d$		
$a_{4,t}$		0°	$0° \leqslant \alpha < 30°$	$30° \leqslant \alpha \leqslant 90°$	0°	$0° \leqslant \alpha < 30°$	$30° \leqslant \alpha \leqslant 90°$
	$d < 5$	$5d$	$6d$	$7d$	$7d$	$8d$	$9d$
	$5 \leqslant d \leqslant 8$	$5d$	$7.5d$	$10d$	$7d$	$9.5d$	$12d$
$a_{4,c}$		$5d$			$7d$		

Notes

a a = spacing or distance (see Figure 6.2 for definitions).

b d = diameter of nail or side length of square nail.

c α = angle of fastener load to grain (see Figure 6.2).

Table 6.3 Minimum spacings and distances for nails and small screws without predrilled holes connecting plywood or OSB to timber[a] (based on EC5 8.3.1.3)

Dimension	**Minimum spacing or distance**		
Spacings between nails	0.85 x (value given in Table 6.1)		
	Type[b]	α[c]	Distance[d]
Distance from loaded edge of panel	Plywood	$0° < \alpha \leqslant 30°$ $30° < \alpha \leqslant 90°$	$5d$ $7d$
	OSB	$0° < \alpha \leqslant 30°$ $30° < \alpha \leqslant 90°$	$7d$ $9d$
Distance from unloaded edge of panel	Plywood OSB		$3d$ $5d$

Notes

a For use as sheathing or decking in walls, roofs and floors a minimum edge distance of 8mm in plywood and OSB has been found to be sufficient and is specified in DD ENV 12872[89].

b Values for OSB are conservatively based on the values for plywood. Strictly EC5 requires the still larger values for solid timber to be used, but these relate to the grain direction which is not clearly defined in OSB, and it is known that nailed OSB and plywood panels with short edge distances have similar racking strengths.

c α = angle of load to edge of panel (0° is parallel to edge).

d d = diameter of nail or side length of square nail.

6.3.2 Laterally loaded nails

6.3.2.1 Effective number of nails, n_{ef}

For pre-calculated values of n_{ef} see Table 6.4. To calculate n_{ef} for predrilled holes or hardwoods, see EC5 8.3.1.1(8). For the use of n_{ef}, see Sections 6.2.3.4 and 6.2.4.

Table 6.4 Values of n_{ef} for connections made with nails or small screws without predrilled holes in softwoods up to strength class C40 (EC5 8.3.1.1(8))

Angle of load to grain[a]		Spacing parallel to grain[a]	Number of nails or small screws in a row parallel to grain for spacing shown[b]								
			2	3	4	6	8	10	12	15	20
d < 5mm	d ⩾ 5mm		Value of n_{ef}								
90°	90°	5*d*	1.52	1.93	2.30	2.93	3.48	3.98	4.44	5.08	6.03
75°	75°	7*d*	1.62	2.16	2.64	3.51	4.29	5.01	5.69	6.66	8.14
60°	75°	8*d*	1.68	2.28	2.83	3.83	4.76	5.62	6.45	7.62	9.46
45°	60°	9*d*	1.74	2.41	3.03	4.19	5.28	6.31	7.30	8.73	11.0
0°	45°	10*d*	1.80	2.54	3.25	4.59	5.86	7.08	8.27	9.99	12.8
0°	45°	11*d*	1.85	2.65	3.42	4.90	6.33	7.72	9.07	11.1	14.3
0°	0°	12*d*	1.90	2.76	3.61	5.25	6.84	8.41	9.96	12.2	16.0
0°	0°	14*d*	2.00	3.00	4.00	6.00	8.00	10.0	12.0	15.0	20.0

Notes

a *d* = diameter of round nail or screw, or side length of square nail.

b For a 'row' parallel to the grain containing only one fastener, $n_{ef} = 1$.

6.3.2.2 Load capacities

Tables 6.5 to 6.7 give characteristic values of the lateral load capacity of nailed connections made with steel nails in some common timber materials.

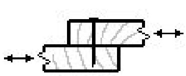


Table 6.5 Characteristic lateral load capacity per fastener in smooth nailed 2-member timber-timber connections, not predrilled. To obtain the design value for the connection multiply by $n_{ef}\,k_{mod}/\gamma_M$

Nail diameter (mm)	Minimum nail length (mm)	Minimum member thickness[a] (mm)	Load capacity (kN)	
			C16	C24, GL24h
2.65	64	32	0.590	0.636
2.80[b]	68	34	0.702	0.758
3.00	72	36	0.726	0.782
3.10[b]	76	38	0.833	0.899
3.35	80	40	0.872	0.940
3.75	90	45	1.060	1.140
4.00	96	48	1.180	1.270
6.00	144	72	2.320	2.510
8.00	192	96	3.770	4.080

Notes

a The minimum thickness of 12*d* applies to both headside and pointside members. It may be reduced to 8*d* with a reduction in the load capacity (see the CD).

b Values for ringed shank machine driven nails with $f_{u,k} = 700\text{N/mm}^2$.

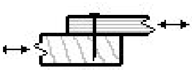


Table 6.6 Characteristic lateral load capacity per fastener in smooth nailed 2-member plywood-timber connections, not predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod}/\gamma_M$

Plywood thickness (mm)	Nail dimensions		Load capacity (kN)	
	Diameter (mm)	Length (mm)	C16	C24, GL24h
6	2.65	40	0.48	0.50
	2.80[b]	40	0.57	0.60
	3.00	50	0.59	0.62
	3.10[b]	50	0.68	0.71
	3.35	50	0.69	0.74
	3.75	75	0.86	0.88
	4.00	75	0.92	0.92
9	2.65	40	0.51	0.54
	2.80[b]	40	0.58	0.62
	3.00	50	0.62	0.65
	3.10[b]	50	0.71	0.74
	3.35	50	0.72	0.76
	3.75	75	0.88	0.92
	4.00	75	0.97	1.01
12	2.65	50	0.58	0.61
	2.80[b]	50	0.67	0.71
	3.00	50	0.67	0.71
	3.10[b]	50	0.76	0.80
	3.35	50	0.76	0.80
	3.75	75	0.94	0.98
	4.00	75	1.02	1.07
15	2.65	50	0.65	0.68
	2.80[b]	50	0.74	0.78
	3.00	65	0.77	0.81
	3.10[b]	65	0.86	0.89
	3.35	65	0.87	0.92
	3.75	75	1.01	1.07
	4.00	75	1.09	1.16
18	2.65	50	0.68	0.72
	2.80[b]	50	0.80	0.84
	3.00	65	0.85	0.89
	3.10[b]	65	0.93	0.98
	3.35	65	0.95	1.00
	3.75	75	1.09	1.15
	4.00	75	1.17	1.24
21	2.65	50	0.67	0.70
	2.80[b]	50	0.75	0.82
	3.00	65	0.85	0.90
	3.10[b]	65	1.00	1.06
	3.35	65	1.02	1.08
	3.75	75	1.18	1.24
	4.00	75	1.26	1.33
29	2.65	50	0.53	0.58
	2.80[b]	50	0.59	0.64
	3.00	65	0.84	0.89
	3.10[b]	65	0.97	1.03
	3.35	65	0.99	1.04
	3.75	75	1.22	1.29
	4.00	75	1.35	1.42

Notes

a Applicable to plywood with a characteristic density of at least 505kg/m³, (525kg/m³ for 6mm thick plywood), i.e. all plywoods listed in Table 3.28. Higher values can be obtained for Finnish birch plywoods using the table for nailed plywood-to-timber connections in the CD. It is assumed that the plywood is the headside member.

b Values for ringed shank machine driven nails with $f_{u,k}$ = 700N/mm².

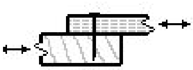


Table 6.7 Characteristic lateral load capacity per fastener in smooth nailed 2-member OSB-timber connections, not predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\,k_{mod}/\gamma_M$

OSB thickness (mm)	Nail dimensions		Load capacity (kN)	
	Diameter (mm)	Length (mm)	C16	C24, GL24h
9	2.65	40	0.51	0.53
	2.80[b]	40	0.59	0.63
	3.00	50	0.61	0.64
	3.10[b]	50	0.70	0.74
	3.35	50	0.70	0.74
	3.75	75	0.85	0.88
	4.00	75	0.93	0.96
11	2.65	50	0.56	0.59
	2.80[b]	50	0.65	0.69
	3.00	50	0.64	0.68
	3.10[b]	50	0.73	0.77
	3.35	50	0.73	0.76
	3.75	75	0.88	0.92
	4.00	75	0.96	1.00
15	2.65	50	0.67	0.70
	2.80[b]	50	0.74	0.78
	3.00	65	0.76	0.81
	3.10[b]	65	0.85	0.90
	3.35	65	0.85	0.89
	3.75	75	0.97	1.02
	4.00	75	1.04	1.10
18	2.65	50	0.69	0.73
	2.80[b]	50	0.83	0.87
	3.00	65	0.85	0.89
	3.10[b]	65	0.93	0.99
	3.35	65	0.93	0.98
	3.75	75	1.05	1.11
	4.00	75	1.12	1.18
23	2.65	50	0.65	0.70
	2.80[b]	50	0.74	0.81
	3.00	65	0.85	0.90
	3.10[b]	65	1.00	1.06
	3.35	65	1.01	1.07
	3.75	75	1.21	1.27
	4.00	75	1.27	1.34

Notes

a It is assumed that the OSB is the headside member.

b Values for ringed shank machine driven nails with $f_{u,k}$ = 700N/mm².

6.3.3 Axially loaded nails

6.3.3.1 Withdrawal resistance

Nails inserted in end grain should not be used to resist axial loading, and smooth nails inserted in side grain should not be used to resist permanent or long-term axial loading. (This means that the 'rope' effect cannot be used when calculating the lateral load resistance of smooth nails subject to shorter-term loads.)

Table 6.8 gives characteristic values for the withdrawal resistance, R_k, per nail per mm of pointside penetration, for smooth nails driven perpendicular to the grain.

EC5 gives no expression to calculate the withdrawal resistance of improved nails, but BS 5268-2 stated that for threaded nails the smooth nail withdrawal resistance may be multiplied by 1.5, provided that a pointside penetration t_{pen} equal to the threaded length in the pointside member is used. For square twisted nails the withdrawal resistance may be calculated as 1.25 times the resistance of a smooth round nail with a diameter equal to the side length of the square nail.

For smooth nails the pointside penetration must be at least 8d, and no additional resistance to axial loading should be assumed for pointside penetrations beyond 18d to avoid the possibility of head pull-through or nail failure.

For improved nails (threaded or twisted) the pointside penetration must be at least 6d, and no additional resistance to axial loading should be assumed for pointside penetrations beyond 9d to avoid the possibility of head pull-through.

Hence the design value of the withdrawal resistance of the connection, R_d, is:

$$R_d = \frac{t_{profile}\, t_{pen}\, n k_{mod} R_k}{\gamma_M}$$

Where	$t_{profile}$	=	1.0 for smooth round nails
		=	1.25 for square twisted nails with a side length of d
		=	1.5 for round threaded nails
	t_{pen}	=	the pointside penetration length or, for threaded nails, the threaded length in the pointside member, in mm
	n	=	number of nails
	k_{mod}	=	service class and duration modification factor (see Table 2.3)
	γ_M	=	material modification factor for connections = 1.3
	R_k	=	characteristic withdrawal resistance per fastener from Table 6.8

For smooth nails with a pointside penetration t_{pen} of $8_d \leqslant t_{pen} < 12d$ R_d should be multiplied by $\frac{t_{pen}}{4d} - 2$.

For improved nails with a pointside penetration t_{pen} of $6d \leqslant t_{pen} < 8d$ R_d should be multiplied by $\frac{t_{pen}}{2d} - 3$.

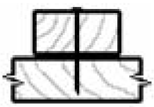


Table 6.8 Characteristic withdrawal resistance of a smooth[a] nail in N per mm of pointside penetration into side grain with a minimum pointside penetration of 12d. To obtain the design value for the connection see formula for R_d

Nail diameter, d (mm)	Withdrawal resistance[b] (N/mm)	
	C16	C24, GL24
2.65	5.09	6.49
2.80[c]	5.38	6.86
3.00	5.77	7.35
3.10[c]	5.96	7.60
3.35	6.44	8.21
3.75	7.21	9.19
4.00	7.69	9.80

Notes

a Smooth nails should not be used to resist permanent or long-term axial loading. The tabulated values may also be used for threaded and square twisted nails in conjunction with the factor $t_{profile}$ given in the text.

b For connections made in timber which is at or near its saturation point (typically green oak, etc.) and which is likely to dry out in service class 1 or 2, the tabulated values should be multiplied by 2/3.

c Machine driven nail size, assumed smooth shank.

6.3.3.2 Head pull-through

For nails with head diameters of 2d or more that observe the limitations in Section 6.3.3.1 it may be assumed that the head pull-through resistance exceeds the withdrawal resistance.

6.4 Screwed connections

6.4.1 Introduction

The design rules for wood screws depend on the screw diameter. In general, for calculating their load-bearing capacity and spacing rules, treat screws with a diameter not exceeding 6mm like nails, and larger screws like bolts. For the calculation of bending strength, however, a reduced diameter must be used to allow for the screw thread, whereas an enhanced rope effect can be applied due to their superior resistance to axial withdrawal.

Tables 6.9 and 6.10 give the most commonly available size ranges of wood screws and coach screws. It may be possible to obtain other sizes to order. Stainless steel coach screws can also be obtained in a more limited range of sizes.

Softwoods need not be predrilled for wood screws of 6mm diameter or less, but screws in predrilled holes have greater resistance to lateral loads and can be spaced more closely. The values given in this *Manual* assume that screw holes will not be predrilled in softwoods. If values for predrilled holes are required see EC5 8.7.1 or TRADA's connections software[8]. For hardwoods EC5 provides spacing rules only for screws in predrilled holes, so in effect hardwoods must be predrilled. For instructions on drilling holes for screws, see Section 12.3.3.

Table 6.9 Common sizes of wood screw		
Screw gauge or number	Length (mm)	
	Minimum	Maximum
3	12.7	15.9
4	6.4	25.4
5	12.7	25.4
6	12.7	50.8
7	12.7	50.8
8	12.7	88.9
10	19.1	101.0
12	19.1	152.0

Table 6.10 Common sizes of coach screw					
Square head			Hexagonal head		
Diameter (inches)	Length (mm)		Diameter (mm)	Length (mm)	
	Minimum	Maximum		Minimum	Maximum
1/4	38	127	6	25	25 - 120
5/16	38	152	8	30 - 150	30 - 150
3/8	38	152	10	30 - 200	30 - 200
1/2	63	152	12	50 - 180	50 - 180
5/8	89	254	–	–	–
3/4	152	152	–	–	–

The outer thread diameter of a wood screw should be in the range 2.4mm to 24.0mm inclusive (see BS EN 14592[46]).

The bending strength of a wood screw depends on the strength of the steel and the root diameter of the thread, d_{root}. The tables assume an ultimate tensile strength for the steel of 540N/mm^2, which was the strength of small traditional wood screws manufactured to the now obsolete *BS 1210 – Specification for wood screws*. Some modern wood screws may have a higher strength. Figure 6.5 shows the profiles of two types of wood screw, a traditional wood screw to BS 1210[90] in which $d_{root} = 0.9d_{nom}$, and a modern wood screw in which $0.6d_{nom} \leqslant d_{root} \leqslant 0.7d_{nom}$ where d_{nom} is the nominal, i.e. outer thread diameter (see BS EN 14592[46]). Calculations of the lateral load capacity of a connection made with wood screws are based on an effective diameter, d_{ef}, calculated as $1.1d_{root}$ (EC5 8.7.1). Hence for a traditional wood screw $d_{ef} = d_{nom}$. For a modern wood screw $d_{ef} = 2d_{nom}/3$ will give a safe but sometimes conservative value.

Fig 6.5 Wood screw profiles

Tables 6.11 to 6.13 and the tables for laterally loaded screwed connections in the CD assume a value of $d_{ef} = 2d_{nom}/3$. However if the actual root diameter is known then an effective diameter of $1.1d_{root}$ may be entered in the CD tables to obtain a more accurate value for the strength of the connection. Alternatively if it can be guaranteed that a traditional wood screw will be used then the nominal diameter may be entered directly in the CD tables.

Spacing and minimum penetration rules are expressed in terms of the nominal screw diameter.

In traditional wood screws the smooth shank extends for a third of the length from the head. Since the withdrawal resistance depends on the threaded length in the pointside member it is necessary, when the connection design depends on the withdrawal resistance, either to specify precisely the type of screw to be used or to assume that the first third will not be threaded. This is particularly relevant in steel-to-timber connections.

Coach screws can be obtained in diameters, *d*, from 6mm to 20mm. They have a square or hexagonal head for a spanner and are particularly useful for attaching steel plates to timber or for general timber connections where it would be difficult to tighten a nut onto a bolt. Hexagonal head coach screws may be specified to a German standard, DIN 571[91]. Square and hexagonal head 'lag screws' to US standard ANSI/ASME B18.2.1[92] are also suitable. For coach screws made in accordance with the above standards it may be assumed that $d_{ef} = d_{nom}$. Members should always be predrilled for coach screws (see Section 12.3.3). When tightened directly onto timber, coach screws should always be used in conjunction with a steel washer with a minimum external diameter or side length of $3d$ and a minimum thickness of $0.3d$ (EC5 10.4.3(2)).

Wood screws should not be used in end grain except to resist lateral loads from secondary structures, or lateral loads from principal structures in service class 1 or 2.

Minimum permitted pointside penetrations in terms of the nominal diameter, *d*, are:

in side grain or in secondary structures in end grain	$6d$
in principal structures in end grain	$10d$

There should be at least two screws in a screwed connection, or three in the case of screws in end-grain resisting lateral loading from principal structures.

For lateral loading, minimum spacings and distances are given for small screws ($d \leqslant$ 6mm) in Tables 6.2 and 6.3 and for large screws ($d >$ 6mm) in Table 6.17.

Structural timber composites may have special limitations on nail diameter and spacing rules which would also apply to screws with a diameter not exceeding 6mm inserted without predrilling. Consult the manufacturer's certification literature for guidance.

6.4.2 Laterally loaded screws

6.4.2.1 Effective number of screws, n_{ef}

For small screws ($d \leqslant$ 6mm), see Section 6.3.2.1. For large screws ($d >$ 6mm), see Section 6.5.2.1.

6.4.2.2 Load capacities

Tables 6.12 to 6.16 give characteristic values of the lateral loading capacity of screwed connections made with steel screws in some common timber materials. Tables 6.12 to 6.14 assume that the screws have an ultimate tensile strength of 540N/mm^2; in Tables 6.15 and 6.16 a value of 400N/mm^2 is assumed (for metric sizes the CD may be used to calculate capacity). The minimum screw length is based on a pointside penetration of 6d, where d is the nominal (outer thread) diameter. The pointside member must be thick enough to accommodate fully the minimum lengths of screw shown.

6.4.3 Axially loaded screws

6.4.3.1 General

The resistance of a screwed connection to axial load is the least of the withdrawal resistance of the screws, their resistance to head pull-through, their tensile strength and the tear-off capacity of the heads. The Code provides a calculation method only for the withdrawal resistance, leaving the other factors to be determined by test. However the calculation method given in the 2004 edition of EC5 is believed to be unsafe, and at the time of writing it has not been replaced. Therefore for the design of axially loaded screws by calculation some assumptions must be made.

6.4.3.2 Withdrawal

It may safely be assumed that the resistance to axial withdrawal of a wood screw of nominal diameter d_{nom}, per mm of threaded length in the pointside member, t_{pen}, is four times the resistance of a smooth nail of the same diameter, provided that $t_{pen} \geqslant 8d$, as required for full-strength threaded nail connections. This assumption is based on a comparison of the withdrawal resistance of nails and wood screws tabulated in BS 5268-2.

Hence the design withdrawal resistance for a wood screw may be calculated as:

$$R_{d,withdrawal} = \frac{4t_{pen} n k_{mod} R_k}{\gamma_M} \text{N/mm}$$

Where	t_{pen}	=	the pointside penetration length in mm
	n	=	number of screws
	k_{mod}	=	service class and duration modification factor (see Table 2.3)
	γ_M	=	material modification factor for connections = 1.3
	R_k	=	characteristic withdrawal resistance per fastener from Table 6.8

provided that $t_{pen} \geqslant 8d_{nom}$ where d_{nom} is the nominal diameter in mm, and $d_{nom} \leqslant$ the outer thread diameter.

6.4.3.3 Head pull-through

For head pull-through it is reasonable to use the EC5 formulae for nail head pull-through on the assumption that the head diameter = 2d.

Hence for screwed connections with a clearance hole drilled in the headside member EC5 8.3.2.(4) leads to a design head pull-through resistance of:

$$R_{\text{d,pull-through}} = \frac{70 \times 10^{-6} \rho_k^2 \, 4d_{\text{nom}}^2 \, nk_{\text{mod}}}{\gamma_M} \text{ N}$$

Where ρ_k = characteristic density of headside member (kg/m^3)
and other factors are as defined in the previous sub-section.

For screwed connections made without predrilling or with a hole in the headside member with a diameter less than $0.75d_{\text{nom}}$, $R_{\text{d,pull-through}}$ calculated as above will be conservative. If required the procedure given for smooth nails in EC5 8.3.2 may be followed with $f_{\text{ax,k}} = 80 \times 10^{-6} \rho_k^2$ N/mm^2, without any reduction for values of t_{pen} between $8d_{\text{nom}}$ and $12d_{\text{nom}}$.

6.4.3.4 Tensile strength and head tear-off

The tensile strength and head tear-off resistance are unlikely to govern the design of a timber connection made with a steel screw unless dense hardwoods are used. In such cases testing is recommended.

Table 6.11 shows minimum spacings and edge distances for axially loaded wood screws.

Table 6.11 Minimum spacings and edge distances for axially loaded wood screws

Screw orientation	**Minimum spacing**	**Minimum edge distance**
Perpendicular to the side grain	$4d$	$4d$
Parallel to the end grain	$4d$	$2.5d$

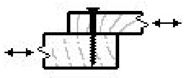


Table 6.12 Characteristic lateral load capacity per fastener in wood-screwed 2-member timber-timber connections, not predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod} / \gamma_M$

Minimum thickness of headside member[b] (mm)	Gauge (No.)	Nominal (outer) diameter d (mm)	Assumed effective diameter d_{ef} (mm)	Minimum screw length[c] (mm)	Timber: Strength class C16 – Characteristic load capacity (kN)	Timber: Strength class C24, GL24h – Characteristic load capacity (kN)
33	8	4.17	2.78	66	0.767	0.840
20(T)	10	4.88	3.25	59	0.869	0.984
39	10	4.88	3.25	78	1.000	1.100
22(T)	12	5.59	3.73	67	1.080	1.230
45	12	5.59	3.73	90	1.260	1.390
32	–	4.00	2.67	64	0.714	0.782
20(T)	–	5.00	3.33	60	0.897	1.020
40	–	5.00	3.33	80	1.050	1.150
24(T)	–	6.00	4.00	72	1.230	1.400
48	–	6.00	4.00	96	1.430	1.570

Notes

a Assumed $f_{u,k}$ = 540N/mm². Selected thicknesses are $4d$ or $8d$. For traditional screws (T) in which a third of the length from the head is unthreaded, the headside thickness should be approximately $4d$.

b For fully threaded screws the values shown for either $4d$ or $8d$ may be used.

c Selected lengths are $12d$ or $16d$. Pointside penetration must be at least $6d$ but preferably at least $8d$.

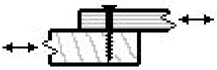


Table 6.13 Characteristic lateral load capacity per fastener in wood-screwed 2-member plywood-timber connections, not predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod} / \gamma_M$

Plywood Thickness (mm)	Screw				Timber	
	Gauge (No.)	Nominal (outer) diameter (mm)	Assumed effective diameter (mm)	Minimum screw length (mm)	Strength class C16, Load capacity (kN)	Strength class C24, GL24h, Load capacity (kN)
6	4	2.74	1.83	28	0.417	0.457
	6	3.45	2.30	34	0.590	0.651
	8	4.17	2.78	39	0.780	0.869
	10	4.88	3.25	45	1.030	1.050
	12	5.59	3.73	51	1.160	1.160
9	6	3.45	2.30	37	0.659	0.701
	8	4.17	2.78	42	0.845	0.923
	10	4.88	3.25	48	1.090	1.170
	12	5.59	3.73	54	1.350	1.460
12	6	3.45	2.30	40	0.743	0.793
	8	4.17	2.78	45	0.941	1.020
	10	4.88	3.25	51	1.180	1.270
	12	5.59	3.73	57	1.450	1.550
15	6	3.45	2.30	43	0.743	0.793
	8	4.17	2.78	48	1.010	1.100
	10	4.88	3.25	54	1.300	1.390
	12	5.59	3.73	60	1.560	1.670
18	6	3.45	2.30	46	0.743	0.793
	8	4.17	2.78	51	1.010	1.100
	10	4.88	3.25	57	1.350	1.450
	12	5.59	3.73	63	1.690	1.810
21	8	4.17	2.78	54	1.010	1.100
	10	4.88	3.25	60	1.350	1.450
	12	5.59	3.73	66	1.720	1.840
29	8	4.17	2.78	62	1.010	1.100
	10	4.88	3.25	68	1.350	1.450
	12	5.59	3.73	74	1.720	1.840

Note

a $f_{u,k}$ = 540N/mm². Values applicable to plywood with a characteristic density of at least 505kg/m³, (527kg/m³ for 6mm thick plywood), i.e. all plywoods listed in Table 3.28. Higher values can be obtained for Finnish birch plywoods using the table for screwed plywood-to-timber connections in the CD. It is assumed that the plywood is the headside member.

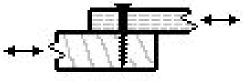


Table 6.14 Characteristic lateral load capacity per fastener in wood-screwed 2-member OSB-timber connections, not predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod} / \gamma_M$[b]

OSB Thickness	Screw				Timber	
	Gauge	Nominal (outer) diameter	Assumed effective diameter	Minimum screw length	Strength class	
					C16	C24, GL24h
					Load capacity	
(mm)	(No.)	(mm)	(mm)	(mm)	(kN)	(kN)
9	4	2.74	1.83	26	0.479	0.523
	6	3.45	2.30	30	0.632	0.697
	8	4.17	2.78	35	0.798	0.885
	10	4.88	3.25	39	1.020	1.140
	12	5.59	3.73	43	1.220	1.220
11	6	3.45	2.30	32	0.690	0.757
	8	4.17	2.78	37	0.853	0.941
	10	4.88	3.25	41	1.070	1.190
	12	5.59	3.73	45	1.310	1.470
15	6	3.45	2.30	36	0.737	0.810
	8	4.17	2.78	41	0.985	1.080
	10	4.88	3.25	45	1.200	1.320
	12	5.59	3.73	49	1.430	1.590
18	6	3.45	2.30	39	0.738	0.812
	8	4.17	2.78	44	1.000	1.100
	10	4.88	3.25	48	1.310	1.440
	12	5.59	3.73	52	1.540	1.700
23	8	4.17	2.78	49	1.000	1.100
	10	4.88	3.25	53	1.320	1.460
	12	5.59	3.73	57	1.670	1.850

Notes

a $f_{u,k}$ = 540N/mm². It is assumed that the OSB is the headside member.

b $k_{mod} = \sqrt{k_{mod,OSB}\, k_{mod,timber}}$.

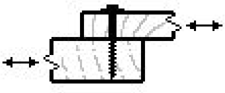


Table 6.15 Characteristic lateral load capacity per fastener in coach screwed 2-member timber-timber connections, predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\ k_{mod} / \gamma_M$

Screw dimensions		Headside	Load parallel to grain (kN)		Load perpendicular to grain (kN)	
Diameter[b] (mm)	Length (mm)	Thickness[c] (mm)	C16	C24, GL24h	C16	C24, GL24h
8	80	22	2.84	3.19	2.34	2.64
	100	35	3.52	3.90	2.77	3.06
	100	38	3.68	4.08	2.87	3.18
	100	44	3.60	3.92	2.85	3.22
	110	47	3.83	4.11	3.18	3.52
10	110	35	4.83	5.33	3.86	4.24
	110	38	4.81	5.43	3.76	4.26
	120	44	5.37	5.96	4.17	4.61
	120	47	5.45	6.14	4.18	4.75
	140	63	5.71	6.14	4.85	5.22
12	120	35	5.91	6.65	4.70	5.29
	130	38	6.51	7.17	5.21	5.72
	130	44	6.58	7.44	5.11	5.79
	140	47	7.11	7.88	5.54	6.12
	150	63	7.69	8.44	6.04	6.89
	160	72	7.79	8.50	6.56	7.19
16	160	44	10.1	11.4	8.02	9.10
	160	47	9.95	11.2	7.74	8.74
	180	63	11.8	13.4	8.97	10.2
	200	72	12.9	14.2	9.70	10.8
	220	97	13.2	14.2	11.0	11.9

Notes

a $f_{u,k}$ = 540N/mm².

b Screw diameters are nominal (outside) diameters.

c The pointside member must be thick enough to accommodate the length of screw shown, normally at least twice the headside thickness.

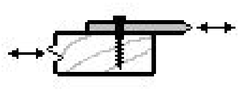


Table 6.16 Characteristic lateral load capacity per fastener in coach screwed 2-member steel-timber connections, predrilled[a]. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod}/\gamma_M$

Screw dimensions		Steel thickness (mm)	Load parallel to grain (kN)		Load perpendicular to grain (kN)	
Diameter[b] (mm)	Length[c] (mm)		C16	C24, GL24h	C16	C24, GL24h
8	60	2.5	4.16	4.01	2.42	2.73
8	70	2.5	4.67	5.07	3.44	3.88
8	80	2.5	4.71	5.07	4.07	4.39
10	70	3.0	5.83	6.29	4.09	4.61
10	80	3.0	6.63	7.32	4.70	5.30
10	90	3.0	6.91	7.44	5.31	5.99
10	100	3.0	6.91	7.44	5.92	6.39
12	80	3.5	7.71	8.27	5.37	6.06
12	90	3.5	8.59	9.38	6.07	6.85
12	100	3.5	9.45	10.2	6.77	7.65
12	120	3.5	9.45	10.2	8.04	8.68
16	120	5.0	14.2	15.5	9.88	11.2
16	140	5.0	16.3	17.7	11.6	13.1
16	150	5.0	16.4	17.7	12.5	14.1
16	175	5.0	16.4	17.7	13.9	15.0

Notes

a $f_{u,k}$ = 540N/mm².

b Screw diameters are nominal (outside) diameters.

c The pointside member must be thick enough to accommodate the length of screw shown, normally at least twice the headside thickness.

6.5 Bolted connections

6.5.1 Introduction

The diameter d of bolts should be in the range 6mm to 30mm inclusive (see BS EN 14592[46]). When tightened directly onto timber, a steel washer with a minimum external diameter or side length of $3d$ and a minimum thickness of $0.3d$ should always be fitted beneath the bolt head and the nut (EC5 10.4.3(2)). Requirements for bolt holes are given in Section 12.3.4.

Simplified rules for minimum spacings and distances are shown in Table 6.17. For more precise rules see EC5 Table 8.2.

Table 6.17 Minimum spacings and distances for connections made with bolts and large screws
(based on EC5 Table 8.2)

	Definition	Minimum spacing or distance in terms of diameters			
a_1	Spacing parallel to grain	$0° \leqslant \alpha < 60°$	$60° \leqslant \alpha < 90°$	$90°$	
		$5d$	$4.5d$	$4d$	
a_2	Spacing perpendicular to grain	$4d$			
$a_{3,t}$	Loaded end	Maximum of $7d$ and 80mm			
$a_{3,c}$	Unloaded end	$0° \leqslant \alpha \leqslant 30°$	$30° < \alpha < 45°$	$45° \leqslant \alpha < 60°$	$60° \leqslant \alpha \leqslant 90°$
		$4d$	$5d$	$6d$	$7d$
$a_{4,t}$	Loaded edge	$0° \leqslant \alpha < 30°$	$30° \leqslant \alpha \leqslant 45°$	$45° < \alpha \leqslant 90°$	
		$3d$	$3.5d$	$4d$	
$a_{4,c}$	Unloaded edge	$3d$			

Notes

a a = spacing or distance (see Figure 6.2).
b d = diameter of fastener.
c α = angle of fastener load to grain.
d The above values apply to solid timber and glulam.
e For LVL use similar values unless the manufacturer's literature states otherwise.
f For plywood or OSB bolted to timber or a structural timber composite the minimum loaded edge distance in the panel product should be $5d$ and the minimum unloaded edge distance $3d$.

6.5.2 Laterally loaded bolts

6.5.2.1 Effective number of bolts, n_{ef}

For pre-calculated values of n_{ef} see Table 6.18, or to calculate n_{ef}, see EC5 8.5.1.1(4). For the use of n_{ef}, see Sections 6.2.3.4 and 6.2.3.5.

6.5.2.2 Load capacities

Tables 6.19 to 6.23 give characteristic values of the lateral loading capacity of bolted connections made with steel bolts in some common timber materials. A minimum tensile strength of 400N/mm² is assumed, but other grades of steel may be specified, as shown in Table 3.29. To obtain the strength of connections made with higher strength bolts see EC5 8.5 or TRADA's fastener software[8].

Table 6.18 Values of n_{ef} for connections made with bolts, dowels and large screws *(EC5 expression 8.5.1.1(4))*

Angle of load to grain		Spacing parallel to grain[a]	Number of bolts or dowels in a row parallel to the grain for spacing shown[b]								
			2	3	4	5	6	8	10	12	15
Bolts	Dowels		Value of n_{ef}								
n/a	90°	3.0d	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
90°	60°	4.0d[c]	1.39	2.00	2.59	3.17	3.74	4.84	5.92	6.97	8.52
60°	45°	4.5d	1.43	2.06	2.67	3.27	3.85	4.98	6.09	7.18	8.78
0°	0°	5.0d	1.47	2.12	2.74	3.35	3.95	5.12	6.26	7.37	9.01
0°	0°	8.0d	1.65	2.38	3.08	3.77	4.44	5.76	7.04	8.29	10.1
0°	0°	12.0d	1.83	2.63	3.41	4.17	4.92	6.37	7.79	9.17	11.2

Notes

a d = diameter of fastener.

b For a 'row' parallel to the grain containing only one fastener n_{ef} = 1.

c Values of n_{ef} are not applicable for bolt loads at 90° to the grain.

6.5.3 Axially loaded bolts

The axial load capacity of a bolt in a timber connection is the minimum of the design tensile capacity of the bolt and the design compression strength of the timber beneath the washer or steel plate.

For the bolt tensile capacity see BS EN ISO 898-1[93].

The design compression strength of the timber (see EC5 8.5.2)

$= 0.75\pi\ (8d^2 - 2d - 1)\ f_{c,90,d}$ N   beneath round washers with an external diameter of 3d

$= 0.75\pi\ (15d^2 - 2d - 1)\ f_{c,90,d}$ N   beneath steel plates

provided that the steel has a thickness of at least 0.3d and a hole diameter of d + 1mm, where d is the diameter of the bolt in mm.

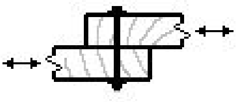


Table 6.19 Characteristic lateral load capacity per fastener for 2-member timber-timber connections made with 4.6 grade bolts. To obtain the design value for the connection multiply by $n_{ef}\, k_{mod}/\gamma_M$

Bolt diameter (mm)	Minimum member thickness (mm)	Load parallel to grain (kN)		Load perpendicular to grain (kN)	
		Strength class		Strength class	
		C16	C24,GL24h	C16	C24,GL24h
8	22	2.13	2.41	1.45	1.64
	35	3.37	3.80	2.31	2.60
	38	3.60	4.06	2.50	2.83
	44	4.06	4.59	2.90	3.27
	47	4.29	4.60	3.10	3.50
	63 or more	4.29	4.60	3.65	3.93
10	22	2.61	2.94	1.74	1.96
	35	4.15	4.68	2.76	3.12
	38	4.50	5.08	3.00	3.39
	44	5.19	5.86	3.47	3.92
	47	5.48	6.18	3.71	4.19
	63	6.40	6.87	4.98	5.62
	72 or more	6.40	6.87	5.41	5.82
12	22	3.06	3.45	2.00	2.26
	35	4.86	5.49	3.18	3.59
	38	5.28	5.96	3.45	3.90
	44	6.12	6.90	4.00	4.51
	47	6.53	7.38	4.27	4.82
	63	8.48	9.51	5.72	6.46
	72	8.85	9.51	6.54	7.38
	97 or more	8.85	9.51	7.44	7.93
16	35	6.19	6.99	3.89	4.40
	38	6.72	7.59	4.23	4.77
	44	7.78	8.79	4.90	5.53
	47	8.31	9.39	5.23	5.90
	63	11.1	12.6	7.01	7.91
	72	12.7	14.4	8.01	9.04
	97	14.7	15.8	10.8	12.2
	145 or more	14.7	15.8	12.0	12.8
20	44	9.27	10.5	5.62	6.34
	47	9.90	11.2	6.00	6.77
	63	13.3	15.0	8.04	9.08
	72	15.2	17.1	9.19	10.4
	97	20.4	23.1	12.4	14.0
	145 or more	21.7	23.4	17.2	18.3
24	47	11.3	12.7	6.60	7.45
	63	15.1	17.1	8.84	9.99
	72	17.3	19.5	10.1	11.4
	97	23.3	26.3	13.6	15.4
	145	29.8	31.7	20.4	23.0
	195 or more	29.8	31.7	22.8	24.3

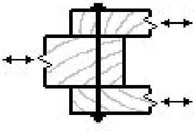


Table 6.20 Characteristic lateral load capacity per shear plane for 3-member timber-timber connections made with 4.6 grade bolts. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Bolt diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Outer (mm)	Inner (mm)	C16	C24,GL24h	C16	C24,GL24h
8	22	44	3.15	3.40	2.49	2.68
	35	70	3.69	4.07	2.85	3.11
	38	76	3.83	4.24	2.96	3.24
	44	88	4.14	4.60	3.19	3.51
	47	94	4.29	4.60	3.31	3.66
	63 or more	126	4.29	4.60	3.65	3.93
10	22	44	4.47	4.80	3.36	3.79
	35	70	5.11	5.58	3.87	4.20
	38	76	5.27	5.80	3.98	4.33
	44	88	5.61	6.20	4.22	4.61
	47	94	5.79	6.41	4.35	4.77
	63	126	6.40	6.87	5.14	5.69
	72 or more	144	6.40	6.87	5.41	5.82
12	22	44	5.91	6.43	3.86	4.36
	35	70	6.61	7.17	5.05	5.44
	38	76	6.81	7.41	5.14	5.56
	44	88	7.25	7.93	5.37	5.83
	47	94	7.46	8.22	5.50	5.99
	63	126	8.61	9.51	6.32	6.96
	72	144	8.85	9.51	6.85	7.57
	97 or more	194	8.85	9.51	7.44	7.93
16	35	70	10.2	10.9	7.52	8.35
	38	76	10.3	11.1	7.88	8.43
	44	88	10.8	11.7	8.05	8.66
	47	94	11.0	12.0	8.16	8.79
	63	126	12.5	13.8	8.92	9.72
	72	144	13.4	14.9	9.46	10.4
	97	194	14.7	15.8	11.2	12.4
	145 or more	290	14.7	15.8	12.0	12.8
20	44	88	14.9	16.0	10.8	12.0
	47	94	15.1	16.3	11.3	12.1
	63	126	16.6	18.1	11.9	12.8
	72	144	17.6	19.3	12.4	13.4
	97	194	20.9	23.2	14.1	15.4
	145 or more	290	21.7	23.4	17.2	18.3
24	47	94	19.8	21.2	12.7	14.4
	63	126	21.1	22.9	15.2	16.3
	72	144	22.1	24.1	15.6	16.8
	97	194	25.5	28.1	17.1	18.7
	145	290	29.8	31.7	21.2	23.4
	195 or more	390	29.8	31.7	22.8	24.3

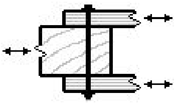


Table 6.21 Characteristic lateral load capacity per shear plane for 3-member plywood-timber-plywood[a] connections made with 4.6 grade bolts. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Member thickness		Load parallel to grain (kN)				Load perpendicular to grain (kN)			
Plywood (mm)	Timber (mm)	Bolt diameter				Bolt diameter			
		M10	M12	M16	M20	M10	M12	M16	M20
C16 timber									
6	35	3.13	3.67	4.67	5.57	2.67	3.07	3.76	4.31
9	38	4.35	5.10	6.49	7.73	2.90	3.33	4.08	4.68
12	47	5.18	6.31	8.03	9.56	3.58	4.12	5.05	5.79
15	63	5.33	7.19	10.76	12.81	4.72	5.53	6.77	7.76
18	63	5.53	7.42	10.76	12.81	4.80	5.53	6.77	7.76
21	63	5.79	7.66	10.76	12.81	4.80	5.53	6.77	7.76
29	72	6.60	8.51	12.30	14.64	5.49	6.32	7.74	8.87
C24 timber									
6	35	3.13	3.67	4.67	5.57	3.01	3.47	4.25	4.87
9	38	4.50	5.28	6.72	8.00	3.27	3.76	4.61	5.29
12	47	5.39	7.04	8.96	10.67	4.05	4.66	5.70	6.54
15	63	5.58	7.46	11.20	13.33	4.92	6.24	7.64	8.77
18	63	5.83	7.70	12.13	14.46	5.17	6.24	7.64	8.77
21	63	6.08	8.01	12.15	14.46	5.42	6.24	7.64	8.77
29	72	6.92	8.94	13.43	16.53	6.20	7.13	8.73	10.02

Note

a The tabulated values are safe for all the plywoods listed in Table 3.28, and for other plywoods listed in BS 5268-2 Groups 1 and 2 but excluding Swedish softwood plywood. However higher values can be obtained for Finnish birch plywoods using the table for bolted plywood-to-timber connections on the CD.

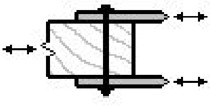


Table 6.22 Characteristic lateral load capacity per shear plane for 3-member steel-timber-steel connections made with 4.6 grade bolts. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Bolt diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Steel (mm)	Timber (mm)	C16	C24,GL24h	C16	C24,GL24h
10	3	35	4.00	4.52	2.67	3.01
		38	4.35	4.91	2.90	3.27
		44	5.03	5.68	3.36	3.79
		47	5.38	6.07	3.58	4.05
		63	6.72	7.14	4.80	5.42
		72 or more	6.72	7.14	5.49	5.83
12	3.6	35	4.70	5.30	3.07	3.47
		38	5.10	5.76	3.33	3.76
		44	5.91	6.67	3.86	4.36
		47	6.31	7.12	4.12	4.66
		63	8.46	9.55	5.53	6.24
		72	9.23	9.80	6.32	7.13
		97 or more	9.23	9.80	7.46	7.93
16	4.8	35	5.98	6.75	3.76	4.25
		38	6.49	7.33	4.08	4.61
		44	7.52	8.49	4.73	5.34
		47	8.03	9.06	5.05	5.70
		63	10.8	12.2	6.77	7.64
		72	12.3	13.9	7.74	8.73
		97	15.1	16.1	10.4	11.8
		120 or more	15.1	16.1	12.0	12.8
20	6	35	7.12	8.04	4.31	4.87
		38	7.73	8.72	4.68	5.29
		44	8.95	10.1	5.42	6.12
		47	9.56	10.8	5.79	6.54
		63	12.8	14.5	7.76	8.77
		72	14.6	16.5	8.87	10.0
		97	19.7	22.3	12.0	13.5
		120	22.1	23.4	14.8	16.7
		145 or more	22.1	23.4	17.2	18.3
24	7.2	35	8.11	9.16	4.75	5.36
		38	8.81	9.95	5.15	5.82
		44	10.2	11.5	5.97	6.73
		47	10.9	12.3	6.37	7.19
		63	14.6	16.5	8.54	9.64
		72	16.7	18.8	9.76	11.0
		97	22.5	25.4	13.2	14.8
		120	27.8	31.4	16.3	18.4
		145	29.9	31.7	19.7	22.2
		195 or more	29.9	31.7	22.8	24.3

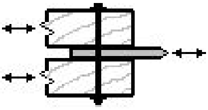


Table 6.23 Characteristic lateral load capacity per shear plane for 3-member timber-steel-timber connections made with 4.6 grade bolts. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Bolt diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Timber (mm)	Steel (mm)	C16	C24,GL24h	C16	C24,GL24h
10	35	3	6.13	6.70	4.93	5.31
	38		6.29	6.90	5.02	5.43
	44		6.65	7.33	5.23	5.72
	47		6.85	7.56	5.34	5.86
	63		8.04	8.94	6.04	6.69
	72		8.63	9.23	6.48	7.20
	97 or more		8.63	9.23	7.23	7.75
12	35	3.6	8.17	8.89	6.14	6.93
	38		8.32	9.07	6.60	7.09
	44		8.67	9.51	6.79	7.34
	47		8.87	9.75	6.92	7.49
	63		10.1	11.2	7.69	8.48
	72		10.9	12.2	8.15	9.03
	97		11.9	12.8	9.62	10.6
	120 or more		11.9	12.8	9.91	10.6
16	35	4.8	12.0	13.5	7.52	8.49
	38		13.0	14.3	8.17	9.22
	44		13.5	14.7	9.45	10.7
	47		13.7	15.0	10.1	11.3
	63		15.0	16.5	11.2	12.1
	72		15.9	17.5	11.7	12.8
	97		18.7	20.8	13.5	15.0
	120		19.7	21.1	15.2	16.9
	145 or more		19.7	21.1	16.2	17.4
20	38	6.0	15.5	17.4	9.37	10.6
	44		17.9	20.2	10.8	12.3
	47		19.1	20.8	11.6	13.1
	63		20.7	22.5	15.2	16.4
	72		21.6	23.7	15.6	16.9
	97		24.5	27.2	17.4	19.0
	120		27.6	30.8	19.4	21.4
	145		29.1	31.1	21.6	24.1
	195 or more		29.1	31.1	23.5	25.3
24	47	7.2	21.8	24.6	12.7	14.4
	63		26.8	28.9	17.1	19.3
	72		27.8	30.1	19.5	21.5
	97		31.0	34.2	21.4	23.3
	120		34.2	38.0	23.3	25.5
	145		38.2	42.5	25.8	28.5
	195 or more		39.7	42.5	31.1	34.1

6.6 Dowelled connections

6.6.1 Introduction

Round steel dowels may be smooth or fluted. Fluted dowels are easier to insert but are more expensive. The inner diameter of the flutes should be at least 0.95d. The external diameter of dowels should be in the range 6mm to 30mm inclusive (see BS EN 14592[46]), with a tolerance of -0/+0.1mm. Holes in the timber members should be bored to produce a diameter not greater than that of the dowel.

Dowelled connections are neater than bolted ones, and if the total length of the dowel is made less than the total thickness of the timber in the connection the resulting recess at each end can be plugged with a timber dowel or epoxy resin to protect the steel from fire. There is less slip in a dowelled connection than a bolted connection but the workmanship must be of a higher standard.

Dowels are not used for axial loading. Where there is likely to be a secondary component of load perpendicular to the grain or a possibility that a dowelled joint might open out, consideration should be given to the addition of supplementary bolts. This detail is typical in a moment-resisting dowel ring or rings in the eaves joint of a portal frame.

For material specifications see Table 3.29. Simplified rules for minimum spacings and distances are shown in Table 6.24. For more precise rules see EC5 Table 8.5.

Table 6.24 Minimum spacings and distances for dowels

<table>
<tr><th></th><th>Definition</th><th colspan="6">Minimum spacing or distance in terms of diameters</th></tr>
<tr><td rowspan="2">a_1</td><td rowspan="2">Spacing parallel to grain</td><td colspan="2">$0° \leqslant \alpha < 60°$</td><td colspan="2">$60° \leqslant \alpha < 90°$</td><td colspan="2">90°</td></tr>
<tr><td colspan="2">5d</td><td colspan="2">4d</td><td colspan="2">3d</td></tr>
<tr><td>a_2</td><td>Spacing perpendicular to grain</td><td colspan="6">3d</td></tr>
<tr><td>$a_{3,t}$</td><td>Loaded end</td><td colspan="6">Maximum of 7d and 80mm</td></tr>
<tr><td rowspan="3">$a_{3,c}$</td><td>Unloaded end</td><td>0°</td><td colspan="2">$0° < \alpha < 30°$</td><td colspan="2">$30° \leqslant \alpha < 45°$</td><td>$60° \leqslant \alpha \leqslant 90°$</td></tr>
<tr><td>$d \leqslant$ 11mm</td><td>3d</td><td colspan="2">40mm</td><td colspan="2">60mm</td><td>80mm</td></tr>
<tr><td>d > 11mm</td><td>3d</td><td colspan="2">3.5d</td><td colspan="2">5d</td><td>7d</td></tr>
<tr><td rowspan="2">$a_{4,t}$</td><td rowspan="2">Loaded edge</td><td colspan="2">$0° \leqslant \alpha < 30°$</td><td colspan="2">$30° \leqslant \alpha \leqslant 45°$</td><td colspan="2">$45° < \alpha \leqslant 90°$</td></tr>
<tr><td colspan="2">3d</td><td colspan="2">3.5d</td><td colspan="2">4d</td></tr>
<tr><td>$a_{4,c}$</td><td>Unloaded edge</td><td colspan="6">3d</td></tr>
</table>

Notes

- **a** a = spacing or distance (see Figure 6.2).
- **b** d = diameter of fastener.
- **c** α = angle of fastener load to grain.
- **d** The above values apply to solid timber and glulam.
- **e** For LVL use similar values unless the manufacturer's literature states otherwise.

6.6.2 Laterally loaded dowels

6.6.2.1 Effective number of dowels, n_{ef}

For values of n_{ef} see Section 6.5.2.1.

6.6.2.2 Load capacities

The lateral load capacity of dowels is a little less than that of bolts of similar diameter because of the 'rope' effect of bolt heads and nuts which helps to hold the connection together.

Tables 6.25 to 6.28 give characteristic values of the lateral loading capacity of dowelled connections made with steel dowels in some common timber materials. The calculated loads assume that the dowels are not recessed and have an ultimate tensile strength of 400N/mm^2 is assumed.

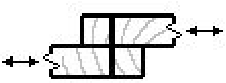


Table 6.25 Characteristic lateral load capacity per fastener for 2-member timber-timber connections made with 4.6 grade dowels To obtain the design value for the connection multiply by $n_{ef}\,k_{mod}/\gamma_M$

Dowel diameter (mm)	Minimum member thickness (mm)	Load parallel to grain (kN)		Load perpendicular to grain (kN)	
		Strength class		Strength class	
		C16	C24,GL24h	C16	C24,GL24h
8	22	1.70	1.92	1.16	1.31
	35	2.71	3.06	1.85	2.08
	38	2.94	3.32	2.00	2.26
	44	3.41	3.85	2.32	2.62
	47	3.64	3.87	2.48	2.80
	63 or more	3.64	3.87	3.00	3.19
10	22	2.08	2.35	1.39	1.57
	35	3.32	3.74	2.21	2.50
	38	3.60	4.07	2.40	2.71
	44	4.17	4.71	2.78	3.14
	47	4.45	5.03	2.97	3.35
	63	5.38	5.71	3.98	4.49
	72 or more	5.38	5.71	4.39	4.66
12	22	2.45	2.76	1.60	1.81
	35	3.89	4.39	2.54	2.87
	38	4.23	4.77	2.76	3.12
	44	4.89	5.52	3.20	3.61
	47	5.23	5.90	3.42	3.86
	63	7.00	7.84	4.58	5.17
	72	7.38	7.84	5.23	5.91
	97 or more	7.38	7.84	5.97	6.34
16	35	4.95	5.59	3.12	3.52
	38	5.38	6.07	3.38	3.82
	44	6.23	7.03	3.92	4.42
	47	6.65	7.51	4.18	4.72
	63	8.92	10.1	5.61	6.33
	72	10.2	11.5	6.41	7.24
	97	12.1	12.9	8.63	9.75
	145 or more	12.1	12.9	9.60	10.2
20	44	7.41	8.37	4.49	5.07
	47	7.92	8.94	4.80	5.42
	63	10.6	12.0	6.43	7.26
	72	12.1	13.7	7.35	8.30
	97	16.3	18.5	9.90	11.2
	145 or more	17.7	18.8	13.7	14.6
24	47	9.0	10.2	5.28	5.96
	63	12.1	13.7	7.08	7.99
	72	13.8	15.6	8.09	9.13
	97	18.6	21.0	10.9	12.3
	145	23.9	25.4	16.3	18.4
	195 or more	23.9	25.4	18.3	19.4

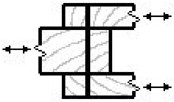


Table 6.26 Characteristic lateral load capacity per shear plane for 3-member timber-timber connections made with 4.6 grade dowels. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Dowel diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Outer (mm)	Inner (mm)	C16	C24,GL24h	C16	C24,GL24h
8	22	44	2.52	2.72	1.99	2.14
	35	70	3.03	3.34	2.28	2.49
	38	76	3.18	3.51	2.36	2.59
	44	88	3.49	3.86	2.55	2.81
	47	94	3.64	3.87	2.65	2.93
	63 or more	126	3.64	3.87	3.00	3.19
10	22	44	3.57	3.84	2.85	3.05
	35	70	4.09	4.46	3.10	3.36
	38	76	4.24	4.65	3.18	3.46
	44	88	4.58	5.05	3.38	3.69
	47	94	4.76	5.26	3.48	3.81
	63	126	5.38	5.71	4.20	4.65
	72 or more	144	5.38	5.71	4.39	4.66
12	22	44	4.82	5.15	3.85	4.10
	35	70	5.29	5.74	4.04	4.35
	38	76	5.44	5.93	4.11	4.44
	44	88	5.80	6.35	4.30	4.67
	47	94	5.99	6.57	4.40	4.79
	63	126	7.29	7.84	5.15	5.67
	72	144	7.38	7.84	5.48	6.05
	97 or more	194	7.38	7.84	5.97	6.34
16	35	70	8.12	8.74	6.25	6.68
	38	76	8.26	8.91	6.30	6.75
	44	88	8.60	9.33	6.44	6.92
	47	94	8.79	9.56	6.52	7.03
	63	126	10.2	11.2	7.23	7.88
	72	144	10.8	12.0	7.57	8.29
	97	194	12.1	12.9	8.96	9.91
	145 or more	290	12.1	12.9	9.60	10.2
20	44	88	11.9	12.8	8.96	9.57
	47	94	12.1	13.0	9.01	9.65
	63	126	13.5	14.7	9.58	10.4
	72	144	14.1	15.5	9.89	10.7
	97	194	16.8	18.6	11.3	12.4
	145 or more	290	17.7	18.8	13.7	14.6
24	47	94	15.8	17.0	11.8	12.6
	63	126	17.1	18.5	12.2	13.1
	72	144	17.7	19.3	12.5	13.4
	97	194	20.4	22.5	13.7	14.9
	145	290	23.9	25.4	16.9	18.7
	195 or more	390	23.9	25.4	18.3	19.4

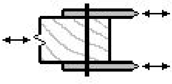


Table 6.27 Characteristic lateral load capacity per shear plane for 3-member steel-timber-steel connections made with 4.6 grade dowels[a]. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Dowel diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Steel (mm)	Timber (mm)	C16	C24,GL24h	C16	C24,GL24h
10	3	35	4.00	4.52	2.67	3.01
		38	4.35	4.91	2.90	3.27
		44	5.03	5.68	3.36	3.79
		47	5.38	5.71	3.58	4.05
		63 or more	5.38	5.71	4.39	4.66
12	3.6	35	4.70	5.30	3.07	3.47
		38	5.10	5.76	3.33	3.76
		44	5.91	6.67	3.86	4.36
		47	6.31	7.12	4.12	4.66
		63	7.38	7.84	5.53	6.24
		72 or more	7.38	7.84	5.97	6.34
16	4.8	35	5.98	6.75	3.76	4.25
		38	6.49	7.33	4.08	4.61
		44	7.52	8.49	4.73	5.34
		47	8.03	9.06	5.05	5.70
		63	10.8	12.1	6.77	7.64
		72	12.1	12.9	7.74	8.73
		97 or more	12.1	12.9	9.60	10.2
20	6	35	7.12	8.04	4.31	4.87
		38	7.73	8.72	4.68	5.29
		44	8.95	10.1	5.42	6.12
		47	9.56	10.8	5.79	6.54
		63	12.8	14.5	7.76	8.77
		72	14.6	16.5	8.87	10.0
		97	17.6	18.8	12.0	13.5
		120 or more	17.6	18.8	13.7	14.6
24	7.2	35	8.11	9.16	4.75	5.36
		38	8.81	9.95	5.15	5.82
		44	10.2	11.5	5.97	6.73
		47	10.9	12.3	6.37	7.19
		63	14.6	16.5	8.54	9.64
		72	16.7	18.9	9.76	11.0
		97	22.5	25.4	13.1	14.8
		120	23.9	25.4	16.3	18.4
		145 or more	23.9	25.4	18.3	19.4

Note

a For use in multiple shear plane connections. The steel plates should not be on the outside unless the dowels are threaded at the ends to take nuts.

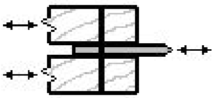


Table 6.28 Characteristic lateral load capacity per shear plane for 3-member timber-steel-timber connections made with 4.6 grade dowels. To obtain the design value for the connection multiply by $2n_{ef}\,k_{mod}/\gamma_M$

Dowel diameter (mm)	Minimum member thickness		Load parallel to grain (kN)		Load perpendicular to grain (kN)	
			Strength class		Strength class	
	Timber (mm)	Steel (mm)	C16	C24,GL24h	C16	C24,GL24h
10	35	3	5.11	5.55	3.94	4.25
	38		5.27	5.74	4.02	4.34
	44		5.63	6.18	4.21	4.58
	47		5.83	6.41	4.32	4.71
	63		7.02	7.79	5.01	5.53
	72		7.60	8.08	5.46	6.05
	97 or more		7.60	8.08	6.21	6.60
12	35	3.6	6.70	7.23	5.23	5.59
	38		6.84	7.41	5.28	5.67
	44		7.20	7.85	5.44	5.87
	47		7.40	8.09	5.53	5.99
	63		8.67	9.58	6.22	6.82
	72		9.47	10.5	6.68	7.36
	97		10.4	11.1	8.14	8.97
	120 or more		10.4	11.1	8.44	8.97
16	35	4.8	10.6	11.3	7.52	8.49
	38		10.6	11.4	8.17	8.87
	44		10.9	11.8	8.38	8.95
	47		11.1	12.0	8.43	9.03
	63		12.4	13.5	8.95	9.71
	72		13.2	14.6	9.39	10.2
	97		16.0	17.8	10.9	12.0
	120		17.1	18.2	12.5	13.9
	145 or more		17.1	18.2	13.6	14.4
20	38	6	15.3	16.3	9.37	10.6
	44		15.4	16.5	10.8	12.2
	47		15.5	16.7	11.6	12.7
	63		16.6	18.0	12.2	13.1
	72		17.5	19.1	12.5	13.5
	97		20.4	22.5	13.9	15.2
	120		23.5	26.2	15.5	17.1
	145		25.0	26.5	17.5	19.5
	195 or more		25.0	26.5	19.4	20.6
24	47	7.2	20.8	22.1	12.7	14.4
	63		21.5	23.1	15.9	17.0
	72		22.2	24.1	16.1	17.2
	97		25.1	27.5	17.1	18.6
	120		28.3	31.3	18.7	20.4
	145		32.3	35.9	20.6	22.8
	195 or more		33.8	35.9	25.2	27.4

6.7 Glued joints

6.7.1 Introduction

Structural glued joints are generally stiffer, require less timber and have a better appearance than mechanically fastened connections. They are resistant to corrosive atmospheres, and joints made with thermosetting resins are safer in fire than mechanically fastened connections. Their chief disadvantages are that they require stringent quality control, which is expensive, may be unsuitable in conditions of fluctuating moisture content if dissimilar materials are involved or if there is a change in the angle of grain at their interfaces, and are unsuitable if there is a significant component of load perpendicular to the plane of adhesion.

EC5 gives little guidance on the design of glued joints, so this section of the *Manual* contains complementary information. Further information can be found in *The structural use of adhesives*[94].

Figure 6.6 shows some common types of glued joint.

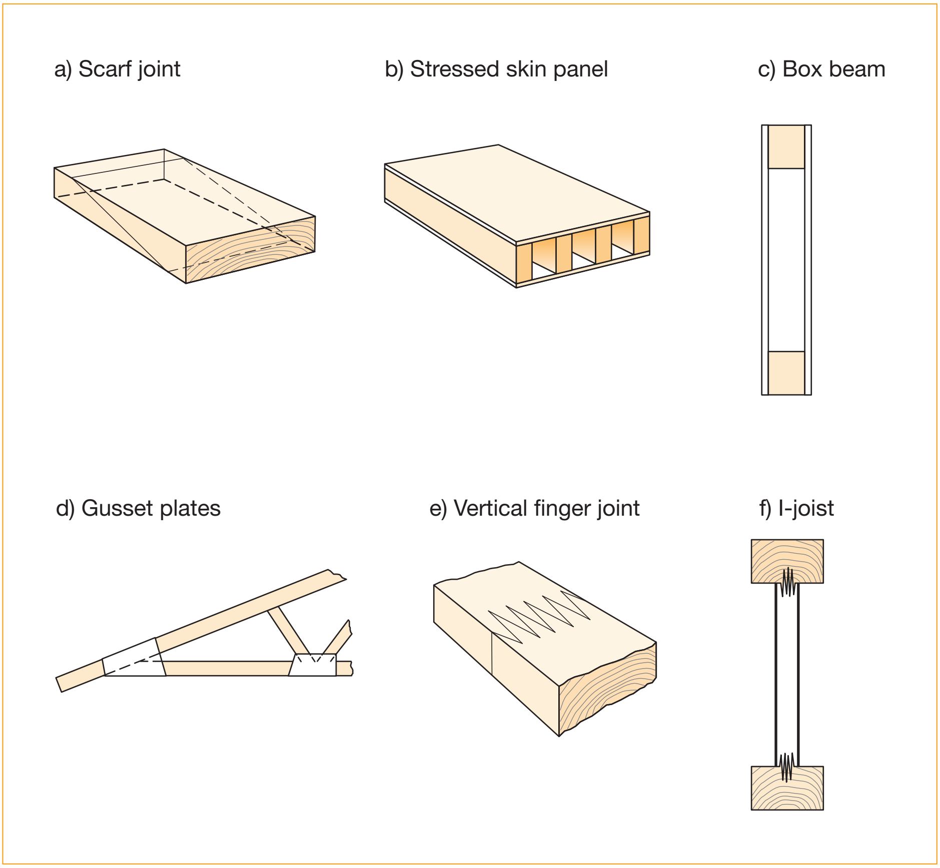


Fig 6.6 Common types of glued timber joint

Unless specialist advice is available, the use of structural timber glued joints should be restricted to joints in dry timber comprising:

- solid or laminated softwood members or structural timber composites not thicker than 50mm in the direction perpendicular to the glued plane
- solid or laminated softwood members or structural timber composites of any thickness joined to plywood, OSB or particleboard no thicker than 29mm
- plywood, OSB or particleboard members of any thickness.

Joint types a) to d) in Figure 6.6 should be manufactured in accordance with BS 6446[95], small finger joints e) in accordance with BS EN 385[96] and large (full depth) finger joints in accordance with BS EN 387[97]. Glued joints in I-joists f) are covered by ETAG 11[98].

All structural glued joints require specialised workmanship and should normally be produced in a workshop within a formal quality control scheme. Full depth finger joints in principal members or in other members acting alone where failure could lead to collapse should not be used unless the joints are manufactured under a third-party quality control scheme.

6.7.2 Design

The design lateral load capacity of joint types a) to e) in Figure 6.6 may be calculated as:

$$R_d = k_{bond}\, k_{mc}\, k_r A f_{v,d}$$

Where	k_{bond}	=	1.0 if a suitable joint cramping pressure as defined in BS 6446[95] is applied by press, airbags, springs or weights
		=	0.9 if cramping pressure is applied by nails, screws or staples in accordance with BS 6446
	k_{mc}	=	factor to allow for the effects of fluctuating moisture content
		=	1.0 in service class 1, and in service class 2 where the grain directions of bonded surfaces are within 30° of each other
		=	0.75 in service class 2 where the grain directions of bonded surfaces differ by more than 30°
		=	0.5 in service class 3 where the grain directions of bonded surfaces are within 30° of each other
		=	0.0 in service class 3 where the grain directions of bonded surfaces differ by more than 30°
	k_r	=	rolling shear factor
		=	0.5 at the junction of the web and flange of a plywood webbed beam
		=	1.0 in all other cases
	A	=	bonded area

$$f_{v,d} = \frac{k_{mod} f_{v,k,\alpha}}{\gamma_M}$$

$f_{v,\alpha,k}$	=	shear strength of timber in direction of load
	=	$f_{v,k}$ (1-0.67sinα) for solid timber, glulam and LVL
	=	$f_{v,r,k}$ for plywood (its rolling shear strength)
α	=	angle between directions of load and grain

Where two dissimilar timber materials are joined, the lower value of $f_{v,d}$ should be used.

In joint types c) to f) made with plywood skins, webs or gussets, alternate veneers of the plywood will be in rolling shear, the weakest of the three timber shear planes. In verifying the strength of the plywood in rolling shear, only the area of contact between the plywood and the framing members should be considered. In the case of box beams and I-beams the k_r value of 0.5 accounts for the stress concentration at the junction between the web and the flanges.

A correctly glued scarf joint with a slope between 1:7 and 1:8 will have similar strength in tension to that of the unjointed timber[99].

Generally adhesively bonded joints should not be loaded in a direction perpendicular to the glue line, so scarf joints should not be used in bending in a plane which would cause the joint to open.

The strength of finger joints depends on the profile and should be determined by testing in accordance with BS EN 385[96]. The strength of the unjointed timber may be used in design provided it is specified that the characteristic bending strength of the jointed timber is not less than that of the grade of timber from which it is made.

6.7.3 Adhesives

Recommended types of structural adhesive are shown in Table 6.29. Suitable specifications for different exposure categories are shown in Table 6.30. For further information see TRADA's *Adhesively-bonded timber connections*[79]. All structural adhesives should conform to the appropriate standard specified in Table 6.30 except epoxy resin, for which the manufacturer's advice should be sought.

Table 6.29 Load-bearing adhesives and their uses

Adhesive	Abbreviation	Type[a]	Uses[b]	Suitable conditions
Resorcinol formaldehyde	RF	Phenolic thermoset resin	Laminating, finger jointing, wood jointing. (Rarely used alone because of high cost)	Fully exterior
Phenol-resorcinol formaldehyde	PRF or PF/RF		Laminating, finger jointing, wood jointing	
Phenol-formaldehyde	PF		Plywood, some particleboard	
Melamine formaldehyde	MF	Aminoplastic thermoset resin	Plywood, particleboard, formwork panels. (Not often used alone in the UK)	Semi-exterior and moist interior
Melamine urea formaldehyde	MUF		Laminating, finger jointing, plywood, particleboard	
Epoxy resin		Multi-component thermoset resin	Structural repairs, timber-to-steel, timber end-jointing	
Polyurethane		Thermoplastic polymer	Laminating	Semi-exterior and moist interior where temperature does not exceed 50°
Urea formaldehyde	UF	Aminoplastic thermoset resin	Plywood, particleboard, wood jointing	Interior
Casein		Milk product	Laminating, finger jointing, wood jointing	

Notes

a An elevated temperature is required to cure PF, MF and MUF adhesives.

b PVA (polyvinyl acetate) adhesives should not be used for structural purposes, but in certain limited circumstances PVAc (cross linked PVA adhesives) may be acceptable.

Table 6.30 Exposure categories for load-bearing adhesives

Category	Description	Examples	Exposure type[a]	Conforming to	Type
Exterior high hazard service class 3	Full exposure to weather	Marine and other exterior structures, exterior components or assemblies in which the glue line is exposed to the elements	Type I	BS EN 301[100]	RF, PRF, PF
Exterior low hazard service class 3/2	External but protected from direct sun and rain	Inside the roofs of open sheds and porches, concrete formwork	Type I	BS EN 301	RF, PRF, PF
			Type II	BS EN 301	MUF[b]
			MR	BS 1204[101]	UF
				BS EN 14080[16]	Thermoplastic polymer
					Epoxy resin
Interior high hazard service class 2/3	Closed buildings with warm and damp conditions where the timber m.c. exceeds 18% and the glue line temperature can exceed 50°C. Chemically polluted atmospheres	Laundries, unventilated roof spaces	Type I	BS EN 301	RF, PRF, PF
					Epoxy resin
		Chemical works, swimming baths			
Interior low hazard service class 1	Heated and ventilated buildings where the timber m.c. does not exceed 18% and the glue line temperature remains below 50°C.	Inside heated buildings	Type I	BS EN 301	RF, PRF, PF
			Type II	BS EN 301	MUF[b]
			MR	BS 1204	UF
				BS EN 14080	Thermoplastic polymer
					Epoxy resin
				BS EN 12436[102]	Casein

Notes

a BS EN 301 divides adhesives into Type I, which are suitable to exposure to high temperature or full exposure to weather, and Type II, which are suitable for use in heated and ventilated buildings, for prolonged exterior use if protected from the weather, or for short periods of full exposure to the weather. MR = moisture resistant.

b Other modified UF adhesives meeting the requirements of Type II may also be used.

6.8 Glued rods

6.8.1 Introduction

Strong, invisible and fire-resistant connections between members made of solid timber, glulam or LVL and members of similar material, steel or concrete can be made by gluing rods, usually made of mild steel, into pre-bored holes in the timber members. Although they are used principally in new structures, such connections are often favoured in the UK for repair and conservation work. The system is suitable for use in service class 1, in service class 2 where the angle between the rods and the timber grain does not exceed 30°, and in service class 3 where the rods are parallel to the grain. Rods can be straight or curved, the latter providing a method of end-jointing large glulam sections at an angle, e.g. in portal frames.

Rods of between 12mm and 20mm diameter are commonly used, with length:diameter ratios of between 10 and 20. For rods loaded in tension a minimum length of d^2 is recommended where d = the outer diameter of the thread in mm. The rods should be threaded to ensure mechanical locking with the adhesive, and be pre-cleaned by grit blasting or solvent wiping in accordance with Health and Safety regulations.

Suitable adhesive types (see Table 6.29) are:

- 2-part thixotropic epoxy resin system (most common). With this type of resin a hole diameter about 5mm larger than the diameter of the rod is suitable.
- PRF resin. May be used if the rod is screwed into a hole with a slightly smaller diameter.
- 2-part polyurethane adhesive. Cheaper than the above, but more suitable for service class 1 or 2 than 3, and not for elevated temperatures.

This is a specialist subject area and reference should be made to TRADA's *Resin repairs to timber structures, Volumes 1*[103] and *2*[104] and *STEP Volume 1 Lecture C14*[6], or to a specialist contractor.

6.8.2 Design

6.8.2.1 Principles

A rod group is normally loaded primarily in shear, tension or bending.

The following limiting strength factors must be verified:

- strength of rods
- strength of adhesive bond with the rod
- strength of adhesive bond with the timber (where rods are loaded in tension or compression)
- strength of connection in lateral shear (where rods loaded in shear)
- block shear strength of timber (where timber is loaded in shear)
- axial tensile or compressive strength of timber (where timber is loaded axially).

The strength of the adhesive bonds should be verified initially by test. The design rules in this Section assume that the strength of the bond between the adhesive, the threaded rods and the timber exceeds the shear strength of the timber. These rules are mostly based on recommendations given in *STEP Volume 1 Lecture C14*[6].

6.8.2.2 Spacings and distances

The minimum recommended spacings and distances in terms of rod diameters are shown in Table 6.31.

Table 6.31 Minimum recommended spacings and distances for glued-in rods in terms of rod diameter or side length, *d*

Orientation of rods to grain	Rod loading	Dimension	Minimum
All	All	Spacing between rods	$2d$
Parallel	Tension	Edge distance	$1.5d$
	Shear	Unloaded edge distance	$2d$
		Loaded edge distance	$4d$
Perpendicular	Tension	Edge distance	$2.5d$
		End distance	$2d$

6.8.2.3 Laterally loaded rods

i) Rods in side grain

For laterally loaded rods inserted in side grain, the design load per rod should not exceed the design capacity of an equivalent dowelled connection, using the expressions given in EC5 8.5 for bolted connections. These expressions will check both the embedding strength of the timber and the bending strength of the rods, provided that there is no gap between the two connected members. However the embedding strength of the timber may be enhanced, due to the adhesive, by a factor of 1.2. This may be allowed for, when using the accompanying CD to calculate the design resistance, by enhancing the timber density by 20%.

When there is a significant component of load perpendicular to the grain, the possibilities of splitting and shear failure should also be checked (see Sections 6.2.4.3 to 6.2.4.6).

ii) Rods in end grain (Figure 6.7)

For rods in end grain the embedding strength is less than for rods inserted perpendicular to the grain. There may also be a gap between connected members, so that the lateral force is applied at a distance from the end of the timber member. Verify that the total design shear force, $F_{v,d}$, exerted by the rods on the timber, is limited to:

$$F_{v,d} \leqslant \left(\sqrt{e^2 + \frac{2M_{y,k}}{d_{ef}\, f_{h,k}\, k_{mod}}} - e\right)\frac{n d_{ef}\, f_{h,k}\, k_{mod}}{\gamma_M}\ \text{N}$$

Where e = distance between the line of action of the load and the end of the timber (mm) (see Figure 6.7)

$M_{y,k}$ = characteristic yield moment of rod

= $0.3 f_{u,k}\, d^{2.6}$ Nmm, with $f_{u,k}$ = 400Nmm^3 for grade 4.6 mild steel

d_{ef} = $\max.\begin{cases} d_{hole} \\ d \end{cases}$ but not more than 1.25d (mm)

d_{hole} = diameter of hole (mm)
d = diameter of rod (mm)

$f_{h,k}$ = $\left(0.0023 + \frac{0.75}{d^{1.5}}\right)\rho_k$ (N/mm^2)

n = number of rods
γ_M = material safety factor for connections = 1.3
ρ_k = characteristic density of timber (kg/m^3)

For the resistance of the timber to block shear, verify that the total design shear force, $F_{v,d}$, is limited to:

$$F_{v,d} \leqslant \frac{0.7 k_{mod}\, h_v\, l_v\, f_{v,90,k}}{\gamma_M} \text{ N}$$

Where h_v = total shear depth of timber members measured in the direction of the load

= $\sum l_{v,i}$ (mm) (see Figure 6.7)

l_v = $\min.\begin{cases} 10d \\ l \end{cases}$ (mm)

$f_{v,90,k}$ = $f_{v,k}/3$ (the rolling shear strength of timber) (N/mm^2)

d = diameter of rods (mm)
l = embedded length of rods (mm)
γ_M = material safety factor for the timber material (Table 3.19)

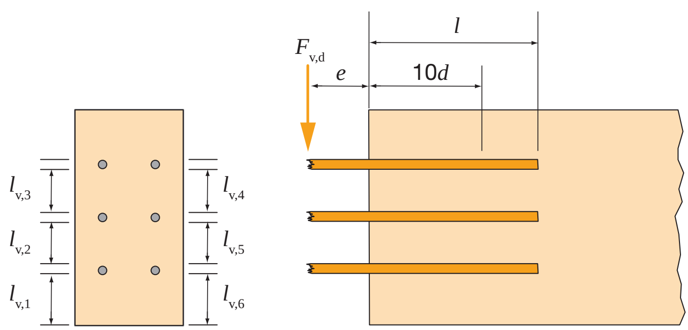


Fig 6.7 Block shear for laterally loaded rods inserted in end grain

6.8.2.4 Axially loaded rods

i) The axial strength of the threaded rod should be checked based on its yield strength and its root diameter (see BS EN ISO 898-1[93] and EC3).

ii) For the strength of the adhesive bond between the timber and rods inserted either parallel or perpendicular to the grain, verify that:

$$F_{ax,d} \leqslant \frac{nk_{mod}\ f_{short}\ \rho_k\ d_{ef}\ l}{\gamma_M} \text{ N, for } l < 200\text{mm} \quad \text{or}$$

$$F_{ax,d} \leqslant \frac{nk_{mod}\ f_{long}\ \rho_k\ d_{ef}\ \sqrt{l}}{\gamma_M} \text{ N, for } l \geqslant 200\text{mm}$$

Where n, ρ_k, d_{ef} and l are defined as previously

f_{short} = 0.037 for epoxy resin, PRF and other brittle adhesives
= 0.046 for 2-part polyurethane and other non-brittle adhesives

f_{long} = 0.520 for epoxy resin, PRF and other brittle adhesives
= 0.650 for 2-part polyurethane and other non-brittle adhesives

γ_M = material safety factor for the timber material (Table 3.19)

iii) For rods inserted parallel to the grain, $F_{ax,d}$ must not exceed the compressive or tensile strength of the effective area of the member behind the rods. Verify that:

$$F_{ax,d} \leqslant \frac{k_{mod}\ f_{ax,k}\ b_{ef}\ h_{ef}}{\gamma_M}$$

Where $f_{ax,k}$ = characteristic tension or compression strength, as appropriate (N/mm²)

b_{ef}, h_{ef} = effective breadth and depth of rod group (mm) (see Figure 6.8a))

γ_M = material safety factor for the timber material (Table 3.19)

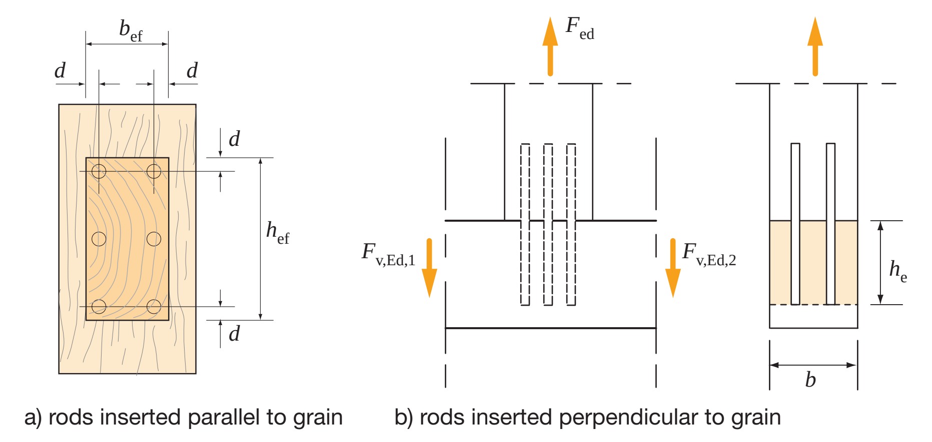


Fig 6.8 Effective cross-sectional dimensions for axially loaded rods

iv) For rods inserted perpendicular to the grain, $F_{v,d}$ should not exceed the shear strength of the timber over the effective cross-section of the timber.
Verify that:

$$F_{v,Ed} \leqslant F_{90,Rd}$$

Where $F_{v,Ed}$ = maximum shear force produced in timber member

$$= \max.\begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases} \text{ (see Figure 6.8b))}$$

$$F_{90,Rd} = \frac{2bh_e \; f_{v,k} \; k_{mod}}{3\gamma_M}$$

b = breadth of member loaded in shear (mm)

h_e = embedded length of rods (mm)

6.8.2.5 Bending

For end connections made with rods inserted parallel to the grain and loaded in bending it is normal to design the rods in tension and compression to resist the bending moment. If the embedded length of the rods is equal to about 4 times the depth of each member the following simplified method may be used.

i) calculate the bending resistance of the rods as:

$$M_{d,rods} = \frac{nM_{y,k}}{\gamma_{M,\,steel}} \text{ Nmm}$$

Where n = number of rods

$M_{y,k}$ = characteristic yield moment of steel as in Section 6.8.2.3 ii)

$\gamma_{M,steel}$ = 1.2

ii) calculate the bending resistance of the bonded rods in the timber as:

$$M_{d,\,embedment} = \frac{nd_{ef}\, l^2 f_{h,k}\, k_{mod}}{6\gamma_M} \text{Nmm}$$

Where l = embedded length of rods in one member (mm)
and the other symbols are as defined in Section 6.8.2.3 ii)

The applied bending moment M_d must not exceed $M_{d,rods}$ or $M_{d,embedment}$.

6.9 3-dimensional nailing plates

3-dimensional nailing plates are made of light-gauge mild steel cut and folded to shape and pre-punched with holes for specified nails. Figure 6.9 shows some examples. The most common kinds are joist hangers, truss clips and angle brackets. 3-dimensional nailing plates should conform to the requirements of ETAG 015[105] which also specifies test methods for obtaining appropriate characteristic values. Currently most manufacturers specify safe working loads for use with BS 5268 but they should be able to provide the original characteristic values for use with EC5 on request.

All 3-dimensional nailing plates should be corrosion resistant, either by means of a specified zinc coating appropriate to the service class for which they are intended or by the specification of stainless steel, in accordance with the recommendations given in ETAG 015.

Characteristic values are derived from tests using the number and type of nails specified by the manufacturer for each product. The Engineer should emphasise the importance of builders complying exactly with the manufacturer's nailing specification.

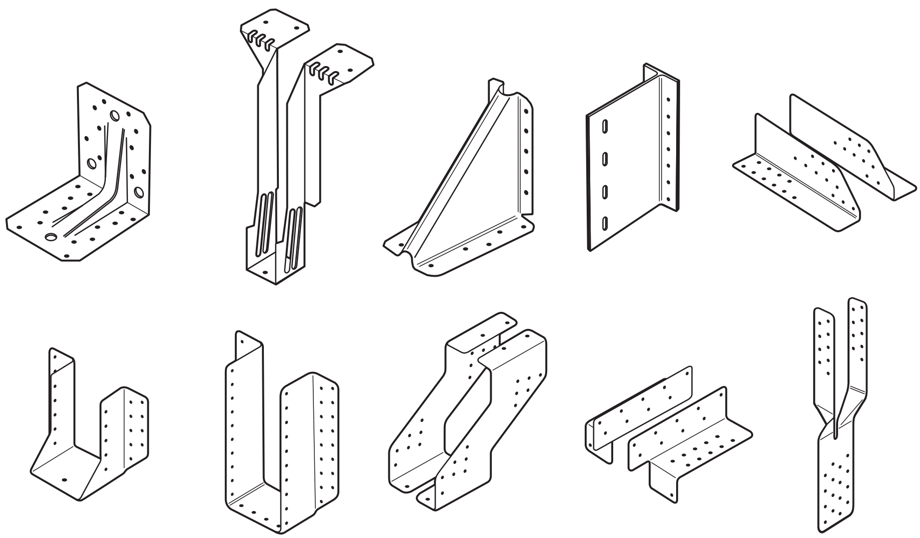


Fig 6.9 Some typical 3-dimensional nailing plates

6.10 Punched metal plate fasteners and nailing plates

6.10.1 Introduction

Punched metal plate fasteners (PMPF) and nailing plates are flat plates made of light-gauge steel, either galvanized mild steel strip or less commonly austenitic stainless steel. PMPF have integral teeth punched out of the material itself, and nailing plates have predrilled nail holes. They are used to joint adjacent solid timber or glulam members of the same thickness in the same plane.

All plate fasteners should conform to the requirements of BS EN 14545[73]. This standard provides specifications for the material and corrosion protection, which should be appropriate to the service class and environment for which they are intended (see Table 3.25). Conformity with BS EN 14545 permits the manufacturer to CE mark the product (see Section 2.1.3).

6.10.2 Punched metal plate fasteners

Also known as nail plates, PMPF are applied in a press or roller to join softwood members together. They are widely used in prefabricated trussed rafters, but are also suitable for timber frame assemblies.

BS EN 14545[73] references test methods (BS EN 1075[85]) for obtaining appropriate characteristic values for PMPF and nailing plates. Most manufacturers are currently updating their product approvals to include characteristic values for use with EC5 in addition to permissible values for use with BS 5268. EC5 8.8 provides a design method for punched metal plate fasteners using these characteristic values. The method is not covered in detail in this *Manual*.

6.10.3 Nailing plates

Nailing plates are commonly used in the form of straps, e.g. to attach joists to a masonry wall, but may also be used in rectangular form to join softwood or hardwood members together. The load capacity can be calculated from the value of an individual steel-to-timber nailed joint taken from the accompanying CD and modified for the number of nails in a line in accordance with Section 6.3.2.1.

6.11 Timber connectors

6.11.1 Introduction

Timber connectors are described in BS EN 912[74]. The three types most commonly used in the UK are toothed plates, shear plates and split rings. They are used in conjunction with bolts to join two members together side by side, enabling higher lateral loads to be carried than with bolts alone. Toothed plates and shear plates can be used in timber-to-timber or steel-to-timber joints, but split rings can be used only in timber-to-timber joints. Toothed plates can be single- or double-sided, shear plates are single-sided, and split rings are double-sided. Toothed plates should be used only in softwoods, in conjunction with a high tensile (e.g. Grade 8.8) steel bolt and nut to embed the teeth. Split rings and shear plates can carry higher loads but require the use of special tools to cut the required recesses in the timber, so the cost of making these connections is higher. Generally only toothplates are suitable for panel products because they are relatively thin. Except for double-sided toothed plates, connectors are demountable.

Table 6.32 lists the types and sizes of timber connector commonly available in the UK and the required bolt diameters. Table 6.33 shows the ways in which the different types can be used. All connectors should conform to a type specified in BS EN 912 and to BS EN 14545[73]. BS EN 912 also gives the dimensions of the grooves which must be cut when using split rings and shear plates.

6.11.2 Design

6.11.2.1 Load-carrying capacity

A comparison of the design values given by EC5 and BS 5268-2[3] indicates that the EC5 rules for the types of connector used in the UK are either unsafe or unproven. Designers are therefore advised not to use them until they have been reviewed. Meanwhile Part 4 *Design values for timber connectors* in the accompanying CD gives a method for converting the basic connector loads given in BS 5268 to design values which can be used in a building otherwise designed to EC5.

Table 6.32 UK connector types and sizes				
Connector (Load capacity)	**Description**	**Type to BS EN 912**[74]	**Nominal diameter or side length (mm)**	
Toothed plate (Low-medium)	Round, double-sided	C.6.1	38, 50, 63, 75	
	Round, single-sided	C.7.1	38, 50, 63, 75	
	Square, double-sided	C.8.1	38, 50, 63, 75	
	Square, single-sided	C.9.1	38, 50, 63, 75	
Split ring (Medium-high)	Parallel sides	A.2.1	64	
	Double bevelled sides	A.3.1	64, 102	
Shear plate (High-very high)	Mild steel	B.2.1	67	
	Cast iron	B.3.1	102	
Required bolt diameters				
Nominal diameter or side length of connector (mm)		38	50 - 75	102
Grade 4.6 bolt diameter (mm)		M10	M12	M20

Note

a For the sizes of bolt holes and washers, see Sections 12.3.4 and 6.5.1 respectively.

6.11.2.2 Spacing rules

See EC5 8.9(9) to 8.9(12), 8.10(7) and 8.10(9).

6.11.2.3 Residual cross-section

For split rings and shear plates the residual cross-section A_{net} should be used to check the strength of the timber in tension, compression or shear. The residual cross-section is the cross-sectional area of the member minus both the projected area of the connector groove(s) and the projected area of the bolt hole lying outside the projected areas of the groove(s), i.e.:

$$A_{net} = A - \left[nA_{groove} + (b - nh_{groove})d_{hole}\right]$$

Where A = gross area of cross-section of member

n = number of connectors on the bolt which penetrate the member (1 or 2)

A_{groove} = projected area of one groove (see Table 6.34)

b = breadth of member measured parallel to bolt

h_{groove} = depth of one groove measured parallel to bolt (see Table 6.34)

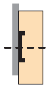
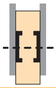
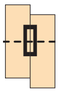
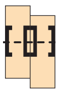
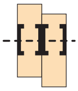
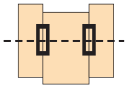
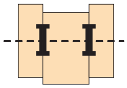
d_{hole} = diameter of bolt hole

Where multiple connectors are used, see Section 6.2.4.7.

6.11.2.4 Block shear and plug failure

The possibility of block shear and shear plug failure (EC5 Annex A) should be considered.

Table 6.33 Uses of timber connectors

Configuration	Materials	Single (S) or double (D) sided		
	S = steel T = timber	Toothed plates	Shear plates	Split rings
	S-T	S	S	
	S-T-S	SS	SS	
	T-T			D
	T-T[a]	SDS		
	T-T[b]	SSSS	SSSS	
	T-T-T	DD		DD
	T-T-T	SSSS	SSSS	

Notes

a The exterior plates are not essential but can resist failure caused by twisting of the bolt.

b Joints made with timber connectors are demountable except in the case of double sided toothplates.

Table 6.34 Projected areas and depth of connector grooves

Connector type	Diameter (mm)	Projected area of one groove A_{groove} (mm²)	Depth of one groove h_{groove} (mm)
Split ring	64	705	9.5
	102	1455	12.7
Shear plate	67	770	11.5
	102	1690	16.5

6.12 Proprietary connectors

Timber engineering hardware manufacturers produce not only a large range of 3-dimensional nailing plates, but also a range of special devices which can simplify assembly, provide neat solutions, and in many cases save money overall. Designers are advised to familiarise themselves with some of the options available by looking at the catalogues of some of the major suppliers. A list of company names is provided on the CD.

6.13 Joint slip

6.13.1 Introduction

Joint slip, the relative movement between two laterally loaded connected members, can result in lateral or rotational displacements of the members. These can produce undesirable displacements in the members themselves, or, in the case of assemblies, a redistribution of stresses. Slip is negligible in glued joints, and EC5 gives no information about the slip in connections made with punched metal plate fasteners.

6.13.2 Slip modulus

For connections made with dowel-type fasteners and timber connectors slip is calculated using a slip modulus, K_{ser} for SLS and $K_u = 2K_{ser}/3$ for ULS. K_{ser} is measured in Newtons per mm per shear plane per fastener. Formulae for calculating K_{ser} are given in Table 6.35.

Table 6.35 Values of K_{ser} for fasteners in timber-to-timber and wood-based panel-to-timber connections, from EC5 Table 7.1[a]

Fastener type	K_{ser} (N/mm)
Nails without predrilling Small wood screws ($d \leqslant$ 6mm) without predrilling	$\frac{\rho_m^{1.5} d^{0.8}}{30}$
Wood screws with predrilling Bolts[b] Dowels	$\frac{\rho_m^{1.5} d}{23}$
Split ring and shear plate connectors	$\frac{\rho_m d}{2}$[c]
Toothed plate connectors	$\frac{\rho_m d}{2.67}$[c]

Notes

a ρ_m = mean density of timber (see Tables 3.14 to 3.18)

d = diameter of round nail or side length of square nail, nominal diameter of screw, diameter of bolt or dowel, or nominal diameter or side length of a timber connector (see BS EN 13271[106])

If the mean densities $\rho_{m,1}$ and $\rho_{m,2}$ of two connected wood-based members differ then $\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}}$. For steel-to-timber or concrete-to-timber connections use ρ_m for the timber member and multiply K_{ser} by 2.

b For bolts the clearance ($d_{hole} - d_{bolt}$) should be added to the calculated slip.

c These expressions from EC5 differ from those given in BS EN 13271.

6.13.3 Instantaneous slip

For SLS verifications the instantaneous slip is calculated as:

$$u_{\text{inst}} = \frac{F_{\text{ser}}}{K_{\text{ser}}} \text{mm}$$

Where F_{ser} = SLS design load per shear plane per fastener (N) (see Section 3.2.2.2i))
K_{ser} = SLS slip modulus (see Section 6.13.1)

For ULS verifications the instantaneous slip is calculated as:

$$u_{\text{inst}} = \frac{F_{\text{u}}}{K_{\text{u}}} \text{mm}$$

Where F_{u} = ULS design load per shear plane per fastener (N) (see Section 3.2.1.3)
K_{u} = ULS slip modulus (see Section 6.13.2)

6.13.4 Final slip

Final slip, the slip after creep occurs, should also be checked, particularly when it may affect serviceability. Like final deflection, it depends on the values of k_{def} for the materials involved (see Section 2.17.2). Although there is very little research evidence to support it, EC5 requires all values of k_{def} to be doubled when calculating the final slip in connections. Hence:

$k_{\text{def,joint}}$ = $2k_{\text{def}}$ in connections between two timber materials having the same creep properties

$k_{\text{def,joint}}$ = k_{def} in connections between a timber material and steel

$k_{\text{def,joint}}$ = $2\sqrt{k_{\text{def,1}}\, k_{\text{def,2}}}$ between two timber materials 1 and 2 having different creep properties.

The final slip can be calculated as:

$$u_{\text{fin}} = u_{\text{inst}} + k_{\text{def, joint}} \left(\sum u_{\text{inst,G,j}} + \sum \psi_{2,\text{i}}\, u_{\text{inst,Q,i}} \right) \text{ (see Section 3.2.2.3).}$$

To determine the final forces, moments and deflections in assemblies with mechanically fastened connections, use final stiffness moduli, $K_{\text{ser,fin}}$ as specified in Section 3.2.3.

6.13.5 Allowing for slip in frame analysis programs

To allow for slip in frame analysis programs, see Section 3.2.4.

6.14 Connections in fire

Adhesively bonded joints may be assumed to retain adequate strength and stiffness in a fire provided that they are properly made with a suitable adhesive (see Table 6.29).

Mechanically fastened connections can fail through loss of strength in the steel or through charring of the timber caused by the conduction of heat through the fastener from the surface. They can also lose their stiffness, allowing increased lateral movement or rotation between connected elements. Unprotected connections can fail in 15 minutes (see EC5-1-2 Table 6.1) and nail plates even faster. Therefore where a specified period of fire resistance is required it will be necessary to cover exposed metal fasteners and other connecting metal-work with an insulating material to prevent an unacceptable rise in temperature. Suitable insulating materials are wood-based panels, gypsum plasterboard type A, F or H, and cork. They may be attached with an adhesive as above, or by nails or screws in accordance with EC5-1-2 Clauses 6.2.1.2(6) and (7). Connections in which the fasteners are protected by gypsum plasterboard type F will have 30 minutes' fire resistance, provided that the fastener diameter is not less than 2.8mm for nails or 3.5mm for screws, and that the timber thickness is not less than 45mm for bolts, dowels and timber connectors. Alternatively the heads of nails, screws and bolts, and washers and nuts, may be countersunk from the timber surface and the recess above them filled with a glued-in timber plug of sufficient thickness to prevent failure in accordance with EC5-1-2 6.2.1.2(3). Steel dowels shorter than the combined thickness of the connected members may be protected in a similar manner. Sealing of the joints between members by means of a suitable intumescent sealant or resin should also be specified to prevent the ingress of hot gases.

Slip moduli (see Section 6.13.2) for mechanically jointed connections in fire should be calculated as the ultimate slip modulus, $K_u = 2K_{ser}/3$, multiplied by 0.2 for connections made with nails or screws, or by 0.67 for connections made with bolts, steel dowels or timber connectors.

For more detailed design rules refer to EC5-1-2 Section 6. This covers:

- how fire resistance of up to 30 minutes can be obtained with unprotected mechanical fasteners
- how to calculate the required thickness of protection for protected mechanical fasteners for a specified period of fire exposure
- rules for minimum edge distances for mechanical fasteners and internal steel plates
- connections made with external steel plates
- axially loaded screws.

7.1 General design requirements

7.1.1 Functions of a roof

A roof has to support the weight of tiles or other cladding and any associated components, its self-weight including the weight of any ceiling and permanent fixtures, specified imposed loads including snow, and the horizontal and uplift forces applied by the wind. It must transfer these loads safely to the building beneath it within agreed deflection limits. It must be stable enough to provide lateral support to the supporting walls and, in the case of pitched roofs on gable walls, to the spandrels. Its attachment to the building must be adequate to resist uplift and sliding forces. It must shelter the exterior of the building from rain and snow, keep the interior dry with adequate ventilation, and prevent excessive loss of heat. It should remain serviceable throughout the design life of the building.

To ensure adequate horizontal diaphragm action there must be sufficient bracing at both ceiling level and in the plane of the rafters, either in the form of diagonal bracing members or an attached sheet material. Slender assemblies such as trussed rafters may require additional bracing to the compression members. Note that the internal edge of rafters may be subject to compression stress when there is roof uplift, and without lateral restraint this could considerably reduce the factor for lateral buckling, k_{crit}.

The ceiling diaphragm generally has to transfer the net horizontal wind load on the roof and the upper half of the top storey to the racking walls at right-angles to the windward face of the building.

7.1.2 Data required

Before designing a roof the Engineer should assemble the following data:

- site location, height, ground roughness and reference to any unusual wind conditions
- overall site plan indicating any adjacent buildings or features which might affect the wind loading
- height of building from ground level to eaves
- building type and whether access to the roof is required for purposes other than maintenance or repair
- intended use of roof space
- reference to any unusual environmental conditions which might affect steel or timber
- the type of any preservative treatment required
- plan and elevations of roof including overhangs and other eaves details, window lights, hatches, stairwells, chimney, and support details (nature, position and breadth) including intermediate supports (e.g. load-bearing walls)
- type and weight of roof tiles or covering
- weight of any sarking, insulation materials and plasterboard
- the size and position of all water tanks

- the weight and position of any permanent ancillary equipment to be supported on the ceiling joists
- preferred spacing of rafters
- any limitations on member size, e.g. to accommodate insulation or to match existing members, or minimum thicknesses for fixing ceiling boards or sarking
- rafter bracing method to be used (solid timber bracing or sarking using a specified panel product, or possibly steel ties in the case of larger roof structures)
- limitations on vertical deflection for rafters and ceilings joists, and on horizontal deflection at the eaves relative to the gable walls
- any unusual site conditions (e.g. low loading limit) which may affect the design and assembly method.

It is advisable to allow an additional 25mm of length on the underside of ceiling ties or joists beyond the outer edge of each wall plate to cater for inaccuracies in construction.

If the roof is to be designed by a specialist roof designer, the information above should be passed on to the designer, preferably in the form of detailed drawings. It is recommended that its design be carried out using the same codes as those used for the main building, i.e. Eurocodes 0, 1 and 5.

If the design and manufacture of the roof are to be carried out by a trussed rafter manufacturer then the number of similar buildings to be built should also be specified. This may affect the quoted cost.

7.1.3 Actions

7.1.3.1 Summary

EC1-1-1 tabulates typical material weights and specifies the minimum imposed loads on roofs. EC1-1-3 specifies snow loads and EC1-1-4 specifies wind loads.

Table 4.5 lists the most of the material weights required to design a roof. A summary of the dead and imposed loads to be considered is given in Table 7.1, and some example load cases are illustrated in Table 7.2.

7.1.3.2 Guidance on imposed loads

For load durations see Table 2.2.

EC1-1-1 6.3.4.2(4) states that roofs, other than those with roof sheeting, should be designed to resist 1.5kN spread over a 50mm sided square. However the intention of this clause is to state that the roofing material should be able to support a concentrated load spread over a 50mm sided square rather than specify a load of 1.5kN. The UK NA specifies a concentrated load of **0.9kN**.

In a load-sharing situation, i.e. when k_{sys} = 1.1 (see Table 3.20), it may be assumed that only 75% of the concentrated load on the rafters or ceiling joists is applied to a single member, the remainder being distributed to the adjacent members.

The water tank load is best spread over three or four ceiling ties or joists by means of suitably sized spread members. For a 230 litre tank, contents and spreader members allow 2.7kN, for a 300 litre tank 3.6kN.

When checking for roof uplift, all variable loads other than wind should be taken as zero. The weight of the roof covering may be included provided that it is heavy enough or is fixed securely enough to the rafters to ensure that it does not lift off itself in any areas of high local pressure. If the weight of the roof covering is not known exactly then the minimum possible value should be used for roof uplift calculations.

7.1.4 Bracing
The requirements for bracing the rafters depend on the type of roof construction and are dealt with in the subsequent sections which relate to roof types. For diaphragm action in the ceiling see Section 5.8.

7.1.5 Raised tie trusses
In both trussed rafters and cut roofs raised tie trusses are sometimes specified to increase the ceiling height. In raised tie trusses the tie is above eaves level, so the ends of the rafters tend to splay outwards. When such trusses are supported on masonry walls their outward horizontal movement should be limited to prevent damage to the walls. A simple rule of thumb which has been found to work in practice is to limit the instantaneous deformation on each side under dead + imposed loads to 6mm, for walls at least 1.8m high measured from floor to eaves. The truss can be analysed with one pinned foot and the other free to move 12mm horizontally. The horizontal deflection will depend on bending in the rafters below the tie, elastic extension of the tie and slip in the tie/rafter connections. For a precise calculation allowance should be made for creep in the rafters and tie and for final slip in the connections.

7.1.6 Serviceability
For recommended deflection limits see Table 3.4.

7.2 Flat roofs
A flat roof is defined in EC1-1-4 7.2.3 for wind loading purposes as a roof with a pitch < 5°. However, to ensure proper drainage a minimum slope of 5° is recommended.

Flat timber roofs, like timber floors, may be supported with joists made of solid timber, glulam, LVL or other structural timber composites, or with engineered timber joists. Timber decking normally consists of plywood or OSB, but P5 or P7 particleboard may also be used. Panel products must be certified for use as roof decking (see Table 3.27 and its note **d**).

Flat roofs may have cold decks or warm decks. In a cold deck roof the insulation is immediately above the ceiling so the decking is cold and must be suitable for service class 2 (humid) conditions. Correct architectural detailing is essential to prevent condensation in the roof void. Ventilation requirements limit the recommended spans for this type of roof to 5m and Scottish Building Regulations prohibit their use. In a warm deck roof the insulation is above the structural decking, which is therefore in service class 1 if the building is heated.

Decking and joists should be designed as normal bending members (see Section 5.2).

The fixing of joists and decking should be carried out in the same way as for floors (see Section 12.6.3).

Table 7.1 Roof loads (based on BS 5268-3)

I.D.	Action	Position
1	Dead load	Full length
2a	Uniformly distributed snow	Full length
2b	Asymmetrically distributed snow if relevant	As specified in EC1
2c	Exceptional snow drifts (Accidental)	Full length
3a	Concentrated load on rafters – access only for maintenance or repair (EC1-1-1 Category H)	Centre of any rafter bay
3b	Concentrated load on rafters – accessible to building users (EC1-1-1 Category I)	Centre of any rafter bay
3c	Distributed load on rafters – access only for maintenance or repair (EC1-1-1 Category H) Measured on plan	Full length
3d	Distributed load on rafters – accessible to building users (EC1-1-1 Category I)	Full length
4	Wind at 0° and 90°	Full length
5	Ceiling dead load	Full length
6a	Water tank point load	Distributed over two ties or joists beneath tank, at nearest node if trussed rafters
6b	Fixed plant and special services	Where applied
7a	Distributed load on ceiling ties not designed to act as floors	Full length
7b	Distributed load on floor joists in attic roof	Full length
8a	Point load on ceiling ties not designed to act as floors	Centre or end of ceiling bay, whichever is more onerous
8b	Point load on floor joists in attic room	Wherever most onerous

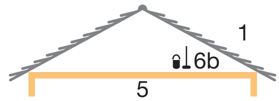
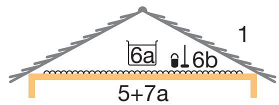
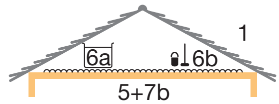
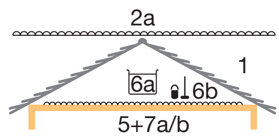
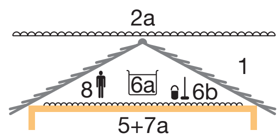
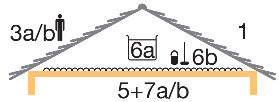
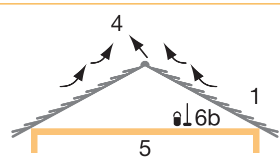
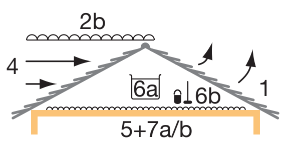
	Value	Duration
	See Table 4.5	Permanent
	From EC1-1-3 and NA	Short-term
	From EC1-1-3 and NA	Short-term
	From EC1-1-3 5.2(3)c and NA	Instantaneous
	0.9kN as specified in EC1-1-1 NA Not at the same time as the distributed load	Short-term
	As specified in EC1-1-1 NA for floors, balconies, etc. in the corresponding building class. Not at the same time as the distributed load	Short-term
	$\alpha < 30°$ q_k = **0.6 kN/m²** $30° \leqslant \alpha < 60°$ q_k = **0.6 [(60-α)/30] kN/m²** $\alpha \geqslant 60°$ q_k = **0.0 kN/m²** as specified in EC1-1-1 NA. (Note that q_k may be zero.)	Short-term
	As specified in EC1-1-1 NA for floors, balconies, etc. in the corresponding building class	Short-term
	From EC1-1-4	Instantaneous
	See Table 4.5	Permanent
	2 x 0.45 kN point loads or actual weight including water if greater	Long-term
	Actual weight	Permanent
	0.25 kN/m². (EC1-1 6.4.3.2(8))	Long-term
	1.5 kN/m² as specified in EC1-1-1 NA [a]	Medium-term
	0.9kN at same time as kN/m² distributed load	Short-term
	2.0kN (EC1-1-1 NA[b]) For local effects only, not simultaneously with 7b	Medium-term

Notes

a For domestic and residential activities, excluding hotel bedrooms, hospital wards and toilet areas.

b 2.0kN on a 100mm square for domestic and residential activities, excluding billiard rooms.

Table 7.2 Example load cases

Case	Duration	Loads from Table 7.1[a]		Illustration
		Rafter	Ceiling tie	
Dead	Permanent	1	5 + 6b	
Dead + long-term	Long-term	1	5 + 6a + 6b + 7a	
Dead + medium-term	Medium-term	1	5 + 6a + 6b + 7b	
Dead + snow	Short-term	1 + 2a/b/c	5 + 6a + 6b + 7a/b	
Person on ceiling joist	Short-term	1 + 2a/b/c	5 + 6a + 6b + 7a + 8	
Person on rafter	Short-term	1 + 3a/b	5 + 6a + 6b + 7a/b	
Wind uplift	Instantaneous	1 + 4	5 + 6b	
Wind sway + asymmetric snow	Instantaneous	1 + 2b + 4	5 + 6a + 6b + 7a/b	

Note

a ψ values may be used for an optimal solution (see Section 3.1.4).

7.3 Trussed rafters

7.3.1 Introduction

7.3.1.1 Trussed rafters

Trussed rafters are structural frames which are individually designed to support roofs and ceilings, principally at a spacing of 600mm. They are manufactured from strength graded timbers which are joined together with punched metal plate fasteners. They can be manufactured in spans up to about 22m, and longer spans can be accommodated by splicing two or more sections together on site. The height can be extended by stacking two or even three sections on top of one another and fastening them securely together. EC5 does not set any maximum span for a specified thickness, nor limit the maximum length of a member by its thickness and depth as BS 5268-3[43] did.

7.3.1.2 Girder trusses

A girder truss or principal trussed rafter is a multiple trussed rafter which:

- supports a length of roof (measured perpendicular to its span) greater than 1.5m or greater than 2.5 times the spacing of the regular trusses in the roof; or
- directly supports other trusses in such as way as to cause eccentricity in the loads applied to it; or
- supports another girder truss.

The trussed rafters which make up a girder truss should be permanently fastened together, preferably at the place of manufacture where they can be kept perfectly flat, with fastenings designed to ensure common action under load. If a load is applied eccentrically to the bottom chord of a multiple truss then the connections between the members should be designed to permit an equal share of the load to be transferred to each member. In determining the radius of gyration of the compression members, account should be taken of the effectiveness of the fastenings used, particularly if these are to be applied on site. For instructions on site fixing see Section 12.6.2.

7.3.2 Design information

Trussed rafter roofs are normally designed by the roof truss manufacturer, who will require the information listed in Section 7.1.2. The Engineer in turn should obtain the following output information from the roof designer:

- the basis of design, including any design assumptions made not covered below
- detailed drawings showing all trussed rafters in the roof and their positions and spacing
- timber strength classes or grades and species, and cross-sectional dimensions
- the type, sizes and positions of all jointing devices with tolerances, or the number of effective teeth or nails required in each member at each joint
- the positions and sizes of all bearings
- the loadings and other conditions for which the trussed rafters have been designed
- the positions, fixings and sizes of any lateral supports necessary to prevent buckling of compression members such as rafters and struts

- the location and support method for tanks and ancillary equipment or loads, plus the capacity and magnitude of any additional loads assumed, e.g. weight of water
- the reactions to be accommodated at the bearings for each separate action (see Table 7.1) or load case (see Table 7.2) including asymmetrical snow loads and exceptional snow drifts where relevant
- maximum initial and final deflections of rafters and ceiling joists
- instructions concerning the fixing of any girder trusses or other special connection details.

7.3.3 Stability

The Engineer is responsible for designing the bracing to provide stability to the roof and the supported walls including gable wall spandrels, and adequate methods for holding the roof down to resist uplift and sliding.

Guidance on bracing in the plane of the rafters and the ceiling of trussed rafter roofs which fall within certain dimensional limits can be found in BS PD 6693[2]. Outside these limits the roof designer should design the rafter bracing in accordance with EC5 9.2.5.3 and the ceiling bracing using the Eurocode 5 method described in Section 5.8.2.

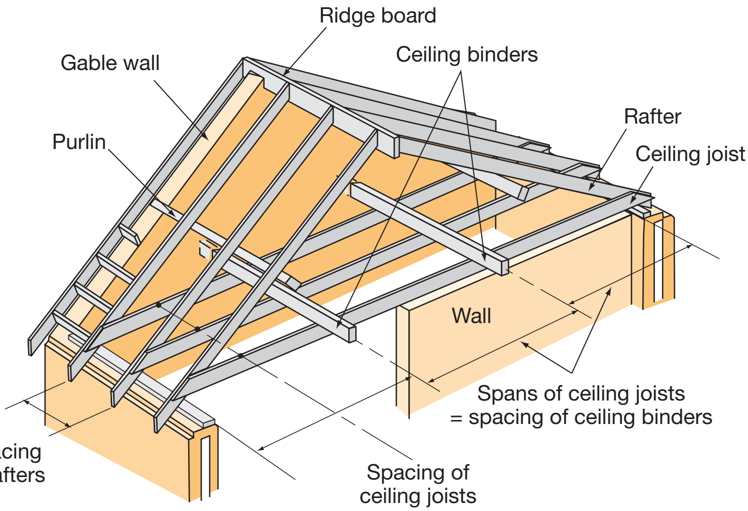
7.3.4 Notching and drilling

Except for the trimming of overhangs and flying rafters, the cutting, notching and drilling of trussed rafters is not permitted unless otherwise advised by the supplier. However holes in the bottom chord of an attic trussed rafter are permitted within strict limitations (for details consult the Trussed Rafter Association).

7.4 Site built cut roofs

Traditional roofs in which the members are cut to size on site are often referred to as cut roofs. They are based on tied rafters at spacings of 400mm, 450mm or 600mm, with purlins to support the rafters at mid- or third-span points and ceiling binders to support the ceiling ties (see Figure 7.1). The rafters are usually 47mm thick, as against the 35mm most commonly used for trussed rafters. Bracing in the plane of the rafters is normally considered unnecessary if a conventional roof covering involving tiles and battens is used.

Rafters are normally designed on the assumption that they support each other at the ridge via only a horizontal reaction equal to $F_{vert,d}\cos\alpha\sin\alpha/2$ for a single-span rafter or $3F_{vert,d}\cos\alpha\sin\alpha/8$ for a 2-span rafter, where $F_{vert,d}$ is the design weight on the rafter or on the span adjacent to the ridge respectively including its self weight, and α is the slope of the roof. This means that the ridge board is not designed as a beam: it is principally for location purposes. However the nailing of the ridge board to the rafter should be sufficient to support the vertical component of the reaction to snow on only one side of the roof, which is $F_{vert,snow,d}\cos^2/2$ for a single-span rafter or $3F_{vert,snow,d}\cos^2\alpha/8$ for a 2-span rafter, where $F_{vert,snow,d}$ is the total design snow load on the rafter or on the span adjacent to the ridge respectively. (Two 3.35mm diameter nails would be a typical minimum requirement.)



Note
Spans of purlins and ceiling binders are not shown

Fig 7.1 A traditional cut roof

The axial compression stress required for combined bending and compression calculations should be calculated as follows:

- for single span rafters at the mid-point of the span

$$\sigma_{c,0,d} = \frac{F_d}{2bh}(\cot\alpha + 3\tan\alpha)$$

- for continuous rafters at a purlin

$$\sigma_{c,0,d} = \frac{3F_d}{8bh}\left(\cot\alpha + \frac{8}{3}\tan\alpha\right)$$

- for continuous rafters at the mid-point of the span

$$\sigma_{c,0,d} = \frac{3F_d}{8bh}\left(\cot\alpha + \frac{13}{3}\tan\alpha\right)$$

Where F_d = component of $F_{vert,d}$ resolved perpendicular to rafter where $F_{vert,d}$ is the design weight on the rafter or on the longer span

α = slope of rafter

b = breadth of rafter

h = depth of rafter

From the Engineer's point of view it is simple to design canted purlins in which the major axis is perpendicular to the rafter axis: these are considered to support the rafter in bending only. In practice it is generally better to set the purlins vertically, notching the rafter in a 'bird's mouth' so that it can transmit both shear and axial compression to the purlin. Vertical purlins can be more easily supported at the ends (e.g. in joist hangers) and do not tend to sag so much at mid-span. In this case the purlin has to be designed as a member in double bending in accordance with Section 5.2.1.1.

The ceiling joists must be designed as ties to resist the horizontal component of the spreading load of the roof.

In hip roofs it is difficult to calculate the bending stress in the hip rafter properly, but in practice it is negligible if the roof slope is 30° or more. This is because the rafters, battens and tiles cause the three adjoining roof planes to act as mutually supporting triangular plates. Although it is not strictly necessary if the three planes are adequately stitched together along their mutual edges by their attachment to the hip rafter, it is advisable to provide horizontal restraint at the lower end of the rafters at the hip end by attaching them to the wall plate and by attaching the wall plate to any ceiling binders. In the case of a single hip end extension or a hip-ended semi-detached house in which the ceiling binders do not span between two opposing hip ends it is advisable to restrain the ceiling binders themselves in some other way.

For roof slopes less than 30° a hip needs to be propped or designed as a bending member otherwise the horizontal reactions at the eaves may cause the walls to move outwards. A simple but conservative approach is to assume that each jack rafter on each side of a hip rafter applies half of the roof weight which it supports and half its own weight to the hip rafter in the form of a vertical point load. Thus there is a series of vertical point loads on the hip rafter which increase in magnitude up the hip rafter as the length of the jack rafters increases. In this case the point loads resolve into a triangular bending load on the hip rafter and an axial compression force down it which must be suitably restrained at the lower end.

Whereas the vertical reaction at the lower end of the jack rafters at the front and back of the building can be provided via the ceiling tie, the only support that the jack rafters at the hip end have is a bird's mouth onto the wall plate. The size of this should be large enough to ensure that the compression strength of the wall plate is not exceeded but not so large that the shear strength of the rafter is compromised.

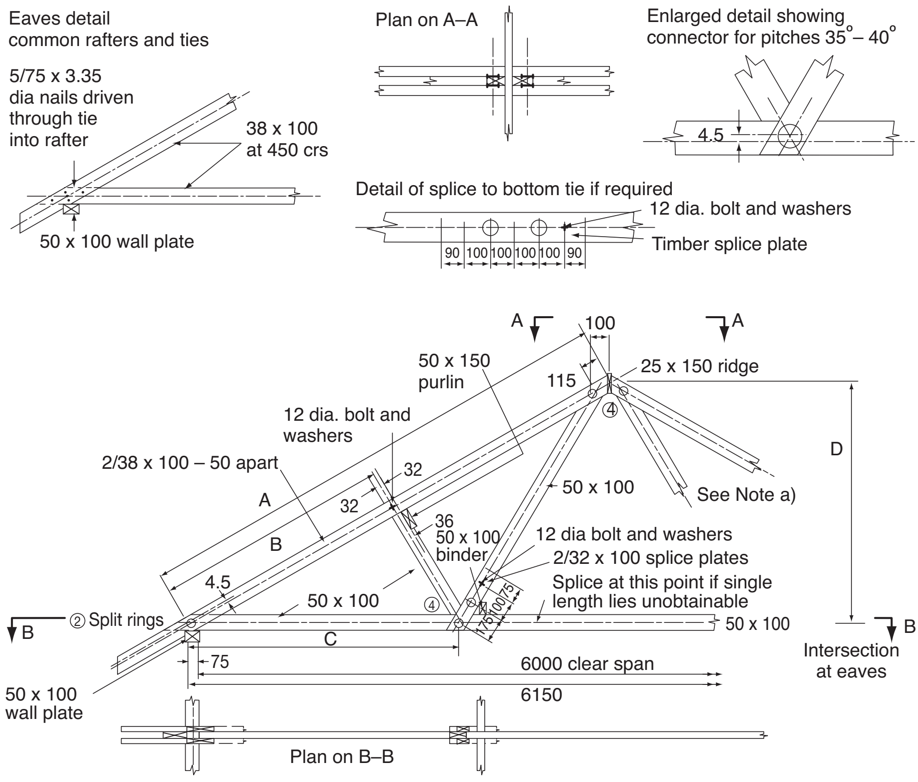
The wall plates at each corner of a hip roof should be connected together so that they can act as a ring beam in tension, as in a pyramid roof. This connection has traditionally been provided by a 'dragon tie'[107].

Valley rafters between the main roof and dormer windows should be designed as bending members. Where the valley is formed by an incoming roof the valley member only has to be designed as a bending member if the rafter lengths covered by the incoming roof have been removed. However because the slope of valley members is less than that of the main roof, roof repairers often walk up them, and since excessive deflection will crack GRP flashing they should be designed to ensure that their instantaneous deformation under a 0.9kN point load does not exceed $l/333$. As a guide they should be twice as thick as the main rafters and 50mm deeper.

Typical construction details for cut roofs can be found in *Roof Construction and Loft Conversion*[107]. Bracing in the plane of the rafters is normally considered unnecessary if a conventional roof covering involving tiles and battens is used.

7.5 Timber trusses

Substantial timber trusses can be built using bolts, dowels, timber connectors, flitch plates and bolts or dowels, or splice plates and bolts to join solid timber or glulam members together, in any of the configurations suggested for trussed rafters in Figure 4.2. With flitch and splice plates all the principal members can be in one plane, but with other types of connection some of the members have to be doubled. For example a single 50mm × 100mm ceiling tie might be fixed between two 38mm × 100mm rafters by means of two split ring connectors (see Figure 7.2). Spans of up to 12m can be achieved by installing timber trusses at 1800mm centres with three intermediate common rafters supported by mid-span purlins and ceiling joists supported by binders. Glulam trusses can span very much further. A frame analysis is generally required to determine the applied forces and bending moments.



Note

a Camber top chords equal to 9mm per 3m of span
Camber bottom chords equal to 12mm per 3m of span

Fig 7.2 Typical engineered timber truss spanning a 6m domestic roof

When used in conjunction with common rafters the system factor k_{sys} is not applicable, but it may be applied for trusses used at centres not exceeding 1200mm in conjunction with tiling battens, purlins or panels which are continuous over at least two spans and have staggered joints. With these fairly substantial trusses bracing in the plane of the rafters is normally considered unnecessary if a conventional roof covering involving tiles and battens is used.

7.6 Pyramid roofs

Pyramid roofs such as the one shown in Figure 7.3 can easily be constructed in timber materials such as glulam or LVL. Diagonal spans of 25m are achievable, or more with large glulam sections. A key requirement is to connect the ring beams at the base of the roof together at each corner so that they can act in tension to prevent the four roof planes from sliding apart. One way to do this is shown in the Figure 7.3. The steel shoe illustrated allows both the horizontal and vertical components of the hip rafter reaction to be resisted by direct bearing while restraining the axial tension in the ring beams via steel dowels and flitch plates.

7.7 Gridshells

Timber has been successfully used to create dramatic gridshell roofs, sometimes using innovative connection techniques. Their design is outside the scope of this *Manual*.

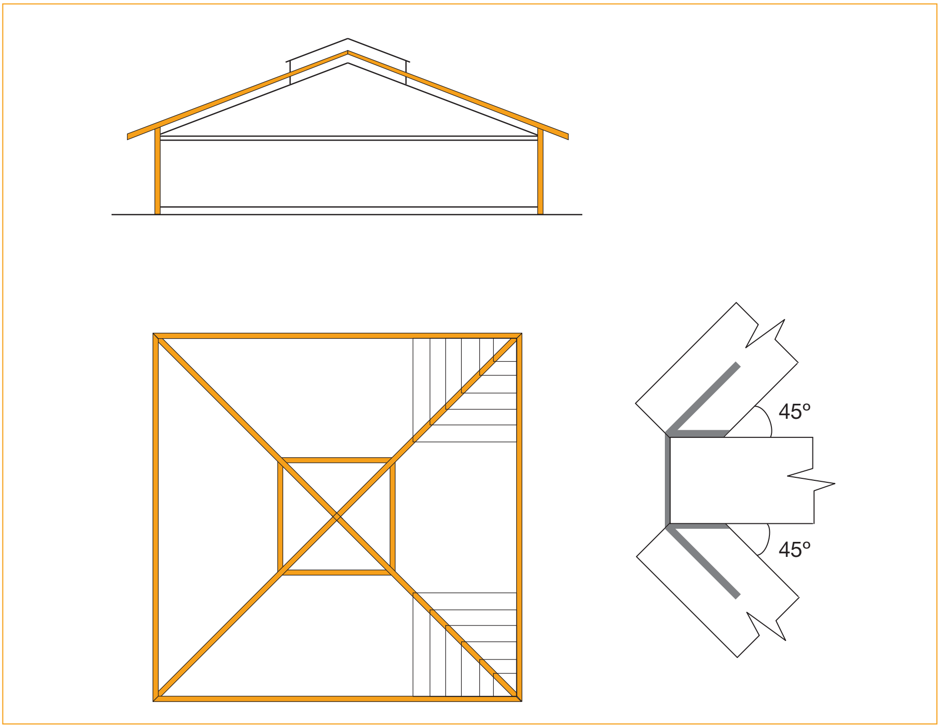


Fig 7.3 A pyramid roof and eaves connection

8.1 General design requirements

8.1.1 Functions of a floor

Floors must resist vertical loads, be satisfactory with respect to vibration, and act as horizontal diaphragms to transmit wind loads to the shear walls. Building Regulations have additional requirements for fire resistance and acoustic attenuation.

8.1.2 Actions

EC1-1-1 tabulates typical material weights and specifies the minimum imposed loads on floors. EC1-1-4 specifies wind loads.

The weight of a timber floor may be determined:

- by precise calculation based on the densities of the materials used from EC1-1 or manufacturer's data
- from values given in Table 4.5
- by using generally accepted values as follows:
  - 0.50 kN/m^2 for timber ground floors and intermediate floors without partitions
  - 0.75 kN/m^2 for an intermediate floor with timber stud partitions in undefined positions
  - 1.25 kN/m^2 for a party floor.

EC1-1-1 Clause 6.2.1(3) states that the imposed point load and the uniformly distributed imposed load should not be considered to act simultaneously. The point load is intended to check local resistance and should normally be considered as acting on a 50mm square. For load durations, see Table 2.2. Where it is possible that moveable partitions with a weight not exceeding 1kN/m may be introduced, EC1-1 states that an additional uniform load of 0.5kN/m^2 may be added to the imposed load. To allow for the effects of partitions weighing no more than 0.636kN/m (i.e. 2.4m high and weighing no more than 27kg/m^2 (see Table 4.5)) it is recommended that an additional imposed load of 0.32kN/m^2 be used instead. Further information on floor design using prefabricated timber joists can be found in *Engineered Wood Products: Code of Practice*[108].

8.1.3 Design information

Before designing a floor the Engineer should assemble the following data:

- the type of building occupancy and the associated imposed loads
- the floor construction, either in terms of material specifications and their dimensions or the dead weight excluding the weight of the joists and beams
- the deflection limits
- the floor plan, including the spans of joists and beams, and the location and size of openings
- the positions of all load-bearing supports and the bearing lengths available
- the required methods of support for joists and beams
- the type of decking and method of fixing
- the position and weight of all internal walls, posts or other structures to be supported by the floor

- any requirements for blocking to support the edges of plasterboard sheets, or for strutting (not normally used with engineered timber joists)
- where it is possible that additional internal partitions may be installed in unspecified positions, the maximum weight should be allowed for (see Table 4.5)
- any particular design requirements, e.g. joist centres, joist depth, extended period of fire resistance, special floor performance criteria
- location and size of any services to be passed through the floor void
- any special features not covered above.

If the floor is to be designed by a prefabricated joist manufacturer or other specialist floor designer, the information above should be passed on to the designer, preferably in the form of detailed drawings. It is recommended that its design be carried out using the same codes as those used for the main building, i.e. Eurocodes 0, 1 and 5.

Specialist floor designers are not generally responsible for designing the lateral restraint to walls provided by the floor or for any non-standard floor support details: these should be provided by the Engineer. The Engineer in turn should obtain the following output information from the floor designer, preferably in the form of detailed drawings:

- the basis of design, i.e. structural codes used and, in the case of proprietary products, details of third party approvals
- design assumptions made where these are not explicitly required by the code or options are given
- confirmation of the loads for which the floor has been designed
- confirmation of the deflection limits and any particular design considerations such as fire resistance or floor performance criteria which have been incorporated
- the size, position, spacing and specification of all the joists and beams
- the positions of all support bearings, the methods of support, the maximum reactions at the supports in terms of characteristic loads, and the minimum required bearing lengths for the joists and beams
- design and specification of all connections between joists, beams and masonry including double trimmer members and other multiple members
- the specification, location and fixing details for any additional members such as web stiffeners and backer blocks
- any special requirements for handling, storage or erection.

8.2 Types of timber floor

The three principal types of timber floor are:

- suspended ground floors – timber joists and decking with thermal insulation, supported on or between external and internal load-bearing walls or on sleeper walls
- intermediate floors – timber joists and decking and plasterboard, supported on or between external and internal load-bearing walls
- separating or compartment floors – timber joists and decking and plasterboard with additional layers of plasterboard, resilient material, sound insulation and sometimes secondary joists and thermal insulation.

Full details of the various types of timber floor are given in TRADA's *Timber Frame Construction*[76]. Figure 10.1 shows some typical details of the floor to wall junctions in timber frame buildings. For separating or compartment floors Building Regulations provide prescriptive solutions, and manufacturers (e.g. of plasterboard or engineered timber joists) provide further solutions based on performance tests. Under Scottish Building Regulations[109] compartment floors cannot be constructed in timber. Composite floors made with timber and concrete or other materials have also been developed. Some typical constructions are described in Table 8.1.

Multiple-span joists should be used only where site supervision and control is reliable, otherwise inaccuracies in the vertical alignment of the supports may result in excessive stress or poor vibrational performance.

Floors may be supplied as prefabricated floor panels with or without plasterboard attached, as pre-cut members, or as members to be cut to size on site. Nailed or glued prefabricated panels may also be designed as stressed skin panels: these are useful for long-span floors in buildings such as offices and shops.

8.3 Materials

8.3.1 Specification

For the general specification of suitable materials, see Table 3.27. In heated buildings floors are in service class 1, so materials such as particleboard Type P4 are adequate. Engineered timber joists are proprietary products and should be specified by make and the manufacturer's product designation. They should have appropriate third party certification for use in construction, and the manufacturer's instructions for installation should be adhered to.

As shown in Table 3.25 steel nails and screws in service class 1 do not require corrosion protection. All nails used to fasten decking and strutting should be ringed shank.

8.3.2 Beams

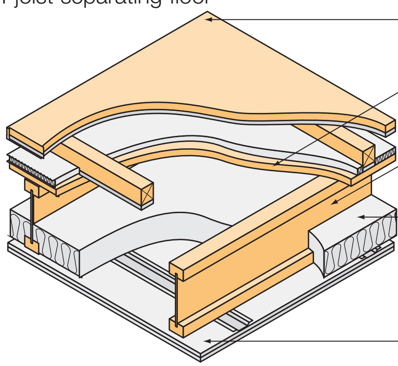
Timber beams may be made of large softwood or hardwood sections, deep or doubled engineered timber joists, glulam, LVL or another structural timber composite. Timber steel flitch beams and nailed or glued timber box beams or I-beams may also be used.

Large solid timber sections may dry out and distort over time, so are best avoided unless a rustic appearance is deliberately sought or they are required for reasons of conservation. The stability of deep slender engineered beams needs to be considered, especially if loads are supported eccentrically. If they are doubled they should be connected together in pairs by fixing web stiffeners between them or by using proprietary fasteners or fixing methods in accordance with the manufacturer's instructions. If LVL is chosen consideration should be given as to whether the 'Q' type, in which some veneers are cross-grained to provide improved shear performance, or the normal type is more appropriate. Steel or reinforced concrete beams should have a depth less than that of the incoming joists if a deck or flat soffit is to be attached across them, to allow for the possibility of shrinkage in the timber. With normal kiln-dried solid timber joists, the top of the beam should be 12mm below the top of the joists and the bottom of the beam 2mm above the bottom of the joists.

Table 8.1 Sample specifications for timber floor constructions for different performance requirements

Floor type	Joists		Decking or floating floor
	Depth (mm)	Spacing (mm)	
Intermediate floor, timber I-joists[a]	240	600	22mm tongue and groove softwood or chipboard floor boarding
Intermediate floor, solid wood joists[a]	195	450	18mm tongue and groove softwood or chipboard floor boarding
Separating floor, timber I-joists[b] (see diagrams)	240	600	18mm tongue and groove softwood or chipboard floor boarding 19mm (13.5kg/m²) Type A gypsum board 70mm deep (when loaded) timber batten with resilient layer Floor decking – see diagrams below
Separating floor, solid wood joists[b] (see diagrams)	220	400	

I-joist separating floor



Notes

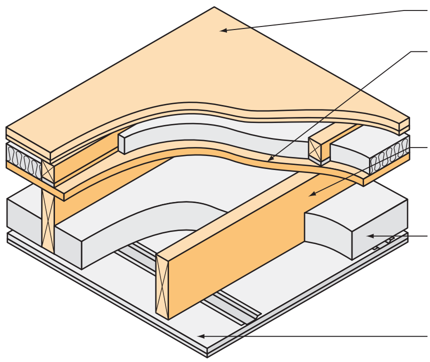
a Fixings must conform to Table 12.1 or the plasterboard manufacturer's specification, or better. Product specifications change: consult manufacturers for details of tested configurations relating to the materials currently manufactured.

b Sound performance from Robust Details. Valid only when used in conjunction with timber frame walls and the junction and fixing details specified in the Handbook[110].

c 10kg/m² plasterboard for the I-joist separating floor or 8kg/m² for the solid timber joist separating floor.

	Ceiling, Type A gypsum plasterboard from BS EN 520[52]	Insulation thickness	Sound insulation
	1 layer 15mm	None	Airborne 40dB (R_w)
	1 layer 12.5mm	100mm laid on top of plasterboard ceiling	Airborne 40dB (R_w)
	1 layer 19mm (13.5kg/m²) + 1 layer 12.5mm[c] or 2 layers of 15mm (each layer 12.5kg/m²) 16mm deep resilient metal ceiling bars at 400mm centres (see *Robust Details Handbook*[110])	10-36kg/m³ mineral wool quilt: 25mm between battens, 100mm between joists	Airborne 47-55dB $D_{nT,w} + C_{tr}$ Impact 46-58dB $L'_{nT,w}$
		10-36kg/m³ mineral wool quilt: 60mm between battens, 100mm between joists	Airborne 47-56dB $D_{nT,w}+C_{tr}$ Impact 48-60dB $L'_{nT,w}$

Solid joist separating floor



Floating floor	See above
Floor decking	11mm thick (min.) OSB or Walker Timber perforated deck system
Joists	220mm (min.) solid timber joists at maximum 400mm centres
Absorbent material	100mm (min.) mineral wool quilt insulation (10-36kg/m³) between joists
Ceiling	See above

Diagrams by permission of Robust Details Ltd.

8.3.3 Joists

For timber floors engineered timber joists have become the most favoured solution, but solid softwood joists are still widely used, sometimes in 'superdried' timber which, like engineered joists, is more stable dimensionally than solid timber. With flange widths of less than 45mm it may be difficult to ensure adequate fixing of sheathing materials.

For trimmers and trimming joists it is usual to fix two standard joists together. When engineered timber I-joists are used in this way web stiffeners in the form of plywood or OSB must be used as specified by the manufacturer at fixing points.

8.3.4 Strutting

Traditionally strutting has been recommended between solid timber joists to help align the joists and limit their twisting, and to improve the floor's vibrational performance. Strutting can take the form of solid timber blocking or 'dwangs' at least 38mm thick and at least three-quarters of the joist depth in depth (with a gap for services at the top) double skew nailed to each joist, or 'herringbone' strutting consisting of pairs of timbers at least 38mm × 38mm in cross-section nailed diagonally in opposing directions to the top and bottom of neighbouring joists, or else proprietary steel herringbone strutting. (Herringbone strutting should not be used if the spacing of the joists exceeds three times their depth.) The recommended number of lines of strutting is shown in Table 8.2.

However even kiln-dried softwood can shrink in a centrally heated building leaving gaps at the ends of solid timber blocking which can cause squeaking and render it ineffective, while herringbone strutting is difficult to install. One solution is to use 'superdried' timber and solid blocking, or to omit strutting altogether but make the floor slightly stiffer. With prefabricated timber I-joists strutting is not normally considered either practical or necessary, except at supports where the manufacturer's recommendations must be followed to ensure stability. With metal open web timber joists a timber member called a 'strongback' running through the joists at mid-span is normally specified by the manufacturers with accompanying fastening instructions.

Table 8.2 Recommended rows of strutting between solid timber joists

Joist span (m)	No. of rows required	Position
$l \leqslant 2.5$	0	N/A
$2.5 < l \leqslant 4.5$	1	Mid-span
$4.5 < l$	2	At one-third span points

8.3.5 Decking

Decking can be of tongue and grooved timber boards or any wood-based panel product certified for use as flooring, subject to satisfactory fire performance. Particleboard is most commonly used, except for the structural decking of separating and compartment floors for which OSB is preferred. In the case of panel products for new-build, products suitable for service class 2 should be specified, unless it can be guaranteed that they will not get wet during delivery or storage on site.

8.3.6 Fixing

8.3.6.1 Proper installation

Complaints about floor performance are most likely to arise from improper installation. The Engineer should make it clear that the specified design requirements must be strictly observed.

8.3.6.2 Joist support

Due to reported failures and accidents with incorrectly fitted joist hangers the following notes, as appropriate, are recommended on drawings, headed ‘SAFETY NOTE’:

i) Timber-to-masonry joist hangers

1. Joist hangers are to be of type <xxx> to EN 845-1[111].
2. Ensure that hangers are fitted tightly against walls and are seated evenly on the masonry.
3. Joists must be cut accurately to length to leave a maximum gap of 6mm to the back plates.
4. The specified fasteners must be used in all the holes provided.
5. Do not load or use this floor until at least <xxx hours (*the curing time*)> after <xxx layers>/<xxx mm height> (*the manufacturer’s requirements*) of blockwork are completed above the hangers.

ii) Timber-to-timber joist hangers

1. Joist hangers are to be of type <xxx> supported by a European Technical Approval written to ETAG 015[105].
2. Ensure that hangers are fitted tightly against walls using only specified fasteners in all the holes provided.
3. Joists must be cut accurately to length to leave a maximum gap of 6mm to the back plates.
4. The specified fasteners must be used in all the holes provided.

iii) Joists supported on a timber wall plate

1. Each joist must have a minimum bearing length of 40mm.
2. Joists must be fixed using all the specified fasteners.

iv) Prefabricated timber I-joists

1. Unbraced joists are highly unstable! Do not walk on joists or store construction materials on them until the floor is fully braced in accordance with the manufacturer’s instructions.

8.3.6.3 Multiple joists or beams

Multiple joists or beams should be fixed together in accordance with Section 8.4.3.

8.3.6.4 Continuous joists
Attention should be drawn to the need to align the supports vertically so that the joist is fully supported at each position. If necessary packing of the same strength and stiffness as the supporting material should be inserted.

8.3.6.5 Tying of floors to masonry walls
For stability reasons the floor should be tied to masonry walls as shown in Figures 12.1 to 12.3. In timber frame buildings the requirements are given in Section 5.11.

8.3.6.6 Decking
Floor boards or decking panels should be fixed to the joists in accordance with Table 12.1. For floors designed to support unusually heavy floor loads special fixing requirements may be applicable. For continuity and diaphragm action board or panel edges not supported on joists must be connected to each other via battens or glued tongue and groove joints in accordance with Table 12.1.

8.4 Floor design
8.4.1 General
Simple beams and joists should be designed in accordance with Section 5.2, with material properties from Tables 3.14 to 3.18. Decking may be designed in a similar way, but it is more common to use permissible span/thickness combinations from the manufacturer's certification literature instead. The minimum thickness of decking may be determined by requirements for minimum mass given in the Building Regulations rather than structural considerations.

8.4.2 Ultimate limit states
Ideally beams and joists should be prevented from rotating at the supports, since then, if lateral displacement of the compressive edge is prevented throughout the length, the value of k_{crit} may be taken as 1.0.

As stated in Section 4.9.1 adequate diaphragm action may be assumed in certain cases (see Section 5.8.1). In other cases it may be necessary to calculate it (see Section 5.8.2).

8.4.3 Serviceability limit states
8.4.3.1 Final deflection
The vertical deflection limits of $l/250$ and $l/150$ recommended in Table 3.4 are to limit curvature rather than absolute deflection. Care must be taken that the absolute deflection of a long floor beam does not cause it to rest on a non load-bearing partition or produce excessive slope in the supported joists. If the $l/150$ limit were used for an exposed 6m beam the final deflection at its mid-point could be 40mm!

Precamber will generally only be used for heavy primary glulam beams supporting floors over long spans.

8.4.3.2 Vibration (EC5 7.3)

EC5 requires floor vibration to be controlled to an acceptable level and it provides a design method for dwellings. The same method should be applicable to floors in other types of building, although the parameters have not been tested for spans above 6m or for floors of unconventional construction. The method is given in EC5 7.3.3(2) and NA 2.6.

The following requirements must be satisfied:

i) Static deflection under a point load (EC5 NA 2.6)

Verify that:

$$w_{\text{inst,Q}} \leqslant a$$

Where $w_{\text{inst,Q}}$ = instantaneous deflection of the whole floor beneath a centrally positioned 1kN point load (mm)

a = **1.8mm for $l \leqslant$ 4000mm**

= **16500/$l^{1.1}$ mm for l > 4000mm**

l = structural span of floor.

$w_{\text{inst,Q}}$ may be calculated as

$$w_{\text{inst,Q}} = \frac{1000 k_{\text{dist}} l_{\text{eq}}^3 k_{\text{amp}}}{48(EI)_{\text{joist}}} \text{ mm} \qquad \text{EC5 NA.2.6.2}$$

Where k_{dist} = proportion of point load acting on a single joist

l_{eq} = effective floor span (mm)

k_{amp} = amplification factor to account for shear deflections or joint slip

$(EI)_{\text{joist}}$ = bending stiffness of a joist (Nmm²) calculated using E_{mean}

$$k_{\text{dist}} = \max.\begin{cases} k_{\text{strut}}\left[0.38 - 0.08\,ln\left(\dfrac{14(EI)_{\text{b}}}{s^4}\right)\right] \\ 0.30 \end{cases}$$

k_{strut} = 0.97 for single or multiple lines of strutting, installed in accordance with Section 8.3.4, otherwise 1.0.

$(EI)_{\text{b}}$ = flexural rigidity of floor decking perpendicular to the joists in Nmm²/m using E_{mean} for E

s = joist spacing (mm)

l_{eq} = span, l, in mm, for simply supported single span joists

= 0.9l for the end spans of continuous joists

= 0.85l for the internal spans of continuous joists

k_{amp} = 1.05 for simply-supported solid timber joists

= 1.10 for continuous solid timber joists

= 1.15 for simply-supported glued thin-webbed joists

= 1.30 for continuous glued thin-webbed joists or mechanically jointed floor trusses

$(EI)_b$ may be increased by adding the flexural rigidity of plasterboard ceilings fastened directly to the soffit of the floor joists, assuming $E_{plasterboard} = 2000N/mm^2$.

$(EI)_b$ may be increased for open web joists with a continuous transverse bracing member fastened to all the joists within $0.1l$ of mid-span, by adding the bending stiffness of the transverse member in Nmm^2 divided by the span l in metres.

ii) *Unit impulse velocity response*

For conventional timber floors made with solid timber joists or engineered timber joist and some form of timber or wood-based decking, the modal damping ratio will be at least 2%, consequently the velocity response will not govern the required stiffness of the joists and its calculation is unnecessary.

iii) *Frequency*

Unless a special investigation is made, e.g. by means of finite element analysis or testing, verify that:

$$f_1 \geqslant 8$$

Where f_1 = the fundamental frequency of vibration of the unloaded floor in Hz (cycles per second)

f_1 may be calculated from the following formula which is derived from EC5 expression (7.5):

$$f_1 = \frac{18}{\sqrt{w_{inst}}} \text{Hz}$$

Where w_{inst} = instantaneous bending deflection of the floor in mm under dead weight alone, assuming that it spans in one direction as a simply supported beam

The mass of the floor alone, without any allowance for partitions, should be used to calculate f_1 (EC5 NA 2.6.2).

EC5 allows the composite stiffness of the decking and joists to be used in calculating an effective floor bending stiffness, but this requires caution. The NA states that for glued decking, the full composite stiffness may be utilized only if the floor is designed in accordance with EC5 9.1.2 with an adhesive meeting the requirements of EC5 3.6 and the detailing and control provisions set out in EC5 10.3. For nailed and screwed decking a design method is given in EC5 Annex B. However tests have demonstrated that the actual composite stiffness is far less than these design methods predict, so it is recommended that if composite action is assumed the basic joist stiffness should be increased by a factor of no more than 1.15 for decking which is nailed or screwed in accordance with Table 12.1, or 1.25 for decking which is fixed as specified above.

When a floor is in use f_1 is likely to be lower than the value calculated as above because the floor will generally support an imposed load in addition to its dead weight.

8.4.3.3 *Joists supported on beams*

Supporting joists on a beam at one or both ends can produce two serviceability problems.

For joists supported on a beam at one end the beam's deflection will increase the slope of the joist at its other end, and this may produce problems with furniture (e.g. drawers sliding open). To deal with this it is recommended that half the deflection of the beam be added to the calculated deflection of the joist when verifying that its deflection does not exceed the specified limits under a UDL.

Secondly a beam support can reduce the fundamental frequency of vibration of the supported joists, f_1, by up to 30%[112]. Therefore where floor joists are supported on a beam, the minimum frequency limit of 8Hz should be applied to the fundamental frequency of the floor system, $f_{1,\text{system}}$. The value of $f_{1,\text{system}}$ may be calculated using Dunkerly's formula as:

$$f_{1,\text{system}} = \sqrt{\frac{f_{1,\text{joist}}{}^2 \times f_{1,\text{beam}}{}^2}{f_{1,\text{joist}}{}^2 + f_{1,\text{beam}}{}^2}}$$

Where $f_{1,\text{joist}}$ and $f_{1,\text{beam}}$ are the frequencies of the joists and supporting beam respectively, each calculated using the dead weight which it supports, and ignoring any composite action with the decking.

8.4.4 Stairwell trimming

Figure 8.1 defines the dimensions for sizing double trimming members around a stairwell or void.

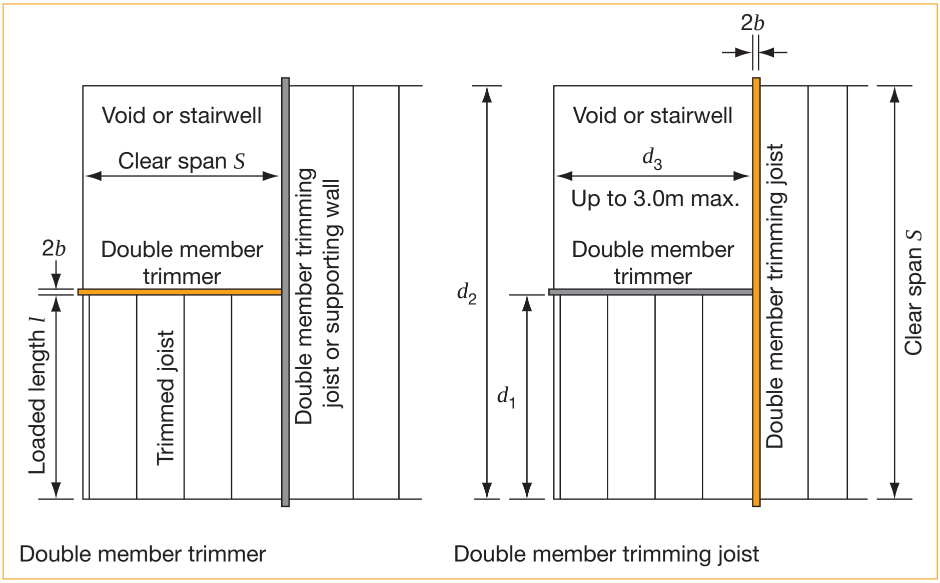


Fig 8.1 Plan showing dimensions for a trimmer and a trimming joist

Double member trimmer

For initial sizing, the maximum clear span for a C16 double member trimmer supporting trimmed joists can be calculated approximately as:

$$S_{max} = 0.165(-0.308l+3.38)\times(0.0214b+1.12)\times(0.0149h-0.073)\text{m}$$

Where l = span of trimmed joists (m) (maximum 6m)
b = breadth of each of the two trimmer members (mm) ($38 \leqslant b \leqslant 75$mm)
h = depth of the trimmer members (mm) ($147 \leqslant h \leqslant 220$mm)

For C24 timber increase S_{max} by 7.5%. This will support:

- a floor with a span of l m weighing 0.5kN/m^2 and supporting an imposed load of 1.5kN/m^2
- a non load-bearing lightweight partition weighing not more than 0.8kN/m run
- a second similar partition supported on and parallel to the trimmed joists.

Double member trimming joist

The maximum clear span for a C16 double member trimming joist supporting a double member trimmer can be calculated approximately as:

$$S_{max} = 0.032(-0.31d_3+3.8)\times\left(\frac{-1.52d_1}{d_2}+3.93\right)\times(0.022b+1.79)$$
$$\times(0.0196h-0.16)\text{m}$$

Where b = breadth of each of the two trimmer members (mm) ($38 \leqslant b \leqslant 75$mm)
h = depth of the trimmer members (mm) ($147 \leqslant \text{h} \leqslant 220$mm)
d_1, d_2 and d_3 are defined in Figure 4.3(b) and are measured in metres

For C24 timber increase S_{max} by 7.5%.
This will support:

- a double member trimmer supporting loads as above
- a non load-bearing lightweight partition weighing not more than 0.8kN/m run over length d_1 only.

8.4.5 Built up beams

Simple built-up beams can be made by nailing or screwing two or more solid timber member or engineered timber joists together.

Where the load is applied to only one member of a multiple-member beam the required fasteners must be calculated. Recommended fixings for double trimmers and trimming joists are given in Table 8.3. When engineered I-joists are used they must be connected together in accordance with the manufacturer's instructions: these usually involve placing a packing piece of timber between the webs before nailing them together with specified nailing.

Where the load is applied equally to all the members fairly nominal fixing is usually adequate, but if two or more solid timber members are securely fastened together so that both

Table 8.3 Fixings for solid timber trimmers and trimming joists

Fixing type	Breadth of single member (mm)	Fastener size (mm x mm)	Fixing spacing (mm)
Plain wire nails	38	3.75ϕ x 75	300
	47	4ϕ x 90	
	75	5ϕ x 125	
Bolts	38 to 75	M12[a]	1200[b]
Double-sided toothed plate connectors	38 to 75	50mm connector + M12 bolt[a]	2400[b]

Notes

a 3mm thick M12 washers should be provided under the head and nut: 38mm ϕ for round connectors or 50mm square for square connectors.

b There should be a minimum of two fasteners at 1/3 span positions.

members can share the load (the fasteners should be capable of transferring 10% of the load on one member to the next one) the strength properties including the bearing strength may be increased by a factor k_{sys} of 1.1 (see Table 3.20 notes **c** and **e**).

For guidance on the design of flitch beams see Section 5.5. The detailed design of box beams and I-beams is outside the scope of this *Manual*, but guidance on the design of mechanically fastened types is given in EC5 Annex B, and a note on glued box beams and I-beams is given in Section 6.8.2.

8.5 Fire resistance

In general intermediate floors require 30 minutes' fire resistance on the underside and party floors require 60 minutes' fire resistance. This is normally provided, as described in Section 5.11.2, by one or two layers of gypsum plasterboard, although other materials may be used if their suitability can be proved by a full scale test or, in the case of timber, by calculation.

All beams which project below the ceiling line will require their own fire protection, either in the form of a protective layer of plasterboard or an equivalent timber material, or alternatively in the case of solid timber and structural timber composites by an appropriate increase in the section size (see Sections 5.10.3 and 5.10.4).

A ceiling should also have the required surface spread of flame (reaction to fire) performance.

8.6 Robustness

To provide catenary action in the case that a floor support fails, wherever possible the structural decking should be continuous throughout each floor level except at any party walls (see Section 5.11).

8.7 Acoustic and thermal requirements

8.7.1 Acoustic

Sound can be transmitted in the form of airborne noise (voices, music centres, etc.) and structure-borne noise (footsteps, plumbing, lifts). UK building regulations require sound transmission to be restricted between attached dwellings and building use categories, and for internal walls between bathrooms or bedrooms and other rooms within a single dwelling. Resistance to both types of sound must be provided by floors which separate dwellings, but the floor of a dwelling above a part of the building which is not a dwelling need have airborne sound resistance only.

Special attention to sound transmission is needed in lightweight timber structures, but by careful design the airborne noise can generally be handled to achieve good sound ratings. The structure borne noise can be more difficult to address, especially in floors. One approach is to isolate the upper part of the floor from the primary structure as much as possible. A simple way to do this is to lay the finished floor on a continuous mineral wool batt (minimum mass 100kg/m^3) or a raft of battens made with a pre-bonded resilient layer which gives even better acoustic performance (see Table 8.1). Separating the ceiling on the underside by attaching it via resilient channels to independent joists at right angles to them (see Table 8.1) will improve both airborne and impact sound attenuation. Impact sound attenuation can be improved by increasing the mass of the floor. This can be accomplished by the use of double layers of heavy sheathing material above and below joists, or by laying 30 to 40mm of concrete, anhydride based screed or plaster on top of the floor, perhaps mixed with plasticizers to inhibit cracking.

The separating floors shown in Table 8.1 may be useful to the engineer for estimating weights. Other solutions and ways of meeting additional acoustic requirements such as resilient flanking strips at floor perimeters and the isolation of service pipes may be found in sources such as *Timber Frame Construction*[76] and *Robust Details Part E*[110].

8.7.2 Thermal

There are currently no thermal insulation requirements for intermediate floors, except where a floor is over an open unheated area such as a porch or garage.

9.1 Forms of open frame construction

9.1.1 Scope

Open frame construction can be used for a variety of building types such as offices, schools, assembly areas and sporting arenas. Domes and grid shells also come into this category but are outside the scope of this *Manual*. Most open frame structures are single storey, but two- and occasionally three-storey structures are also built. Single storey buildings can have roof spans up to 80m (see Table 4.6) whilst two- and three-storey buildings can have floor spans up to 10m. The spacing of frames may be limited by the economic span of secondary members such as floor joists and purlins.

9.1.2 Construction principles

Open frames can be constructed as:

- pin jointed structures relying on horizontal bracing systems or diaphragms to transfer lateral and instability forces to vertical bracing systems or vertical timber shear walls and thence to the foundation
- stiff jointed structures capable of resisting lateral and instability forces, e.g. portal frames
- a combination of pinned and stiff jointed frames together with the appropriate bracing systems.

Modern machining techniques allow steel plates to be set within deep timber members thus giving the facility for concealed stiff joints.

9.1.3 Selection of type of frame and materials

The selection of the framing method and the types of components and materials used can depend on the required appearance, aesthetics and economics of the completed structure. Table 4.6 gives typical limiting dimensions and the timber-based materials for various components that can be used in open frame construction. The list is by no means exhaustive. The components illustrated can also be incorporated in concrete, masonry and steel structures.

The dimensions of an open frame building may be dictated by the available sizes and appearance of the materials proposed. The length and weight of components can be limited by fabrication and transport restrictions, with site conditions occasionally being a further constraint.

Solid timber, structural timber composites and stock sizes of glulam are generally used for straight elements, since shaping or profiling can be costly and wasteful. Glulam and built-up components such as thin web box beams, trusses etc., can be more easily formed to predetermined profiles and shapes.

It is quite common to specify large sections of solid timber for frame construction, but reference should be made to Section 2.17.3 in relation to the effects of drying out.

9.2 Design

9.2.1 Principles

Open frame constructions, members and connections should be designed in accordance with the principles set out in this *Manual*.

In addition plane frames and arches should be separately checked for prescribed dimensional imperfections arising from deviations from the assumed geometry of members and fabrication and erection tolerances.

With open frame construction particular attention has to be paid to robustness and disproportionate collapse as the uses of the buildings often come within critical occupancy categories e.g. single storey educational buildings and assembly buildings (see Section 2.10.1).

9.2.2 Frame imperfections

Frame imperfections are allowed for by defined rotations of members and frames of ϕ (in radians) and curvatures between points of contraflexure corresponding to an eccentricity e (EC5 5.4.4):

Where ϕ = 0.005 for $h \leqslant 5.0$m where h is the storey or building height

= $0.005\sqrt{\frac{5}{h}}$ for $h > 5.0$m

e = $0.0025l$ where l is the distance between points of contraflexure

The effects of these imperfections should be assessed by a second order (P–Δ) analysis with the normal design loads applied. In this assessment, the design values of the stiffness properties, E_d and G_d should be used (see EC5 2.2.2(1)P and 2.4.1(2)P) without adjustment for duration of load (i.e. without creep):

Where $E_d = \frac{E_{mean}}{\gamma_M}$

and $G_d = \frac{G_{mean}}{\gamma_M}$

For checking stresses the value of k_{mod} for the relative duration of load should be used.

Figure 9.1 shows some examples of assumed initial deviations in the geometry of a portal frame and an arch, and Figure 9.2 shows some similar examples for a rectangular frame.

For rectangular frames the sway imperfection and local bow imperfection may be replaced by equivalent horizontal forces as shown in Figure 9.3, where N_d is the design axial load for the relevant load duration. If the magnitude of applied horizontal forces due to, say wind, is greater than the equivalent horizontal force (Figure 9.3d) then the equivalent horizontal force may be neglected.

9.2.3 Stability and bracing

For stability principles see Section 2.6. For the bracing of compression members and beam systems, see Section 5.8.

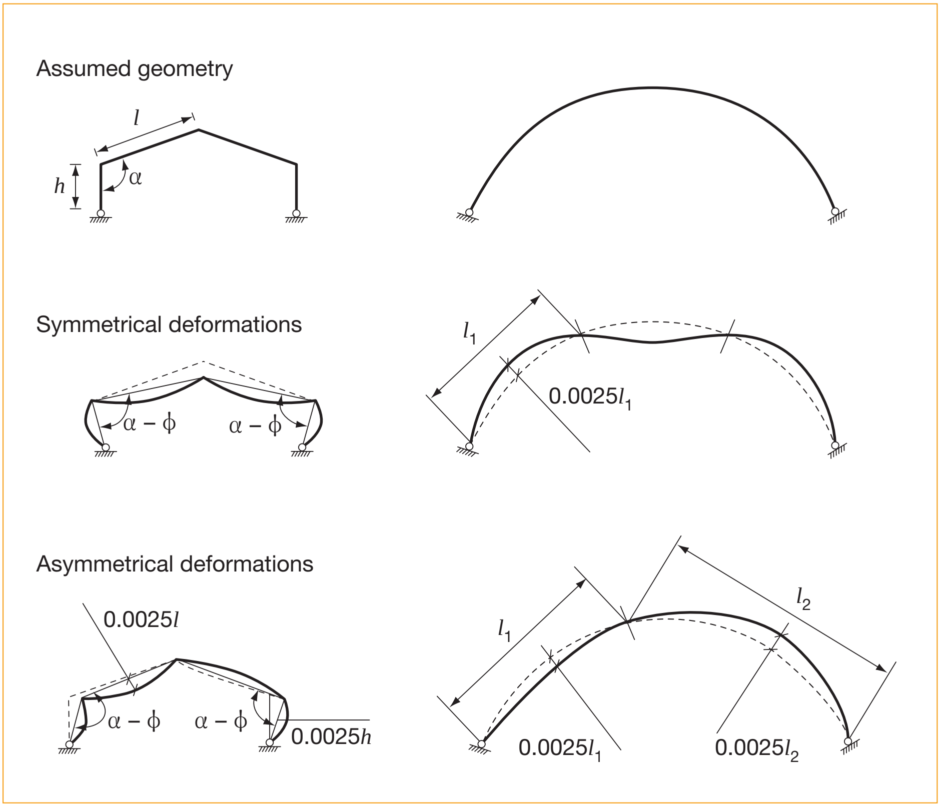


Fig 9.1 Examples of assumed deviations in geometry of a portal frame and arch

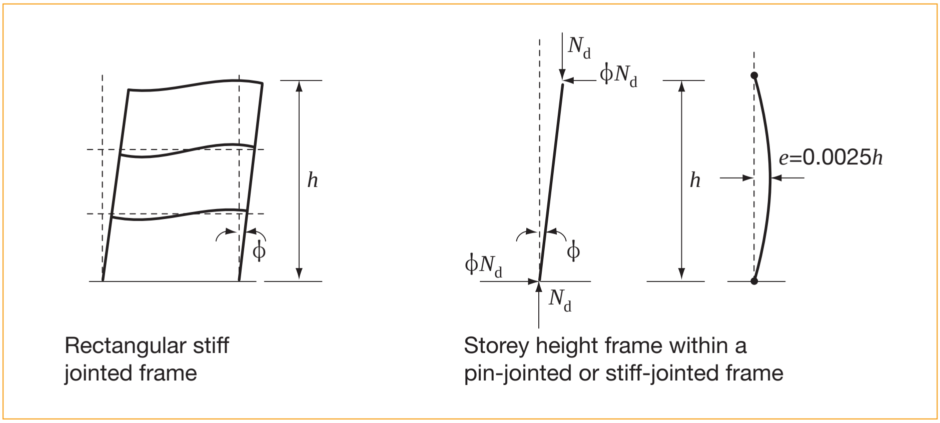


Fig 9.2 Examples of assumed deviations in the geometry of a rectangular frame

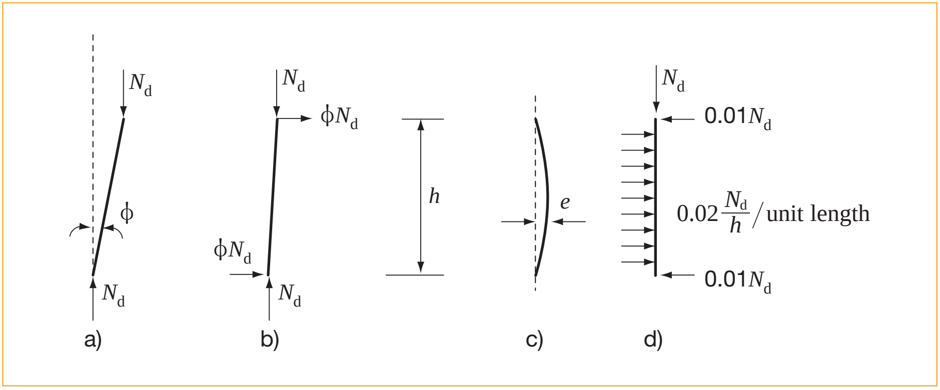


Fig 9.3 Equivalent horizontal forces for rectangular frames

9.2.4 Base connections

Usually the connection to the foundation or supporting structure is assumed to be pinned. If there is a risk of water collecting at the foot of the frame it is essential it is kept clear of moisture and in particular that any form of shoe connection will not collect water.

9.3 Construction details

9.3.1 Situations to be avoided

Problems of splitting and possible loss of strength of timber members can be avoided by careful detailing. Specified spacings, edge and end distances of fastenings must always be maintained and undue restraint of any moisture movements of timber and wood based products should be avoided. Wood-based materials have low strength in tension perpendicular to the grain, and adhesives have low strength perpendicular to the glue line, so no reliance can be placed on members stressed in these directions.

Figure 9.4 shows three ways to end-join timber members loaded in shear. The first method is deprecated because it can cause the timber to split as it is loaded in tension perpendicular to the grain.

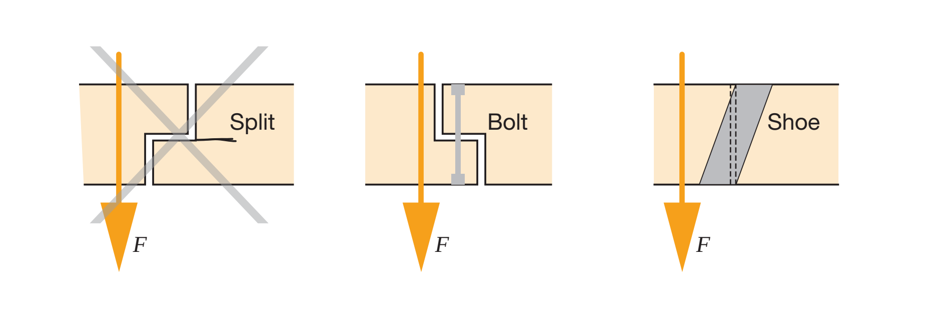


Fig 9.4 Half lap end connections

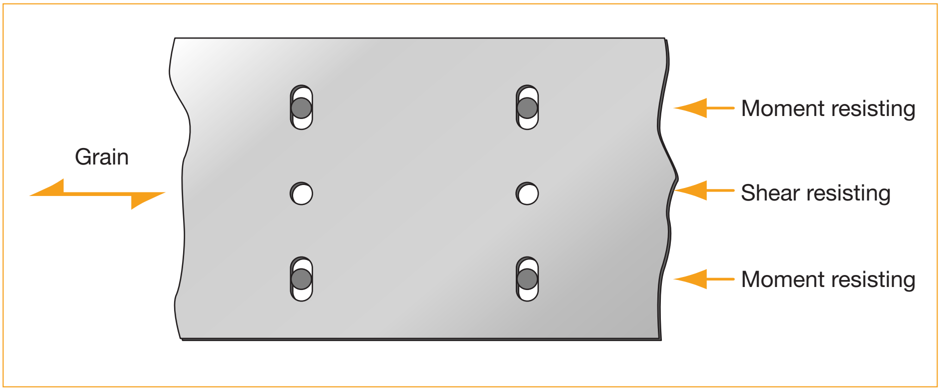


Fig 9.5 Slotted holes in deep steel plates

Site gluing is not recommended because, to produce adequate load-carrying joints, the same care and attention must be exercised as would be needed in factory conditions. So consideration of facilities for storage and mixing of adhesives; control of glue spreading; open and closed assembly times; application of some form of pressure to achieve close contact between surfaces; maintaining a minimum ambient temperature (10ºC) of the timber and the glue line for a number of hours and then ensuring time for curing of the adhesive before loads are applied, will be difficult to achieve on most sites. If necessary reference should be made to TRADA's *Adhesively bonded timber connections*[79].

Where deep steel plates are used in connections, care must be taken that they do not cause splitting of the timber if cross grain shrinkage of the timber occurs. This risk can be avoided by the appropriate slotting of holes to allow timber movement, as shown in Figure 9.5.

9.3.2 Base details

The foot of a plain frame is usually carried in some form of shoe. For external use this should provide at least 150mm clearance above the surrounding ground. Frames can sometimes bear on a sole plate being retained in position by brackets, provided that the sole plate is isolated from ground contact by a permanent damp proof membrane (see Figure 10.6a).

9.3.3 Pin jointing techniques

The simplest kind of connection is the direct bearing of one component on another, with dowels, gussets or housing in a mortice serving mainly to locate the members in position. This type of joint does not generally create eccentricity of loading.

Alternatively pinned joints can be obtained by joining components together with mechanical fastenings such as bolts which are confined to a relatively small area.

The use of extended brackets or shoes (either proprietary or specially designed) can create eccentricities that must be allowed for in design.

Stability must be provided by means of triangulation or diaphragm action.

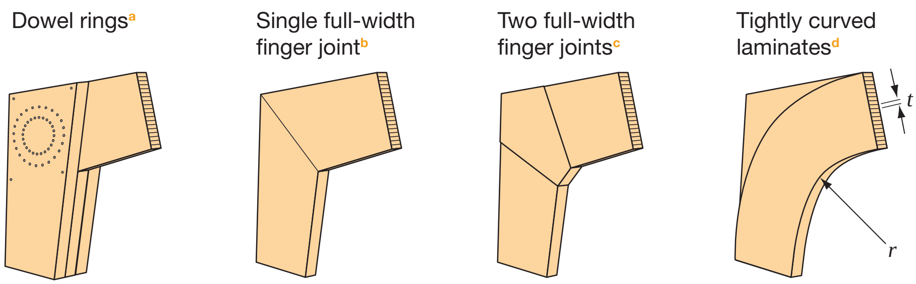
9.3.4 Stiff jointing techniques

Large gusset plates fastened with nails, screws, bolts dowels or adhesive can be designed to give stiff joints. Gussets can be external plates of either timber, wood based material or steel. Alternatively with modern machining methods, single or multiple steel plates can be slotted into the members to be joined and secured with mechanical fastenings.

In portal frames the haunch can be formed with gussets or fin plates as above, or else by means of mechanically fastened moment-resisting lap joints (Figure 9.6a), large finger joints (Figures 9.6b and 9.6c) or curved laminates (Figure 9.6d). Each method has its advantages and limitations.

9.4 Movement

See Sections 2.8 and 10.11.



Notes

a Suitable for glulam or LVL. A durable connection which can be site-assembled. Reinforcement of the timber may be required to avoid splitting.

b A neat connection, suitable for glulam or LVL. The tightness of the corner reduces its strength; it cannot be used in service class 3.

c As **b** but with improved strength; still cannot be used in service class 3.

d Very neat appearance and no site assembly required. Suitable only for glulam. Internal radius limited to:

$$r \geqslant \frac{tE_{\text{mean}}}{70}$$

Where r = radius of curvature (mm)

E_{mean} = mean modulus of elasticity (N/mm²)

t = thickness of individual glulam laminates (mm)

Fig 9.6 Haunch details

10.1 Introduction

This section covers the principal structural requirements and design procedures for platform timber frame buildings. Subsidiary requirements given in the Building Regulations, such as requirements for cavity barriers and air tightness, are not covered in this *Manual*.

For the general detailing of timber frame construction the Engineer is recommended to read TRADA's *Timber frame construction*[76], or for more detailed engineering guidance TRADA's *Timber frame housing: Structural recommendations*[80].

10.2 Design procedure

Figure 10.1 shows the overall design procedure for platform timber frame.

10.3 Actions

10.3.1 Wind loads

10.3.1.1 Wind pressure

Wind pressures on different parts of the walls and roof are calculated from EC1-1-4. For the timber frame buildings covered by this *Manual* the structural factor c_sc_d may normally be taken as 1.0 (EC1-1-4 6.2(c)). For simplicity the following recommendations are made for timber frame buildings.

- Use a single reference height z_e equal to the total height of the building above the ground (EC1-1-4 Figure 7.4). This means that the wind pressure acting on an external or internal surface of the building is calculated simply as q_pc_{pe} or q_pc_{pi} respectively.
- Base the external pressure coefficients for walls on the height of the wall to the eaves.
- For overturning, sliding, roof uplift and racking resistance calculations involving more than one value of c_{pe} on the roof, first apply a single conservative value to the whole roof. If the structure fails, calculate the overturning moment or the sliding, uplift or racking force more accurately.

For all structures in the execution phase the seasonal factor c_{season} may be used to modify the basic wind velocity (EC1-1-4 4.2(3)). For the execution phase it is expected that a value for c_{season} based on a 2 year erection period will be specified in the National Annex to BS EN 1991-1-6: *Eurocode 1. Actions on structures. General actions. Actions during erection*[113]. However, for smaller timber frame projects a 1-year period might be considered more appropriate, for which the corresponding value of c_{season} is 0.749. This reduces the wind pressure by a factor of $0.749^2 = 0.56$.

10.3.1.2 Masonry shielding

EC5 makes no allowance for the shielding effect of masonry, but both testing and experience in the UK have demonstrated that within certain limits masonry walls will reduce the wind load onto the timber frame of buildings. The resulting reduced wind load is considered to act uniformly

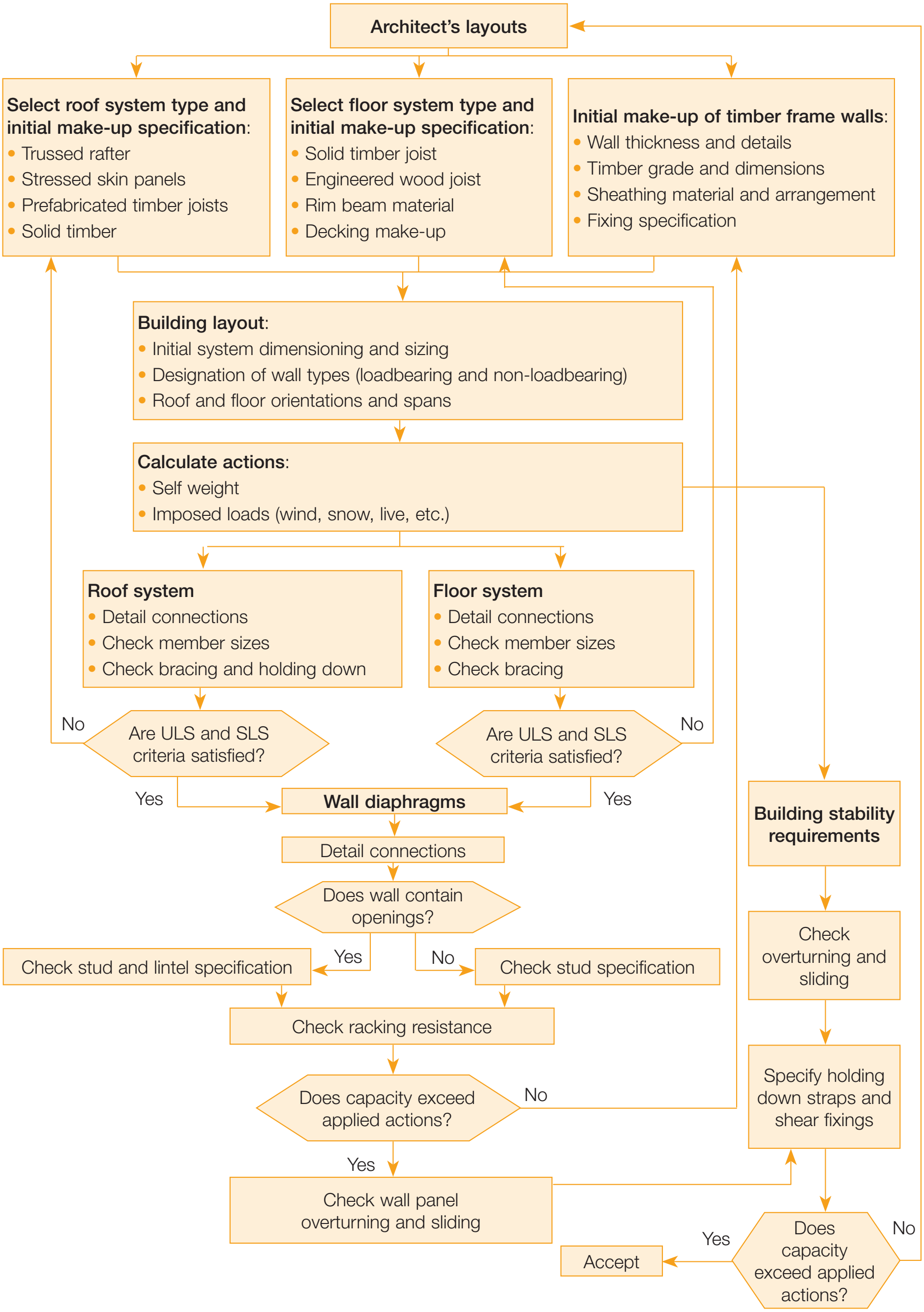


Fig 10.1 Flow chart illustrating platform timber frame design procedure

over the entire area of the adjacent timber frame wall up to and including the fourth storey above ground level.

When the wind blows on or off a gable wall the total wind load on or off the adjacent timber frame wall should be calculated as:

$$F_{v,k} = k_{masonry} F_{v,masonry,k} + F_{v,spandrel,k}$$

Where $k_{masonry}$ = wind shielding reduction factor from Table 10.1

$F_{v,masonry,k}$ = total wind load on or off the masonry wall excluding the spandrel area

$F_{v,spandrel,k}$ = wind load on or off spandrel

On other types of masonry-shielded wall the wind load should be calculated as:

$$F_{v,k} = k_{masonry} F_{v,masonry,k}$$

Where $k_{masonry}$ = wind shielding reduction factor from Table 10.1

$F_{v,masonry,k}$ = total wind load on or off the masonry wall

Since $k_{masonry}$ depends on the proportion of openings in the wall it may differ on windward and leeward faces, therefore it must be used in conjunction with the surface pressure method of EC1 (see EC1-1-4 5.3(3)).

$k_{masonry}$ may be used only in accordance with the following conditions:

- only the first four storeys of masonry not exceeding 10m in total height can be considered to contribute wind shielding
- the external dimensions of the masonry walls are used to calculate the wind loads
- the masonry walls are constructed in accordance with BS EN 1996-1-1: *Eurocode 6. Design of masonry structures. General rules for buildings. Rules for reinforced and unreinforced masonry* (EC6-1-1)[114] and BS EN 1996-2: *Eurocode 6. Design of masonry structures. Design, selection of materials and execution of masonry* (EC6-2)[115] from a material designated in EC6-1-1
- the mortar conforms to the relevant part of EC6-1-1 with a minimum strength class of M4
- the masonry walls are at least 100mm thick and have a minimum mass of 75kg/m^2
- the masonry cladding is connected to the timber frame with wall ties that have sufficient strength and stiffness to transfer wind forces to the timber frame wall manufactured in accordance with BS EN 845-1[111]. See Section 10.8.4
- $k_{masonry}$ is applied to the wall as a whole up to eaves level, to the top of the fourth storey of masonry or up to 10m of masonry, whichever is less.

$k_{masonry}$ should not be applied to the design of individual elements, for example studs. $k_{masonry}$ should not be used when checking the execution phase.

Table 10.1 Values of $k_{masonry}$

Percentage of shielded wall occupied by openings	Number of storeys shielded by masonry								
	1 and 2			3			4		
	A	B	C	A	B	C	D	E	F
0	0.45	0.60	0.75	0.50	0.68	0.85	0.60	0.74	0.88
10	0.50	0.64	0.78	0.55	0.71	0.87	0.64	0.77	0.89
20	0.56	0.68	0.80	0.60	0.74	0.88	0.69	0.80	0.91
30	0.61	0.72	0.83	0.65	0.78	0.90	0.73	0.83	0.93
40	0.66	0.76	0.85	0.70	0.81	0.92	0.77	0.86	0.95
50	0.71	0.80	0.88	0.75	0.84	0.93	0.81	0.89	0.96
60	0.77	0.84	0.90	0.80	0.87	0.94	0.86	0.92	0.98
70	0.82	0.88	0.93	0.85	0.91	0.96	0.90	0.95	1.00
>70	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Notes

A For masonry walls with buttresses or returns of length ⩾ 550mm and spaced at not more than 9m centres.

B For masonry walls with buttresses or returns of length ⩾ 550mm at one end only, wall length ⩽ 4.5m.

C For masonry walls other than A and B.

D For masonry walls with buttresses or returns of length ⩾ 950mm and spaced at not more than 9m centres.

E For masonry walls with buttresses or returns of length ⩾ 950mm at one end only, wall length ⩽ 4.5m.

F For masonry walls other than D or E.

a In calculating the percentage of wall occupied by openings, the height of the wall should be taken as the height to the eaves, the top of the fourth storey of masonry or 10m, whichever is less.

b Values for intermediate percentages of wall occupied by openings may be obtained by linear interpolation.

c For walls longer than 9m the tabulated values may be used provided that additional buttresses or returns are added to the masonry wall spaced at not more than 9m centres.

d If the selected support conditions do not extend to the full shielded height of the wall in question then the number of storeys and percentage of loaded wall should be based on the height to which the selected support conditions reach.

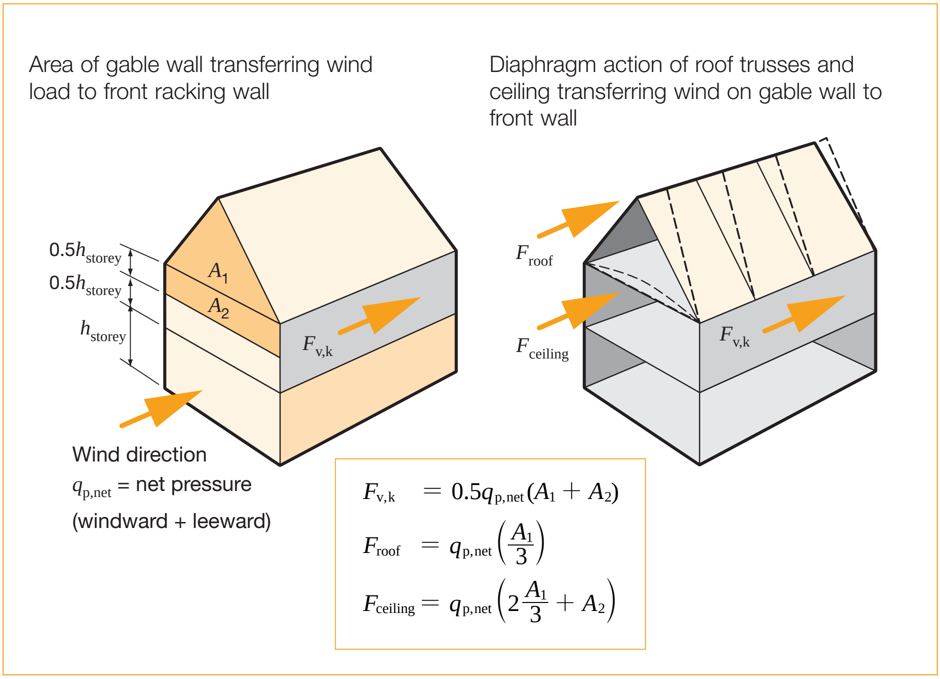


Fig 10.2 Racking load on first floor front wall from wind on gable wall

10.3.1.3 Wind transfer to racking walls

Figure 10.2 illustrates an example of how wind load is resisted in a timber frame house. It shows the racking load on the front first floor wall produced by wind on a gable wall, and the required diaphragm actions in the roof and ceiling.

10.3.2 Other actions

Dead and imposed loads on timber frame buildings and their design values are derived in the same way as they are for other structures (see Section 3).

10.4 Material selection

10.4.1 Roofs

For roofing materials, see Section 7.

10.4.2 Floors

For flooring materials, see Section 8.

10.4.3 Walls

10.4.3.1 Specification

For the specification of materials, see Section 3.5.

10.4.3.2 External walls

External timber frame wall panels (see Figure 10.3) most commonly consist of a structural sheathing material (usually OSB/3) and plasterboard nailed or screwed onto opposite sides of softwood framing and studwork with the spaces filled by a suitable insulating material. For economy, standard sheet sizes should be used wherever possible, typically 2400mm × 1200mm or 2440mm × 1220mm. A typical construction for walls without openings in buildings up to 4 storeys is:

- structurally graded C16 framing members, specified with 'no wane', cross-section 38mm × 89mm, 44 × 97mm or 38 × 140mm (depth governed by thermal insulation requirements and method of insulation – 140mm is increasingly common)
- stud spacing 600mm (maximum); where possible spacing should match joist centres which are normally 600mm but may be 400mm or 450mm to reduce joist depth
- top and bottom rails nailed to studs with a minimum of 2 no. nails of 3.0mm ϕ galvanized smooth round steel wire nails or 3.1mm machine-driven galvanized steel nails, 75mm long
- external sheathing 9.0mm thick OSB/3
  - for class 1 buildings (Table 5.5) fastened to studs with 3.0mm ϕ galvanized smooth round steel wire nails or 2.8mm ϕ galvanized machine-driven steel nails
  - for class 2 buildings fastened to studs with 3.35mm ϕ galvanized smooth round steel wire nails or 3.1mm ϕ galvanized machine-driven steel nails; all at least 50mm long, spaced at 150mm on perimeter, 300mm on internal studs
- insulating material between the studs of a type and thickness necessary to achieve the required U-value, depending on the form of the external cladding and the thermal efficiency of the building (see Table 10.2)
- 12.5mm thick gypsum plasterboard suitable for 30 minutes' fire resistance fastened to the internal face with 2.65mm ϕ plasterboard nails or plasterboard screws at least 40mm long, maximum fastener spacing 150mm around perimeter and on internal studs if relevant
- edge studs of adjacent wall panels nailed together with 3.35mm ϕ galvanized smooth round steel wire nails or 3.1mm ϕ machine-driven galvanized steel nails, 75mm long at 250mm centres
- tops of wall panels linked by a head binder member or wall plate similar to the studwork, nailed to the top rails and continuous across the panel joints fastened with 1 no. 3.0mm ϕ galvanized smooth round steel wire nails or 1 no. 3.1mm ϕ machine-driven galvanized steel nails, 75mm long at 300mm centres; for building occupancy classes higher than 1 denser fastening may be required (see Appendix A)
- bottoms of wall panels fastened through the bottom rail to sole plate or floor below with 4mm diameter galvanized steel wire nails (the length depends on the construction details but the pointside penetration should be at least 37mm):

– in open wall panels, to which plasterboard has not yet been attached, one nail between each pair of studs in class 1 buildings (Table 5.5)
– in closed wall panels, to which the plasterboard has already been attached, skew nailed at 300mm centres.
– for building occupancy classes higher than 1 denser fastening may be required (see Appendix A).

For normal site practice it is preferable to use one nail type throughout. In most circumstances this will be a 3.1ϕ machine-driven galvanized steel nail 90mm long.

The minimum stud sections should be 38mm × 89mm on external walls and 38mm × 63mm on internal walls, but see Table 10.3.

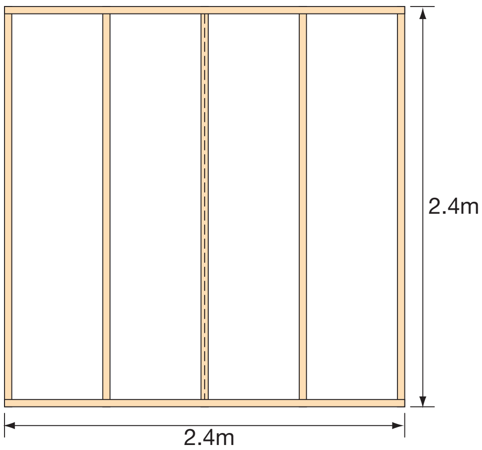


Fig 10.3 Standard timber frame wall panel

10.4.3.3 *Thermal insulation*

Table 10.2 shows most of the commonly available types of insulation which are suitable for timber frame wall panels. There are also proprietary types of wall panel such as structural insulated wall panels, or SIPS, in which a rigid insulating foam is glued between structural board materials and a proprietary connection system is used between the panels. Alternatively where very high thermal performance is required the depth of the wall can be increased by the use of prefabricated timber I-joists. The strength and stiffness of such specialised forms of panel is generally determined by test rather than calculation.

To prevent air leakage a sealing bead should be applied along junction between the inner face of the sole plate and the foundation, or between the inner face of the bottom rail of an upper floor wall and the floor deck.

Table 10.2 Insulation materials suitable for timber frame walls

Insulation type	Environmental rating[a]	Mean density (kg/m³)	Thermal conductivity, λ (W/m²K)
Glass wool insulation	A	10 - 32	0.040 – 0.033
Rock wool insulation	A	23 - 45	0.040 – 0.033
Expanded polystyrene (EPS)	A		0.040 – 0.032
Recycled cellulose insulation	A	35 - 45	0.044 – 0.038
Corkboard insulation	B	120	0.050 – 0.040
Foamed glass insulation	B		0.042
Polyurethane insulation (PU)(HCFC free)	B	30 - 40	0.028 – 0.022
Extruded polystyrene (XPS) (HCFC free)	C	< 40	0.036 – 0.027
Hemp insulation	X[b]	35	0.039
Wool insulation	X[b]	25	0.039

Notes

a Rating from *Green Guide to Housing Specification*[116], based on 12 different environmental impact types. A = best, B = middle, C = worst.

b X = not rated, but probably would be rated 'A'.

10.4.3.4 Internal walls

Internal walls are constructed in a similar manner to external walls except that 12.5mm plasterboard is used on both sides and the stud size may be reduced to 38mm × 63mm. If they are required to carry vertical or horizontal loads the stud depth should be at least 72mm. If necessary an additional layer of structural sheathing materials may be introduced beneath the plasterboard to provide additional racking resistance.

10.4.3.5 Party walls

Both separating walls and compartment walls are required to provide 60 minutes of fire resistance from either side independently. Separating walls must also provide airborne sound insulation.

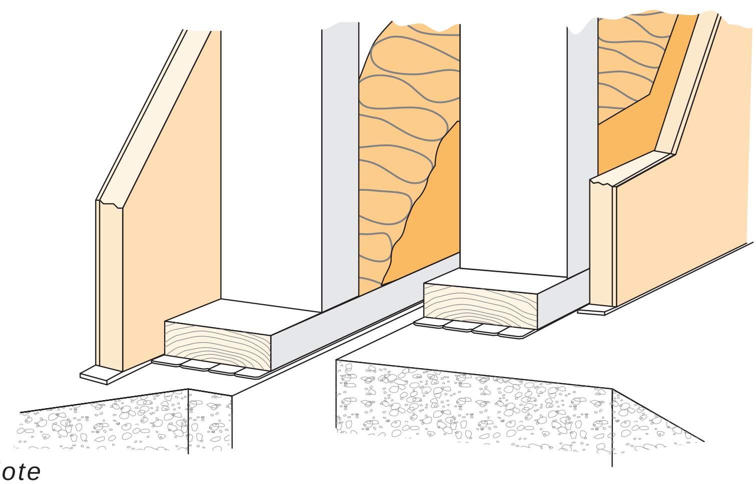
Timber frame party walls consist of two separate wall panels with a gap between them. Normally the frames are each sheathed only on the room face with two layers of plasterboard, 19.5mm thick and 12.5mm thick respectively, the joints being staggered. It is particularly important for structural reasons that the inner layer is fixed to the framing with specified fasteners at the specified spacings. To limit sound transmission the distance between the cavity-facing surfaces of the plasterboard should be at least 200mm and preferably 250mm. There should also be a gap in the cavity of at least 50mm between the framing members, with some designers specifying 75mm. A structural connection between the two sides may be made by means of light steel ties, but to limit sound transmission these should be no more than 2.5mm or 3mm thick and no more than 40mm wide, fastened at 1.2m centres beneath the top rail at the eaves and beneath each intermediate floor. The strength of these connections can be critical when it is necessary to transfer a proportion of the racking forces on an end-of-terrace house into the adjacent property.

If the plasterboard is to provide racking resistance in a party wall, only the board attached directly to the framing should be considered in the calculations and this should be a moisture-resistant grade. However the fasteners used to attach the second layer of plasterboard will contribute to the racking resistance of the first layer provided that the spacing in the two layers is staggered so that minimum permitted spacings are maintained in the first layer. Their additional contribution may then be calculated using the appropriate pointside penetration depth into the solid timber.

A second option is to attach a layer of full height structural sheathing to each leaf. If it is attached to the cavity side a gap of at least 50mm should be preserved between each leaf. Often a length of 0.5h is specified at each end, but no individual piece should be shorter than 0.25h. If such sheathing is attached to the room side of the framing then it may extend for the full length of the wall and be covered later by two layers of plasterboard as specified above, but in this case only the racking resistance of the structural sheathing may be utilised.

The provision of any additional bracing must be accompanied by adequate holding-down arrangements to prevent party wall panels from sliding and overturning. For further details see TRADA's *Timber Frame Construction*[76].

Figure 10.4 illustrates one common type of timber frame party wall. To prevent air and sound leakage a sealing bead should be applied along junction between the bottom of the plasterboard and the foundation, or between the inner face of the bottom rail of an upper floor wall and the floor deck.



Note

Designed as two individual wall units separated by a cavity, the sound performance is comparable to that of a 240mm thick concrete wall. Each wall unit has plasterboard linings on its sides and is filled with insulation between the wall studs. (It is perhaps more common to install a timber sole plate between the bottom rail of the wall panel and the foundation.)

Fig 10.4 Timber frame party wall on concrete slab

Since the plasterboard is usually attached at a fairly late stage in the building it cannot contribute racking resistance during the execution phase. Therefore if no other permanent bracing is specified for party walls it is generally necessary to specify temporary diagonal bracing fastened with removable power driven screws to the cavity face of the framing.

The stability of the wall studs during the execution phase should also be verified. If necessary thicker studs should be specified, or else structural sheathing on the room side of each leaf (with the plasterboard attached on top of it later). In the latter instance, it may be necessary to skew nail at least one leaf to the sole plate or floor below through the sheathing, unless the framing is fastened down before the sheathing is attached.

In medium-rise buildings measures for stabilising the wall studs may be necessary for the finished construction (see Section 10.8.5.1).

10.4.3.6 Plasterboard, fire resistance and insulation requirements

The thickness and type of plasterboard and insulation used for both walls and floors depends on the requirements for fire resistance and thermal and acoustic insulation specified in the relevant Building Regulations, and also on the requirements of the plasterboard within the building. For example boards with a vapour control layer may be specified for external timber frame walls to avoid the need to fix a separate vapour control barrier. In general one layer of 12.5mm thick Type A plasterboard will provide a standard 30 minutes' fire resistance, while two layers of 12.5mm plasterboard Type A with the joints staggered will provide 60 minutes' fire resistance. Maximum fastener centres should be 150mm for nails or 230mm (9 inches) for screws, with all joints filled and taped. Table 10.3 gives some typical values obtained mainly from test data, but for exact specifications and installation requirements the manufacturer's literature should be consulted. Since specifications for both plasterboard and insulation have changed in recent years it is important to establish that test results, particularly for fire resistance, relate to the products currently manufactured.

For further information on meeting energy conservation requirements with timber frame walls, see *Part L Brochure: Comply without costing the earth*[117].

10.5 Overall stability calculations

10.5.1 General (EC0 6.4.1(1)(a))

Since platform timber frame buildings are relatively lightweight, it is necessary to verify their overall stability under wind loading with respect to overturning, sliding and roof uplift, both during the execution phase and after completion. During the execution phase the weight of the roof tiles should be excluded, and the contribution of plasterboard to racking resistance should be excluded unless factory-built units with pre-fitted plasterboard are used.

All imposed loads should be taken as zero, and wind pressures should be calculated as described in Section 10.3.1. For the design values of the loads see Section 3.2.1.3. As stated in Section 10.3.1.1 it may be possible to justify the design by applying the most severe pressure coefficient for each verification to the entire roof. If that is not possible then the more precise approach described in the following sections should be followed.

Table 10.3 Illustrative specifications for timber frame stud walls for particular performance requirements – not to be used without confirmation from the specified product manufacturers[a]

Wall type[b]	Minimum stud sizes[c] (mm)	Plasterboard[d]	Insulation thickness within frame	Fire resistance (minutes)	Thermal transmittance U (W/m²K)	Sound insulation R_w[e] (dB)
External	38 x 89	1 layer 12.5mm Type A	90mm	30	0.35	50
External	38 x 140	1 layer 12.5mm Type A	140mm	30	0.31	55
Internal	38 x 63[f]	1 layer 12.5mm Type A	65mm	30		40
Internal	44 x 75	1 layer 15mm Type D[g] with staggered joints	None	30		43
Internal, party[h]	38 x 89	1 layer 19mm + 1 layer 12.5mm Type A with staggered joints on each leaf	65mm in each frame	60		47 – 65, mean value of 55

Notes

a The values shown relate to walls insulated with Isowool, a glass mineral wool, and are reproduced by kind permission of the manufacturer. Other types may also be used (see Table 10.2). Insulating materials and plasterboards are produced in different densities, and the manufacturers should be consulted on the product types to which such test data apply.

b External walls have 1 or 2 layers of plasterboard on the inner face; internal walls have 1 or 2 layers on both faces. For thermal and sound values 'External walls' assumes 100mm brickwork or blockwork, a 50mm cavity, and 9mm thick plywood or OSB sheathing on the outer face.

c Minimum stud centres 600mm.

d Plasterboard types from BS EN 520[51].

e Airborne sound. Improved sound performance can be obtained by the use of proprietary resilient bars.

f Not to be used for load-bearing purposes.

g Type D is a high density, sound-insulating board.

h Two separate timber frames spaced 50mm apart, consisting of timber studs with mid-height noggings. Two layers of plasterboard on the internal face of each frame.

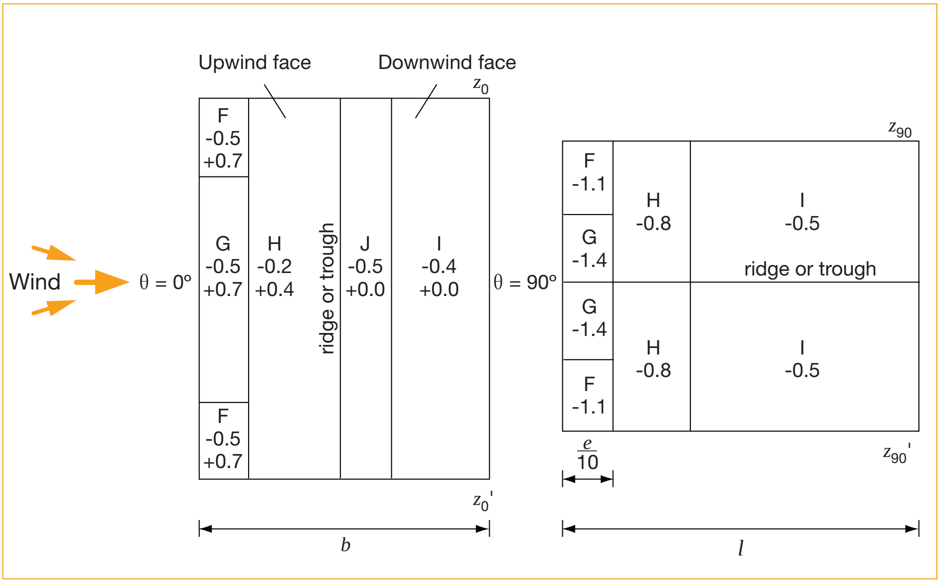


Fig 10.5 Wind zones on a 30° duopitch gable roof (EC1-1-4 7.2.5)

For certain pitches of roof two sets of external pressure coefficients are given, and the critical coefficients may differ for different verifications. Figure 10.5 illustrates the $c_{pe,10}$ values for a duopitch gable roof with a pitch of 30° and Table 10.4 illustrates the corresponding $c_{pe,10}$ values which would be used to check the stability and racking resistance of a timber frame building.

10.5.2 Overturning

The overturning calculation can be illustrated using the examples in 10.5.1, where:

A_F, A_G	=	total area of roof region *F, G* etc.
z-z'	=	axis of rotation – centre line of base of structural wall
a_F,a_G, etc.	=	horizontal distances between z and centre of areas A_F, A_G etc.
h_F,h_G, etc.	=	vertical distances between z and centre of areas A_F, A_G etc.
b, l	=	plan dimensions of structural walls
α	=	roof pitch
q_p	=	peak velocity wind pressure (without masonry shielding reduction)
$F_{building,k}$	=	characteristic dead weight of building (excluding the tile weight in the execution phase)

Table 10.4 Illustrative values of $c_{pe,10}$ for overall stability and racking resistance verifications

Verification	Wind coefficient zone					Comments
	F	G	H	I	J	
	Wind perpendicular to the ridge θ = 0°					
Overturning about *z-z'*	-0.5	-0.5	-0.2	-0.4	-0.5	
Sliding	+0.7	+0.7	+0.4	-0.4	-0.5	Wind friction forces may generally be disregarded (see EC1-1-4 5.3(4))
Roof uplift	N/A	N/A	N/A	-0.4	-0.5	Calculate uplift on more severe side of ridge, resisted by half the roof weight[a]
Racking	+0.7	+0.7	+0.4	-0.4	-0.5	Use for horizontal racking load and for uplift which reduces vertical load on wall panels
	Wind parallel to the ridge θ = 90°					
Overturning about *z-z'*	-1.1	-1.4	-0.8	-0.5		
Sliding	N/A	N/A	N/A	N/A		Wind friction forces may generally be disregarded (see EC1-1-4 5.3(4))
Roof uplift	-1.1	-1.4	N/A	N/A		Assume roof trusses are separate members and check worst case
Racking	-1.1	-1.4	-0.8	-0.5		Use for horizontal racking load and for uplift which reduces vertical load on wall panels

Note

a If necessary a more accurate calculation using the moments about the opposite eaves exerted by all the wind coefficient zones may be used in conjunction with the restoring moment of the whole roof.

For wind perpendicular to the ridge the clockwise design overturning moment about $z\text{-}z'$ is:

$$M_{90,d} = \gamma_Q \begin{bmatrix} q_p \cos\alpha(0.5A_F a_F + 0.5A_G a_G + 0.2A_H a_H + 0.4A_I a_I + 0.5A_J a_J) \\ + q_p \sin\alpha(-0.5A_F h_F - 0.5A_G h_G - 0.2A_H h_H + 0.4A_I h_I + 0.5A_J h_J) \\ + \dfrac{F_{v,k} h_e}{2} \end{bmatrix}$$

Where γ_Q = partial factor for variable loads = 1.5

$F_{v,k}$ = total characteristic wind force on windward and leeward walls, after allowing for any masonry shielding in completed phase (see Section 10.3.1.2). With masonry shielding the maximum value of $F_{v,k}$ will normally depend on the wind direction. If the masonry shielding itself is more than 4 storeys high a higher wind pressure will be applicable from the fifth storey of masonry upwards (see Section 10.3.1.2).

The design restoring moment about $z_{90} - z_{90}'$ is:

$$M_{R,90,d} = \frac{\gamma_G F_{building,k} b}{2} = 0.45 F_{building,k} b \ \text{(see Table 3.1)}$$

For wind parallel to the ridge the clockwise design overturning moment about $z_0 - z_0'$ is:

$$M_{0,d} = \gamma_Q \left[\begin{array}{l} q_p \cos\alpha (1.1 A_F a_F + 1.4 A_G a_G + 0.8 A_H a_H + 0.5 A_I a_{IJ}) \\ + F_{v,spandrel,k} \left(h_e + \dfrac{h_r}{3} \right) + \dfrac{F_{v,k} h_e}{2} \end{array} \right]$$

Where γ_Q = partial factor for variable loads = 1.5

$F_{v,k}$ = characteristic wind force on windward wall for end-of-terrace building or total wind force on windward and leeward walls for detached building, excluding the spandrel area of the gable walls, after allowing for any masonry shielding in the completed phase – see Section 10.3.1.2. With masonry shielding the maximum value of $F_{v,k}$ will normally depend on the wind direction

$F_{v.spandrel,k}$ = characteristic wind force on windward spandrel for end-of-terrace building or net wind force on windward and leeward spandrels for a detached building. For hip-ended buildings or buildings with a flat roof $F_{v,spandrel,k} = 0$

h_e = height to eaves

h_r = height of ridge from eaves

The design restoring moment about $z_0 - z_0'$ is:

$$M_{R,0,d} = \frac{\gamma_G F_{building,k} l}{2} = 0.45 F_{building,k} l \ \text{with } \gamma_G = 0.9 \ \text{(see Table 3.1)}$$

If $M_d > M_{R,d}$ specify galvanized anchor bolts or screws, or galvanized sole plate fixing brackets, spaced at 600mm to 1200mm centres to attach the sole plate to the concrete foundations along the wall $z - z'$ and the opposite wall. The fixing brackets may be attached to the edge of the sole plate as in Figure 10.6a) or preferably as in Figure 10.6b). Where there is a masonry wall, galvanized steel or stainless steel ‘timber frame holding down straps’ are often specified as shown in Figure 10.6c). Although commonly employed, these work best if at least 150mm concrete cavity fill is placed in the trench above them, and in practice this is usually omitted. The Engineer should consider durability requirements within a cavity (thin galvanized mild steel is unlikely to be suitable) and whether the strap should be designed to resist bending. Since such straps provide little lateral restraint against sliding, direct sole plate anchorage may be preferable.

The restraints should provide a total design restraining force along each wall of:

$$\left(\frac{M_{90,d} - M_{R,90,d}}{b} \right) \text{or} \left(\frac{M_{0,d} - M_{R,0,d}}{l} \right) \quad \text{as appropriate.}$$

A 1.5mm thick steel sole plate fixing bracket with 4 no. 2.65mm $\times$ 40mm diameter nails attaching it to the side of a 38mm C16 sole plate gives an instantaneous design load in service class 2 of $4 \times 0.505 = 2.02$kN. The clip is fastened to the concrete slab or footing using ballistic or masonry nails, or expanding screw fixings. Such fasteners may be used to locate the sole plates during setting out, so additional restraints may not be necessary.

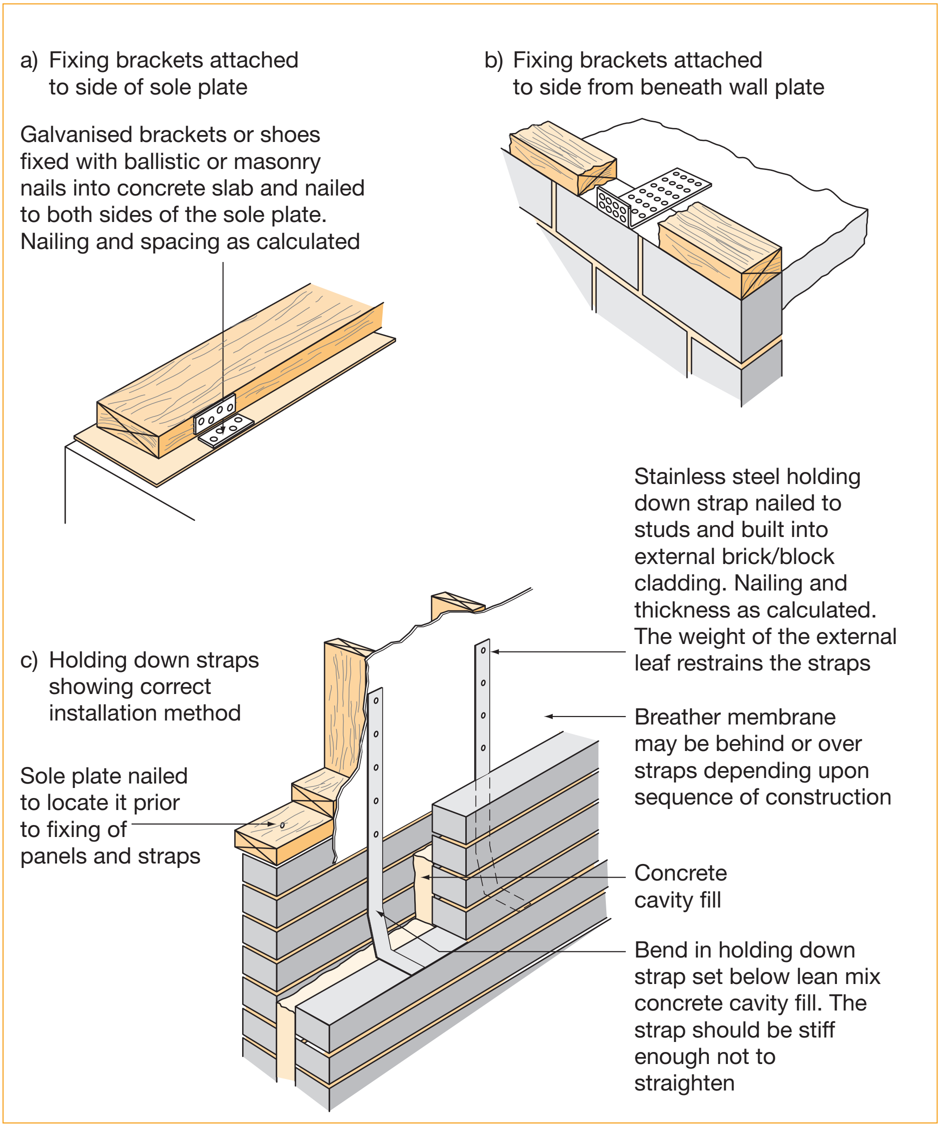


Fig 10.6 Timber frame holding down methods

10.5.3 Sliding

The sliding calculation can be illustrated using the examples in Section 10.5.1, where $F_{v,k}$ and $F_{v,spandrel,k}$ are defined as in Section 10.5.2.

For wind perpendicular to the ridge the design sliding force is:

$$F_{v,90,d} = \gamma_Q\left[q_p \sin\alpha(-0.7A_F h_F - 0.7A_G h_G - 0.4A_H h_H + 0.4A_I h_I + 0.5A_J h_J) + F_{v,k}\right]$$

and for wind parallel to the ridge the design sliding force is:

$$F_{v,0,d} = \gamma_Q\left[F_{v,k} + F_{v,spandrel,k}\right]$$

Where $\gamma_Q = 1.5$

The sliding force, $F_{v,d,max}$, is the greater of the two values.

It is common practice to level sole plates by inserting proprietary plastic shims beneath them. This reduces the frictional resistance to sliding to an unknown value. The Engineer is therefore recommended to specify positive restraints around all the structural perimeter walls, as previously recommended in BS 5268-6-1[118] and as described in Section 10.5.2. These should provide a total design restraining force of at least $F_{v,d,max}$. If, however, friction is utilised, a coefficient of 0.25 is recommended and a partial factor of 0.9 should be applied to the characteristic dead weight of the building; any further lateral resistance still required may then be provided by more positive restraints.

10.5.4 Roof uplift

It is recommended to attach every trussed rafter to the wall plate with truss clips, whether or not there is a possibility of roof uplift, because this makes a significant contribution to the strength of the horizontal diaphragm in the ceiling plane.

The roof uplift calculation can be illustrated using the examples in 10.5.1, where $F_{roof,k}$ is the characteristic weight of the roof, excluding the tile weight when the execution phase is being considered.

For wind perpendicular to the ridge the simplest approach is to calculate the uplift force on the more severely loaded side of the ridge and compare this with half the roof weight. In this case:

$$F_{90,d} = \gamma_Q q_p \cos\alpha(0.4A_I + 0.5A_J) \text{ with } \gamma_Q = 1.5$$

The design resisting force applied by half the roof weight is:

$$R_d = 0.5\gamma_G F_{roof,k} = 0.45F_{roof,k} \text{ with } \gamma_G = 0.9 \text{ (see Table 3.1)}$$

If necessary a more accurate calculation can be undertaken using the moments exerted by all the wind coefficient zones in conjunction with the restoring moment of the whole roof.

If $F_{90,d} > R_d$ specify truss clips to attach the roof trusses to the head binder or top rail of the wall panels. These should provide a total design restraining force of at least $(F_{90,d} - R_d)$

on each side of the roof. Truss clips should be tested to ETAG 015[105], but currently few have been, and characteristic load capacities for truss clips are generally not available. Some manufacturers quote a safe working load for uplift, in which case it should be possible to calculate a safe characteristic value for the resistance to uplift as $R_k = 1.34 \times \text{SWL}$. However tests carried out by TRADA[119] demonstrated that an additional horizontal component of load can reduce the vertical load capacity by a factor as high as 2.8. In practice there may be little horizontal load on the truss clips, but it is suggested that an additional safety factor of 2 should be applied to characteristic values for uplift resistance, whether they are derived directly from tests or from quoted safe working loads. An alternative approach is to calculate the characteristic load capacity of the nails in the wall plate and the truss, but once again the resulting values should be halved since a) the edge distances in the wall plate are considerably sub-standard and b) the most common mode of failure is not in the nailed connections but in the bending strength of the clip at the junction between the roof truss and the wall plate which is difficult to calculate reliably.

The design resistance to uplift of a truss clip with a characteristic resistance R_k is:

$$R_d = \frac{k_{mod} k_{sys} R_k}{\gamma_M} = \frac{1.1 \times 1.1 \times R_k}{1.3}$$

Although truss clips generally fail in the steel, for which $\gamma_M = 1.1$, it is possible that the nails in the wall plate may fail, so a value of $\gamma_M = 1.3$ for connections is suggested.

For wind parallel to the ridge the design uplift force should be calculated for one side of a single truss in the most severely loaded zone. In the example shown in Figure 10.5:

$$F_{0,d} = \gamma_Q q_p \cos\alpha (1.1 A_F + 1.4 A_G) \times \frac{10s}{2e}$$

Where s = trussed rafter spacing

e = the cross-wind building width or twice its height, whichever is smaller

The design resisting force applied by the roof weight on one truss is:

$$R_d = \frac{0.5 s \gamma_G F_{roof,k}}{2l} = \frac{0.225 s F_{roof,k}}{l} \text{ with } \gamma_G = 0.9 \text{ (see Table 3.1)}$$

Each truss, at least in the most severely loaded roof zones, should be restrained by a truss clip at each eaves point with a design resistance to uplift of at least $(F_{0,d} - R_d)$, determined as for wind perpendicular to the ridge.

10.5.5 Roof sliding

Although horizontal wind loads on the roof can be calculated in a similar way it is even harder to obtain data for the resistance of truss clips to horizontal loading. It is recommended that if the stability in this direction cannot be demonstrated by friction using a coefficient of friction of 0.25 then trussed clips to every rafter should be specified.

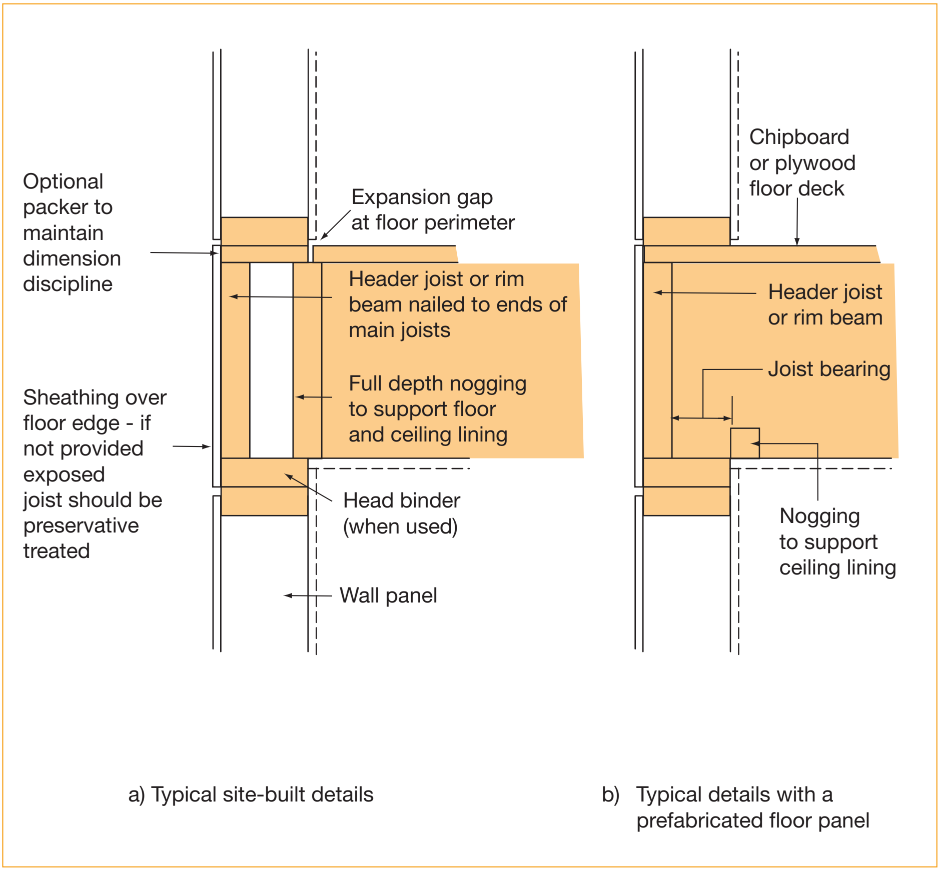


Fig 10.7 Typical intermediate floor/external timber frame wall junction details: joists at right angles to wall

10.6 Roof design

For the design of the roof, see Section 7. Platform timber frame buildings nearly always use a prefabricated trussed rafter roof designed by the supplier. In this case the Engineer should provide the roof designer with the information listed in Section 7.1.2.

10.7 Floor design

Floor design is covered in Section 8. Figures 10.7 and 10.8 shows some typical details for the floor to wall junction in timber frame buildings.

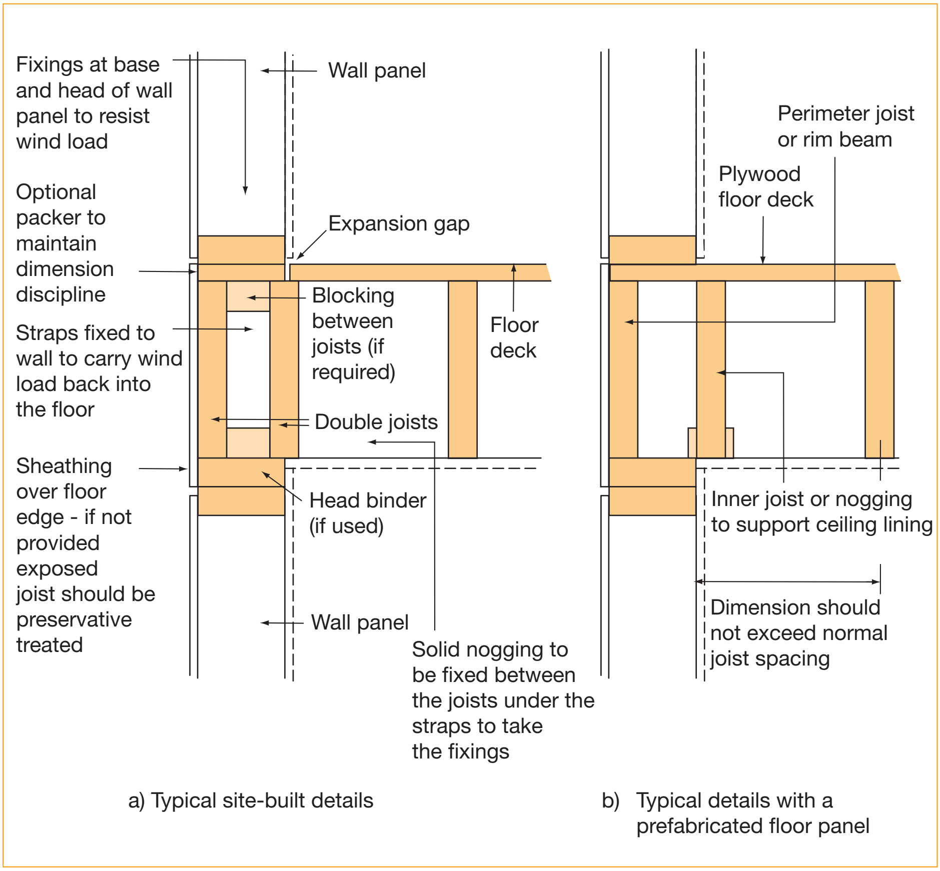


Fig 10.8 Typical intermediate floor/external timber frame wall junction details: joists parallel to wall

10.8 Wall design

10.8.1 Racking resistance of timber frame walls

10.8.1.1 Design method

The method for designing wall timber frame diaphragms given in this section is based on Method B in EC5 9.2.4.3, as specified in the NA. This covers requirements for strength, stability and stiffness. EC5 method B is derived from BS 5268-6[118], so compliance should ensure that the horizontal deflection of individual wall panels does not exceed 0.003*h* or *h*/333, where *h* is the storey height. However for medium-rise buildings tighter deflection limits may be more suitable (see Table 3.5).

The design method applies with the following definitions.

- A *wall panel* comprises a unit of timber framing sheathed on one or both sides with a panel product. The framing consists of a top and bottom timber rail nailed or screwed to the top and bottom ends of well fitting vertical timber studs. A typical layout is shown in Figure 10.3.
- A *wall* is a series of one or more wall panels fastened together side by side by nails or screws through their adjoining studs and bounded at each end by a wall in another plane or an opening for a window, door or arch.
- A *wall assembly* is a series of adjacent walls and openings in one plane.
- h = overall height of the wall in m.
- b_i = total length of wall i in m.
- $F_{i,v,Ed}$ = design racking load applied to wall i in kN.

The method is applicable to external walls (which normally have structural sheathing on one side and gypsum plasterboard on the other), to internal load-bearing walls (which normally have plasterboard on both sides), and to party walls (which often have two layers of plasterboard on one side only). Where more than one sheet of plasterboard is attached to the same side of a timber frame, only the layer attached to the frame should be considered as contributing racking resistance, but the fasteners for the secondary boards may be included provided that their penetration into the framing is adequate.

For each floor and for each of the two principal wind directions verify that:

$$F_{v,Ed} \leqslant \sum F_{v,Rd}$$

Where $F_{v,Ed}$ = total design racking force in the direction considered, calculated in accordance with Section 10.8.1

$$F_{v,Rd} = \sum F_{i,v,Rd}$$

Where $F_{i,v,Rd}$ = design racking resistance of a constituent wall, i

$F_{i,v,Rd}$ depends on the shear strength of wall i and its resistance to rotation (see Sections 10.8.1.4 and 10.8.1.5).

10.8.1.2 Limitations

The guidance given in Section 10.8.1.1 is applicable with the following provisions.

- The wall panels are assembled as described in Section 10.4.3.2 (with permitted variations as described below).
- $2.1\text{m} \leqslant h \leqslant 2.7\text{m}$.
- $b_{net} \leqslant 100t$.

Where b_{net} = clear distance between the studs
t = thickness of the principal sheathing material.
(This is to prevent buckling of the sheathing.)

- If the studs are spaced more than 650mm apart additional horizontal framing members at 650mm centres or less are provided to support plasterboard.
- In any one wall assembly the spacing of the fasteners around the perimeter of each sheet of sheathing material is constant, although it may differ on each side of the wall panels if there is sheathing on both sides.
- The spacing of the fasteners on intermediate studs is constant and no greater than twice the spacing of the corresponding fasteners around the perimeter of the sheet.
- The fastener spacing around the perimeter of the plasterboard does not exceed 150mm; the fastener spacing s around the perimeter of each structural sheathing panel is within the range 50mm $\leqslant s \leqslant$ 150mm for nails or 50mm $\leqslant s \leqslant$ 200mm for screws and the spacing on intermediate studs is $2s$.
- Only wall panels with a width of at least $h/4$ are considered to contribute to racking resistance.
- The end studs of adjacent panels, including panels which meet at a corner, are connected together with fasteners which have a design resistance to lateral load per unit length at least equal to $F_{i,v,d}/b_i$ per unit length – this is to transfer from one panel to the next one the vertical restraint force required to prevent rotation. In addition EC5 9.2.4.3.1(4) requires a minimum design resistance under wind loading of 2.5kN/m, which can be achieved in 38mm thick C16 framing by using 3.35mm ϕ galvanized smooth round steel wire nails or 3.1mm ϕ machine-driven galvanized steel nails at 250mm centres, or 3.75mm ϕ galvanized smooth round steel wire nails or 3.25mm ϕ machine-driven galvanized steel nails at 300mm centres, all 75mm long.
- The tops of the panels are linked by a member or assembly which spans the joint – this is to distribute any point loads and to improve load transfer between panels.
- Studs which support a lintel over an opening are connected to the end studs on adjacent panels as above.
- The bottom rail of each panel is connected to the substrate as above.
- The framing, sheathing, plasterboard and fasteners are suitable for structural use in timber frame wall panels, in accordance with Tables 3.27 to 3.29. Fasteners for use in the framing of external wall panels and to attach the external sheathing to the framing should have the corrosion protection specified for service class 2 (see Table 3.25).

10.8.1.3 Fastener data and basic racking resistances

Design values for the lateral load capacities of nailed and screwed connections can be found in the CD. For convenience Tables 10.5 to 10.8 give values for wind loading for some common joint specifications used in timber frame walls. The basis on which these values were calculated can be found in the CD. Table 10.9 gives the basic racking resistance of some complete wall assemblies.

Table 10.5 Design lateral load capacities under wind loading in service class 2 for some common connections made with smooth round nails driven into C16 timber (kN)

Headside member		Nail diameter (mm) x length (mm)					
Material	Thickness (mm)	2.65 x 50 ps[a]	3.00 x 50	3.35 x 50	3.35 x 65 ps[a]	3.35 x 75	3.75 x 75 ps[a]
Galvanized steel	1.0 – 1.6			0.748	0.748		0.904
	1.9 – 5.0			1.029	1.032		1.246
C16 timber	38				0.603	0.723	0.851
OSB/3	9	0.400	0.463	0.534	0.552		0.649
	11	0.431	0.492	0.555	0.578		0.677

Note

a ps = preferred size (most commonly available).

Table 10.6 Design lateral load capacities under wind loading in service class 2 for some common connections made with machine driven nails driven into C16 timber (kN)[a]

Headside member		Nail diameter (mm) x length (mm)					
		Smooth shank			Threaded shank		
Material	Thickness (mm)	2.8 x 51	3.1 x 75	3.25 x 75	2.8 x 51	3.1 x 75	3.25 x 75
Galvanized steel	1.0 – 1.4			0.763			0.763
	1.7 – 5.0			1.054			1.054
C16 timber	38		0.696	0.744		0.705	0.763
OSB/3	9	0.449	0.533	0.568	0.471	0.576	0.612
	11	0.477	0.564	0.597	0.498	0.600	0.635

Note

a Assumed ultimate tensile strength of 700N/mm².

Table 10.7 Design lateral load capacities under wind loading for Type A plasterboard attached to C16 timber with plasterboard screws[a]

Plasterboard thickness (mm)	Screw dimensions Diameter x length (mm x mm)	Service class	Design capactity (kN)
12.5	3.5 x 45	1	0.399
	3.9 x 55		0.417
15.0	3.5 x 60	2 (Party wall)	0.517
	3.9 x 55		0.542
19.0	3.5 x 60	2 (Party wall)	0.567
	3.9 x 55		0.596

Note

a Assumed tensile strength = 540kg/m³.

Table 10.8 Design axial load capacities under wind loading in service class 2 for some common connections made with fasteners driven into C16 timber (kN)

Headside member		Fastener type							
		Ringed shank hand driven nails		Machine driven nails[a]		Wood screws[b]			
				Smooth	Threaded	Clearance hole headside[c]		No clearance hole headside[d]	
		Fastener diameter (mm) x length (mm)[e]							
Material	Thick-ness (mm)	3.35 x 65	3.75 x 75	3.25 x 75	3.25 x 75	No. 8 (4.17) x 75	No. 10 (4.88) x 100	No. 8 (4.17) x 75	No. 10 (4.88) x 100
C16	38	0.221[f]	0.320[g]	0.165[g]	0.240[g]	0.396[g]	0.542[g]	1.00[f]	1.75[h]
Steel	1	–	–	–	–	1.00[f]	1.97[f]	1.00[f]	1.97[f]

Notes

a Manufacturers may have higher independently verified test values.

b These values are calculated as 4 times the smooth nail withdrawal resistance because the EC5 expressions for screw withdrawal have been reported to be unsafe. They apply to conventional wood screws, but should also be safe for modern screws with a thread root diameter of 0.6d or more. In softwoods EC5 allows screws up to 6mm in diameter to be inserted with or without predrilling, but if predrilling is not specified it is advisable to specify a modern screw profile.

c A clearance hole in the headside is required if a conventional wood screw with a smooth shank in the headside is specified.

d These values apply to modern screws threaded for their full length and driven either without predrilling or with a predrilled hole in both headside and pointside having a diameter $d \leqslant 0.75d$.

e It is assumed that the pointside length is fully embedded in C16 timber.

f Governed by withdrawal resistance from the pointside.

g Governed by head pull-through resistance.

h Governed by head pull-through and withdrawal from the headside.

Table 10.9 Basic racking resistances of some common C16 grade timber frame wall configurations[a] (kN/m)

Wall type	Party Walls		Interior Walls	
	A:Party	B:Party	C:Int	D:Int
Primary board[b]	GYP + GYP	GYP + GYP	GYP	GYP
Service class	2	2	1	1
t_1 (mm)	19+12.5	15+15	12.5	12.5
Fastener type[c]	S	S	S	S
d_1 (mm)	3.9+3.9	3.5+3.5	3.9	3.5
l_1 (mm)	55+55	60+60	55	45
s_1 m[d]	0.15+0.15	0.15+0.15	0.15	0.15
$F_{f,Rd,1}$ (kN)	0.488[e]	0.476[e]	0.417	0.399
$k_{s,1}'$ (m^{-1})	7.48	7.90	5.05	5.23
$F_{f,Rd,1}k_{s,1}'$ (kN/m)	3.65	3.76	2.10	2.09
Secondary board[b]	n/a	n/a	GYP	GYP
Service class	–	–	1	1
t_2 (mm)	–	–	12.5	12.5
Fastener type[c]	–	–	S	S
d_2 (mm)	–	–	3.9	3.5
l_2 (mm)	–	–	55	45
s_2 (m)[d]	–	–	0.15	0.15
$F_{f,Rd,2}$ (kN)	–	–	0.417	0.399
$k_{s,2}'$ (m^{-1})	–	–	5.05	5.23
$F_{f,Rd,2}\,k_{s,2}'$ (kN/m)	–	–	2.10	2.09
$F_{i,Rd}$[f] (kN/m)	3.65	3.76	3.16	3.13

Key

A Party wall, one layer of 19mm gypsum wallboard with a layer of 12.5mm plasterboard on top, each fixed with 55mm long plasterboard screws. The perimeter spacings on both boards are 150mm with the screws in the outer board positioned between the screws in the inner board. The outer board contributes to the racking resistance only by helping the inner wallboards to act as a continuous diaphragm, but the screws in it are treated like additional (55 – 12.5) = 42.5mm screws through the inner board. 0.488kN is the mean of the resistance of the two sets of screws, and the resultant spacing of 75mm is used to calculate $k_{s,1}'$.

B Party wall, like A, but with two layers of 15mm plasterboard each fixed with 60mm long screws. The outer board itself does not contribute to the racking resistance, but the screws in it are treated like additional (60 – 15) = 45mm screws through the inner board. 0.476kN is the mean of the resistance of the two sets of screws.

C Interior wall with one 12.5mm layer of plasterboard on each side, 3.9mm plasterboard screws.

D Interior wall, like C, but with 3.5mm plasterboard screws.

E Exterior wall, low strength.

F Exterior wall, standard strength.

G Exterior wall, extra strength.

H Exterior wall, high strength.

I Exterior wall, very high strength. This wall type has a layer of OSB on the exterior and interior face, with an additional layer of 12.5mm plasterboard on the interior face. On the interior face it is in service class 1 so the connections are stronger, hence the interior OSB is the 'primary' sheathing. The perimeter spacings are 50mm

Exterior Walls						
E:Ext	F:Ext	G:Ext	H:Ext	I:Ext	J:Ext	K:Ext
OSB	OSB	OSB	OSB	OSB+ GYP	OSB	PLY
2	2	2	2	1	2	2
9	9	9	9	9 +12.5	11	9.5
N	N	N	N	N + S	N	N
2.8	3.1	3.1	3.1	3.1+3.9	3.1	3.1
50	50	50	50	50 + 55	75	75
0.15	0.1	0.075	0.05	0.1+0.1	0.05	0.05
0.495	0.51	0.51	0.51	0.595[e]	0.564	0.61
5.60	7.09	8.37	10.20	10.20	10.20	10.20
2.77	3.62	4.27	5.20	6.07	5.76	6.22
GYP	GYP	GYP	GYP	OSB	GYP	GYP
1	1	1	1	2	1	1
12.5	12.5	12.5	12.5	9	12.5	12.5
S	S	S	S	N	S	S
3.5	3.9	3.9	3.9	3.1	3.5	3.5
45	55	55	55	50	45	45
0.15	0.15	0.15	0.15	0.05	0.15	0.15
0.399	0.417	0.417	0.417	0.51	0.399	0.399
5.23	5.05	5.05	5.05	10.20	5.23	5.23
2.09	2.10	2.10	2.10	5.20	2.09	2.09
3.81	4.67	5.32	6.26	8.67	6.80	7.27

on the exterior OSB board and 100mm on each of the two interior boards, with the plasterboard screws positioned between the OSB nails. The plasterboard itself does not contribute to the racking resistance, but the screws in it are treated like additional (55 - 12.5) = 42.5mm screws through the OSB. 0.595kN is the mean of the resistance of the nails and the screws in the OSB, and the resultant spacing of 50mm is used to calculate $k_{s,1}'$. With spacing on the interior boards closer than 100mm it is difficult to ensure that the plasterboard screws are not inserted too close to the OSB nails unless the process is computer controlled.

J Exterior wall, high strength with 11mm OSB.

K Exterior wall, high strength with 9.5mm plywood.

Notes

a See Section 10.8.1.4 iii) for definitions of the symbols. The tabulated values of $F_{i,Rd}$ may be used in Section 10.8.1.4 iii) expression 3) to obtain the shear resistance of a wall.

b OSB = OSB Type 3 or 4 to BS EN 300[20], characteristic density 550kg/m³
PLY = softwood plywood, exterior grade, characteristic density 505kg/m³ (any plywood listed in Table 3.28).
GYP = Type A gypsum plasterboard or better, characteristic density 640kg/m³.

c N = smooth machine driven nail, ultimate tensile strength 700N/mm².
S = plasterboard screw, ultimate tensile strength 540N/mm².

d Spacing around board perimeter. On intermediate studs fastener spacing should be 2s.

e Mean value of $F_{f,Rd}$ for the fasteners for the two boards in the inner board only (see A, B and I).

f $F_{i,Rd} = F_{f,Rd,1}\, k_{s,1}' + 0.5\, F_{f,Rd,2}\, k_{s,2}'$ where $F_{f,Rd,1}\, k_{s,1} \geqslant F_{f,Rd,2}\, k_{s,2}'$.

10.8.1.4 Resistance to panel shear

i) Design one floor at a time. Generally the same wall construction should be specified for all the external walls within a single floor except party walls, in order to avoid construction errors. Check the resistance to panel shear for each of the two orthogonal wind directions.

Choose initial constructions for the external and internal load-bearing walls, including framing (material and cross-section), primary and secondary sheathing (materials and thicknesses), and fasteners (type, diameter, length and spacing). See Section 10.4.3.2 for typical details. For structures of building occupancy class 1 (see Table 5.5) the fastener spacing around the perimeter of the primary sheathing should not exceed 300mm. For building occupancy class 2 the maximum spacing is normally determined by requirements for robustness (Appendix A).

ii) Obtain the design value under wind loading of the lateral load capacity of each fastener in the primary and any secondary sheathing, $F_{f,Rd,1}$ and $F_{f,Rd,2}$ respectively, in kN. Tables 10.5 to 10.8 give some typical values. Alternatively obtain the basic racking resistance of the wall assembly directly from Table 10.9. External timber frame walls and party walls are considered to be in service class 2; internal walls are normally in service class 1.

iii) Calculate the basic design shear resistance of the panels in each wall assembly in the floor.

The basic design shear strength, $F_{i,Rd}$, of a panel in a wall assembly, *i*, is the summation of the shear strength of the primary sheathing (e.g. 9mm OSB/3) and half the shear strength of the secondary sheathing (e.g. 12.5mm plasterboard) if any.

$$F_{i,Rd} = F_{f,Rd,1} k_{s,1}' + 0.5 F_{f,Rd,2} k_{s,2}' \text{ kN/m}$$

Where 1 and 2 refer to the primary and secondary sheathing (if any) respectively

$F_{f,Rd,j}$ = design lateral load capacity of a single fastener under wind loading in sheathing *j* (1 or 2) (kN)

$k_{s,j}'$ = combined fastener spacing factor (= k_s/s_0 in EC5)

$$= \frac{1}{0.86 s_j + \dfrac{5.5 d_j}{\rho_k}} \text{ m}^{-1}$$

s_j = fastener spacing around board perimeter (m)

d_j = fastener diameter where $d_j \leqslant$ 4mm (mm)

ρ_k = characteristic density of timber frame (kg/m^3)

Notes

- The primary sheathing is defined as the sheathing for which $F_{f,Rd,1}k_s'$ is greater.
- Values of $F_{f,Rd}$ for nails are given in Tables 10.5 to 10.7. Pre-calculated values of $F_{i,v,Rd}$ for some typical wall panels are given in Table 10.9.
- s_j and the spacing on intermediate studs should be within the limits specified in Section 10.8.1.2.
- The expression for $k_{s,j}'$ includes a correction to the EC5 expression for s_0 which is too large by a factor of 1000 if d is entered in mm.

iv) Calculate the design shear resistance of each wall assembly in the floor.
For each wall, i, in the assembly, calculate its shear resistance as:

$$F_{i,v,Rd} = F_{i,Rd}\, b_i\, k_{i,d}\, k_{i,q} \text{ (kN)}$$

Where $F_{i,Rd}$ = basic design shear strength of the panels from iii) (kN/m)
b_i = total length of wall i in m, ignoring any panels with a length less than $h/4$ where h = height of wall (m)
$k_{i,d}$ = wall dimension factor

$$= \begin{cases} \dfrac{b_i}{h} & \text{for } 0.25 \leqslant \dfrac{b_i}{h} \leqslant 1.0 \\ \left(\dfrac{b_i}{h}\right)^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i \leqslant 4.8\text{m} \\ \left(\dfrac{4.8}{h}\right)^{0.4} & \text{for } \dfrac{b_i}{h} > 1.0 \text{ and } b_i > 4.8\text{m} \end{cases}$$

$k_{i,q}$ = vertical load factor

$$= 1 + \left(0.067 q_{i,d} - 0.00082 q_{i,d}^{\,2}\right)\left(\frac{2.4}{b_i}\right)^{0.4}$$

Where $q_{i,d}$ = net vertical design load per unit length on top of wall (kN/m)
= permanent loads – wind uplift (if any), with a maximum value of 10.5kN/m

A concentrated vertical load, $F_{vert,Ed}$ kN, acting at distance a metres from the leeward end of a wall, i, is equivalent to a UDL of $\dfrac{2F_{vert,Ed}\,a}{b_i^{\,2}}$ kNm, and may be added to $q_{i,d}$.

Note: The expression for $k_{i,q}$ is more conservative than that given in EC5 to match its derivation from BS 5268-2[3]. The 10.5kN/m is considered necessary for a safe design as in BS 5268-6[118].

v) Calculate the total shear resistance of all the n racking walls parallel to the wind direction in the floor under consideration as: $\sum_{i=1}^{n} F_{i,v,Rd}$ (kN), for each of the two principal wind directions.

If $\sum F_{v,Rd} < \sum F_{v,Ed}$ or $\sum F_{v,Rd} >> \sum F_{v,Ed}$ for either direction, adjust the fastener spacings or diameters or sheathing material accordingly, and repeat steps i) to iv). ($F_{v,Ed}$ is the design racking force in kN.) Otherwise continue to Section 10.8.1.5.

vi) Calculate the total shear resistance of all the wall assemblies resisting wind load in the floor under consideration as $\sum F_{v,Rd}$, for each of the two principal wind directions. If $\sum F_{v,Rd} < \sum F_{v,Ed}$ or $\sum F_{v,Rd} >> \sum F_{v,Ed}$ for either direction, adjust the fastener spacings or diameters or sheathing material accordingly and repeat steps i) to v). ($F_{v,Ed}$ is the racking force in kN.) Otherwise continue to Section 10.8.1.5.

10.8.1.5 Resistance to rotation/overturning (EC0 6.4.1(1)(a))

Introduction

At each floor level it must be demonstrated that the racking walls can resist the rotational or overturning moment produced by the wind acting above that level. The overturning moment on a storey j is calculated as:

$$M_J = F_{wind,j}\, h_j$$

Where $F_{wind,j}$ = the wind load applied between the base of the storey under consideration and the top of the building

h_j = the vertical distance between the centroid of that wind load and the base of that storey

It may be assumed that M_j is divided between the n racking walls parallel to the wind direction in proportion to their racking resistance (kN), so that each wall i resists a rotational moment of $\phi_i M_j$,

Where $\phi_i = \dfrac{F_{i,v,Rd}}{\sum_{1}^{n} F_{i,v,Rd}}$

A series of individual wall panels may be considered to act together to form a single wall (as defined in Section 10.8.1.1) if the fasteners in the vertical joints between the panels have a design resistance per unit length not less than $q_{v,d}$, where $q_{v,d}$ is the racking load per unit length in kN/m, and not less than 2.5kN/m (EC5 9.2.4.3.1(4)). (The standard fixing schedule in Section 10.4.3.2 provides 2.88kN/m.) In addition the heads of the panels should be linked by means of a head binder or wall plate nailed in accordance with Section 10.4.3.2, which provides a splice across the panel joints to resist additional overturning moment. These requirements apply to external and load-bearing internal walls and party walls.

A wall i of length b_i can then be restrained against uplift at its windward end by means of a vertical restraint force of $\phi_i M_j / b_i$ kN applied to its end stud, by an equivalent vertical load applied to the top of the wall by the structure above it, by the attachment of its bottom rail to the foundation or the wall beneath it, or by a combination of these.

Overturning moments increase the axial compression in wall studs at the ends of walls. Allowance should be made for this when checking their buckling strength and the bearing strength of the bottom rail.

Restraint by a force applied to the windward end of the wall

EC5 gives little guidance on how to provide a restraint force to the end of a wall, but any of the following may be utilised.

- Connection of an end wall to an adjacent return wall. The weight of a limited length of return wall and any vertical load on it and its attachment to the foundation or the wall beneath it may all be utilised to hold down the end of a wall.
- Connection of an intermediate wall to a cripple stud supporting the lintel over a doorway or window opening. Half the vertical load on the lintel and the weight of a limited length of wall panel beneath a window and its attachment to the foundation or the wall beneath it may all be utilised.
- Straps and brackets. Vertical restraint straps or bolted brackets with adequate strength and stiffness may be specified to attach the ends of the wall to the foundation or the wall beneath.

The connections between the end studs of each wall and the restraining structure must be able to transmit the required restraining force. In particular any straps or brackets used to restrain the ends of ground floor walls must be able to transmit the required restraining force into the foundation.

More detailed information can be found in Part 3 of the CD.

Restraint by vertical loads on top of the wall

Vertical loads defined as in Section 10.8.1.4 iv) can be utilised to provide an equivalent vertical restraint at the end of the wall.

A net UDL of q_d is equivalent to a vertical restraint of $\frac{q_d b_i}{2}$.

A permanent concentrated vertical load, $F_{vert,Ed}$, acting at a distance of a metres from the leeward end of a wall, i, is equivalent to a vertical restraint of:

$$\frac{F_{vert,Ed}\, a}{b_i}$$

A value of 0.9 should be used for γ_G when calculating the design values of permanent actions loads resisting (see Table 3.1).

Restraint by connection of the bottom rail to the substrate

The expression for racking strength given in EC5 9.2.4.3.2(2) is valid when a panel which resists only a racking load of $F_{i,v,Ed}$ with no additional imposed rotational moments is restrained against the rotation induced by the racking load by fixing its bottom rail to a substrate. Therefore a rotational moment up to $F_{i,v,Ed}h$ can be resisted by a wall with a height of h, provided that the attachment of the bottom rail to the substrate is properly designed to resist such a moment.

More detailed information can be found in Part 3 of the CD.

Wind direction

Where concentrated vertical loads, return walls or openings are utilised to provide restraint against overturning, adequate restraint must be demonstrated in opposite wind directions.

10.8.2 Racking resistance in asymmetric buildings

For many buildings it is sufficient to verify the racking resistance in a particular storey by considering the total racking force on that storey and the total racking resistance of the walls in that storey parallel to the wind direction. This implies that the shear walls share the load in proportion to their strength, on the assumptions that the strength of a wall is proportional to its stiffness and that the horizontal diaphragms create a stiff structure. Hence:

$$F_{v,d,i} = \frac{F_{v,d} R_{d,i}}{\sum R_{d,i}}$$

Where $F_{v,d,i}$ = design load on racking wall i

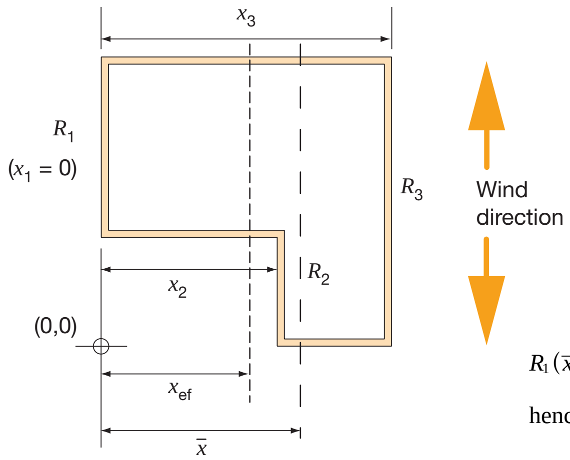
$F_{v,d}$ = total racking load

$R_{d,i}$ = design racking resistance of wall i

For most timber frame buildings the above assumptions are adequate. However if the shear walls on one side of a building are significantly less strong and stiff than those on the other side then the share of the load which they carry may be greater than the load calculated as above. Examples are:

- an L-shaped building
- an end terrace house with a full gable wall on one side and a plasterboard party wall on the other
- a wall with a large glazed area
- a built-in garage where the opening provides little shear resistance in the front wall.

In such cases it may be assumed that the building acts like a rigid box which resists both the shear force of the wind load and a torsional moment. This torsional moment is equal to the wind load multiplied by the distance between the line of force of the wind and the building's centre of rotation (CR) measured perpendicular to the wind direction.



$$R_1(\overline{x} - x_1) + R_2(\overline{x} - x_2) = R_3(x_3 - \overline{x})$$

$$\text{hence } \overline{x} = \frac{R_1 x_1 + R_2 x_2 + R_3 x_3}{R_1 + R_2 + R_3}$$

Fig 10.9 Axis of rotation for wind perpendicular to the *x*-axis

For building plans on an *x*-*y* grid with an origin (0,0) in one corner, the distance of the CR from the origin for wind perpendicular to the *x*-axis (see Figure 10.9) is calculated from the formula:

$$\overline{x} = \frac{\sum R_{d,i} x_i}{R_{d,i}}$$

Where $R_{d,i}$ = design resistance of racking wall *i* which is parallel to wind direction

$\overline{x}$ = distance of CR from origin, measured along *x*-axis

x_i = distance of wall *i* from origin, measured along *x*-axis

The resulting torsional moment, $F_{v,d}(\overline{x} - x_{ef})$ is resisted by all the walls, with each wall contributing to the total moment in proportion to its (stiffness)×(lateral displacement)×(perpendicular distance to the centre of rotation), i.e.:

$$F_{v,d}(\overline{x} - x_{ef}) = k_x \sum R_{d,i} z_i^2$$

Where $F_{v,d}$ = design racking load on building (sum of wind force on windward and leeward walls)

x_{ef} = distance of the line of force of wind from the origin, measured along *x*-axis

k_x = a constant calculated from the above equation

z_i = perpendicular distance of any racking wall *i* from CR, i.e. $(\overline{x} - x_i)$ or $(\overline{y} - y_i)$ or as appropriate

$\overline{y}$ may be calculated in a similar manner, with y_i as the distance of each racking wall *i* perpendicular to the wind direction.

The additional load which each wall perpendicular to the *x*-axis takes to resist the torsional moment is then:

$$F_{\mathrm{tor,d,i}} = k_{\mathrm{x}} R_{\mathrm{d,i}}\, x_{\mathrm{i}}$$

Hence the total load carried by each wall perpendicular to the *x*-axis is:

$$F_{\mathrm{d,i}} = F_{\mathrm{v,d,i}} + |F_{\mathrm{tor,d,i}}|$$

The positive value of $F_{\mathrm{tor,d,i}}$ should be used, for although $F_{\mathrm{tor,d}}$ will be negative for some walls it will be positive when the wind is in the reverse direction.

Check that $F_{\mathrm{d,i}} \leqslant R_{\mathrm{d,i}}$ for each wall parallel to the wind, then repeat the exercise for wind at 90°.

If any walls fail, either increase their strength or take additional measures (see Section 10.8.3).

This method should not be used for seismic design.

10.8.3 Alternative methods of providing racking resistance

If the racking resistance of the system is not sufficient and additional racking resistance is required the following options are available.

- A layer of OSB or other structural sheathing can be attached beneath the plasterboard on internal or external walls. In this case the contribution of the plasterboard should be discounted. This solution will be most effective on internal walls, but in this case the minimum stud depth should be increased from 63mm to 72mm.
- For buildings with masonry cladding an additional contribution to racking resistance can be obtained from the masonry. See Section 10.8.4.
- In end-of-terrace houses where there is often little racking resistance to wind blowing on the gable wall, a proportion of the wind load can be transferred to the adjacent houses via tiling battens and/or steel party wall straps. For further information, see TRADA's *Timber frame housing: UK Structural recommendations*[80].
- Additional resistance to wind forces can also be achieved through the addition of a goal post or portal frame to the system, usually made of steel. Its connection to the timber frame should be adequately detailed. Typically goal posts are securely fastened to timber frame walls using 3 no. 2.5mm × 200mm galvanized mild steel straps nailed to panels and shot-fired to goal posts at low, medium and high level.

10.8.4 Masonry wall ties

Masonry cladding conforming to Section 10.3.1.2 with a minimum height of 2.4m and a minimum width of 600mm attached by suitable wall ties to storey height timber frame walls can increase their racking resistance. The walls ties and their fasteners should have a design horizontal shear strength of at least 225N at deformations of 5mm or more and a characteristic horizontal shear stiffness of at least 30N/mm for deformations up to 5mm.

The additional racking resistance, $F_{v,masonry,Rd}$, provided by the masonry subject to the conditions above, is:

$$F_{v,masonry,Rd} = \text{minimum of } \begin{cases} 0.25F_{v,Rd} \\ l_{masonry}\, q_{masonry} \end{cases}$$

Where $F_{v,Rd}$ = design racking resistance of attached timber frame wall (kN)

$l_{masonry}$ = length of masonry wall (m)

$q_{masonry}$ = 0.75 kN/m for 4.4 ties/m^2 (e.g. 600mm horizontally, 380mm vertically)

= 0.6 kN/m for 3.7 ties/m^2 (e.g. 600mm horizontally, 450mm vertically)

10.8.5 Design of wall studs

10.8.5.1 Axial strength and stability

Wall studs are designed in accordance with Section 5.3.2. For simplicity it is normally assumed that a stud resists the full vertical load and full net wind load (external pressure minus internal pressure times its share of the wall area), i.e. any contribution from the sheathing to strength and stiffness is normally ignored. For the calculation of k_{crit} (Figures 3.2 to 3.4) about the stronger y-y axis a value of $0.85l$ may be used for the effective length, where l is the length of the stud within the frame. In the traditional UK design of buildings not exceeding four storeys it has been assumed that wall studs are fully restrained against buckling about their weaker axis by their connection to the sheathing, provided it is fixed in accordance with Section 10.8.1.2.

However there is evidence that when studs are braced in this way on one side only, as in a party wall, the load capacity is reduced, so some caution is recommended. particularly for buildings above four storeys. In such buildings it is good practice to specify mid-height noggings between the studs in party walls.

To support the ends of lintels single or multiple studs will be required at each end. If they are made of the same material and section as the main wall studs the total number required is at least equal to the number of wall studs removed by the opening. If necessary the strength properties of multiple studs may be increased by a factor k_{sys} of 1.1 provided that they are fastened together at 300mm centres maximum so that they can share the load (see Table 3.20 note **c**). The outer stud on each side of the opening must be attached to the adjacent wall panel in accordance with Section 10.8.1.2.

Beneath a window sill studs are normally provided in the position that the full height wall studs would have been.

10.8.5.2 Notching and drilling of studs

Wall studs should not be notched.

Unless otherwise justified by calculation, drilling of studs should conform to the following requirements.

- Holes should be drilled on the centreline, avoiding knots.
- Hole diameters should not exceed one quarter of the stud depth, e.g. 22mm ϕ max. in a 38×89mm stud.
- Holes should be no nearer than 150mm and no further than a quarter of the stud length from either the top or bottom of the stud.
- Centre-to-centre hole spacing should be at least 4 hole diameters.

10.8.5.3 Deflection

The effect of axial load on the horizontal deflection of a wall stud subject to wind loading may be generally be ignored, except in the case of slender studs subject to high wind loads, when ignoring axial load may result in excessive deflection. If in doubt either choose a tight deflection limit or include the effect of vertical load when calculating the deflection. A suitable formula is given in TRADA's *Timber frame housing: UK Structural Recommendations*[80]. Instantaneous deflection limits of $0.003l$ and $0.005l$ have both been used, the latter on the grounds that calculations based on the stud alone do not allow for the stiffening effect of the sheathing.

10.8.5.4 Bearing strength of bottom rails

The bearing strength of the bottom rail should be verified in accordance with Section 5.2.1.3. The value of $k_{c,90}$ is 25% greater for intermediate studs than for edge studs in a conventional wall panel, but the edge studs take only half the load that the intermediate studs take because they share the load with the edge stud in the adjacent panel. Therefore the bearing strength may be checked using the load on an intermediate stud and the corresponding value of $k_{c,90}$. For single studs with the same cross-section as the bottom rail with $h \geqslant 2.5b$ some values of $k_{c,90}$ are given in Table 10.10. $k_{sys} = 1.0$ because there is only one bottom rail.

Table 10.10 Values of $k_{c,90}$ for matching single wall studs and bottom rails

Stud thickness (mm)	38	44	47
$k_{c,90}$	2.88	2.85	2.83

For multiple studs supporing lintels the value of $k_{c,90}$ should be determined from Figure 3.8.

10.8.6 Design of lintels

Lintels above windows, doors and patio windows may consist of two solid timber members fastened together with nails, screws, dowels or bolts, a single LVL or hardwood member, or where necessary a bolted steel flitch beam.

Lintels supporting door or window openings should be designed in accordance with Section 5.2.1. For lintels consisting of two or more solid timber members securely fastened together so that both members can share the load the strength properties including the bearing strength may be increased by a factor k_{sys} of 1.1 (see Table 3.20 notes **c** and **e**). A deflection limit of $w_{fin} \leqslant 250l$ under dead + imposed load is recommended (see Table 3.4) but this is unlikely to govern the required size.

10.8.7 Horizontal deflection of shear walls

EC5 does not provide a method for calculating the in-plane stiffness of timber-frame shear walls. The design method in Section 10.8.1 will normally ensure that the deflection is not excessive.

10.9 Foundations

Foundation design should be in accordance with EC7. For detailing see *Timber Frame Construction*[76], but note that the recommended depths of footings may need to be increased for medium-rise timber frame construction. Holding down methods are illustrated in Figure 10.6.

10.10 Fire resistance

Fire resistance for walls and ceilings in timber frame buildings is normally provided by plasterboard (see Section 4.3.2). Extended periods of fire resistance can be provided by timber frame party walls as described in Sections 10.4.3.5 and 10.4.3.6.

In addition there are important requirements in the Building Regulations for cavity barriers and firestops in timber frame buildings with cavity walls. For further information see *Timber frame construction*[76].

10.11 Movement

10.11.1 Introduction

Failure to make provision for movement in the design of multi-storey timber buildings can cause serious serviceability problems. However by the careful specification of materials, buildings up to seven storeys in height can be designed to accommodate it.

The overall height of a timber building reduces with time due to:

- gradual shrinkage of the timber as it dries out (predominantly across the grain)
- initial compression due to the building's self weight and occupancy
- initial settlement at the interfaces of roofs, walls and floors under load.

Meanwhile the height of any attached masonry made with fired clay or cementitious products also changes over time as the material absorbs or loses moisture. The resulting differential movement between the two parts of the building can be as much as 10mm per storey in timber frame structures unless measures are taken to restrict it.

Differential movement can also occur where concrete lift shafts or metal balcony structures or services are attached to timber frame structures.

10.11.2 Moisture movement

With solid timber joists and wall framing members installed at a typical 18% moisture content the height of a platform frame structure can diminish by 5mm per storey due to moisture movement. See Section 2.8 for further information.

10.11.3 Movement from induced stresses

At eaves level a 7-storey timber frame building may be subject to 3mm to 4mm of shortening by the end of its design life, due to elastic and creep compression in the load-bearing members caused by the dead and imposed loads. This movement comprises both the relatively low compression of vertical studs and columns and the relatively high compression of the horizontal members such as ring beams and the top and bottom rails of wall panels in which the modulus of elasticity perpendicular to the grain is only 4% to 6% of its value parallel to the grain, depending on the material.

10.11.4 Interface settlement

Settlement at the member interfaces can occur when the natural wind or twist of horizontal members flattens out under load. This can lead to movements of up to 3mm per storey when the complete roof has been installed. It is therefore advisable to complete the timber frame erection and roof covering before any cladding is applied to the frame, in order to minimise any relative movement.

10.11.5 Masonry expansion

Claddings of clay products will expand after construction due to the absorption of atmospheric moisture until equilibrium is reached. The amount of movement will vary from 0.25mm to 1.0mm per metre of height, depending on the type of clay from which the product is made.

10.11.6 Differential movement

The total differential movement is the combination of all the previously described effects. Figure 10.10 shows the movement which can be expected in a typical building made with various materials. This is taken from *Multi-storey timber frame buildings: a design guide*[80] which gives more detailed guidance on handling differential movement between timber and other materials.

Options for reducing the differential movement between the timber frame and the external masonry include:

- timber joists and headers delivered at a low moisture content, preferably 12% or less, e.g. 'super-dried' timber, structural timber composites or prefabricated timber joists
- minimise the amount of timber loaded perpendicular to the grain
- use clay bricks with low movement characteristics
- specify non-masonry claddings such as timber boarding or tiles.

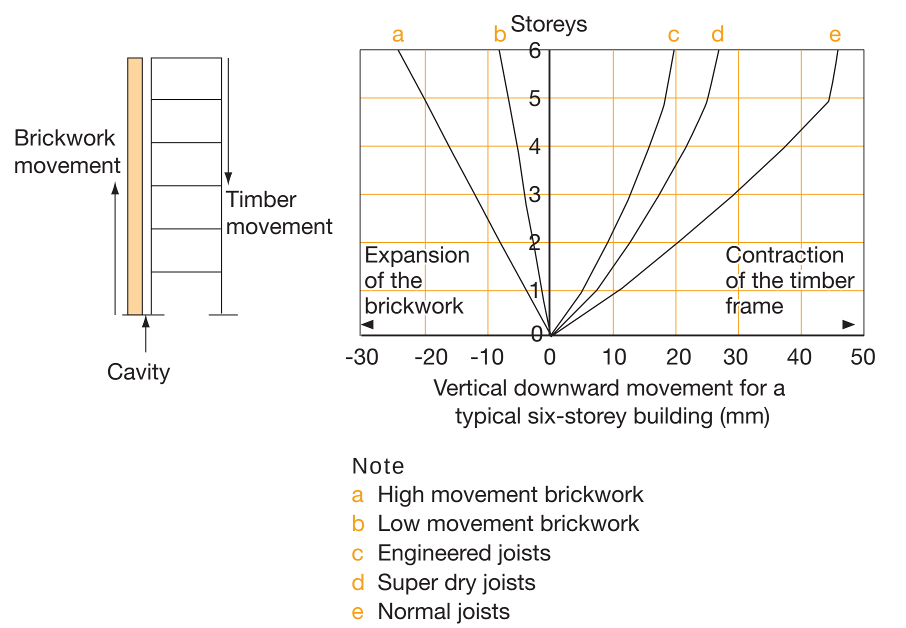


Fig 10.10 Vertical movement in a typical platform timber frame building

Differential movement between timber frame walls and the external masonry can be accommodated by means of:

- separate lintels for outer and inner leafs
- the use of vertically-sliding wall ties
- the fixing of windows to the timber frame (inner leaf)
- the provision of soft joints or gaps between sills and masonry (outer leaf)
- the provision of a plastic fascia beneath the masonry at the head of the window to cover any downward movement of the frame
- the use of suitable wall head details.

For standard detail drawings see TRADA's *Standard details for houses and flats*[120].

10.11.7 Movement joints

Movement joints in claddings are required as in non-timber buildings. For the spacing of expansion gaps in wood based horizontal diaphragms refer to Table 12.1.

10.12 Other requirements

10.12.1 Acoustic

Section 8.7.1. describes how timber floors can meet requirements for sound attenuation.

For walls, an imperforate timber framed, plasterboard lined internal wall without insulation provides at least 30dB of sound insulation which is usually considered adequate for domestic or similar purposes. Improved insulation (e.g. around bathrooms) can be obtained by the use of two layers of 12.5mm thick plasterboard on each side, insulation, or high density plasterboard, glass wool and resilient bars. Plasterboard and insulation manufacturers can provide tested wall specifications (see Table 10.2.).

Adequate sound attenuation between dwellings is provided by timber frame party walls as described in Section 10.4.3.6.

10.12.2 Thermal

Energy conservation requirements in the Building Regulations are usually achieved in timber frame construction by placing fibre insulation between the wall studs, ceiling joists and roof joists/rafters. 89mm of wall insulation with a reflective breather membrane in a building clad with masonry may comply with requirements, but in some circumstances 150mm of insulation, an efficient boiler system, and/or a supplementary supply of renewable energy may be required with fuel types other than gas.

10.13 Robustness of platform timber frame

Platform timber frame is an inherently robust structure when correctly designed and built. Provided that the requirements set out in Appendix A are met then a timber frame structure will satisfy current requirements for building occupancy classes up to and including 2: higher risk group.

10.14 Platform timber frame above 4 storeys

The following matters require special consideration for buildings of more than 4 storeys.

For more than 4 storeys above the ground, EC1-1-7 or the relevant Building Regulations may have more stringent requirements for demonstrating robustness (see Table 5.5).

- Only the first four storeys of suitably buttressed masonry can provide wind shielding (see Section 10.3.1.2).
- Periods of required fire resistance may increase, in which case additional layers of plasterboard will be needed. When two layers are specified the Engineer should highlight the necessity of fastening both layers, especially the inner layer, with specified fasteners at specified centres, and of staggering the joints.
- Relative vertical movement between the timber frame and masonry cladding or balconies increases as the number of storeys increases. See Section 2.8.1.

11.1 General checking requirements

11.1.1 Local Authority checking

The primary function of a Local Authority Checking Engineer is to ensure that the design of a building meets the requirements of the applicable Building Regulations in force at the time. This function will be carried out either by the actual Local Authority for the geographic area of the site or by its appointed consultants. In Scotland the same function may be carried out by any certified member of the Scheme for Certification of Design (Building Structures) administered by the Scottish Building Standards Agency.

The detailed knowledge of the Local Authority Engineers may be limited in the specialised area of timber construction. Such engineers or their consultants might only be able to undertake a basic form of checking, i.e. to ensure that, as perceived, the structure accords with the statutory requirements of the law. They would attempt to check the basic elements.

- Force and load analysis.
- Load distribution throughout any frame construction.
- Load and force distribution to any supporting structure, and to the foundation.
- Any elements of the structure which are referenced as being taken from generally accepted tables of acceptable values.

Consideration should also be given concerning the location. There may be unusual environmental conditions of which only the local authority may be aware.

11.1.2 General checking

This section summarises the general checks which should be carried out by the Engineer to ensure that a completed design will be safe, serviceable, robust and durable. It includes checks which Local Authority Engineers or Approved Certifiers of Design (Building Structures) are recommended to carry out in addition to those directly related to the Building Regulations.

- Ensure that all relevant actions likely to be exerted on a structure in the locality in which it is to be erected have been taken into consideration. The actions should include fundamental actions during the execution phase and the finished structure, and the robustness of the structure to resist accidental actions.
- Within all the parameters of design, ensure that all forces, moments and deflections are considered and can be adequately resisted by the structure as a whole and by its individual members and components, in accordance with all relevant codes, standards and practical considerations.
- Check that interfaces with the structural elements designed by others have been properly designed.
- Check that the calculated deflections will not distort the structure in such a way that it imposes additional forces on any adjacent structure.

- Check that the required periods of fire resistance will be met.
- Check the adequacy of measures to provide adequate durability within the required service environment.
- Check that material specifications are complete, correct and appropriate for the service environment and intended use.
- Check the practicality of the proposed method of transportation of any final structure, considering restrictions on height, width and length. If necessary, can it be transported in sections and be assembled on site?
- Check that any additional requirements imposed by insurance companies have been satisfied.

11.2 Codes and standards

The design codes used should be listed.

A Checking Engineer should note significant differences between BS 5268 and EC5 as detailed in Section 2.1.2.2.

11.3 Material specifications

Material specifications should be checked in accordance with Sections 2.1.3 and 3.5.

Because the strength and stiffness of timber depends on its moisture content, check that the correct service class has been specified for each part of the structure (Section 2.15), and that any wood-based panel products specifications are appropriate for the service class in which they are to be used (Section 3.5.3). Check that corrosion protection specifications are appropriate (Section 3.4.5).

11.4 Protection against decay and insect attack

Check that the requirements in Section 3.4 concerning protection against timber decay, steel corrosion and insect attack and any regulatory requirements for preservative treatment have been met.

11.5 Deflection and creep

Confirm that deflection limits have been agreed between the client and the designer and that these are clearly defined as initial or final deflections.

Check that creep and slip have been properly allowed for in assemblies in which they can affect the distribution of forces and moments. This includes composite components such as prefabricated I-joists and steel flitch beams.

11.6 Trussed rafter roofs

Where a trussed rafter roof has been designed by the roof truss manufacturer, check that the designer has provided all the information listed in Section 7.3.2. Check that the Engineer has designed the bracing and roof fixings, and that appropriate methods of holding it down have been specified.

11.7 Floors

Where a floor has been designed by a floor manufacturer or specialist designer, check that the designer has provided all the information listed in Section 8.1.3. Check that the Engineer has designed and specified the restraints between the floor and the walls which are required for a robust structure (see Section 5.11).

11.8 Platform timber frame

Because platform timber frame is a lightweight form of building, there are certain matters which assume more importance than in other forms of structure.

- Check that interfaces with the structural elements have been properly designed.
- Check that the specified roof cladding including any sarking board is of the weight assumed in the design. (Some generic types, e.g. concrete interlocking tiles, are produced in a wide range of weights.)
- Check the stability of the building with respect to roof uplift, overturning and sliding and transmission of shear forces to the foundation (Section 10.5). In particular note the warning in Section 10.5.2 about holding down straps.
- Check that the strength and stability of the structure during the construction phase has been addressed, when for example there will be no masonry walls to shield the timber frame from some of the wind (Section 2.6).
- Check the provisions for the temporary bracing of party walls during the construction phase (Section 10.4.3.5).
- Check that proper allowance has been made for differential movement between the timber structure and any non-timber materials attached to it (Section 2.8.1).
- Check that all necessary cavity barriers and fire stops have been correctly specified.

11.9 Checking aids

It may be possible to use span tables and other design aids produced for BS 5268 designs to confirm that no major mistakes have been made. In general designs to BS 5268 and EC5 should not differ significantly.

Other useful publications are listed in Section 1.3.6.

12.1 General

This section gives recommendations for the preparation, fabrication, storage, handling, installation, control and maintenance of materials and components. These recommendations are necessary conditions for the applicability of the design rules given in the *Manual*.

Workmanship in the preparation, fabrication and installation of materials should conform in all respects to accepted good practice. Where necessary reference may be made to BS 8000, in particular part 5[121].

The specified materials should be installed in such a way as to perform adequately the functions for which they are designed.

For the structure to conform to the principles and practical considerations of the design there should be adequate supervision throughout its preparation, construction and maintenance.

12.2 Members

12.2.1 Condition of timber members

Members which are damaged, crushed or split beyond the limits permitted for similar defects in the grading should be rejected or repaired to the satisfaction of the designers and approving authority.

The deviation from straightness measured midway between the supports should, for columns and beams where lateral instability can occur, or members in frames, be limited to 1/500 times the length of glued laminated timber or LVL members and to 1/300 times the length of solid timber.

12.2.2 Moisture content

Before being used in construction, solid timber and other structural timber products should be dried as near as practicable to the moisture content appropriate to their climatic condition in the completed structure (see Table 12.1).

As a guide, at the time of erection the moisture content of solid timber members designed for use in service class 1 or 2 should not exceed 18%, and the moisture content of wood-based panels for use in service class 1 should not exceed 12% (see Section 12.5).

The determination of moisture content by a properly calibrated moisture meter used in accordance with the manufacturer's instructions will normally be considered sufficiently accurate if insulated probes inserted 20mm or one quarter of the timber thickness (whichever is less) are used. Where a more accurate average determination of moisture content has to be made, and for meter calibration, the oven dry method detailed in EC5 NA 4.1 should be used.

12.2.3 Dimensions

The size, shape and finish of all members and materials should conform to the detailed drawings and specifications. Dimensions and spacings should not be scaled from drawings.

12.2.4 Modifications

The cutting, notching, drilling or other modification of members, other than that allowed by the drawings and specification or by Sections 5.2.1.4, 5.2.1.5, 7.3.4 or 10.8.5.2, should not be permitted. Care should be taken to ensure that notches and holes are not so positioned in a member that the remainder of the cross-section contains a knot or other defect which will significantly affect its strength.

Where service holes are drilled in wall studs, the services should be protected if necessary to avoid penetration by plasterboard nails.

12.2.5 Treatment of cut surfaces

The cutting of timber after preservative treatment should be avoided. When it is unavoidable, exposed untreated timber should be given a liberal application of a permitted preservative in accordance with *Manual on Preservative Treatment*[122].

12.3 Connections

12.3.1 General

Wane, fissures, knots or other defects which have not been allowed for in the design, and which may affect significantly the load-carrying capacity, should not be permitted at a connection.

The spacing and predrilling of holes for nails, screws, bolts and dowels should conform to the design drawings.

It is essential that the fastener types and corrosion protection specified in the design are used, particularly square twisted nails, ringed shank nails, screws with a specified minimum root diameter, or high strength steel bolts.

12.3.2 Nailed connections

If predrilling is specified in the design, holes with a diameter not exceeding 0.8 times the nail diameter must be drilled before inserting the nail.

12.3.3 Screwed connections

Screws should be turned, not hammered.

Screws should be tightened so that the members fit closely together.

Coach screws with a timber headside member should be fitted with a special steel washer in accordance with Table 3.29.

Predrilling is not required:

- for special screws if their manufacturer can provide characteristic load capacities from tests on complete connections carried out in accordance with BS EN 26891[88] and matching the connection configuration used in the design
- for wood screws with a smooth shank diameter $\leqslant$ 6mm in softwood, unless the Engineer has specified predrilling on the drawings

In other cases predrilling is required, with the following requirements:

- the lead hole for the smooth shank should have the same diameter as the shank and the same depth as the length of the shank

- for $\rho_k \leqslant 500\text{kg/m}^3$ the hole for the threaded portion should have a diameter of approximately 70% of the shank diameter; while for $\rho_k > 500\text{kg/m}^3$ the predrilling diameter should be determined by tests where ρ_k = the characteristic timber density.

12.3.4 Bolted connections

The maximum diameter of bolt holes should be:

$d + 1\text{mm}$ in timber

$$\max.\begin{cases} d + 2\text{mm} \\ 1.1d\,\text{mm} \end{cases} \quad \text{in steel plates}$$

Where d = diameter of bolt (mm)

Special steel washers in accordance with Table 3.29 must be fitted between timber and the bolt head or nut.

Bolts should be tightened so that the members fit closely together.

12.3.5 Dowelled connections

The minimum dowel diameter should be 6mm. The tolerances on the dowel diameter should be -0/+0.1mm.

The diameter of pre-bored holes in the timber members should not exceed that of the dowels.

12.3.6 Connections made with timber connectors

Toothed plate, split ring and shear plate should be fitted in accordance with the design drawings.

Members connected with toothed plates should be drawn tightly together by means of a high tensile steel bolt. With other types of connector recesses should be accurately cut concentric with the bolt hole, and all chips and shavings removed.

The diameters of bolts used with timber connectors should conform to Table 6.32.

12.3.7 Adhesively bonded joints

Joints in structural components made from separate pieces of timber, plywood or other wood based board that are fastened together with adhesive (e.g. box beams, single web beams, stressed skin panels, glued gussets) should be manufactured in accordance with BS 6446[95]. The manufacturing process should be subject to quality control.

Large (full depth) finger joints should be manufactured in accordance with BS EN 387[97].

The adhesive manufacturer's recommendations with respect to mixing, environmental conditions for application and curing, moisture content of members and all factors relevant to the proper use of the adhesive should be followed.

For adhesives which require a conditioning period after initial set to attain their full strength, the application of load to the joint should be delayed for the necessary time.

12.4 Trussed rafters

Requirements for the fabrication of trusses are given in BS EN 14250[123].

12.5 Storage and handling

12.5.1 Protection of materials and components

Precautions should be taken during storage prior to delivery, and on site, to minimise changes in moisture content due to the weather. Rain, damp and direct sunlight are all potentially harmful to timber and wood-based components.

Materials and components should be stored clear of the ground on dry bases, and stacks should be evenly supported on bearers with spacer sticks at regular intervals. Stacks should be protected from the weather either by a roof, or by tarpaulins or other impervious materials so arranged to give full cover but at the same time to permit free passage of air around and through the stack.

Solid timber delivered packaged, i.e. strapped, should not be stored in packaged form for lengthy periods. Where early use is not possible, packages should be opened and the timber should be open-piled and protected from the weather as above. Care should be taken not to deform stacked material.

For wood-based panel products any shrink-wrapping should be left untouched until 24 to 48 hours before installation (see Table 12.1).

Where it is essential that materials and components have low moisture contents, it may not be possible to maintain suitable conditions on site other than for short periods, and deliveries should be arranged accordingly.

12.5.2 Handling

Undue distortion of components during transportation and handling should be avoided. Similarly, damage from chafing or slings should also be avoided. Where design assumptions for long, flexible or heavy components dictate certain methods of handling, lifting points should be marked on the components and methods of lifting should be shown on the design drawings.

12.6 Assembly, erection and installation

12.6.1 General

The method of assembly and erection should be such as to ensure that the geometry of assembled components, as specified by the designer, is achieved correctly within the specified tolerances.

During assembly and erection, no forces should be applied to a component that could cause the permissible stresses to be exceeded in that or any other component. Special care is necessary when handling framed arches and shaped beams. Any instructions about methods and sequences of erection included in the health and safety plan or shown on the design drawings should be rigorously followed.

12.6.2 Trussed rafters

Trusses should be checked for straightness and vertical alignment prior to the fixing of any bracing.

At the time of fabrication, the truss members should be free from distortion within the limits given in BS EN 14250[123]. However, if members become distorted during the period between fabrication and erection but can be straightened without damage to the timber or the joints and can be maintained straight, then the truss may be considered satisfactory for use.

The maximum bow in any truss member after erection should be limited. Provided that it is adequately secured in the completed roof to prevent the bow from increasing, the permitted value of the maximum bow should be taken as **10mm**.

The maximum deviation $a_{\text{dev,perm}}$ of a truss from true vertical alignment after erection should be limited to:

$$a_{\text{dev,perm}} = \min. \begin{cases} 10 + 5\,(H - 1)\,\text{mm} \\ 25\text{mm} \end{cases}$$

Where *H* = height of truss (m)

If a girder truss has to be assembled on site, the first truss should be laid out on level supports to ensure that it is flat. The ceiling tie members must be fastened together as specified by the designer. Rafter and web members should be bolted or nailed together in accordance with the designer's directions or as recommended in BS PD 6693[2]. Gaps greater than 3mm between chord members should be packed with suitable material before fastening.

12.6.3 Floors

Requirements for the installation of floor decking are given in Table 12.1.

Requirements for the anchoring of timber floors to masonry walls are given in Figures 12.1 to 12.3.

When installing engineered timber joists the manufacturer's instructions should be followed. See Section 8.3.6.2 for safety notes relating to joist supports.

Table 12.1 Requirements for the installation of wood-based decking for floors and roofs			
Item	**Solid timber boards**	**Plywood decking**	**Particleboard and OSB decking**
Storage	All decking materials must be fully protected from the weather before installation		
Time of installation	Decking should not be installed until the building is weathertight unless it is factory-faced with a protective film		
Moisture content[a]	m.c. $\leqslant$ 18% at installation. Floor boarding intended as a decorative feature must dry out to 12% ± 2% m.c. before being laid, and should not be laid until the building has dried out	No special requirement but must be dry	Boards should be conditioned by laying them in place at least 24 hours before fixing down. They should not be fixed to joists or noggings which have a m.c. > 20%
Dust and debris	Before fixing decking, all dust and debris should be brushed or blown from the joist surfaces		
Fastener type and size	Decking should be attached to joists with flat head annular ringed shank nails, or countersunk head traditional wood screws in predrilled holes, 3.0mm to 3.35mm in diameter[b]; or with countersunk head self-drilling wood screws, No.8 screw gauge or 4.00mm in diameter. Nails should have a minimum length of 2.5t and screws a minimum length of 2.0t where t is the decking thickness. Nail heads should be punched and screw heads driven 2-3mm below the surface. Where necessary a countersinking hole should be drilled for screws		
Access provision	Screws should be used where access to underfloor services is required. With panel products a purpose-designed screwed access panel should be provided		
Recommended maximum fastener spacing[c]	1 nail at each joist crossing, or 3 for boards wider than 175mm	150mm around the edges of a board, 300mm wherever boards cross a joist (EC5 10.8.1)	
Position of fasteners	Nails should be 15 to 20mm from the edges of the board. (If there is a third nail it should be in the centre)	Fasteners should be positioned at least 3d from the edges of boards, joists and noggings	

Table 12.1 Continued

Item	Solid timber boards	Plywood decking	Particleboard and OSB decking
Gluing	In addition to nailing or screwing as above, it is recommended that all decking should be glued to clean, dry, frost-free joists using a PVAc adhesive of durability class D3 to BS EN 204[124]. (This is to improve the floor performance and reduce the risk of creaking. The site gluing of floor decking should not be treated as a substitute for mechanical fixings)		
Orientation of boards	Boards should be laid across the joists	Board should be laid with the face grain at right-angles to the joists (see Table 3.17 note **a**)	Square-edge chipboard is normally laid with the long edges along the joists. In all other cases the long edge should be at right-angles to the joists
Continuity over joists	Boards should span over at least 3 joists		
Support on joists	Floorboard ends should meet in the centre of a joist	Edges supported on joists should meet on the centre of joists at least 38mm thick	
End joints	End joints supported on the same joist should be at least 2 board widths apart	The joints between the short ends of adjacent boards should be staggered. (This improves the strength of the floor as a horizontal diaphragm. See Figure 5.4)	
Support of edges	Before fixing, square-edged boards should be butted tightly. Square edged solid timber boards should not be used in timber frame buildings or in any buildings where floor diaphragm action is required (EC5 10.8.1). T & G boards should be glued with a PVAc adhesive of durability class D3 to BS EN 204 or an equivalent adhesive, and be cramped tightly together	All edges should be tongued and grooved to adjacent boards, or supported on joists or noggings, or (at the walls supporting the joists) be supported on header joists or perimeter noggings. T & G joints should be glued with a generous bead of PVAc adhesive of durability class D3 to BS EN 204 or an equivalent adhesive, applied to both tongue and groove before fixing. All boards should be tightly pulled together before fixing with nails or screws. Softwood noggings minimum 38mm wide x 38mm deep should be skew nailed at each end to the joists with two 3mm ϕ nails. Panel edges should be fastened and glued to noggings as to joists	

Table 12.1 Continued

Item	Solid timber boards	Plywood decking	Particleboard and OSB decking
Expansion gap		Timber floors in masonry buildings – An expansion gap of at least 10mm should be allowed between the decking and the perimeter wall in each room area to prevent possible buckling as it absorbs moisture. On floors longer than 10m the gap should be 2mm per metre run of deck, divided between each end of the floor, but manufacturers may recommend intermediate expansion gaps instead. The gap may be masked by skirting board or loose cover strip, or filled with an easily compressible material. It must be kept clear of plaster and other building debris	
		Timber floors in timber frame buildings – The decking must be connected to the outer edge of the perimeter joists to provide adequate diaphragm action. It is therefore imperative that the decking be conditioned to the in-service temperature and humidity before it is fastened down	

Notes

a Moisture content for roof decking will be dependent on the form of construction and building control requirements.

b 3.00mm diameter is preferable for 38mm thick joists to reduce the possibility of splitting.

c For service class 2 buildings the maximum fastener spacing around the perimeter of a floor may be determined by tying requirements (see Appendix A).

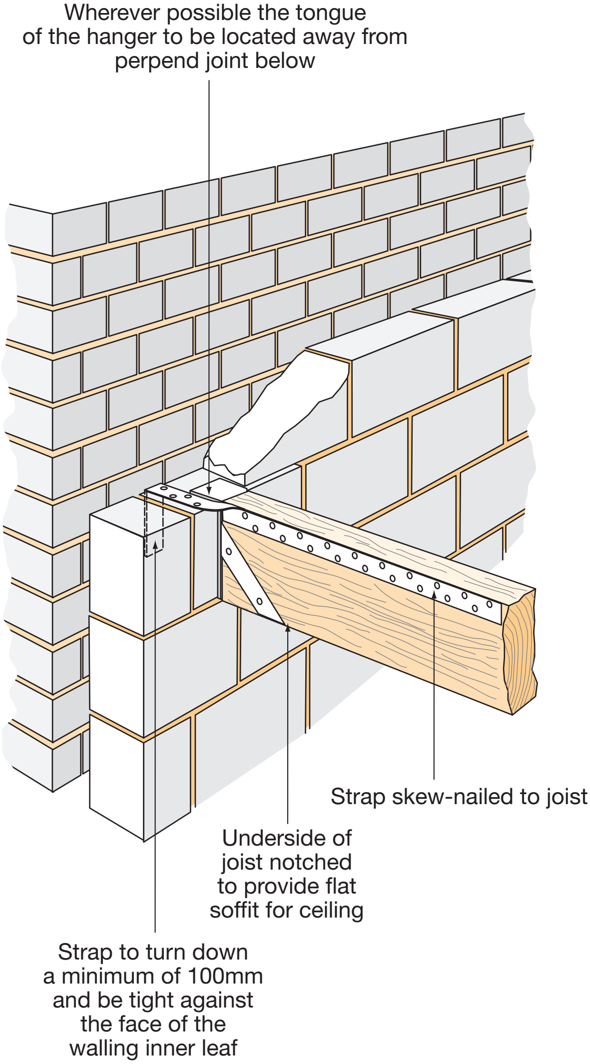
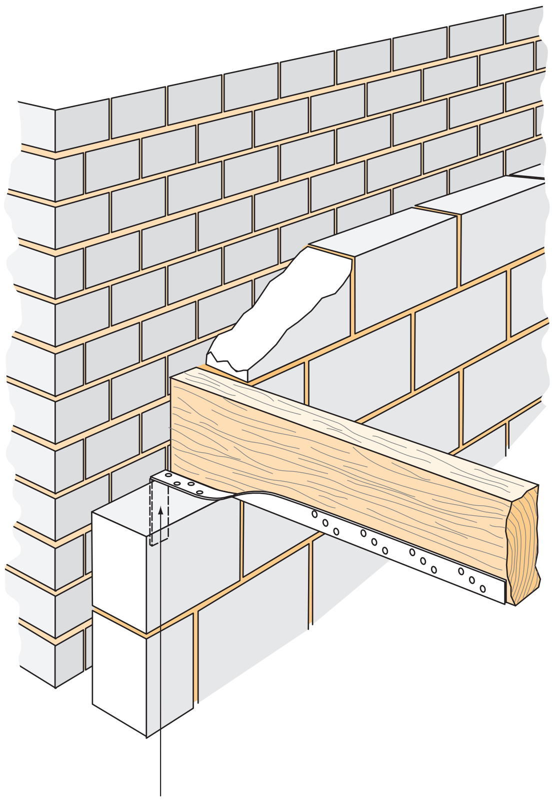


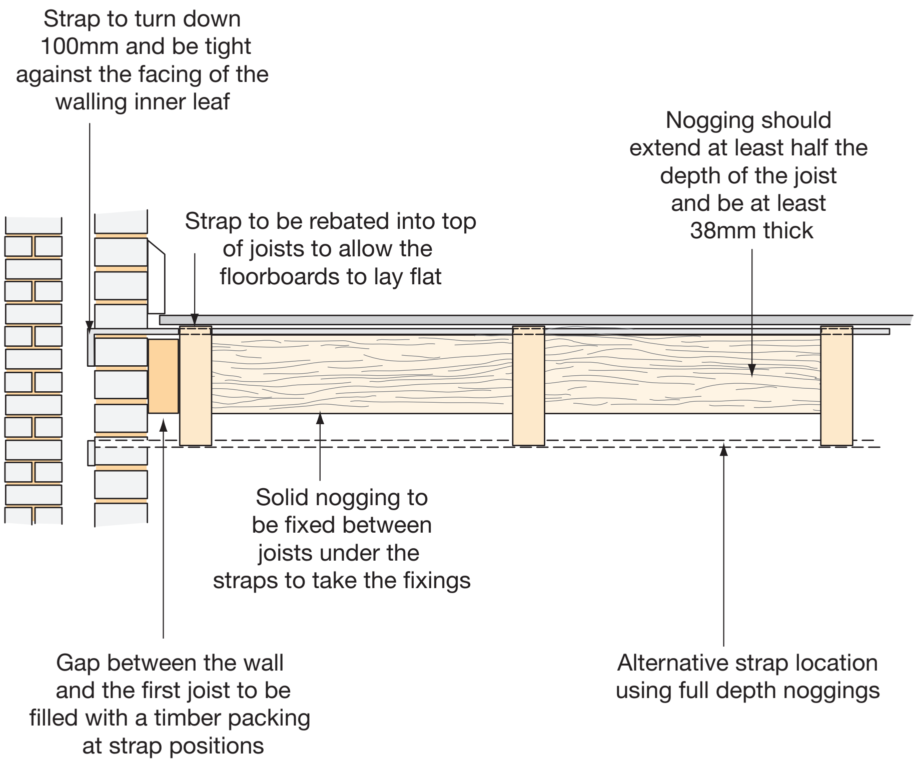
Fig 12.1 Timber joists supported by joist hangers on blockwork



Note

No straps are necessary in dwellings up to 3 storeys if the joist spacing is 1200mm or less and the joists bear at least 90mm into the wall or are supported on specially designed joist hangers. Otherwise strap as shown – either on top of the joist with the strap turned up, or on one side of the joist with the strap turned sideways – 2m apart up to 3 storeys or 1.25m apart above 3 storeys. Straps to be galvanized mild steel 5mm x 30mm, nailed as indicated in Table 5.9 to resist the forces specified in Appendix A.

Fig 12.2 Timber joists supported on blockwork



Note

The strap should be carried over at least three joists and be secured with four fixings of which at least one should be in the third joist or in a nogging beyond the third joist. Strap specification, spacing and nailing as for Figure 12.2.

Fig 12.3 Timber joists spanning parallel to a blockwork wall

12.7 Treatments

12.7.1 Preservative and flame retardant treatments

Preservative and flame retardant treatments should be applied in accordance with the manufacturer's instructions. Flame retardants should be used under such conditions and in such a manner that they will not adversely affect other materials or processes. Leach resistant retardants should be used for materials exposed to the weather or to regular wetting, and they may need to be renewed after a period recommended by the manufacturer.

12.7.2 Anti-corrosive treatments

The anti-corrosive treatment of metal fasteners and fittings should be sufficient to ensure their satisfactory performance and structural integrity throughout the intended life of the structure. The degree of protection required will depend on the dimensions of the fittings and the environment to which they are to be exposed (see Table 3.25).

12.7.3 Decorative treatments

Where structural timber is painted, varnished or otherwise decorated, the work should be in accordance with BS 6150[125]. Care should be taken to ensure that paints, preservatives, flame retardants and adhesives are compatible. Advice can be obtained from the Wood Protection Association.

12.8 Production and site control

12.8.1 Control plan

A control plan should be drawn up to ensure that the construction is built in accordance with the design specification. As a minimum the control plan should cover:

- any necessary preliminary tests, e.g. tests for suitability of materials and production methods
- checking of delivered materials and their identification, e.g.
  - for wood and wood-based materials: species, grade, marking, treatments and moisture content
  - for glued constructions: adhesive type, production process, glue-line quality
  - for fasteners: type, corrosion protection
  - for all materials, checking of correct dimensions and quantities
- transport, handling of materials and site storage
- checking of assembly and erection
- checking of structural details, e.g.
  - splitting
  - number of nails, bolts etc.
  - sizes of holes, correct predrilling
  - spacings and distances to end and edge of members
- final checking of the result of the production process, e.g. by visual inspection or proof loading.

12.8.2 Inspection

Reasonable facilities and access for inspection should be provided during and at completion of fabrication and erection of a structure. These facilities and access conditions should be agreed between the parties concerned.

12.9 Maintenance

12.9.1 Responsibility

The responsibility for maintenance should be agreed between the parties concerned. A maintenance schedule should be drawn up to ensure that the building continues to function properly.

12.9.2 Tightening of bolts

It is advisable to check the tightness of bolts some six weeks to eight weeks after completion of the structure, when the timber may have reached equilibrium moisture content. Where tightening is to be carried out access for this purpose should be provided. A second inspection of large solid timber members about 12 months after completion is recommended.

12.9.3 Ancillary components

It is imperative that features of the construction which are essential to the structural performance of timber and timber-based components, e.g. vapour barriers, ventilators, seals, etc., are maintained in an effective condition throughout the intended life of the structural timberwork.

12.9.4 Structural metal-work

Any corroded fittings should be treated with an anti-corrosive or, if necessary, replaced.

12.9.5 Other matters

The maintenance schedule should include inspection (and renewal when necessary) of wood stains or other protective treatments on exterior woodwork, the removal of any debris or vegetation which could retain moisture, and normal building maintenance on drains, gutters, flat-roofing, etc., to ensure that the structural timberwork remains well ventilated and either remains dry or does not remain wet for prolonged periods.

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Table A.1 specifies the tying forces required to produce a robust timber structure when using Method i) as described in Section 5.11. The values shown are to be included in a revision of BS 5268-2[3], and are considered adequate to satisfy the Building Regulations in England and Wales, Northern Ireland and Scotland. NDPs for EC1-1-7 are not yet available. Once tying forces appropriate to a timber structure have been published as NDPs in the NA to EC1-1-7 the latter should be used instead.

Tables A.2 and A.3 give fastener spacings required to provide a distributed tie force of 0.5kN/m. For other values the spacings may be adjusted proportionately (but see Table 12.1 for maximum recommended values). Finally Table A.4 gives nail capacities for use with steel anchor straps to tie timber floors and ceilings to masonry walls.

Table A.1 Required tying forces for internal ties for robust detailing of a timber building or a building with timber floors[a]

Horizontal design tie force, $F_{t,hor,d}$[b]		**Vertical design tie force, $F_{t,vert,d}$[c]**
Distributed ties (kN/m)[d]	Concentrated ties (kN)[e]	Distributed ties (kN/m) or concentrated ties (kN)
$\max.\begin{cases}3.5\\0.5(g_k+q_k)l\end{cases}$	$\max.\begin{cases}10\\0.5(g_k+q_k)s_t l\end{cases}$	A tensile force equal to the maximum vertical characteristic load from any one storey normally supported by the wall or column

Notes

a For peripheral ties use half the tabulated values. The values shown have been taken from draft recommendations for BS 5268-2, because the values recommended in Informative Annex A of BS EN 1991-1-7 are not appropriate for a timber structure.

b g_k = characteristic dead weight of floor or roof per unit area (kN/m²)
q_k = full characteristic imposed load on floor or roof per unit area (kN/m²)
l = full length of tied area of floor or roof (m)
s_t = mean spacing of ties (m).

c Each column or wall supporting vertical load should be tied continuously from the lowest to the highest level. There should be an effective connection between vertical and horizontal ties at each level.

d Provision for a horizontal tie force in a platform frame timber construction is required at the connection between the head of the supporting external wall and the supported floor joist construction, between the floor joist and the floor decking horizontal diaphragm, and between the floor diaphragm/floor joists and the external wall above.

e Ties for use in post and beam type structures.

Table A.2 Nail spacings required to achieve 5kN/m lateral design capacity in C16 softwood under accidental instantaneous loading[a]

Headside member	Min. length (mm)	Application	Service class	Smooth hand driven nail[b]		Machine driven nail[c] Smooth	Machine driven nail[c] Threaded
				Diameter (mm)			
				3.0	3.35	3.1	3.1
				Maximum spacing (mm)			
9mm OSB/3	50	Timber frame wall	1 or 2	120	140	130	140
11mm OSB/3	50	Timber frame wall	1 or 2	130	145	140	145
18mm OSB/3	50	Floor perimeter	1	160	170	165	175
22mm particleboard[d]	50	Floor perimeter	1	140	155	150	155
22mm particleboard[d]	63	Floor perimeter	1	185	220	210	220
				Smooth hand driven nail[b]		**Machine driven ringed shank nail[c]**	
				Diameter (mm)			
				3.35	4.0	3.1	3.1
38mm C16	75	Wall panel to top binder	1 or 2	185		180	
38mm C16	90	Wall panel to floor header joist	1 or 2		255		180

Notes

a May be adjusted pro-rata for other required tying forces.
b Assumed minimum tensile strength = 600N/mm².
c Assumed minimum tensile strength = 700N/mm².
d The values for particleboard are for any structural grade.

Table A.3 Wood screw spacings required to achieve 5kN/m lateral design capacity in C16 softwood under accidental instantaneous loading – no predrilling[a]

Headside member	Length[b] (mm)	Application	Service class	Nominal diameter of wood screw[c]			
				No. 6	No. 8	Metric sizes	
				3.45mm	4.17mm	3.5mm	4mm
				Maximum spacing (mm)			
9mm OSB/3	50	Timber frame wall	1 or 2	140	175	140	165
11mm OSB/3	50		1 or 2	150	180	150	175
18mm OSB/3	60	Floor perimeter	1	175	235	180	225
22mm particleboard[d]	60		1	175	230	180	215
				Nominal diameter of wood screw[c]			
				No. 8	No 10	Metric sizes	
				4.17mm	4.88mm	4mm	5mm
38mm C16	80	Wall panel to C16 top binder	1 or 2	145		135	
38mm C16	100	Wall panel to C16 floor header joist	1 or 2		190		200

Notes

a May be adjusted pro-rata for other required tying forces.

b The pointside penetration must be at least $6d$. Penetrations less than $8d$ will give reduced load capacities.

c Thread root diameter assumed to be 60% of nominal diameter, giving an effective diameter of $d_{ef} = 0.6 \times 1.1d$. Assumed minimum tensile strength = 540N/mm².

d The values for particleboard are for any structural grade.

Table A.4 Lateral design capacity per nail in C16 softwood under accidental loading through 5mm steel anchor straps – no predrilling

Type	Diameter (mm)	Length (mm)	Design load (kN)
Square twisted	3.75	32	1.09
Smooth round	3.75	75	1.58
Smooth round	4.00	75	1.76
Machine driven round – smooth or threaded	3.10	75	1.24

Appendix B – Useful UK organisations

IStructE
T: +44 (0) 20 7235 4535 W: www.istructe.org

TRADA
T: +44 (0) 1494 569600 W: www.trada.co.uk

Building Research Establishment
Bucknalls Lane, Watford WD25 9XX
T: +44 (0) 1923 664200; W: www.bre.co.uk/timber

Central Point of Expertise on Timber Procurement (CPET)
Department for Environment, Food & Rural Affairs,
Nobel House, 17 Smith Square, London SW1P 3JR
T: +44 (0) 1865 243766; W: www.proforest.net/cpet

Timber Trades Federation (for solid timber)
Clareville House, 26/27 Oxendon Street, London SW1Y 4EL
T: +44 (0) 20 7839 1891; W: www.ttf.co.uk

Trussed Rafter Association
31 Station Road, Sutton cum Lound, Retford, Nottinghamshire DN22 8PZ
T: +44 (0) 1246 230036; W: www.tra.org.uk

Glued Laminated Timber Association (for glulam)
Chiltern House, Stocking Lane, Hughenden Valley,
High Wycombe, Buckinghamshire HP14 4ND
T: +44 (0) 1494 569758; W: www.glulam.co.uk

UK Timber Frame Association
The e-Centre, Cooperage Way Business Village, Alloa, FK10 3LP
Tel: +44 (0) 1259 272140. W: www.timber-frame.org

Wood Panel Industries Federation (for wood-based panel products)
28 Market Place, Grantham, Lincolnshire NG31 6LR
T: +44 (0) 1476 563707; W: www.wpif.org.uk

Wood Protection Association
1 Gleneagles House, Vernongate, South Street, Derby DE1 1UP
W: www.wood-protection.org

Manual for the design of timber building structures to Eurocode 5

This *Manual* supports the design of structures to BS EN 1995-1-1:2004 for construction in the UK. The basic format of manuals published by the Institution for other structural materials is followed.

Guidance is provided on the design of structures of single-storey and medium-rise multi-storey buildings using common forms of structural timberwork. Structures designed in accordance with this *Manual* will normally comply with EC5. Nationally Determined Parameters from the UK National Annex have been taken into account in the design formulae that are presented.

The *Manual* is laid out for hand calculation, but the procedures are equally suitable for computer-based applications. Timber is a relatively complex structural material but designers should find the *Manual* concise and useful in practical design. Methods are given for initial sizing design as well as for final design.

The accompanying CD provides valuable connection design software to facilitate the calculation of the lateral load capacity of nails, screws, bolts and dowels, in addition to more extensive tables of material properties.

The Institution of Structural Engineers
International HQ
11 Upper Belgrave Street
London SW1X 8BH
United Kingdom
T: +44 (0) 20 7235 4535
F: +44 (0) 20 7235 4294
E: mail@istructe.org
W: www.istructe.org

The Institution of Structural Engineers
TRADA

Manual for the design of timber building structures to Eurocode 5 – CD

December 2007

CD Contents

For material properties of plywood reference should be made to BS 5268-2 Tables 40 to 56, and to Section 3.3.3 of the *Manual* for conversion formulae.

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CD1 Material Properties

CD1.1 Properties of solid timber strength classes

Table CD1.1 Properties of solid timber grades BS EN 338:2003[a]

		Poplar and softwood species[b]										Hardwoods					
		C14	**C16**	**C18**	**C20**	**C22**	**C24**	**C27**	**C30**	**C35**	**C40**	**D30**	**D35**	**D40**	**D50**	**D60**	**D70**
Strength properties (N/mm²)																	
Bending	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40	30	35	40	50	60	70
Tension parallel to grain	$f_{t,0,k}$	8	10	11	12	13	14	16	18	21	24	18	21	24	30	36	42
Tension perpendicular to grain	$f_{t,90,k}$	0.4	0.5	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Compression parallel to grain	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26	23	25	26	29	32	34
Compression perpendicular to grain	$f_{c,90,k}$	2.0	2.2	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	8.0	8.4	8.8	9.7	10.5	13.5
Shear parallel to grain	$f_{v,k}$	1.7	1.8	2.0	2.2	2.4	2.5	2.8	3.0	3.4	3.8	3.0	3.4	3.8	4.6	5.3	6.0
Stiffness properties (N/mm²)																	
Mean MOE parallel to grain	$E_{0,mean}$	7000	8000	9000	9500	10000	11000	11500	12000	13000	14000	10000	10000	11000	14000	17000	20000
5th percentile MOE parallel to grain	$E_{0,05}$	4700	5400	6000	6400	6700	7400	7700	8000	8700	9400	8000	8700	9400	11800	14300	16800
Mean MOE perpendicular to grain	$E_{90,mean}$	230	270	300	320	330	370	380	400	430	470	640	690	750	930	1130	1330
Mean shear modulus	G_{mean}	440	500	560	590	630	690	720	750	810	880	600	650	700	880	1060	1250
Density (kg/m³)																	
Characteristic density	ρ_k	290	310	320	330	340	350	370	380	400	420	530	560	590	650	700	900
Mean density	ρ_{mean}	350	370	380	390	410	420	450	460	480	500	640	670	700	780	840	1080

Notes

a These properties are compatible with timber conditioned at a temperature of 20°C and a relative humidity of 65%.

b BS EN 338 also gives values for C45 and C50.

CD1.2 Properties of strength-graded oak

BS 5756:1997 gives grading rules for temperate hardwoods such as oak and sweet chestnut. Grades THA and THB are designated as 'heavy structural' grades which are for use when the cross-sectional area is at least 20000mm² and the minimum cross-sectional dimension is 100mm. Grade THA is the higher grade. Grades TH1 and TH2 are for smaller sections, with grade TH1 being the higher grade.

BS 5268-2:2002 gives properties for oak and sweet chestnut graded as above, but these properties cannot be used directly with EC5. However it specifies equivalent solid timber strength classes for oak graded to TH1, THA and THB, and gives the densities for all four grades. The table below reproduces the properties for the BS EN 338:2003 strength classes to which BS 5268-2 designates the three BS 5756 grades of oak mentioned above, and tabulates the corresponding densities given in BS 5268-2 for all four grades of oak. It also gives properties for TH2 which have been scaled from the values for TH1 using the corresponding values given in BS 5268-2 for TH1 and TH2.

The values tabulated may be conservative compared with the values directly assigned to BS 5756 grades given in BS 5268-2 Table 15.

Table CD1.2 Properties of oak grades[a] BS 5268-2 and BS EN 338

		Grades to BS 5758			
		TH1	TH2	THA	THB
Strength properties (N/mm²)					
Bending	$f_{m,k}$	30	24.4	40	30
Tension parallel to grain	$f_{t,0,k}$	18	14.6	24	18
Tension perpendicular to grain	$f_{t,90,k}$	0.6	0.6[b]	0.6	0.6
Compression parallel to grain	$f_{c,0,k}$	23	20.8	26	23
Compression perpendicular to grain	$f_{c,90,k}$	8.0	8.0	8.8	8.0
Shear parallel to grain	$f_{v,k}$	3.0	3.0	3.8	3.0
Stiffness properties (N/mm²)					
Mean MOE parallel to grain	$E_{0,mean}$	10000	6720	11000	10000
5th percentile MOE parallel to grain	$E_{0,05}$	8000	6590	9400	8000
Mean MOE perpendicular to grain	$E_{90,mean}$	640	450[b]	750	640
Mean shear modulus	G_{mean}	600	420[b]	700	600
Density (kg/m³)					
Characteristic density	ρ_k	569	598	595	584
Mean density	ρ_{mean}	680	704	713	692

Notes

a These properties are compatible with timber conditioned at a temperature of 20°C and a relative humidity of 65%.

b See BS EN 338 Annex A.

CD1.3 Properties of grade TH1 American hardwoods

Four species of American hardwood are exported to the UK, which are graded to the USA National Hardwood Lumber Association rules for appearance. These may be used structurally if they are first regraded for strength to BS 5756:2007, grade TH1, by a certified timber grader in the UK. They are available in thicknesses from 19mm to 100mm, in widths of 102mm and wider, and in lengths from 1.8m to 4.8m. Larger sections can be obtained by glued lamination.

For further information contact the American Hardwood Export Council, 3 St Michael's Alley, London, EC3V 9DS, UK. T: +44 (0)20 7626 4111 F: +44 (0)20 7626 4222.

Table CD1.3 Properties of grade TH1 American hardwoods

		Species			
		American white oak (Quercus spp.)	**American red oak (Quercus spp.)**	**American ash (Fraxinus spp.)**	**Tulipwood (Liriodendron tulipfera)**
Strength properties (N/mm²)					
Bending parallel	$f_{m,k}$	51.8	53.7	37.8	41.7
Tension parallel	$f_{t,0,k}$	31.1	32.2	22.7	25.0
Tension perpendicular	$f_{t,90,k}$	0.6	0.6	0.6	0.5
Compression parallel	$f_{c,0,k}$	29.5	30.0	25.6	26.8
Compression perpendicular	$f_{c,90,k}$	10.3	9.2	9.2	6.8
Shear parallel	$f_{v,k}$	4.7	4.8	3.7	4.0
Stiffness properties (N/mm²)					
Mean MOE parallel to grain	$E_{0,mean}$	15000	13000	12800	11900
5th percentile MOE parallel to grain	$E_{0,05}$	12600	10900	10700	10000
Mean MOE perpendicular to grain	$E_{90,mean}$	1000	870	850	800
Mean shear modulus	G_{mean}	940	810	800	750
Density (kg/m³)					
Mean density	ρ_{mean}	811	680	667	552
5% density	ρ_k	688	615	616	456
Equivalent strength class[a]		D50	D40	D35	D40[b]
Heartwood durability rating to BS EN 350-2:1994		2/3 (Durable/ moderately durable)	4 (Slightly durable)	5 (Not durable)	5 (Not durable)

Notes

a Particular properties may exceed the requirements for the designated strength class. It is generally preferable to use the tabulated properties rather than those for the strength class.

b The density of tulipwood is less than that specified for D40, so for the calculation of self-weight and connection capacities the densities shown in this table should be used.

CD1.4 Properties of glued laminated softwood strength classes

Table CD1.4 Properties of glued laminated softwood[a] strength classes (BS EN 1194:1999)

		Homogenous glulam				Combined glulam			
		GL24h	GL28h	GL32h	GL36h	GL24c	GL28c	GL32c	GL36c
Strength properties (N/mm^2)									
Bending	$f_{m,g,k}$	24	28	32	36	24	28	32	36
Tension parallel to grain	$f_{t,0,g,k}$	16.5	19.5	22.5	26	14	16.5	19.5	22.5
Tension perpendicular to grain	$f_{t,90,g,k}$	0.4	0.45	0.5	0.6	0.35	0.4	0.45	0.5
Compression parallel to grain	$f_{c,0,g,k}$	24	26.5	29	31	21	24	26.5	29
Compression perpendicular to grain	$f_{c,90,g,k}$	2.7	3.0	3.3	3.6	2.4	2.7	3.2	3.8
Shear parallel to grain	$f_{v,g,k}$	2.7	3.0	3.8	4.3	2.2	2.7	3.0	3.3
Stiffness properties (N/mm^2)									
Mean MOE parallel to grain	$E_{0,g,mean}$	11600	12600	13700	14700	11600	12600	13700	14700
5th percentile MOE parallel to grain	$E_{0,g,05}$	9400	10200	11100	11900	9400	10200	11100	11900
Mean MOE perpendicular to grain	$E_{90,g,mean}$	390	420	460	490	320	390	420	460
Mean shear modulus	$G_{g,mean}$	720	780	850	910	590	720	780	850
Density (kg/m^3)									
Characteristic density	$\rho_{g,k}$	380	410	430	450	350	380	410	430
Mean density	$\rho_{g,mean}$ [b]	420	450	475	495	395	430	460	485

Notes

a These properties are compatible with timber conditioned at a temperature of 20°C.

b Calculated as $1.1\rho_k$ for homogenous glulam, $1.125\rho_k$ for combined glulam.

CD1.5 Properties of some common types of LVL

Table CD1.5 Properties of some common types of LVL[a] (source VTT Certificate number 184/03 March 2004)				
		LVL-S[b] Thickness 21 – 90mm	LVL-Q[c] Thickness 21 – 24mm	LVL-Q[c] Thickness 27 – 29mm
Strength properties (N/mm²)				
Bending strength				
Edgewise	$f_{m,0,edge,k}$	44.0	28.0	32.0
Size effect parameter	s	0.12	0.12	0.12
Flatwise	$f_{m,0,flat,k}$	50.0	32.0	36.0
Tensile strength				
Parallel to grain	$f_{t,0,k}$	35.0	19.0	26.0
Perpendicular to grain, edgewise	$f_{t,90,edge,k}$	0.8	6.0	6.0
Perpendicular to grain, flatwise	$f_{t,90,flat,k}$	-	-	-
Compressive strength				
Parallel to grain	$f_{c,0,k}$	35.0	19.0	26.0
Perpendicular to grain, edgewise	$f_{c,90,edge,k}$	3.4	9.0	9.0
Perpendicular to grain, flatwise	$f_{c,90,flat,k}$	1.7	1.7	1.7
Shear strength				
Edgewise	$f_{v,0,edge,k}$	5.7	5.7	5.7
Flatwise	$f_{v,0,flat,k}$	4.4	1.3	1.3
Stiffness properties (N/mm²)				
Modulus of elasticity				
Mean parallel to grain	$E_{0,mean}$	13500	10000	10500
Mean perpendicular to grain	$E_{90,mean}$	-	-	-
5th percentile parallel to grain	$E_{0,k}$	11600	8300	8800
5th percentile perpendicular to grain	$E_{90,k}$	-	-	-
Shear modulus				
Mean, edgewise	$G_{0,edge,mean}$	600	600	600
Mean, flatwise	$G_{0,flat,mean}$	600	-	-
5th percentile, edgewise	$G_{0,edge,k}$	400	400	400
5th percentile, flatwise	$G_{0,flat,k}$	400	-	-
Density (kg/m³)				
Mean	ρ_{mean}	510	510	510
Minimum	ρ_k	480	480	480

Notes

a For precise values refer to the manufacturer's literature.

b LVL-S: LVL: the grain direction of all the veneers is the same.

c LVL-Q: LVL: some veneers are cross-grained to increase its dimensional stability in large panels.

CD1.6 Properties of OSB/3 and OSB/4

Table CD1.6 Properties of OSB/3 and OSB/4[a] from BS EN 12369-1:2001

		OSB/3 Load bearing boards for use in humid conditions			OSB/4 Heavy duty load bearing boards for use in humid conditions		
		Board thickness (mm)			Board thickness (mm)		
		6+ to 10	10+ to 18	18+ to 25	6+ to 10	10+ to 18	18+ to 25
Strength properties (N/mm²)							
Bending parallel to span	$f_{m,0,k}$	18.0	16.4	14.8	24.5	23.0	21.0
Bending perpendicular to span	$f_{m,90,k}$	9.0	8.2	7.4	13.0	12.2	11.4
Tension parallel to span	$f_{t,0,k}$	9.9	9.4	9.0	11.9	11.4	10.9
Tension perpendicular to span	$f_{t,90,k}$	7.2	7.0	6.8	8.5	8.2	8.0
Compression parallel to span	$f_{c,0,k}$	15.9	15.4	14.8	18.1	17.6	17.0
Compression perpendicular to span	$f_{c,90,k}$	12.9	12.7	12.4	14.3	14.0	13.7
Panel shear (as in a racking panel)	$f_{v,k}$	6.8	6.8	6.8	6.9	6.9	6.9
Planar shear (as in floor decking)	$f_{r,k}$	1.0	1.0	1.0	1.1	1.1	1.1
Stiffness properties (N/mm²)[b]							
MOE in bending parallel to span	$E_{m,0}$	4930	4930	4930	6780	6780	6780
MOE in bending perpendicular to span	$E_{m.90}$	1980	1980	1980	2680	2680	2680
MOE in tension parallel to span	$E_{t,0}$	3800	3800	3800	4300	4300	4300
MOE in tension perpendicular to span	$E_{t.90}$	3000	3000	3000	3200	3200	3200
MOE in compression parallel to span	$E_{c,0}$	3800	3800	3800	4300	4300	4300
MOE in compression perpendicular to span	$E_{c.90}$	3000	3000	3000	3200	3200	3200
Shear modulus in panel shear	G_v	1080	1080	1080	1090	1090	1090
Shear modulus in planar shear	G_r	50	50	50	60	60	60
Density (kg/m³)							
Characteristic density	ρ_k	550	550	550	550	550	550
Mean density[c]	ρ_{mean}	650	650	650	650	650	650

Notes

a These properties are compatible with timber conditioned at a temperature of 20°C and a relative humidity of 65%.

b 5th percentile characteristic values for stiffness may be taken as 0.85 × the mean values given in the table.

c Calculated as ρ_k/0.85 based on Note **b**.

CD1.7 Properties of wood particleboard (chipboard)

Table CD1.7 Properties of wood particleboard (chipboard)[a] from BS EN 312:2003 (types 5 and 6) and BS EN 12369-1:2001

		BS EN 312-5 Load bearing boards for use in humid conditions						BS EN 312-6 Heavy duty load bearing boards for use in humid conditions					
		Board thickness (mm)						Board thickness (mm)					
		6+ to 13	13+ to 20	20+ to 25	25+ to 32	32+ to 40	40+	6+ to 13	13+ to 20	20+ to 25	25+ to 32	32+ to 40	40+
Strength properties (N/mm²)													
Bending	$f_{m,0,k}$	15.0	13.3	11.7	10.0	8.3	7.5	18.3	16.7	15.4	14.2	13.3	12.5
Tension	$f_{t,0,k}$	9.4	8.5	7.4	6.6	5.6	5.6	11.5	10.6	9.8	9.4	9.0	8.0
Compression	$f_{c,0,k}$	12.7	11.8	10.3	9.8	8.5	7.8	15.5	14.7	13.7	13.5	13.2	13.0
Panel shear (as in a racking panel)	$f_{v,k}$	7.0	6.5	5.9	5.2	4.8	4.4	8.6	8.1	7.9	7.4	7.2	7.0
Planar shear (as in floor decking)	$f_{r,k}$	1.9	1.7	1.5	1.3	1.2	1.0	2.4	2.2	2.0	1.9	1.9	1.8
Stiffness properties (N/mm²)[b]													
MOE in bending	$E_{m,0}$	3500	3300	3000	2600	2400	2100	4600	4200	4000	3900	3500	3200
MOE in tension and compression	$E_{t,0}$	2000	1900	1800	1500	1400	1300	2600	2500	2400	2300	2100	2000
Shear modulus in panel shear	G_v	960	930	860	750	690	660	1250	1200	1150	1100	1050	1000
Density (kg/m³)													
Characteristic density	ρ_k	650	600	550	550	500	500	650	600	550	550	500	500
Mean density[c]	ρ_{mean}	815	750	690	690	625	625	815	750	690	690	625	625

Notes

a These properties are compatible with timber conditioned at a temperature of 20°C and a relative humidity of 65%.

b 5th percentile characteristic values for stiffness may be taken as 0.8 $\times$ the mean values given in the table.

c Calculated as ρ_k/0.8 based on Note **b**.

CD2 Laterally loaded fastener spreadsheets

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The values calculated in the spreadsheets are provided in good faith that they are in accordance with the rules given in BS EN 1995-1-1:2004 and the assumptions detailed in this document. The copyright owners give no guarantee of the reliability of the calculated values, and users should check all such values independently before using them in a final design.

CD2.1 Calculation basis for fastener spreadsheets

CD2.1.1 Introduction

The accompanying fastener spreadsheets cover timber connections made with laterally loaded nails, wood screws, coach screws, bolts and steel dowels. The member materials covered comprise the softwood and hardwood grades in BS EN 338:2003, the softwood glulam grades in BS EN 1194:1999, Kerto LVL in British Board of Agrément Certificate Number 00/3717, the softwood plywoods in BS 5268-2:2002, wood based panels in BS EN 12369-1:2001 (OSB, particleboards and hardboards), and steel. To reduce the size of the tables the solid timber strength classes have been amalgamated into six larger groups. For strength classes which are not the lowest one in the group, the values will be mostly conservative.

The spreadsheets give the load-carrying capacity per shear plane of single nails, screws, bolts and dowels loaded in lateral shear. These capacities have been calculated using the formulae given in Clauses 8.2 (Lateral load-carrying capacity of metal dowel-type fasteners), 8.3 (Nailed connections), 8.5 (Bolted connections), 8.6 (Dowelled connections) and 8.7 (Screwed connections) of EC5.

The load-carrying capacities are displayed in Microsoft Excel© spreadsheets which allow the user to specify the dimensions of the fasteners and members, the service class and the load duration. Design values can be displayed for both normal and accidental loading conditions; also characteristic values can be displayed. So far as possible 'illegal' entries such as bolt diameters of more than 30mm are prevented. However before finalising a design the user should always check compliance with any limitations on diameter, pointside penetration and permissible service class given in EC5 or in the certification literature of proprietary materials.

In common with most published fastener load tables, the tables give the total load capacity per fastener per shear plane. This means that the values given for 3-member connections should be multiplied by 2 to obtain the load capacity per fastener. For multiple fastener connections the effective number of fasteners may be less than the actual number if there is a component of load parallel to the grain: in this instance reference should be made to the *Manual* or EC5.

The types of connection covered are shown in Table CD2.1. The spreadsheets should be used in accordance with the following explanatory notes.

CD2.1.2 Nailed connections

General notes

- The nail tables are applicable to 2-member connections made with smooth round wire with a minimum tensile strength of 600N/mm^2.
- The values given are for a single nail driven at right-angles to the grain and loaded in shear.
- It is assumed that holes in softwoods and wood-based panel products are not pre-drilled, and that holes in steel and hardwoods are pre-drilled. Pre-drilled holes should have a diameter no greater than 0.8 times the nail diameter.
- The minimum spacings and distances specified in EC5 8.3.1.2 should be observed.
- The contribution of the rope effect ($F_{ax,Rk}/4$) to the load-carrying capacity is limited to 15% of the principal 'Johanssen' part. For permanent and long-term loading contribution of the rope effect is taken as zero because smooth nails may not be used to resist axial loads under permanent and long-term loading.
- For load durations other than permanent and long-term, the characteristic withdrawal capacity $F_{ax,Rk}$ is calculated as the lower of the capacities in the two members, found from the following expressions:

$$F_{ax,Rk} \min. \begin{cases} f_{ax,point,k}\, d\, t_{pen} \\ f_{ax,head,k}\, d\, t_{pen} + f_{head,k}\, d_h^2 \end{cases}$$

Where

$f_{ax,point,k}$ = characteristic pointside withdrawal strength
$f_{ax,head,k}$ = the characteristic headside withdrawal strength
$f_{head,k}$ = the characteristic headside pull-through strength
t_{pen} = the pointside penetration length
d = the nail diameter
t = the thickness of the headside member
d_h = the nail head diameter (assumed to be equal to 2d)

For smooth nails with a pointside penetration of at least 12d, the characteristic values of the withdrawal and pull-through strengths are found from the following expressions:

$f_{ax,point,k}$ or $f_{ax,head,k} = 20 \times 10^{-6}\rho_k^2$
$f_{head,k} = 70 \times 10^{-6}\rho_k^2$

Where

ρ_k is the characteristic timber density in kg/m^3

Table CD2.1 Connections covered

Fastener	Number of members	Material A	Material B	Material A′
Nails	2	Timber	Timber	-
		Plywood	Timber	-
		Particleboard	Timber	-
		OSB	Timber	-
		Hardboard	Timber	-
		LVL	LVL	-
Wood screws	2	Timber	Timber	-
		Plywood	Timber	-
		OSB	Timber	-
Coach screws	2	Timber	Timber	-
		Steel	Timber	-
Bolts and dowels	2	Timber	Timber	-
		Glulam	Glulam	-
		LVL	LVL	-
	3	Timber	Timber	Timber
		Glulam	Glulam	Glulam
		LVL	LVL	LVL
		Plywood	Timber	Plywood
		Timber	Steel	Timber
		Glulam	Steel	Glulam
		LVL	Steel	LVL
		Steel	Timber	Steel
		Steel	Glulam	Steel
		Steel	LVL	Steel

- For smooth nails, the pointside penetration t_{pen} should be at least $8d$. For nails with a pointside penetration smaller than $12d$ the withdrawal capacity is multiplied by $(t_{pen}/4d - 2)$. For nails with a pointside penetration smaller than $8d$, the withdrawal capacity is taken as 0.

Timber-to-timber connections

- Nail penetrations refer to the embedded length of nail in either the headside or the pointside member, whichever is less.

Wood-based panels-to-timber connections

- The nails should be fully embedded and should have overall lengths not less than the values tabulated.
- The plywood groups used in the spreadsheets are the same as those defined in BS 5268-2 Annex G, except that Swedish softwood plywood is assigned to a third group because its characteristic density is less than that of other softwood plywoods in group I. So the plywood groups comprise:

  Plywood group I
  - American construction and industrial plywood
  - Canadian Douglas fir plywood
  - Canadian softwood plywood
  - Finnish conifer plywood

  Plywood group II
  - Finnish birch-faced plywood
  - Finnish birch plywood

  Plywood group III
  - Swedish softwood plywood

  The corresponding densities used in the spreadsheets are shown in Table CD2.2.
- OSB, particleboard and hardboard should comply with the specifications given in BS EN 12369-1. The characteristic densities shown in Table CD2.3 have been used to calculate head pull-through strength for the ‘rope’ effect.

LVL connections

- Member thickness refers to the embedded length of nail in either the headside or the pointside member, whichever is less.
- In LVL the maximum diameter of nails inserted parallel to the glue line should be 4mm.
- BBA Certificate No 00/3717 specifies that Kerto LVL should be classed as C27 for connections. Since this is for UK use, the characteristic density of C27 given in BS 5268-2 has been used, namely 370kg/m³.

Table CD2.2 Characateristic density of plywood at 12% moisture content			
Plywood thickness (mm)	Characteristic density (kg/m³)		
	Plywood group I	Plywood group II	Plywood group III
6	527	641	420
9	505	641	420
12	505	628	420
15	505	614	420
18	505	599	420
21	505	593	420
29	505	548	436

Table CD2.3 Characteristic density of boards	
Thickness (mm)	Characteristic density (kg/m³)
OSB	
From 6 to 25	550
Particleboard	
9	650
13	650
15	600
18	600
22	550
Hardboard	
3.2	900
4.8	850
6.4	800
8	800

CD2.1.3 Screwed connections

General notes

- The screw tables are applicable to 2-member connections made with wood screws made from steel with a minimum tensile strength of 540N/mm², or made with coach screws made from steel with a minimum tensile strength of 400N/mm².
- The values given are for a single fastener driven at right-angles to the grain and loaded in shear.
- For wood screws it is assumed that holes in softwoods and wood-based panel products are not pre-drilled, and that holes in steel and hardwoods are pre-drilled. For coach screws it is assumed that holes are pre-drilled in both members.
- For screws with a nominal diameter d > 6mm the rules in EC5 8.5.1 apply. These are assumed to be coach screws. The yield moment is based on an effective diameter of 0.9d, except where the headside member is metal in which case an effective diameter of d has been used.
- For screws with a diameter of 6mm or less, the rules in EC5 8.3.1 apply. These are assumed to be wood screws. The yield moment is based on an effective diameter of 2d/3, assuming a minimum root diameter of 0.6d as specified in prEN 14592.
- The minimum spacings and end distances specified in EC5 8.5.1.1 and 8.3.1.2 should be observed.
- The contribution of the rope effect ($F_{ax,Rk}$ /4) to the load-carrying capacity is limited to 100% of the principal 'Johanssen' part. The characteristic withdrawal capacity $F_{ax,Rk}$ is taken as the lower of the capacities in the two members.
- Because of acknowledged problems with the EC5 screw withdrawal strength design method at the time of publication, a value of 4 times the smooth nail withdrawal strength has been used. Hence the characteristic withdrawal strength is calculated as:

$$f_{ax,point,k} = 80 \times 10^{-6} \rho_k^2$$

Where ρ_k is the characteristic timber density (kg/m³)

- For wood screws the characteristic withdrawal capacity $F_{ax,Rk}$ is calculated as the lower of the capacities in the two members, found from the following expressions:

$$F_{ax,Rk} \text{ min.} \begin{cases} f_{ax,point,k}\ d\ t_{pen} \\ f_{ax,head,k}\ d_h^2 \end{cases}$$

  with the symbols other than $f_{ax,point,k}$ defined as for nails. No withdrawal resistance has been included for the headside member, in case a clearance hole is used (which is essential for coach screws). Only the head pull-through contribution has been included.
- For coach screws with washers the characteristic pull-through resistance is based on the timber bearing strength beneath a washer with a diameter of $3d$ and a net bearing area of $2\pi d^2$. The compressive strength of the timber is taken as $3.0 \times 0.007\rho_k$, according to Annex A of BS EN 338, and EC5 8.5.2(2).

  Hence $F_{ax,head,Rk} = 3.0 \times 0.007\rho_k \times 2\pi d^2$.
- For coach screws inserted through steel plates, the characteristic pull-through resistance is based on the timber bearing beneath a circular washer with a diameter equal to the smaller of $12t_{steel}$ and 4_d, where t_{steel} is the thickness of the steel, in accordance with EC5 8.5.2(3).

  Hence $F_{ax,head,Rk} = 3.0 \times 0.007\rho_k \times \pi$ (MIN[$12t_{steel}$,$4d$]^2–d^2)/4.

  Hence for coach screws the characteristic withdrawal capacity $F_{ax,Rk}$ is calculated as the lower of the capacities in the two members, found from the following expressions:

$$F_{ax,Rk} \text{ min.} \begin{cases} f_{ax,point,k}\ d\ t_{pen} \\ f_{ax,head,Rk} \end{cases}$$

  Where the symbols other than $f_{ax,point,k}$ and $F_{ax,head,Rk}$ are defined as for nails.
- It is assumed that the tensile strength of the steel does not govern the characteristic withdrawal capacity.
- The pointside penetration t_{pen} should be at least $6d$, as for threaded nails (EC5 8.3.2(7)). For screws with a pointside penetration smaller than $8d$ the withdrawal capacity is multiplied by ($t_{pen}/2_d - 3$). For screws with a pointside penetration smaller than $6d$, the withdrawal capacity is taken as 0.

Timber-to-timber connections

- It is assumed that the pointside penetration depth is twice the tabulated headside thickness.

Plywood and OSB-to-timber connections

- The thickness of the pointside member should be sufficient to accommodate the full length of the screw.
- Plywood and OSB should comply with the specifications quoted in CD2.2.

Steel-to-timber coach screw connections

The thickness of steel plates should preferably comply with BS 5268-2: Section 6 in Clause 6.6.5.1, which states that they should have a minimum thickness of 2.5mm or 0.3 times the coach screw diameter, whichever is the greater. Otherwise a warning message "Check the bearing strength of the steel plate" will be displayed.

CD2.1.4 Bolted and dowelled connections

General notes

- The bolt and dowel tables are applicable to connections made with 4.6 steel with a minimum tensile strength of 400N/mm^2.
- The values given are for a single fastener loaded in shear, per shear plane.
- The minimum spacings and distances specified EC5 8.5.1.1 and 8.6 should be observed.
- For bolted connections, bolt holes should have a diameter not greater than d + 1mm, where d is the diameter of the bolt. A steel washer with a side length or diameter of at least $0.3d$ should be used between the bolt head and nut and the timber.
- For dowelled connections the tolerance on the dowel should be –0/+0.1mm, and the diameter of the pre-bored holes should not exceed the diameter of the dowel.
- For 2-member connections not involving steel it is assumed that both members have the same thickness. For 3-member connections not involving steel it is assumed that both the outer members have the same thickness and the inner member is twice as thick as each outer member.
- As specified in EC5 8.2.2, the contribution of the rope effect ($F_{ax,Rk}$/4) to the load-carrying capacity is limited to 25% of the principal 'Johanssen' part for bolts, and 0% for dowels.
- For bolts with washers the characteristic withdrawal capacity is based on the timber bearing strength beneath a washer with a diameter of $3d$ and a net bearing area of $2\pi d^2$. The compressive strength of the timber is taken as $3.0 \times 0.007\rho_k$ for softwoods and plywood, and $3.0 \times 0.015\rho_k$ for hardwoods, according to Annex A of BS EN 338, and EC5 8.5.2(2).

  Hence $F_{ax,Rk} = 3.0 \times 0.007\rho_k \times 2\pi d^2$ for softwoods and plywood

  $= 3.0 \text{ x } 0.015\rho_k \times 2\pi d^2$ for hardwoods.
- For bolts inserted through steel plates, the characteristic withdrawal capacity is based on the timber bearing beneath a circular washer with a diameter equal to the smaller of $12t_{steel}$ and $4d$, where t_{steel} is the thickness of the steel, in accordance with EC5 8.5.2(3).

  Hence $F_{ax,\,Rk} = 3.0 \times 0.007\rho_k \times \pi$(MIN.[$12t_{steel}$,$4d$]^2–$d^2$)/4 for softwoods and plywood

  $= 3.0 \times 0.015\rho_k \times \pi$(MIN.[$12t_{steel}$,$4d$]^2–$d^2$)/4 for hardwoods.

- In multiple member connections the total load-carrying capacity should be determined by calculating the sum of the load-carrying capacity for each shear plane, treating the connection as a series of three member connections.

CD2.1.5 Plywood-to-timber connections

- Plywood should comply with the specifications quoted in Section CD2.1.2.

CD2.1.6 Connections involving steel plates

- The thickness of steel plates should preferably comply with BS 5268-2 Section 6 in Clause 6.6.5.1, which states that they should have a minimum thickness of 2.5mm or 0.3 times the coach screw diameter, whichever is the greater. Otherwise a warning message “Check the bearing strength of the steel plate” will be displayed.

CD2.2 NAILED CONNECTIONS

In accordance with BS EN 1995-1-1:2004 and accompanying "Calculation basis"

2-MEMBER TIMBER JOINT

2-MEMBER LVL JOINT

2-MEMBER PLYWOOD-TO-TIMBER JOINT

2-MEMBER OSB-TO-TIMBER JOINT

2-MEMBER PARTICLEBOARD-TO-TIMBER JOINT

2-MEMBER HARDBOARD-TO-TIMBER JOINT

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Lateral design loads for one smooth round wire nail in a 2-member timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Nail diameter	Member thickness (Note a)	Strength classes					
		Softwoods - not predrilled				Hardwoods - predrilled	
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2.65	32	0.392	0.408	0.440	0.456	0.650	0.723
3	36	0.483	0.503	0.542	0.561	0.814	0.902
3.35	40	0.580	0.604	0.651	0.674	0.991	1.099
3.75	45	0.701	0.730	0.788	0.817	1.213	1.343
4	48	0.781	0.814	0.878	0.910	1.361	1.507
4.5	54	0.952	0.992	1.071	1.110	1.678	1.858
5	60	1.136	1.184	1.279	1.326	2.023	2.240
5.6	67	1.372	1.431	1.546	1.604	2.472	2.738
6	72	1.542	1.608	1.739	1.805	2.793	3.093
6.7	80	1.853	1.933	2.091	2.170	3.394	3.759
8	96	2.501	2.610	2.827	2.936	4.638	5.136

Note a
Member thickness refers to the embedded length of nail in either the headside or the pointside member, whichever is less. For smooth nails the standard pointside penetration of 12d (12 nail iameters) may be reduced to a minimum of 8d with some loss of load capacity.

Lateral design loads for one smooth round wire nail in a 2-member LVL joint

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Note a
Member thickness refers to the embedded length of nail in either the headside or the pointside member, whichever is less. For smooth nails the standard pointside penetration of 12d (12 nail diameters) may be reduced to a minimum of 8d with some loss of load capacity.

Member thickness (Note a)	Not predrilled
	KERTO LVL
(mm)	(kN)
32	0.415
36	0.510
40	0.612
45	0.737
48	0.820
54	0.995
60	1.184
67	1.428
72	1.600
80	1.919
96	2.572

Lateral design loads for one smooth round wire nail in a 2-member plywood-to-timber joint

Plywood group	Group 1	See table on right	
Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

Design values | Characteristic values | Reset tabl | Return to menu

Group 1	Group 2	Group 3
American construction and industrial plywood	Finnish birch-faced plywood Finnish birch plywood	Swedish sofwood plywood
Canadian Douglas fir plywood	For smooth nails the standard pointside penetration of 12d (12 nail diameters) may be reduced to a minimum of 8d with some loss of load capacity.	
Canadian softwood plywood		
Finnish conifer plywood		

Plywood thickness	Nail dimensions		Strength classes					
			Softwoods - not predrilled				Hardwoods - predrilled	
	Diameter	Length	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(N)	(N)	(N)	(N)	(N)	(N)
6	2.65	40	320	330	350	359	430	450
6	3	50	393	406	433	441	520	520
6	3.35	50	464	480	510	524	561	561
6	3.75	75	579	593	607	607	607	607
6	4	75	635	635	635	635	635	635
9	2.65	40	343	352	370	379	458	477
9	3	50	415	427	452	465	549	572
9	3.35	50	482	497	526	541	649	676
9	3.75	75	593	612	635	645	774	807
9	4	75	651	674	700	711	859	896
12	2.65	50	393	404	425	436	513	533
12	3	50	454	467	492	504	602	625
12	3.35	50	514	528	555	568	698	725
12	3.75	75	629	650	680	691	818	850
12	4	75	685	708	743	755	899	935
15	2.65	50	441	452	474	485	580	601
15	3	65	516	531	560	575	669	693
15	3.35	65	584	601	634	651	765	793
15	3.75	75	677	698	740	752	883	916
15	4	75	732	755	800	815	962	998

CD2.2
Note this is a snapshot of an interactive spreadsheet

18	2.65	50	460	474	500	513	647	676
18	3	65	571	586	616	631	746	772
18	3.35	65	640	657	691	708	843	873
18	3.75	75	734	755	797	818	962	996
18	4	75	789	812	857	879	1041	1077
21	2.65	50	450	464	488	499	620	675
21	3	65	573	591	625	642	804	843
21	3.35	65	684	705	746	765	929	961
21	3.75	75	796	818	860	881	1050	1086
21	4	75	852	875	921	943	1130	1168
29	2.65	65	463	477	504	518	650	682
29	3	65	566	583	615	631	804	843
29	3.35	65	660	683	718	735	925	1006
29	3.75	75	822	847	896	920	1178	1233
29	4	75	905	932	984	1008	1301	1377

Lateral design loads for one smooth round wire nail in a 2-member OSB-to-timber joint

OSB group: OSB 3/4

Service Class: 1 — $k_{mod,combined}$ = 0.90

Load duration: Short term — γ_M = 1.30

Design situation: Normal

Design values | Characteristic values | Reset table | Return to menu

For smooth nails the standard pointside penetration of 12d (12 nail diameters) may be reduced to a minimum of 8d with some loss of load capacity.

OSB thickness	Nail dimensions		Strength classes					
			Softwoods - not predrilled				Hardwoods - predrilled	
	Diameter	Length	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(N)	(N)	(N)	(N)	(N)	(N)
9	2.65	50	351	361	383	393	456	474
9	3	50	407	419	444	456	535	557
9	3.35	50	468	483	511	525	624	649
9	3.75	75	572	587	608	618	735	750
9	4	75	626	644	668	678	765	765
11	2.65	50	379	390	412	422	495	514
11	3	50	433	445	470	482	570	592
11	3.35	50	488	502	529	542	654	679
11	3.75	75	592	612	634	643	760	788
11	4	75	643	665	690	701	831	863
13	2.65	50	413	424	446	456	542	562
13	3	50	465	477	502	514	614	637
13	3.35	50	512	525	551	563	694	719
13	3.75	75	619	639	668	677	795	824
13	4	75	668	690	722	732	863	894
15	2.65	50	450	461	484	494	594	616
15	3	65	513	528	557	572	665	689
15	3.35	65	570	586	619	635	742	768
15	3.75	75	651	671	708	718	839	868
15	4	75	698	720	760	771	904	936

CD2.2
Note this is a snapshot of an interactive spreadsheet

18	2.65	50	464	478	504	517	655	692
18	3	65	573	588	618	633	749	775
18	3.35	65	628	645	678	695	824	852
18	3.75	75	706	727	767	788	917	948
18	4	75	752	774	817	839	980	1012
23	2.65	50	427	450	487	498	614	663
23	3	65	573	591	626	643	810	850
23	3.35	65	681	701	741	761	968	1014
23	3.75	75	814	836	877	898	1071	1106
23	4	75	858	881	925	947	1131	1168

Lateral design loads for one smooth round wire nail in a 2-member particleboard-to-timber joint

Particleboard group: Particleboard 4/5

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.87

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

For smooth nails the standard pointside penetration of 12d (12 nail diameters) may be reduced to a minimum of 8d with some loss of load capacity.

Particleboard thickness	Nail dimensions		Strength classes					
			Softwoods - not predrilled				Hardwoods - predrilled	
	Diameter	Length	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(N)	(N)	(N)	(N)	(N)	(N)
9	2.65	40	331	341	358	367	443	461
9	3	50	395	407	431	443	520	541
9	3.35	50	455	469	496	510	606	630
9	3.75	75	556	571	591	601	715	729
9	4	75	608	626	649	659	743	743
13	2.65	45	397	408	428	437	527	546
13	3	50	451	464	488	500	597	619
13	3.35	50	498	511	535	547	675	699
13	3.75	75	601	621	649	658	772	800
13	4	75	649	670	701	712	839	869
15	2.65	50	437	448	470	481	577	599
15	3	65	499	513	542	556	646	669
15	3.35	65	554	570	602	617	721	747
15	3.75	75	632	652	688	698	815	844
15	4	75	679	700	739	750	879	909
18	2.65	50	451	464	490	503	637	673
18	3	65	557	572	601	615	728	754
18	3.35	65	610	627	659	675	801	828
18	3.75	75	687	706	746	765	891	921
18	4	75	731	752	794	815	952	984

CD2.2
Note this is a snapshot of an interactive spreadsheet

22	2.65	50	428	451	476	487	605	656
22	3	65	558	575	609	626	787	825
22	3.35	65	662	682	721	740	921	952
22	3.75	75	769	790	830	850	1009	1042
22	4	75	812	834	877	898	1068	1102

Lateral design loads for one smooth round wire nail in a 2-member tempered hardboard-to-timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

Grade HB.LA and HB.HLA 1 or 2 to BS EN 622-2

$k_{mod,combined}$ = 0.87

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

For smooth nails the standard pointside penetration of 12d (12 nail diameters) may be reduced to a minimum of 8d with some loss of load capacity.

Hardboard thickness	Nail dimensions		Strength classes					
			Softwoods - not predrilled				Hardwoods - predrilled	
	Diameter	Length	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(N)	(N)	(N)	(N)	(N)	(N)
3.2	2.65	40	257	257	257	257	257	257
3.2	3	50	280	280	280	280	280	280
3.2	3.35	50	303	303	303	303	303	303
3.2	3.75	65	327	327	327	327	327	327
3.2	4	75	343	343	343	343	343	343
4.8	2.65	40	322	332	353	364	445	469
4.8	3	50	395	409	437	447	536	536
4.8	3.35	50	467	484	516	532	579	579
4.8	3.75	65	572	594	626	626	626	626
4.8	4	75	644	655	655	655	655	655
6.4	2.65	40	355	367	388	399	495	521
6.4	3	50	428	443	471	485	595	627
6.4	3.35	50	499	516	549	565	706	745
6.4	3.75	65	602	624	664	677	845	892
6.4	4	75	673	698	732	746	939	991
8	2.65	40	404	416	439	451	568	598
8	3	50	477	492	522	537	670	705
8	3.35	50	548	565	600	617	780	822
8	3.75	65	650	673	718	738	919	969
8	4	75	720	746	792	807	1011	1067

CD2.3 SCREWED CONNECTIONS

In accordance with BS EN 1995-1-1:2004 and accompanying CD2.1 *Calculation basis*

WOOD SCREWS

2-MEMBER TIMBER JOINT

2-MEMBER PLYWOOD-TO-TIMBER JOINT

2-MEMBER OSB-TO-TIMBER JOINT

COACH SCREWS

2-MEMBER TIMBER JOINT

2-MEMBER STEEL-TO-TIMBER JOINT

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Lateral design loads for one steel woodscrew in a 2-member timber joint

Service Class: 1

Softwood predrilled? Yes

Load duration: Short term

k_{mod} = 0.90

Design situation: Normal

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Screw nominal diameter	Headside thickness	Strength classes					
		Softwoods - drilling as specified				Hardwoods - predrilled	
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
3.45	14	0.386	0.412	0.465	0.492	0.671	0.808
4.17	17	0.556	0.593	0.670	0.709	0.950	1.146
4.88	20	0.752	0.803	0.908	0.946	1.268	1.532
5.59	22	0.951	1.016	1.149	1.210	1.626	1.969

Note
The tabulated values may be used for a connection with a headside member at least as thick as the headside thickness shown, and a screw pointside penetration equal to at least twice this thickness. This spreadsheet does not cover screws with a diameter greater than 6 mm.

Screw gauge sizes and typical lengths

No. 3 = 2.39mm 13mm - 16mm
No. 4 = 2.74mm 6mm - 25mm
No. 5 = 3.10mm 13mm - 25mm
No. 6 = 3.45mm 13mm - 51mm
No. 7 = 3.81mm 13mm - 51mm
No. 8 = 4.17mm 13mm - 89mm
No. 10 = 4.88mm 19mm - 102mm
No. 12 = 5.59mm 19mm - 152mm

Lateral design loads for one steel woodscrew in a 2-member plywood-to-timber joint

Plywood group: Group 1

Service Class: 1

Softwood predrilled? Yes

Load duration: Short term

$k_{mod,combined}$ = 0.90

Design situation: Normal

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Plywood thickness	Screw dimensions: Nominal diameter	Screw dimensions: Length	Strength classes: Softwoods - drilling as specified: C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	Hardwoods - predrilled: D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
6	3.45	34	0.415	0.436	0.478	0.482	0.509	0.523
6	4.17	40	0.565	0.596	0.654	0.654	0.654	0.654
6	4.88	46	0.730	0.730	0.730	0.730	0.730	0.730
6	5.59	51	0.803	0.803	0.803	0.803	0.803	0.803
9	3.45	37	0.464	0.485	0.514	0.519	0.547	0.561
9	4.17	43	0.611	0.642	0.682	0.687	0.724	0.742
9	4.88	49	0.780	0.821	0.872	0.880	0.925	0.949
9	5.59	54	0.964	1.018	1.089	1.099	1.155	1.155
12	3.45	40	0.531	0.553	0.584	0.589	0.620	0.636
12	4.17	46	0.681	0.713	0.753	0.760	0.799	0.819
12	4.88	52	0.850	0.892	0.944	0.952	0.999	1.024
12	5.59	57	1.032	1.086	1.159	1.168	1.225	1.254
15	3.45	43	0.534	0.558	0.593	0.600	0.645	0.668
15	4.17	49	0.750	0.785	0.833	0.843	0.887	0.909
15	4.88	55	0.933	0.976	1.030	1.039	1.090	1.116
15	5.59	60	1.118	1.172	1.246	1.256	1.316	1.347
18	3.45	46	0.534	0.558	0.593	0.600	0.645	0.668
18	4.17	52	0.750	0.785	0.833	0.843	0.903	0.935
18	4.88	58	0.994	1.042	1.105	1.117	1.191	1.219

Note
The tabulated values may be used for a connection with a screw at least as long as the length shown. The thickness of the timber must be sufficient to accommodate its full length. This spreadsheet does not cover screws with a diameter greater than 6 mm.

Screw gauge sizes and typical lengths

No. 3 = 2.39mm 13mm - 16mm
No. 4 = 2.74mm 6mm - 25mm
No. 5 = 3.10mm 13mm - 25mm
No. 6 = 3.45mm 13mm - 51mm
No. 7 = 3.81mm 13mm - 51mm
No. 8 = 4.17mm 13mm - 89mm
No. 10 = 4.88mm 19mm - 102mm
No. 12 = 5.59mm 19mm - 152mm

CD2.3
Note this is a snapshot of an interactive spreadsheet

18	5.59	63	1.214	1.269	1.345	1.356	1.421	1.454
21	3.45	49	0.534	0.558	0.593	0.600	0.645	0.668
21	4.17	55	0.750	0.785	0.833	0.843	0.903	0.935
21	4.88	61	0.994	1.042	1.105	1.117	1.194	1.235
21	5.59	66	1.262	1.323	1.409	1.425	1.521	1.570
29	4.17	63	0.750	0.785	0.833	0.843	0.903	0.935
29	4.88	69	0.994	1.042	1.105	1.117	1.194	1.235
29	5.59	74	1.262	1.323	1.409	1.425	1.521	1.571

Lateral design loads for one steel woodscrew in a 2-member OSB-to-timber joint

OSB group: OSB 2

Service Class: 1

Softwood predrilled? No

Load duration: Short term

$k_{mod,combined}$ = 0.87

Design situation: Normal

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

OSB thickness	Screw dimensions: Nominal diameter	Screw dimensions: Length	Strength classes: Softwoods - drilling as specified: C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	Hardwoods - predrilled: D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(N)	(N)	(N)	(N)	(N)	(N)
9	3.45	37	404	425	469	484	533	545
9	4.17	42	509	537	596	626	665	682
9	4.88	48	649	688	769	789	789	789
9	5.59	54	804	822	822	822	822	822
13	3.45	41	471	495	544	561	602	619
13	4.17	46	587	616	676	708	751	770
13	4.88	52	722	761	844	878	922	944
13	5.59	58	869	920	1027	1066	1117	1143
15	3.45	43	472	496	545	562	606	630
15	4.17	48	633	663	724	756	802	823
15	4.88	54	766	806	890	924	971	995
15	5.59	60	911	962	1071	1110	1163	1190
18	3.45	46	473	497	546	563	608	632
18	4.17	51	640	672	739	774	836	868
18	4.88	57	839	881	966	1002	1053	1079
18	5.59	63	983	1035	1145	1185	1242	1272
23	3.45	51	474	498	548	565	610	635
23	4.17	56	642	674	741	776	840	871
23	4.88	62	847	891	984	1022	1095	1135
23	5.59	68	1068	1126	1246	1291	1379	1426

Note

The tabulated values may be used for a connection with a screw at least as long as the length shown. The thickness of the timber must be sufficient to accommodate its full length. This spreadsheet does not cover screws with a diameter greater than 6 mm.

Screw gauge sizes and typical lengths

No. 3 = 2.39mm 13mm - 16mm
No. 4 = 2.74mm 6mm - 25mm
No. 5 = 3.10mm 13mm - 25mm
No. 6 = 3.45mm 13mm - 51mm
No. 7 = 3.81mm 13mm - 51mm
No. 8 = 4.17mm 13mm - 89mm
No. 10 = 4.88mm 19mm - 102mm
No. 12 = 5.59mm 19mm - 152mm

CD2.3
Note this is a snapshot of an interactive spreadsheet

Lateral design loads for one steel coach screw in a 2-member timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Screw diameter	Headside thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	35	2.30	2.44	2.70	2.84	3.65	4.13	1.82	1.92	2.12	2.22	3.62	4.10
8	38	2.40	2.54	2.83	2.94	3.65	4.13	1.88	1.99	2.20	2.30	3.62	4.10
8	44	2.55	2.65	2.85	2.94	3.65	4.13	2.01	2.13	2.36	2.48	3.62	4.10
8	47	2.55	2.65	2.85	2.94	3.65	4.13	2.08	2.20	2.44	2.52	3.62	4.10
8	50	2.55	2.65	2.85	2.94	3.65	4.13	2.15	2.26	2.44	2.52	3.62	4.10
10	35	2.91	3.10	3.48	3.68	5.24	6.18	2.28	2.43	2.73	2.89	5.08	6.07
10	38	3.28	3.46	3.83	4.01	5.45	6.18	2.60	2.74	3.02	3.16	5.30	6.07
10	44	3.51	3.71	4.12	4.33	5.45	6.18	2.73	2.89	3.19	3.35	5.35	6.07
10	47	3.63	3.85	4.25	4.39	5.45	6.18	2.81	2.97	3.29	3.45	5.35	6.07
10	63	3.80	3.95	4.25	4.39	5.45	6.18	3.23	3.36	3.62	3.74	5.35	6.07
12	38	3.50	3.69	4.07	4.27	5.87	7.13	2.65	2.79	3.07	3.21	5.55	6.75
12	44	4.39	4.69	5.28	5.54	7.56	8.59	3.43	3.67	4.14	4.33	7.19	8.35
12	47	4.65	4.92	5.45	5.72	7.57	8.59	3.64	3.84	4.23	4.43	7.35	8.35
12	50	4.79	5.07	5.63	5.90	7.57	8.59	3.72	3.92	4.34	4.54	7.35	8.35
12	63	5.26	5.48	5.89	6.09	7.57	8.59	4.09	4.33	4.81	5.05	7.35	8.35
16	50	5.73	6.04	6.66	6.97	9.55	11.56	4.21	4.43	4.86	5.08	8.66	10.48
16	63	7.91	8.36	9.28	9.74	12.65	14.38	6.05	6.39	7.06	7.39	12.05	13.71
16	72	8.45	8.96	9.81	10.15	12.65	14.38	6.35	6.72	7.44	7.81	12.05	13.71

Note

The values may be used for a connection with a headside member at least as thick as the headside thickness shown, and a screw pointside penetration equal to at least twice this thickness. The timber must be predrilled.

Typical coach screw sizes

6mm	25mm - 120mm
8mm	30mm - 150mm
10mm	30mm - 200mm
12mm	50mm - 180mm
1/4"	1½" - 5"
5/16"	1½" - 6"
3/8"	1½" - 6"
1/2"	2½" - 6"
5/8"	3½" - 10"
3/4"	6"

16	97	8.76	9.12	9.81	10.15	12.65	14.38	7.30	7.61	8.20	8.49	12.05	13.71
16	97	8.76	9.12	9.81	10.15	12.65	14.38	7.30	7.61	8.20	8.49	12.05	13.71
16	97	8.76	9.12	9.81	10.15	12.65	14.38	7.30	7.61	8.20	8.49	12.05	13.71
20	63	8.56	9.04	10.00	10.48	14.49	17.64	6.16	6.48	7.13	7.47	12.70	15.47
20	63	8.56	9.04	10.00	10.48	14.49	17.64	6.16	6.48	7.13	7.47	12.70	15.47
20	72	10.71	11.47	13.03	13.84	18.77	21.37	8.02	8.59	9.80	10.43	17.06	20.02
20	97	12.96	13.49	14.52	15.02	18.77	21.37	9.61	10.17	11.30	11.86	17.56	20.02
20	97	12.96	13.49	14.52	15.02	18.77	21.37	9.61	10.17	11.30	11.86	17.56	20.02
20	97	12.96	13.49	14.52	15.02	18.77	21.37	9.61	10.17	11.30	11.86	17.56	20.02

Lateral design loads for one steel coach screw in a 2-member steel-timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Note
The values may be used for a connection with a screw at least as long as the length shown. The thickness of the timber must be sufficient to accommodate its full length. The timber must be predrilled and the steel thickness should not exceed 0.5d.

Typical coach screw sizes

6mm	25mm - 120mm
8mm	30mm - 150mm
10mm	30mm - 200mm
12mm	50mm - 180mm
1/4"	1½" - 5"
5/16"	1½" - 6"
3/8"	1½" - 6"
1/2"	2½" - 6"
5/8"	3½" - 10"
3/4"	6"

Steel thickness	Screw dimensions		Direction of loading											
			Parallel to the grain						Perpendicular to the grain					
			Strength classes											
	Diameter	Length	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
Default values			(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
2.5	8	55	2.54	2.68	2.88	2.98	3.75	4.34	1.73	1.85	2.09	2.21	3.72	4.30
2.5	8	60	2.75	2.88	3.14	3.27	4.36	5.20	1.90	2.03	2.29	2.42	4.32	5.16
2.5	8	65	2.96	3.12	3.44	3.61	4.56	5.20	2.06	2.20	2.49	2.63	4.53	5.16
2.5	8	70	3.06	3.24	3.51	3.63	4.56	5.20	2.23	2.38	2.69	2.84	4.53	5.16
2.5	8	75	3.11	3.26	3.51	3.63	4.56	5.20	2.39	2.56	2.88	3.05	4.53	5.16
2.5	8	80	3.13	3.26	3.51	3.63	4.56	5.20	2.56	2.73	3.04	3.15	4.53	5.16
2.5	8	90	3.13	3.26	3.51	3.63	4.56	5.20	2.70	2.82	3.04	3.15	4.53	5.16
3	10	63	3.56	3.72	3.96	4.07	4.87	5.39	2.37	2.53	2.86	3.02	4.75	5.26
3	10	65	3.67	3.80	4.06	4.18	5.11	5.75	2.45	2.62	2.96	3.13	4.99	5.62
3	10	70	3.87	4.03	4.35	4.51	5.78	6.76	2.65	2.83	3.19	3.38	5.66	6.63
3	10	75	4.10	4.30	4.69	4.89	6.55	7.61	2.84	3.04	3.43	3.63	6.43	7.48
3	10	80	4.36	4.59	5.07	5.31	6.68	7.61	3.04	3.25	3.67	3.88	6.56	7.48
3	10	90	4.59	4.78	5.15	5.33	6.68	7.61	3.44	3.67	4.15	4.39	6.56	7.48
3	10	100	4.59	4.78	5.15	5.33	6.68	7.61	3.83	4.10	4.42	4.58	6.56	7.48
3	10	110	4.59	4.78	5.15	5.33	6.68	7.61	3.93	4.10	4.42	4.58	6.56	7.48
3.6	12	80	5.14	5.33	5.72	5.90	7.34	8.38	3.47	3.71	4.19	4.43	7.08	8.10
3.6	12	90	5.67	5.94	6.49	6.76	9.10	10.60	3.93	4.20	4.74	5.01	8.85	10.32
3.6	12	100	6.29	6.63	7.15	7.40	9.29	10.60	4.38	4.68	5.29	5.59	9.04	10.32
3.6	12	110	6.37	6.63	7.15	7.40	9.29	10.60	4.84	5.17	5.84	6.17	9.04	10.32

CD2.3
Note this is a snapshot of an interactive spreadsheet

3.6	12	120	6.37	6.63	7.15	7.40	9.29	10.60	5.29	5.66	6.11	6.33	9.04	10.32
3.6	12	130	6.37	6.63	7.15	7.40	9.29	10.60	5.42	5.66	6.11	6.33	9.04	10.32
3.6	12	140	6.37	6.63	7.15	7.40	9.29	10.60	5.42	5.66	6.11	6.33	9.04	10.32
3.6	12	150	6.37	6.63	7.15	7.40	9.29	10.60	5.42	5.66	6.11	6.33	9.04	10.32
4.8	16	110	8.67	9.02	9.73	10.07	12.84	14.97	5.86	6.26	7.07	7.47	12.14	14.20
4.8	16	120	9.39	9.85	10.78	11.25	15.26	17.82	6.41	6.85	7.74	8.18	14.56	17.05
4.8	16	130	10.23	10.81	11.96	12.39	15.59	17.82	6.97	7.45	8.41	8.89	14.89	17.05
4.8	16	140	10.62	11.09	11.96	12.39	15.59	17.82	7.53	8.04	9.08	9.60	14.89	17.05
4.8	16	150	10.64	11.09	11.96	12.39	15.59	17.82	8.08	8.64	9.75	10.31	14.89	17.05
4.8	16	160	10.64	11.09	11.96	12.39	15.59	17.82	8.64	9.23	10.12	10.49	14.89	17.05
4.8	16	170	10.64	11.09	11.96	12.39	15.59	17.82	8.96	9.36	10.12	10.49	14.89	17.05
6	20	130	12.11	12.55	13.41	13.82	16.94	19.15	7.92	8.46	9.56	10.10	15.55	17.60
6	20	140	12.91	13.47	14.58	15.13	19.63	23.18	8.56	9.15	10.33	10.92	18.23	21.64
6	20	150	13.83	14.52	15.92	16.63	22.70	26.57	9.20	9.83	11.10	11.73	21.31	25.03
6	20	160	14.87	15.70	17.43	18.31	23.22	26.57	9.83	10.51	11.87	12.55	21.82	25.03
6	20	170	15.64	16.45	17.76	18.40	23.22	26.57	10.47	11.19	12.64	13.36	21.82	25.03
6	20	180	15.78	16.45	17.76	18.40	23.22	26.57	11.11	11.88	13.41	14.18	21.82	25.03
6	20	190	15.78	16.45	17.76	18.40	23.22	26.57	11.75	12.56	14.18	14.99	21.82	25.03
6	20	200	15.78	16.45	17.76	18.40	23.22	26.57	12.39	13.24	14.89	15.45	21.82	25.03

CD2.4 BOLTED CONNECTIONS

In accordance with BS EN 1995-1-1:2004 and accompanying CD2.1 *Calculation basis*

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Lateral design loads for one 4.6 grade steel bolt in a 2-member timber joint

Service Class	1	k_{mod} = 0.90
Load duration	Short term	γ_M = 1.30
Design situation	Normal	

Design values | Characteristic values | Reset table | Return to menu

Bolt diameter	Minimum member thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	16	1.00	1.07	1.21	1.28	1.83	2.25	0.68	0.73	0.82	0.87	1.80	2.21
8	22	1.38	1.48	1.67	1.76	2.52	3.09	0.94	1.00	1.13	1.20	2.47	3.03
8	35	2.18	2.33	2.63	2.78	4.01	4.56	1.49	1.60	1.80	1.91	3.93	4.51
8	44	2.63	2.81	3.18	3.29	4.12	4.56	1.88	2.01	2.27	2.40	4.08	4.51
8	47	2.78	2.97	3.19	3.29	4.12	4.56	2.01	2.14	2.42	2.56	4.08	4.51
8	60	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	72	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	97	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	147	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
10	16	1.23	1.31	1.48	1.57	2.24	2.75	0.82	0.87	0.99	1.04	2.14	2.62
10	22	1.69	1.80	2.04	2.15	3.08	3.78	1.13	1.20	1.36	1.44	2.94	3.60
10	35	2.69	2.87	3.24	3.43	4.91	6.02	1.79	1.91	2.16	2.28	4.67	5.73
10	44	3.36	3.59	4.06	4.29	6.08	6.74	2.25	2.41	2.72	2.87	5.88	6.58
10	47	3.55	3.79	4.28	4.53	6.08	6.74	2.40	2.57	2.90	3.07	5.94	6.58
10	60	4.26	4.43	4.75	4.91	6.08	6.74	3.07	3.28	3.70	3.92	5.94	6.58

CD2.4
Note this is a snapshot of an interactive spreadsheet

10	72	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
10	97	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
10	147	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
12	16	1.44	1.54	1.74	1.84	2.63	3.23	0.94	1.01	1.14	1.20	2.44	2.99
12	22	1.98	2.12	2.39	2.53	3.62	4.44	1.29	1.38	1.56	1.65	3.35	4.11
12	35	3.15	3.37	3.80	4.02	5.76	7.06	2.06	2.20	2.49	2.63	5.33	6.54
12	44	3.96	4.23	4.78	5.05	7.24	8.88	2.59	2.77	3.12	3.30	6.70	8.22
12	47	4.23	4.52	5.11	5.40	7.73	9.25	2.77	2.96	3.34	3.53	7.16	8.78
12	60	5.27	5.64	6.37	6.73	8.35	9.25	3.53	3.77	4.26	4.50	8.04	8.90
12	72	5.90	6.13	6.58	6.80	8.35	9.25	4.24	4.53	5.11	5.40	8.04	8.90
12	97	5.90	6.13	6.58	6.80	8.35	9.25	4.95	5.15	5.49	5.64	8.04	8.90
12	147	5.90	6.13	6.58	6.80	8.35	9.25	4.95	5.15	5.49	5.64	8.04	8.90
16	16	1.83	1.96	2.21	2.34	3.35	4.11	1.15	1.23	1.39	1.47	2.94	3.60
16	22	2.52	2.69	3.04	3.22	4.61	5.65	1.59	1.69	1.91	2.02	4.04	4.96
16	35	4.01	4.29	4.84	5.12	7.33	8.99	2.52	2.70	3.04	3.22	6.43	7.88
16	44	5.04	5.39	6.08	6.43	9.21	11.30	3.17	3.39	3.83	4.04	8.08	9.91
16	47	5.38	5.76	6.50	6.87	9.84	12.07	3.39	3.62	4.09	4.32	8.63	10.59
16	60	6.87	7.35	8.30	8.77	12.56	15.17	4.32	4.62	5.22	5.52	11.02	13.51
16	72	8.25	8.82	9.96	10.52	13.70	15.17	5.19	5.55	6.26	6.62	12.83	14.21
16	97	9.80	10.19	10.95	11.32	13.70	15.17	6.99	7.47	8.44	8.92	12.83	14.21
16	147	9.80	10.19	10.95	11.32	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
20	16	2.18	2.33	2.63	2.78	3.99	4.89	1.32	1.41	1.60	1.69	3.32	4.08
20	22	3.00	3.21	3.62	3.83	5.48	6.73	1.82	1.94	2.19	2.32	4.57	5.60
20	35	4.77	5.10	5.76	6.09	8.72	10.70	2.89	3.09	3.49	3.69	7.27	8.92
20	44	6.00	6.41	7.24	7.66	10.97	13.45	3.64	3.89	4.39	4.64	9.14	11.21
20	47	6.41	6.85	7.74	8.18	11.71	14.37	3.88	4.15	4.69	4.96	9.76	11.97
20	60	8.18	8.75	9.88	10.44	14.96	18.34	4.96	5.30	5.99	6.33	12.46	15.28
20	72	9.82	10.50	11.85	12.53	17.95	22.01	5.95	6.36	7.18	7.59	14.96	18.34
20	97	13.23	14.14	15.97	16.69	19.97	22.12	8.02	8.57	9.68	10.23	18.23	20.19
20	147	14.47	15.05	16.18	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19
24	16	2.49	2.66	3.00	3.17	4.55	5.58	1.45	1.56	1.76	1.86	3.61	4.43
24	22	3.42	3.66	4.13	4.36	6.25	7.67	2.00	2.14	2.41	2.55	4.96	6.08
24	35	5.44	5.82	6.57	6.94	9.95	12.20	3.18	3.40	3.84	4.06	7.89	9.68
24	44	6.84	7.31	8.26	8.73	12.50	15.33	4.00	4.28	4.83	5.10	9.92	12.17
24	47	7.31	7.81	8.82	9.32	13.36	16.38	4.27	4.57	5.16	5.45	10.60	13.00
24	60	9.33	9.97	11.26	11.90	17.05	20.91	5.46	5.83	6.58	6.96	13.53	16.59

24	72	11.19	11.97	13.51	14.28	20.46	25.09	6.55	7.00	7.90	8.35	16.24	19.91
24	97	15.08	16.12	18.20	19.24	27.03	29.93	8.82	9.43	10.64	11.25	21.88	26.67
24	147	19.81	20.62	21.97	22.58	27.03	29.93	13.37	14.29	16.13	17.05	24.08	26.67

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Bolt diameter	Minimum member thickness		Direction of loading											
			Parallel to the grain						Perpendicular to the grain					
			Strength classes											
	Outer members	Inner member	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	16	32	1.94	2.06	2.20	2.27	2.81	3.20	1.32	1.41	1.59	1.68	2.78	3.16
8	22	44	2.09	2.18	2.35	2.44	3.12	3.62	1.66	1.73	1.85	1.92	3.08	3.57
8	35	70	2.42	2.55	2.82	2.95	4.05	4.56	1.88	1.97	2.15	2.24	3.99	4.51
8	44	88	2.71	2.87	3.19	3.29	4.12	4.56	2.10	2.21	2.43	2.54	4.08	4.51
8	47	94	2.81	2.97	3.19	3.29	4.12	4.56	2.17	2.29	2.53	2.65	4.08	4.51
8	60	120	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	72	144	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	97	194	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
8	147	294	2.86	2.97	3.19	3.29	4.12	4.56	2.43	2.53	2.72	2.81	4.08	4.51
10	16	32	2.37	2.53	2.86	3.02	4.02	4.52	1.58	1.69	1.91	2.02	3.91	4.39
10	22	44	2.98	3.09	3.32	3.43	4.30	4.93	2.17	2.32	2.62	2.72	4.17	4.77
10	35	70	3.36	3.54	3.86	4.02	5.31	6.26	2.57	2.68	2.91	3.02	5.10	6.01
10	44	88	3.67	3.88	4.29	4.50	6.08	6.74	2.79	2.92	3.19	3.33	5.91	6.58
10	47	94	3.79	4.01	4.44	4.65	6.08	6.74	2.87	3.01	3.30	3.44	5.94	6.58
10	60	120	4.26	4.43	4.75	4.91	6.08	6.74	3.27	3.45	3.82	4.00	5.94	6.58

10	72	144	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
10	97	194	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
10	147	294	4.26	4.43	4.75	4.91	6.08	6.74	3.60	3.75	4.03	4.15	5.94	6.58
12	16	32	2.78	2.97	3.36	3.55	5.08	6.07	1.82	1.94	2.19	2.32	4.71	5.77
12	22	44	3.82	4.09	4.45	4.60	5.67	6.44	2.50	2.67	3.02	3.19	5.41	6.13
12	35	70	4.38	4.58	4.97	5.16	6.69	7.82	3.36	3.49	3.76	3.90	6.32	7.36
12	44	88	4.78	5.02	5.49	5.73	7.61	9.01	3.56	3.72	4.04	4.20	7.15	8.45
12	47	94	4.90	5.17	5.69	5.94	7.94	9.25	3.64	3.81	4.15	4.32	7.45	8.84
12	60	120	5.48	5.80	6.44	6.76	8.35	9.25	4.05	4.26	4.68	4.89	8.04	8.90
12	72	144	5.90	6.13	6.58	6.80	8.35	9.25	4.49	4.74	5.24	5.49	8.04	8.90
12	97	194	5.90	6.13	6.58	6.80	8.35	9.25	4.95	5.15	5.49	5.64	8.04	8.90
12	147	294	5.90	6.13	6.58	6.80	8.35	9.25	4.95	5.15	5.49	5.64	8.04	8.90
16	16	32	3.54	3.78	4.27	4.52	6.47	7.93	2.23	2.38	2.69	2.84	5.68	6.96
16	22	44	4.87	5.20	5.87	6.21	8.90	10.03	3.06	3.27	3.69	3.91	7.80	9.31
16	35	70	6.76	7.03	7.56	7.82	9.86	11.34	4.87	5.21	5.78	5.96	9.04	10.36
16	44	88	7.13	7.44	8.07	8.38	10.83	12.64	5.36	5.57	5.99	6.20	9.84	11.44
16	47	94	7.28	7.61	8.27	8.60	11.20	13.12	5.42	5.65	6.09	6.30	10.15	11.85
16	60	120	8.05	8.46	9.28	9.69	12.94	15.17	5.79	6.06	6.59	6.86	11.62	13.76
16	72	144	8.80	9.31	10.32	10.83	13.70	15.17	6.23	6.55	7.17	7.48	12.83	14.21
16	97	194	9.80	10.19	10.95	11.32	13.70	15.17	7.34	7.75	8.57	8.98	12.83	14.21
16	147	294	9.80	10.19	10.95	11.32	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
20	16	32	4.21	4.51	5.09	5.38	7.70	9.45	2.55	2.73	3.08	3.26	6.42	7.87
20	22	44	5.80	6.19	6.99	7.39	10.59	12.99	3.51	3.75	4.24	4.48	8.83	10.82
20	35	70	9.22	9.86	10.66	11.00	13.58	15.43	5.59	5.97	6.74	7.13	12.18	13.76
20	44	88	9.90	10.30	11.09	11.48	14.51	16.72	7.02	7.51	8.28	8.54	12.85	14.72
20	47	94	10.03	10.45	11.28	11.69	14.87	17.21	7.50	7.80	8.35	8.62	13.13	15.10
20	60	120	10.76	11.27	12.27	12.77	16.70	19.61	7.80	8.12	8.76	9.08	14.54	16.98
20	72	144	11.62	12.21	13.39	13.97	18.62	22.09	8.19	8.56	9.30	9.66	16.06	18.96
20	97	194	13.64	14.45	16.06	16.69	19.97	22.12	9.26	9.74	10.70	11.17	18.23	20.19
20	147	294	14.47	15.05	16.18	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19
24	16	32	4.80	5.14	5.80	6.13	8.78	10.77	2.81	3.00	3.39	3.58	6.97	8.55
24	22	44	6.61	7.06	7.97	8.43	12.07	14.81	3.86	4.13	4.66	4.93	9.58	11.75
24	35	70	10.51	11.23	12.68	13.41	17.84	20.07	6.15	6.57	7.42	7.84	15.24	17.57
24	44	88	13.08	13.58	14.54	15.01	18.63	21.24	7.73	8.26	9.33	9.86	16.18	18.32
24	47	94	13.18	13.69	14.69	15.18	18.97	21.72	8.25	8.82	9.96	10.53	16.40	18.64
24	63	126	14.00	14.62	15.84	16.45	21.22	24.75	10.11	10.51	11.28	11.67	17.97	20.81

24	72	144	14.63	15.31	16.68	17.36	22.73	26.71	10.35	10.78	11.63	12.05	19.07	22.26
24	97	194	16.78	17.68	19.47	20.36	27.03	29.93	11.31	11.85	12.92	13.46	22.61	26.67
24	147	294	19.81	20.62	21.97	22.58	27.03	29.93	14.01	14.80	16.37	17.15	24.08	26.67

Lateral design loads for one 4.6 grade steel bolt in a 2-member softwood glulam joint

Service Class	1
Load duration	Short term
Design situation	Normal
k_{mod} =	0.90
γ_M =	1.30

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Bolt diameter	Minimum member thickness	Direction of loading															
		Parallel to the grain								Perpendicular to the grain							
		Strength classes															
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	65	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	90	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	115	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	140	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	165	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	190	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	215	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
10	65	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.01	4.37	4.21	4.47	4.37	4.58	4.47
10	90	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	115	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	140	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	165	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	190	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	215	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
12	65	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.01	4.62	5.41	5.01	5.67	5.41	5.93	5.67
12	90	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08

CD2.4
Note this is a snapshot of an interactive spreadsheet

12	115	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	140	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	165	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	190	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	215	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
16	65	9.76	8.99	10.53	9.76	11.04	10.53	11.56	11.04	6.14	5.65	6.62	6.14	6.94	6.62	7.27	6.94
16	90	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	8.50	7.83	9.17	8.50	9.62	9.17	10.01	9.62
16	115	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	140	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	165	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	190	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	215	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
20	65	11.62	10.70	12.53	11.62	13.14	12.53	13.76	13.14	7.04	6.48	7.60	7.04	7.97	7.60	8.34	7.97
20	90	16.08	14.81	17.35	16.08	17.99	17.35	18.40	17.99	9.75	8.98	10.52	9.75	11.03	10.52	11.54	11.03
20	115	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	12.46	11.47	13.44	12.46	14.01	13.44	14.33	14.01
20	140	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	165	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	190	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	215	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
24	65	13.24	12.20	14.29	13.24	14.98	14.29	15.68	14.98	7.74	7.13	8.36	7.74	8.76	8.36	9.17	8.76
24	90	18.34	16.89	19.78	18.34	20.75	19.78	21.71	20.75	10.72	9.88	11.57	10.72	12.13	11.57	12.70	12.13
24	115	22.89	21.58	23.77	22.89	24.35	23.77	24.91	24.35	13.70	12.62	14.78	13.70	15.50	14.78	16.23	15.50
24	140	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	16.68	15.36	18.00	16.68	18.62	18.00	19.05	18.62
24	165	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
24	190	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
24	215	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member softwood glulam joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Bolt diameter	Minimum member thickness: Outer members	Minimum member thickness: Inner member	Direction of loading: Parallel to the grain — Strength classes								Perpendicular to the grain							
			GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	65	130	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	90	180	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	115	230	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	140	280	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	165	330	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	190	380	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
8	215	430	3.34	3.19	3.50	3.34	3.59	3.50	3.69	3.59	2.86	2.72	2.99	2.86	3.06	2.99	3.13	3.06
10	65	130	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.02	4.37	4.21	4.47	4.37	4.58	4.47
10	90	180	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	115	230	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	140	280	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	165	330	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	190	380	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
10	215	430	4.99	4.75	5.22	4.99	5.37	5.22	5.51	5.37	4.21	4.03	4.37	4.21	4.47	4.37	4.58	4.47
12	65	130	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.25	4.91	5.58	5.25	5.81	5.58	6.03	5.81
12	90	180	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08

12	115	230	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	140	280	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	165	330	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	190	380	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
12	215	430	6.91	6.58	7.23	6.91	7.43	7.23	7.64	7.43	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
16	65	130	10.37	9.71	11.03	10.37	11.47	11.03	11.91	11.47	7.25	6.82	7.67	7.25	7.95	7.67	8.23	7.95
16	90	180	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	8.73	8.16	9.31	8.73	9.69	9.31	10.01	9.69
16	115	230	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	140	280	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	165	330	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	190	380	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
16	215	430	11.50	10.95	12.03	11.50	12.34	12.03	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
20	65	130	13.52	12.72	14.31	13.52	14.84	14.31	15.37	14.84	9.48	8.97	9.97	9.48	10.31	9.97	10.64	10.31
20	90	180	16.37	15.29	17.45	16.37	17.99	17.45	18.40	17.99	10.94	10.27	11.60	10.94	12.05	11.60	12.49	12.05
20	115	230	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	12.70	11.86	13.54	12.70	14.01	13.54	14.33	14.01
20	140	280	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	165	330	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	190	380	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
20	215	430	16.91	16.18	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
24	65	130	16.95	16.02	17.88	16.95	18.49	17.88	19.11	18.49	11.94	11.35	12.51	11.94	12.89	12.51	13.27	12.89
24	90	180	19.88	18.63	21.12	19.88	21.95	21.12	22.78	21.95	13.27	12.52	14.01	13.27	14.51	14.01	15.00	14.51
24	115	230	22.89	21.74	23.77	22.89	24.35	23.77	24.91	24.35	15.00	14.07	15.93	15.00	16.55	15.93	17.17	16.55
24	140	280	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	16.97	15.84	18.09	16.97	18.62	18.09	19.05	18.62
24	165	330	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
24	190	380	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
24	215	430	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62

Lateral design loads for one 4.6 grade steel bolt in a 2-member LVL joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Bolt diameter	Minimum member thickness	Direction of loading: Parallel to the grain, LVL, KERTO Q	Direction of loading: Parallel to the grain, LVL, KERTO S	Direction of loading: Perp. to the grain, LVL, KERTO Q	Direction of loading: Perp. to the grain, LVL, KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	2.16	2.16	1.47	1.47
8	33	2.64	2.64	1.80	1.80
8	39	3.04	3.04	2.12	2.12
8	45	3.29	3.29	2.45	2.45
8	51	3.29	3.29	2.76	2.76
8	57	3.29	3.29	2.81	2.81
8	63	3.29	3.29	2.81	2.81
8	75	3.29	3.29	2.81	2.81
10	27	2.64	2.64	1.76	1.76
10	33	3.23	3.23	2.15	2.15
10	39	3.82	3.82	2.54	2.54
10	45	4.37	4.37	2.94	2.94
10	51	4.84	4.84	3.33	3.33
10	57	4.91	4.91	3.72	3.72
10	63	4.91	4.91	4.11	4.11
10	75	4.91	4.91	4.15	4.15
12	27	3.10	3.10	2.03	2.03
12	33	3.79	3.79	2.48	2.48

12	39	4.48	4.48	2.93	2.93
12	45	5.17	5.17	3.38	3.38
12	51	5.86	5.86	3.83	3.83
12	57	6.45	6.45	4.28	4.28
12	63	6.80	6.80	4.73	4.73
12	75	6.80	6.80	5.63	5.63
16	27	3.95	3.95	2.48	2.48
16	33	4.82	4.82	3.03	3.03
16	39	5.70	5.70	3.59	3.59
16	45	6.58	6.58	4.14	4.14
16	51	7.45	7.45	4.69	4.69
16	57	8.33	8.33	5.24	5.24
16	63	9.21	9.21	5.79	5.79
16	75	10.93	10.93	6.89	6.89
20	27	4.70	4.70	2.85	2.85
20	33	5.74	5.74	3.48	3.48
20	39	6.79	6.79	4.11	4.11
20	45	7.83	7.83	4.75	4.75
20	51	8.87	8.87	5.38	5.38
20	57	9.92	9.92	6.01	6.01
20	63	10.96	10.96	6.64	6.64
20	75	13.05	13.05	7.91	7.91
24	27	5.36	5.36	3.13	3.13
24	33	6.55	6.55	3.83	3.83
24	39	7.74	7.74	4.52	4.52
24	45	8.93	8.93	5.22	5.22
24	51	10.12	10.12	5.92	5.92
24	57	11.31	11.31	6.61	6.61
24	63	12.50	12.50	7.31	7.31
24	75	14.88	14.88	8.70	8.70

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member LVL joint

Service Class	1	k_{mod} =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Bolt diameter	Minimum member thickness: Outer members	Minimum member thickness: Inner member	Direction of loading: Parallel to the grain, LVL: KERTO Q	Parallel to the grain, LVL: KERTO S	Perp. to the grain, LVL: KERTO Q	Perp. to the grain, LVL: KERTO S
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	54	2.64	2.64	2.02	2.02
8	33	66	2.87	2.87	2.18	2.18
8	39	78	3.12	3.12	2.37	2.37
8	45	90	3.29	3.29	2.58	2.58
8	51	102	3.29	3.29	2.78	2.78
8	57	114	3.29	3.29	2.81	2.81
8	63	126	3.29	3.29	2.81	2.81
8	75	150	3.29	3.29	2.81	2.81
10	27	54	3.62	3.62	2.81	2.81
10	33	66	3.92	3.92	2.96	2.96
10	39	78	4.25	4.25	3.15	3.15
10	45	90	4.55	4.55	3.37	3.37
10	51	102	4.87	4.87	3.61	3.61
10	57	114	4.91	4.91	3.86	3.86
10	63	126	4.91	4.91	4.13	4.13

CD2.4
Note this is a snapshot of an interactive spreadsheet

10	75	150	4.91	4.91	4.15	4.15
12	27	54	4.76	4.76	3.71	3.71
12	33	66	5.05	5.05	3.84	3.84
12	39	78	5.40	5.40	4.02	4.02
12	45	90	5.80	5.80	4.24	4.24
12	51	102	6.20	6.20	4.48	4.48
12	57	114	6.57	6.57	4.75	4.75
12	63	126	6.80	6.80	5.04	5.04
12	75	150	6.80	6.80	5.64	5.64
16	27	54	7.49	7.49	4.79	4.79
16	33	66	7.72	7.72	5.86	5.86
16	39	78	8.05	8.05	6.05	6.05
16	45	90	8.45	8.45	6.23	6.23
16	51	102	8.92	8.92	6.46	6.46
16	57	114	9.42	9.42	6.72	6.72
16	63	126	9.97	9.97	7.00	7.00
16	75	150	11.07	11.07	7.65	7.65
20	27	54	9.07	9.07	5.50	5.50
20	33	66	10.93	10.93	6.72	6.72
20	39	78	11.19	11.19	7.94	7.94
20	45	90	11.55	11.55	8.56	8.56
20	51	102	11.99	11.99	8.74	8.74
20	57	114	12.49	12.49	8.96	8.96
20	63	126	13.05	13.05	9.21	9.21
20	75	150	14.29	14.29	9.82	9.82
24	27	54	10.34	10.34	6.05	6.05
24	33	66	12.64	12.64	7.39	7.39
24	39	78	14.78	14.78	8.74	8.74
24	45	90	15.06	15.06	10.08	10.08
24	51	102	15.45	15.45	11.30	11.30
24	57	114	15.91	15.91	11.46	11.46
24	63	126	16.45	16.45	11.67	11.67
24	75	150	17.69	17.69	12.19	12.19

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member plywood-timber-plywood joint

Plywood group	Group 1		
Service Class	1	$k_{\text{mod,combined}}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Minimum thickness		Direction of loading							
		Parallel to the grain				Perpendicular to the grain			
		Bolt diameter (mm)							
Plywood	Timber	10	12	16	20	10	12	16	20
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
Strength class		**C14**							
6	21	1.56	1.83	2.32	2.77	1.04	1.19	1.46	1.68
9	31	2.30	2.69	3.43	4.08	1.53	1.76	2.16	2.47
12	43	3.19	3.74	4.76	5.66	2.12	2.44	2.99	3.43
15	52	3.58	4.52	5.75	6.85	2.57	2.95	3.62	4.15
18	62	3.73	4.99	6.86	8.17	3.06	3.52	4.31	4.95
21	73	3.90	5.15	8.08	9.61	3.52	4.15	5.08	5.83
29	101	4.46	5.74	8.74	12.11	4.03	5.16	7.03	8.06
Strength classes		**C16, C18, C20 & C22**							
6	35	2.17	2.54	3.24	3.85	1.85	2.13	2.60	2.99
9	29	2.30	2.69	3.43	4.08	1.53	1.76	2.16	2.47
12	39	3.09	3.62	4.61	5.49	2.06	2.37	2.90	3.33
15	48	3.69	4.46	5.68	6.76	2.53	2.92	3.57	4.10
18	58	3.83	5.13	6.86	8.17	3.06	3.52	4.31	4.95

21	68	4.01	5.30	8.04	9.57	3.59	4.13	5.06	5.80
29	94	4.57	5.89	8.98	12.36	4.14	5.32	6.99	8.02
Strength class			**C24**						
6	18	1.61	1.89	2.40	2.86	1.07	1.23	1.51	1.73
9	25	2.24	2.62	3.34	3.97	1.49	1.71	2.10	2.41
12	34	3.04	3.57	4.54	5.40	2.03	2.33	2.86	3.28
15	43	3.84	4.51	5.74	6.84	2.56	2.95	3.61	4.14
18	51	4.03	5.33	6.81	8.11	3.04	3.50	4.28	4.91
21	60	4.21	5.55	8.01	9.54	3.58	4.11	5.04	5.78
29	72	4.79	6.19	9.30	11.44	4.29	4.94	6.05	6.94
Strength classes			**C27, C30, C35 & C40**						
6	17	1.61	1.89	2.40	2.86	1.07	1.23	1.51	1.73
9	24	2.27	2.66	3.39	4.03	1.51	1.74	2.13	2.44
12	32	3.02	3.55	4.52	5.38	2.02	2.32	2.84	3.26
15	40	3.78	4.44	5.65	6.72	2.52	2.90	3.55	4.07
18	49	4.11	5.42	6.92	8.23	3.09	3.55	4.35	4.99
21	57	4.31	5.63	8.05	9.58	3.59	4.13	5.06	5.80
29	79	4.89	6.33	9.44	13.01	4.47	5.67	7.01	8.05
Strength classes			**D30, D35 & D40**						
6	11	1.49	1.75	2.22	2.65	1.42	1.62	1.95	2.21
9	17	2.30	2.70	3.44	4.09	2.19	2.50	3.02	3.41
12	22	2.98	3.49	4.45	5.30	2.84	3.24	3.90	4.41
15	28	3.79	4.45	5.66	6.74	3.61	4.12	4.97	5.62
18	34	4.50	5.40	6.87	8.18	4.38	5.00	6.03	6.82
21	40	4.73	6.18	8.09	9.63	4.68	5.88	7.09	8.02
29	55	5.50	6.96	10.34	13.24	5.44	6.84	9.75	11.03
Strength classes			**D50, D60 & D70**						
6	9	1.49	1.75	2.23	2.66	1.42	1.62	1.96	2.21
9	13	2.16	2.53	3.22	3.84	2.06	2.35	2.83	3.20
12	18	2.99	3.51	4.46	5.31	2.85	3.25	3.92	4.43
15	23	3.82	4.48	5.70	6.79	3.64	4.15	5.00	5.66
18	27	4.48	5.26	6.70	7.97	4.27	4.87	5.87	6.64
21	32	4.95	6.23	7.93	9.45	4.90	5.77	6.96	7.87
29	45	5.73	7.27	10.82	13.28	5.68	7.15	9.79	11.07

Lateral design loads per shear plane for
one 4.6 grade steel bolt in a 3-member timber-steel-timber joint

Service Class	1	$k_{\text{mod,combined}}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Bolt diameter	Minimum timber thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	22	3.26	3.48	3.93	4.16	5.47	6.21	2.17	2.32	2.62	2.77	5.31	6.02
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	35	4.04	4.24	4.64	4.84	6.50	7.63	3.28	3.41	3.68	3.81	6.26	7.34
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	38	4.14	4.35	4.77	4.98	6.81	8.02	3.33	3.48	3.76	3.90	6.55	7.71
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	47	4.50	4.74	5.24	5.48	7.80	9.30	3.51	3.70	4.06	4.24	7.49	8.91
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	63	5.25	5.56	6.19	6.50	8.60	9.53	3.95	4.18	4.63	4.85	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	75	5.75	5.97	6.39	6.60	8.60	9.53	4.34	4.60	5.11	5.37	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	100	5.75	5.97	6.39	6.60	8.60	9.53	4.82	5.01	5.37	5.54	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	150	5.75	5.97	6.39	6.60	8.60	9.53	4.82	5.01	5.37	5.54	8.40	9.30

CD2.4
Note this is a snapshot of an interactive spreadsheet

Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	200	5.75	5.97	6.39	6.60	8.60	9.53	4.82	5.01	5.37	5.54	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	250	5.75	5.97	6.39	6.60	8.60	9.53	4.82	5.01	5.37	5.54	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	300	5.75	5.97	6.39	6.60	8.60	9.53	4.82	5.01	5.37	5.54	8.40	9.30
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
12	22	3.82	4.09	4.62	4.88	6.99	8.23	2.50	2.67	3.02	3.19	6.47	7.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	25	4.35	4.65	5.25	5.55	7.47	8.46	2.84	3.04	3.43	3.62	7.14	8.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	38	5.49	5.76	6.28	6.54	8.59	10.03	4.32	4.57	4.91	5.07	8.12	9.46
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	47	5.84	6.14	6.75	7.06	9.66	11.42	4.59	4.79	5.19	5.38	9.08	10.72
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	63	6.64	7.02	7.78	8.16	11.81	13.08	5.05	5.32	5.87	6.14	11.05	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	75	7.33	7.77	8.65	9.09	11.81	13.08	5.45	5.76	6.38	6.69	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	100	7.94	8.25	8.83	9.11	11.81	13.08	6.39	6.79	7.36	7.60	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	150	7.94	8.25	8.83	9.11	11.81	13.08	6.60	6.86	7.36	7.60	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	200	7.94	8.25	8.83	9.11	11.81	13.08	6.60	6.86	7.36	7.60	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	250	7.94	8.25	8.83	9.11	11.81	13.08	6.60	6.86	7.36	7.60	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	300	7.94	8.25	8.83	9.11	11.81	13.08	6.60	6.86	7.36	7.60	11.37	12.59
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
16	22	4.87	5.20	5.87	6.21	8.90	10.91	3.06	3.27	3.69	3.91	7.80	9.57
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	25	5.53	5.91	6.68	7.06	10.11	12.40	3.48	3.72	4.20	4.44	8.87	10.88
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	38	8.41	8.99	9.88	10.21	12.75	14.62	5.29	5.65	6.38	6.75	11.73	13.38
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	47	9.06	9.49	10.35	10.77	13.83	16.09	6.54	6.99	7.81	8.07	12.59	14.59
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	63	9.85	10.37	11.41	11.93	16.24	19.24	7.42	7.75	8.40	8.72	14.60	17.24
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8

16	75	10.60	11.19	12.39	12.98	18.29	21.45	7.88	8.27	9.03	9.41	16.35	19.48
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	100	12.41	13.16	14.64	15.11	19.37	21.45	8.99	9.51	10.54	11.06	18.14	20.09
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	150	13.16	13.66	14.64	15.11	19.37	21.45	10.79	11.21	12.03	12.43	18.14	20.09
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	200	13.16	13.66	14.64	15.11	19.37	21.45	10.79	11.21	12.03	12.43	18.14	20.09
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	250	13.16	13.66	14.64	15.11	19.37	21.45	10.79	11.21	12.03	12.43	18.14	20.09
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	300	13.16	13.66	14.64	15.11	19.37	21.45	10.79	11.21	12.03	12.43	18.14	20.09
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	22	5.80	6.19	6.99	7.39	10.59	12.99	3.51	3.75	4.24	4.48	8.83	10.82
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	25	6.59	7.04	7.95	8.40	12.04	14.76	3.99	4.27	4.82	5.09	10.03	12.30
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	38	10.01	10.70	12.08	12.77	17.74	20.05	6.07	6.48	7.32	7.74	15.24	17.97
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	47	12.38	13.23	14.43	14.91	18.65	21.40	7.50	8.02	9.06	9.57	16.60	18.92
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	63	13.66	14.34	15.61	16.22	21.06	24.66	10.06	10.54	11.32	11.71	18.40	21.41
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	72	14.19	14.93	16.40	17.14	22.69	26.78	10.39	10.83	11.69	12.12	19.66	23.08
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	100	16.30	17.24	19.12	20.06	28.25	31.28	11.61	12.19	13.35	13.93	24.23	28.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	150	19.36	20.11	21.56	22.26	28.25	31.28	14.41	15.28	17.02	17.89	25.79	28.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	200	19.36	20.11	21.56	22.26	28.25	31.28	15.66	16.29	17.49	18.08	25.79	28.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	250	19.36	20.11	21.56	22.26	28.25	31.28	15.66	16.29	17.49	18.08	25.79	28.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	300	19.36	20.11	21.56	22.26	28.25	31.28	15.66	16.29	17.49	18.08	25.79	28.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
24	22	6.61	7.06	7.97	8.43	12.07	14.81	3.86	4.13	4.66	4.93	9.58	11.75
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	25	7.51	8.02	9.06	9.58	13.72	16.83	4.39	4.69	5.30	5.60	10.89	13.35
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	38	11.41	12.20	13.77	14.56	20.85	25.58	6.67	7.13	8.05	8.51	16.55	20.30

Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	47	14.11	15.09	17.03	18.01	24.18	27.41	8.25	8.82	9.96	10.53	20.47	23.80
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	63	17.86	18.59	20.03	20.74	26.36	30.51	11.06	11.83	13.35	14.12	22.53	25.86
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	75	18.64	19.48	21.15	21.97	28.54	33.43	13.17	13.98	15.01	15.51	24.06	27.97
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	100	20.59	21.72	23.98	25.12	33.95	40.41	14.34	14.99	16.29	16.94	28.05	33.19
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	150	25.44	26.99	29.46	30.42	38.23	42.33	17.29	18.24	20.14	21.08	34.05	37.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	200	26.44	27.47	29.46	30.42	38.23	42.33	20.59	21.86	23.61	24.41	34.05	37.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	250	26.44	27.47	29.46	30.42	38.23	42.33	21.11	21.96	23.61	24.41	34.05	37.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	300	26.44	27.47	29.46	30.42	38.23	42.33	21.11	21.96	23.61	24.41	34.05	37.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member steel-timber-steel joint

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Bolt diameter	Minimum timber thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	22	1.63	1.74	1.97	2.08	2.98	3.65	1.09	1.16	1.31	1.39	2.84	3.48
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	35	2.59	2.77	3.13	3.31	4.74	5.81	1.73	1.85	2.09	2.21	4.51	5.54
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	38	2.82	3.01	3.40	3.59	5.14	6.31	1.88	2.01	2.27	2.39	4.90	6.01
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	47	3.48	3.72	4.20	4.44	6.08	6.74	2.32	2.48	2.80	2.96	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	63	4.50	4.65	4.94	5.08	6.08	6.74	3.11	3.33	3.76	3.97	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	75	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	100	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	150	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58

Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	200	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	250	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	300	4.50	4.65	4.94	5.08	6.08	6.74	3.67	3.80	4.04	4.15	5.94	6.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
12	22	1.91	2.04	2.31	2.44	3.49	4.29	1.25	1.34	1.51	1.59	3.24	3.97
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	25	2.17	2.32	2.62	2.77	3.97	4.87	1.42	1.52	1.71	1.81	3.68	4.51
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	38	3.30	3.53	3.99	4.21	6.04	7.40	2.16	2.31	2.61	2.75	5.59	6.86
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	47	4.09	4.37	4.93	5.21	7.47	9.16	2.67	2.85	3.22	3.41	6.91	8.48
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	63	5.48	5.85	6.61	6.98	8.35	9.25	3.58	3.83	4.32	4.57	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	75	6.18	6.39	6.79	6.98	8.35	9.25	4.26	4.55	5.14	5.44	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	100	6.18	6.39	6.79	6.98	8.35	9.25	5.00	5.16	5.49	5.64	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	150	6.18	6.39	6.79	6.98	8.35	9.25	5.00	5.16	5.49	5.64	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	200	6.18	6.39	6.79	6.98	8.35	9.25	5.00	5.16	5.49	5.64	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	250	6.18	6.39	6.79	6.98	8.35	9.25	5.00	5.16	5.49	5.64	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	300	6.18	6.39	6.79	6.98	8.35	9.25	5.00	5.16	5.49	5.64	8.04	8.90
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6

16	22	2.43	2.60	2.94	3.11	4.45	5.46	1.53	1.64	1.85	1.95	3.90	4.79
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	25	2.77	2.96	3.34	3.53	5.05	6.20	1.74	1.86	2.10	2.22	4.43	5.44
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	38	4.20	4.49	5.07	5.36	7.68	9.42	2.64	2.83	3.19	3.37	6.74	8.27
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	47	5.20	5.56	6.28	6.63	9.50	11.65	3.27	3.50	3.95	4.17	8.34	10.22
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	63	6.97	7.45	8.41	8.89	12.74	15.17	4.38	4.69	5.29	5.59	11.17	13.70
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	75	8.30	8.87	10.01	10.59	13.70	15.17	5.22	5.58	6.30	6.66	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	97	10.13	10.48	11.13	11.44	13.70	15.17	6.75	7.21	8.15	8.61	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	150	10.13	10.48	11.13	11.44	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	200	10.13	10.48	11.13	11.44	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	250	10.13	10.48	11.13	11.44	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	300	10.13	10.48	11.13	11.44	13.70	15.17	8.03	8.31	8.83	9.08	12.83	14.21
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	22	2.90	3.10	3.50	3.70	5.30	6.49	1.76	1.88	2.12	2.24	4.41	5.41
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	25	3.29	3.52	3.97	4.20	6.02	7.38	2.00	2.13	2.41	2.55	5.01	6.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	38	5.00	5.35	6.04	6.39	9.15	11.22	3.03	3.24	3.66	3.87	7.62	9.35
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	47	6.19	6.62	7.47	7.90	11.31	13.87	3.75	4.01	4.53	4.79	9.43	11.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	63	8.30	8.87	10.01	10.59	15.16	18.60	5.03	5.38	6.07	6.42	12.64	15.50
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	72	9.48	10.14	11.44	12.10	17.33	21.25	5.75	6.14	6.94	7.33	14.44	17.71
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	100	13.17	14.08	15.90	16.69	19.97	22.12	7.98	8.53	9.63	10.18	18.23	20.19
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	150	14.77	15.28	16.23	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	200	14.77	15.28	16.23	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19

Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	250	14.77	15.28	16.23	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	300	14.77	15.28	16.23	16.69	19.97	22.12	11.50	11.89	12.64	12.99	18.23	20.19
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
24	22	3.30	3.53	3.99	4.21	6.04	7.40	1.93	2.06	2.33	2.46	4.79	5.88
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	25	3.75	4.01	4.53	4.79	6.86	8.41	2.20	2.35	2.65	2.80	5.44	6.68
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	38	5.71	6.10	6.89	7.28	10.43	12.79	3.34	3.57	4.03	4.26	8.28	10.15
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	47	7.06	7.54	8.52	9.00	12.90	15.82	4.13	4.41	4.98	5.27	10.24	12.55
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	63	9.46	10.11	11.42	12.07	17.29	21.20	5.53	5.91	6.68	7.06	13.72	16.83
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	75	11.26	12.04	13.59	14.37	20.58	25.24	6.59	7.04	7.95	8.40	16.33	20.03
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	100	15.01	16.05	18.12	19.16	27.03	29.93	8.78	9.39	10.60	11.20	21.78	26.67
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	150	19.99	20.67	21.97	22.58	27.03	29.93	13.17	14.08	15.90	16.80	24.08	26.67
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	200	19.99	20.67	21.97	22.58	27.03	29.93	15.29	15.81	16.80	17.27	24.08	26.67
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	250	19.99	20.67	21.97	22.58	27.03	29.93	15.29	15.81	16.80	17.27	24.08	26.67
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	300	19.99	20.67	21.97	22.58	27.03	29.93	15.29	15.81	16.80	17.27	24.08	26.67
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member glulam-steel-glulam joint

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

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Bolt diameter	Minimum glulam thickness	Direction of loading															
		Parallel to the grain								Perpendicular to the grain							
		Strength classes															
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	65	6.70	6.32	6.99	6.70	7.18	6.99	7.37	7.18	5.05	4.71	5.40	5.05	5.63	5.40	5.86	5.63
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	90	6.70	6.39	6.99	6.70	7.18	6.99	7.37	7.18	5.63	5.37	5.88	5.63	6.04	5.88	6.21	6.04
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	115	6.70	6.39	6.99	6.70	7.18	6.99	7.37	7.18	5.63	5.37	5.88	5.63	6.04	5.88	6.21	6.04
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	140	6.70	6.39	6.99	6.70	7.18	6.99	7.37	7.18	5.63	5.37	5.88	5.63	6.04	5.88	6.21	6.04
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	165	6.70	6.39	6.99	6.70	7.18	6.99	7.37	7.18	5.63	5.37	5.88	5.63	6.04	5.88	6.21	6.04
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	190	6.70	6.39	6.99	6.70	7.18	6.99	7.37	7.18	5.63	5.37	5.88	5.63	6.04	5.88	6.21	6.04
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
12	65	8.51	7.92	9.09	8.51	9.48	9.09	9.87	9.48	6.37	5.95	6.79	6.37	7.06	6.79	7.33	7.06
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	90	9.25	8.83	9.66	9.25	9.93	9.66	10.19	9.93	7.62	7.08	8.07	7.62	8.30	8.07	8.52	8.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	115	9.25	8.83	9.66	9.25	9.93	9.66	10.19	9.93	7.72	7.36	8.07	7.72	8.30	8.07	8.52	8.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6

12	140	9.25	8.83	9.66	9.25	9.93	9.66	10.19	9.93	7.72	7.36	8.07	7.72	8.30	8.07	8.52	8.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	165	9.25	8.83	9.66	9.25	9.93	9.66	10.19	9.93	7.72	7.36	8.07	7.72	8.30	8.07	8.52	8.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	190	9.25	8.83	9.66	9.25	9.93	9.66	10.19	9.93	7.72	7.36	8.07	7.72	8.30	8.07	8.52	8.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6

16	65	12.37	11.57	13.17	12.37	13.70	13.17	14.23	13.70	8.99	8.50	9.49	8.99	9.82	9.49	10.15	9.82
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	90	14.75	13.72	15.79	14.75	16.47	15.79	16.91	16.47	10.62	9.95	11.30	10.62	11.75	11.30	12.20	11.75
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	115	15.34	14.64	16.03	15.34	16.47	16.03	16.91	16.47	12.26	11.39	13.12	12.26	13.58	13.12	13.95	13.58
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	140	15.34	14.64	16.03	15.34	16.47	16.03	16.91	16.47	12.63	12.03	13.21	12.63	13.58	13.21	13.95	13.58
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	165	15.34	14.64	16.03	15.34	16.47	16.03	16.91	16.47	12.63	12.03	13.21	12.63	13.58	13.21	13.95	13.58
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	190	15.34	14.64	16.03	15.34	16.47	16.03	16.91	16.47	12.63	12.03	13.21	12.63	13.58	13.21	13.95	13.58
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	65	16.74	15.80	17.68	16.74	18.30	17.68	18.93	18.30	11.98	11.39	12.57	11.98	12.96	12.57	13.35	12.96
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	90	19.38	18.09	20.68	19.38	21.55	20.68	22.41	21.55	13.48	12.69	14.26	13.48	14.78	14.26	15.30	14.78
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	115	22.33	20.75	23.62	22.33	24.28	23.62	24.93	24.28	15.43	14.44	16.42	15.43	17.08	16.42	17.74	17.08
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	140	22.61	21.56	23.62	22.61	24.28	23.62	24.93	24.28	17.58	16.34	18.82	17.58	19.64	18.82	20.26	19.64
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	165	22.61	21.56	23.62	22.61	24.28	23.62	24.93	24.28	18.37	17.49	19.22	18.37	19.77	19.22	20.26	19.77
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	190	22.61	21.56	23.62	22.61	24.28	23.62	24.93	24.28	18.37	17.49	19.22	18.37	19.77	19.22	20.26	19.77
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
24	65	21.29	20.19	22.38	21.29	23.10	22.38	23.82	23.10	14.96	13.78	16.07	14.96	16.52	16.07	16.96	16.52
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	90	24.37	22.90	25.84	24.37	26.82	25.84	27.79	26.82	16.58	15.70	17.46	16.58	18.04	17.46	18.62	18.04
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	115	27.59	25.70	29.47	27.59	30.72	29.47	31.98	30.72	18.42	17.32	19.51	18.42	20.24	19.51	20.97	20.24
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	140	30.89	28.80	32.29	30.89	33.20	32.29	34.10	33.20	20.61	19.28	21.94	20.61	22.82	21.94	23.70	22.82
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	165	30.89	29.46	32.29	30.89	33.20	32.29	34.10	33.20	23.02	21.47	24.58	23.02	25.62	24.58	26.66	25.62
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	190	30.89	29.46	32.29	30.89	33.20	32.29	34.10	33.20	24.75	23.61	25.71	24.75	26.33	25.71	26.94	26.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member steel-glulam-steel joint

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Bolt diameter	Minimum glulam thickness	Direction of loading: Parallel to the grain — Strength classes								Perpendicular to the grain							
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	65	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	3.87	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	90	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	4.04	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	115	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	4.04	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	140	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	4.04	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	165	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	4.04	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	190	5.15	4.94	5.35	5.15	5.48	5.35	5.61	5.48	4.21	4.04	4.37	4.21	4.47	4.37	4.58	4.47
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
12	65	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	4.84	4.46	5.22	4.84	5.48	5.22	5.73	5.48
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	90	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08

Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	115	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	140	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	165	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	190	7.07	6.79	7.35	7.07	7.52	7.35	7.70	7.52	5.72	5.49	5.94	5.72	6.08	5.94	6.22	6.08
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
16	65	9.42	8.68	10.17	9.42	10.66	10.17	11.16	10.66	5.93	5.46	6.39	5.93	6.71	6.39	7.02	6.71
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	90	11.60	11.13	12.05	11.60	12.34	12.05	12.62	12.34	8.21	7.56	8.85	8.21	9.29	8.85	9.72	9.29
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	115	11.60	11.13	12.05	11.60	12.34	12.05	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	140	11.60	11.13	12.05	11.60	12.34	12.05	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	165	11.60	11.13	12.05	11.60	12.34	12.05	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	190	11.60	11.13	12.05	11.60	12.34	12.05	12.62	12.34	9.20	8.83	9.55	9.20	9.78	9.55	10.01	9.78
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	65	11.22	10.33	12.10	11.22	12.69	12.10	13.28	12.69	6.80	6.26	7.34	6.80	7.69	7.34	8.05	7.69
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	90	15.53	14.31	16.76	15.53	17.58	16.76	18.39	17.58	9.41	8.67	10.16	9.41	10.65	10.16	11.15	10.65
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	115	16.91	16.23	17.57	16.91	17.99	17.57	18.40	17.99	12.03	11.08	12.98	12.03	13.61	12.98	14.24	13.61
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	140	16.91	16.23	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	165	16.91	16.23	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	190	16.91	16.23	17.57	16.91	17.99	17.57	18.40	17.99	13.17	12.64	13.68	13.17	14.01	13.68	14.33	14.01
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
24	65	12.79	11.78	13.80	12.79	14.47	13.80	15.14	14.47	7.48	6.89	8.07	7.48	8.46	8.07	8.86	8.46
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	90	17.71	16.31	19.10	17.71	20.04	19.10	20.97	20.04	10.35	9.54	11.17	10.35	11.72	11.17	12.26	11.72

Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	115	22.63	20.84	23.77	22.63	24.35	23.77	24.91	24.35	13.23	12.19	14.28	13.23	14.97	14.28	15.67	14.97
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	140	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	16.11	14.84	17.38	16.11	18.23	17.38	19.05	18.23
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	165	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	190	22.89	21.97	23.77	22.89	24.35	23.77	24.91	24.35	17.50	16.80	18.18	17.50	18.62	18.18	19.05	18.62
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member LVL-steel-LVL composite joint

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

Design values | Characteristic values | Reset table | Return to menu

Bolt diameter	Minimum LVL thickness	Direction of loading: Parallel to the grain, LVL, KERTO Q	KERTO S	Perp. to the grain, KERTO Q	KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	3.17	3.17	2.56	2.56
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	33	3.41	3.41	2.72	2.72
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	39	3.68	3.68	2.88	2.88
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	45	3.99	3.99	3.06	3.06
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	51	4.31	4.31	3.26	3.26
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	57	4.43	4.43	3.47	3.47
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	63	4.43	4.43	3.69	3.69
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	75	4.43	4.43	3.75	3.75
Steel thickness (mm) >		2.5	2.5	2.5	2.5

10	27	4.52	4.52	3.40	3.40
Steel thickness (mm) >		3	3	3	3
10	33	4.75	4.75	3.75	3.75
Steel thickness (mm) >		3	3	3	3
10	39	5.04	5.04	3.93	3.93
Steel thickness (mm) >		3	3	3	3
10	45	5.37	5.37	4.16	4.16
Steel thickness (mm) >		3	3	3	3
10	51	5.73	5.73	4.38	4.38
Steel thickness (mm) >		3	3	3	3
10	57	6.11	6.11	4.61	4.61
Steel thickness (mm) >		3	3	3	3
10	63	6.50	6.50	4.85	4.85
Steel thickness (mm) >		3	3	3	3
10	75	6.60	6.60	5.37	5.37
Steel thickness (mm) >		3	3	3	3
12	27	5.99	5.99	3.91	3.91
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	33	6.32	6.32	4.78	4.78
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	39	6.59	6.59	5.10	5.10
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	45	6.93	6.93	5.30	5.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	51	7.31	7.31	5.56	5.56
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	57	7.73	7.73	5.84	5.84
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	63	8.16	8.16	6.14	6.14
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	75	9.09	9.09	6.69	6.69
Steel thickness (mm) >		3.6	3.6	3.6	3.6
16	27	7.62	7.62	4.79	4.79
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	33	9.32	9.32	5.86	5.86
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	39	10.26	10.26	6.92	6.92
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	45	10.62	10.62	7.99	7.99
Steel thickness (mm) >		4.8	4.8	4.8	4.8

16	51	11.03	11.03	8.20	8.20
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	57	11.46	11.46	8.44	8.44
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	63	11.93	11.93	8.72	8.72
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	75	12.98	12.98	9.41	9.41
Steel thickness (mm) >		4.8	4.8	4.8	4.8
20	27	9.07	9.07	5.50	5.50
Steel thickness (mm) >		6	6	6	6
20	33	11.09	11.09	6.72	6.72
Steel thickness (mm) >		6	6	6	6
20	39	13.11	13.11	7.94	7.94
Steel thickness (mm) >		6	6	6	6
20	45	14.80	14.80	9.17	9.17
Steel thickness (mm) >		6	6	6	6
20	51	15.17	15.17	10.39	10.39
Steel thickness (mm) >		6	6	6	6
20	57	15.65	15.65	11.50	11.50
Steel thickness (mm) >		6	6	6	6
20	63	16.22	16.22	11.71	11.71
Steel thickness (mm) >		6	6	6	6
20	75	17.42	17.42	12.28	12.28
Steel thickness (mm) >		6	6	6	6
24	27	10.34	10.34	6.05	6.05
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	33	12.64	12.64	7.39	7.39
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	39	14.94	14.94	8.74	8.74
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	45	17.24	17.24	10.08	10.08
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	51	19.54	19.54	11.43	11.43
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	57	20.26	20.26	12.77	12.77
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	63	20.74	20.74	14.12	14.12
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	75	21.97	21.97	15.51	15.51
Steel thickness (mm) >		7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel bolt in a 3-member steel-LVL-steel joint

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

Design values | Characteristic values | Reset table | Return to menu

Bolt diameter	Minimum LVL thickness	Direction of loading			
		Parallel to the grain		Perp. to the grain	
		LVL			
		KERTO Q	KERTO S	KERTO Q	KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	2.09	2.09	1.42	1.42
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	33	2.55	2.55	1.74	1.74
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	39	3.01	3.01	2.05	2.05
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	45	3.44	3.44	2.37	2.37
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	51	3.44	3.44	2.68	2.68
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	57	3.44	3.44	2.84	2.84
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	63	3.44	3.44	2.84	2.84
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	75	3.44	3.44	2.84	2.84
Steel thickness (mm) >		2.5	2.5	2.5	2.5

CD2.4
Note this is a snapshot of an interactive spreadsheet

10	27	2.55	2.55	1.70	1.70
Steel thickness (mm) >		3	3	3	3
10	33	3.12	3.12	2.08	2.08
Steel thickness (mm) >		3	3	3	3
10	39	3.69	3.69	2.46	2.46
Steel thickness (mm) >		3	3	3	3
10	45	4.25	4.25	2.84	2.84
Steel thickness (mm) >		3	3	3	3
10	51	4.82	4.82	3.21	3.21
Steel thickness (mm) >		3	3	3	3
10	57	5.08	5.08	3.59	3.59
Steel thickness (mm) >		3	3	3	3
10	63	5.08	5.08	3.97	3.97
Steel thickness (mm) >		3	3	3	3
10	75	5.08	5.08	4.15	4.15
Steel thickness (mm) >		3	3	3	3
12	27	2.99	2.99	1.96	1.96
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	33	3.66	3.66	2.39	2.39
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	39	4.33	4.33	2.83	2.83
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	45	4.99	4.99	3.26	3.26
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	51	5.66	5.66	3.70	3.70
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	57	6.32	6.32	4.13	4.13
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	63	6.98	6.98	4.57	4.57
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	75	6.98	6.98	5.44	5.44
Steel thickness (mm) >		3.6	3.6	3.6	3.6
16	27	3.81	3.81	2.40	2.40
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	33	4.66	4.66	2.93	2.93
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	39	5.50	5.50	3.46	3.46
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	45	6.35	6.35	3.99	3.99
Steel thickness (mm) >		4.8	4.8	4.8	4.8

16	51	7.20	7.20	4.53	4.53
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	57	8.05	8.05	5.06	5.06
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	63	8.89	8.89	5.59	5.59
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	75	10.59	10.59	6.66	6.66
Steel thickness (mm) >		4.8	4.8	4.8	4.8
20	27	4.54	4.54	2.75	2.75
Steel thickness (mm) >		6	6	6	6
20	33	5.55	5.55	3.36	3.36
Steel thickness (mm) >		6	6	6	6
20	39	6.55	6.55	3.97	3.97
Steel thickness (mm) >		6	6	6	6
20	45	7.56	7.56	4.58	4.58
Steel thickness (mm) >		6	6	6	6
20	51	8.57	8.57	5.19	5.19
Steel thickness (mm) >		6	6	6	6
20	57	9.58	9.58	5.80	5.80
Steel thickness (mm) >		6	6	6	6
20	63	10.59	10.59	6.42	6.42
Steel thickness (mm) >		6	6	6	6
20	75	12.60	12.60	7.64	7.64
Steel thickness (mm) >		6	6	6	6
24	27	5.17	5.17	3.02	3.02
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	33	6.32	6.32	3.70	3.70
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	39	7.47	7.47	4.37	4.37
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	45	8.62	8.62	5.04	5.04
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	51	9.77	9.77	5.71	5.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	57	10.92	10.92	6.39	6.39
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	63	12.07	12.07	7.06	7.06
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	75	14.37	14.37	8.40	8.40
Steel thickness (mm) >		7.2	7.2	7.2	7.2

CD2.5 DOWELLED CONNECTIONS

In accordance with BS EN 1995-1-1:2004 and accompanying CD2.1 *Calculation basis*

2-MEMBER TIMBER JOINT

3-MEMBER TIMBER JOINT

2-MEMBER SOFTWOOD GLULAM JOINT

3-MEMBER SOFTWOOD GLULAM JOINT

2-MEMBER LVL JOINT

3-MEMBER LVL JOINT

3-MEMBER TIMBER-STEEL-TIMBER JOINT

3-MEMBER STEEL-TIMBER-STEEL JOINT

3-MEMBER GLULAM-STEEL-GLULAM JOINT

3-MEMBER STEEL-GLULAM-STEEL JOINT

3-MEMBER LVL-STEEL-LVL JOINT

3-MEMBER STEEL-LVL-STEEL JOINT

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Lateral design loads for one 4.6 grade steel dowel in a 2-member timber joint

Service Class: 1 | k_{mod} = 0.90

Load duration: Short term | γ_M = 1.30

Design situation: Normal

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Dowel diameter	Minimum member thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	16	0.80	0.86	0.97	1.02	1.47	1.80	0.55	0.58	0.66	0.70	1.44	1.76
8	22	1.10	1.18	1.33	1.41	2.02	2.47	0.75	0.80	0.91	0.96	1.98	2.43
8	35	1.76	1.88	2.12	2.24	3.21	3.65	1.19	1.28	1.44	1.52	3.15	3.61
8	44	2.21	2.36	2.67	2.75	3.29	3.65	1.50	1.61	1.81	1.92	3.26	3.61
8	47	2.36	2.52	2.68	2.75	3.29	3.65	1.60	1.72	1.94	2.05	3.26	3.61
8	60	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	72	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	97	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	147	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
10	16	0.98	1.05	1.19	1.25	1.79	2.20	0.65	0.70	0.79	0.84	1.71	2.10
10	22	1.35	1.44	1.63	1.72	2.47	3.03	0.90	0.96	1.09	1.15	2.35	2.88
10	35	2.15	2.30	2.59	2.74	3.93	4.81	1.43	1.53	1.73	1.83	3.74	4.59
10	44	2.70	2.89	3.26	3.45	4.87	5.39	1.80	1.92	2.17	2.30	4.70	5.26
10	47	2.88	3.08	3.48	3.68	4.87	5.39	1.92	2.06	2.32	2.45	4.75	5.26

10	60	3.60	3.72	3.96	4.07	4.87	5.39	2.45	2.62	2.96	3.13	4.75	5.26
10	72	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
10	97	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
10	147	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
12	16	1.15	1.23	1.39	1.47	2.11	2.58	0.75	0.80	0.91	0.96	1.95	2.39
12	22	1.58	1.69	1.91	2.02	2.90	3.55	1.04	1.11	1.25	1.32	2.68	3.29
12	35	2.52	2.69	3.04	3.22	4.61	5.65	1.65	1.76	1.99	2.10	4.27	5.23
12	44	3.17	3.39	3.82	4.04	5.79	7.10	2.07	2.21	2.50	2.64	5.36	6.58
12	47	3.38	3.62	4.08	4.32	6.19	7.40	2.21	2.36	2.67	2.82	5.73	7.02
12	60	4.32	4.62	5.21	5.51	6.68	7.40	2.82	3.02	3.41	3.60	6.43	7.12
12	72	4.94	5.11	5.43	5.58	6.68	7.40	3.39	3.62	4.09	4.32	6.43	7.12
12	97	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
12	147	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
16	16	1.47	1.57	1.77	1.87	2.68	3.29	0.92	0.99	1.11	1.18	2.35	2.88
16	22	2.02	2.16	2.43	2.57	3.68	4.52	1.27	1.36	1.53	1.62	3.23	3.96
16	35	3.21	3.43	3.87	4.09	5.86	7.19	2.02	2.16	2.43	2.57	5.14	6.31
16	44	4.03	4.31	4.87	5.15	7.37	9.04	2.54	2.71	3.06	3.24	6.46	7.93
16	47	4.31	4.60	5.20	5.50	7.87	9.65	2.71	2.90	3.27	3.46	6.91	8.47
16	60	5.50	5.88	6.64	7.02	10.05	12.13	3.46	3.70	4.17	4.41	8.82	10.81
16	72	6.60	7.05	7.96	8.42	10.96	12.13	4.15	4.44	5.01	5.30	10.26	11.36
16	97	8.11	8.38	8.90	9.16	10.96	12.13	5.59	5.98	6.75	7.13	10.26	11.36
16	147	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
20	16	1.75	1.87	2.11	2.23	3.19	3.91	1.06	1.13	1.28	1.35	2.66	3.26
20	22	2.40	2.57	2.90	3.06	4.39	5.38	1.45	1.56	1.76	1.86	3.66	4.48
20	35	3.82	4.08	4.61	4.87	6.98	8.56	2.31	2.47	2.79	2.95	5.82	7.13
20	44	4.80	5.13	5.79	6.13	8.77	10.76	2.91	3.11	3.51	3.71	7.31	8.97
20	47	5.13	5.48	6.19	6.54	9.37	11.49	3.11	3.32	3.75	3.97	7.81	9.58
20	60	6.55	7.00	7.90	8.35	11.96	14.67	3.97	4.24	4.79	5.06	9.97	12.23
20	72	7.86	8.40	9.48	10.02	14.36	17.61	4.76	5.09	5.75	6.07	11.96	14.67
20	97	10.58	11.31	12.77	13.35	15.98	17.70	6.41	6.86	7.74	8.18	14.59	16.15
20	147	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15
24	16	1.99	2.13	2.40	2.54	3.64	4.46	1.16	1.24	1.40	1.48	2.89	3.54
24	22	2.74	2.93	3.30	3.49	5.00	6.13	1.60	1.71	1.93	2.04	3.97	4.87
24	35	4.35	4.65	5.25	5.55	7.96	9.76	2.55	2.72	3.07	3.25	6.31	7.74
24	44	5.47	5.85	6.61	6.98	10.00	12.27	3.20	3.42	3.86	4.08	7.94	9.74
24	47	5.85	6.25	7.06	7.46	10.68	13.10	3.42	3.65	4.13	4.36	8.48	10.40

24	60	7.46	7.98	9.01	9.52	13.64	16.73	4.36	4.67	5.27	5.57	10.82	13.28
24	72	8.96	9.57	10.81	11.43	16.37	20.07	5.24	5.60	6.32	6.68	12.99	15.93
24	97	12.07	12.90	14.56	15.39	21.62	23.95	7.06	7.54	8.52	9.00	17.50	21.33
24	147	16.00	16.54	17.57	18.07	21.62	23.95	10.69	11.43	12.90	13.64	19.26	21.33

Lateral design loads per shear plane for one 4.6 grade steel dowel in a 3-member timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Dowel diameter	Minimum member thickness		Direction of loading											
			Parallel to the grain						Perpendicular to the grain					
			Strength classes											
	Outer members	Inner member	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	16	32	1.59	1.65	1.76	1.82	2.25	2.56	1.30	1.34	1.43	1.47	2.22	2.53
8	22	44	1.67	1.74	1.88	1.95	2.50	2.90	1.33	1.38	1.48	1.53	2.46	2.85
8	35	70	2.00	2.10	2.31	2.41	3.24	3.65	1.50	1.58	1.72	1.79	3.19	3.61
8	44	88	2.28	2.41	2.67	2.75	3.29	3.65	1.68	1.77	1.95	2.03	3.26	3.61
8	47	94	2.39	2.52	2.68	2.75	3.29	3.65	1.74	1.84	2.03	2.12	3.26	3.61
8	60	120	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	72	144	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	97	194	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
8	147	294	2.44	2.52	2.68	2.75	3.29	3.65	2.01	2.08	2.21	2.27	3.26	3.61
10	16	32	2.33	2.41	2.56	2.64	3.21	3.62	1.58	1.69	1.91	2.02	3.13	3.51
10	22	44	2.38	2.47	2.66	2.75	3.44	3.95	1.91	1.98	2.11	2.18	3.34	3.82
10	35	70	2.70	2.83	3.09	3.22	4.24	5.01	2.05	2.15	2.32	2.41	4.08	4.81
10	44	88	3.01	3.17	3.49	3.65	4.87	5.39	2.23	2.34	2.55	2.66	4.73	5.26
10	47	94	3.13	3.30	3.64	3.81	4.87	5.39	2.29	2.41	2.64	2.76	4.75	5.26
10	60	120	3.60	3.72	3.96	4.07	4.87	5.39	2.62	2.76	3.05	3.20	4.75	5.26

CD2.5

Note this is a snapshot of an interactive spreadsheet

10	72	144	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
10	97	194	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
10	147	294	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
12	16	32	2.78	2.97	3.36	3.55	4.34	4.85	1.82	1.94	2.19	2.32	4.17	4.65
12	22	44	3.22	3.33	3.56	3.68	4.54	5.15	2.50	2.67	2.84	2.92	4.33	4.91
12	35	70	3.50	3.66	3.97	4.13	5.35	6.25	2.69	2.79	3.01	3.12	5.05	5.89
12	44	88	3.82	4.01	4.39	4.58	6.09	7.21	2.85	2.98	3.23	3.36	5.72	6.76
12	47	94	3.94	4.15	4.55	4.75	6.35	7.40	2.91	3.05	3.32	3.45	5.96	7.07
12	60	120	4.53	4.78	5.29	5.55	6.68	7.40	3.24	3.41	3.74	3.91	6.43	7.12
12	72	144	4.94	5.11	5.43	5.58	6.68	7.40	3.59	3.79	4.19	4.39	6.43	7.12
12	97	194	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
12	147	294	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
16	16	32	3.54	3.78	4.27	4.52	6.47	7.84	2.23	2.38	2.69	2.84	5.68	6.96
16	22	44	4.87	5.20	5.75	5.92	7.16	8.02	3.06	3.27	3.69	3.91	6.67	7.45
16	35	70	5.41	5.62	6.05	6.26	7.89	9.07	4.18	4.33	4.62	4.77	7.23	8.29
16	44	88	5.70	5.95	6.46	6.71	8.67	10.12	4.29	4.46	4.79	4.96	7.88	9.15
16	47	94	5.82	6.09	6.62	6.88	8.96	10.50	4.34	4.52	4.87	5.04	8.12	9.48
16	60	120	6.44	6.77	7.43	7.75	10.35	12.13	4.64	4.85	5.27	5.49	9.30	11.00
16	72	144	7.11	7.50	8.28	8.67	10.96	12.13	4.99	5.24	5.74	5.98	10.26	11.36
16	97	194	8.11	8.38	8.90	9.16	10.96	12.13	5.87	6.20	6.86	7.19	10.26	11.36
16	147	294	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
20	16	32	4.21	4.51	5.09	5.38	7.70	9.45	2.55	2.73	3.08	3.26	6.42	7.87
20	22	44	5.80	6.19	6.99	7.39	10.32	11.48	3.51	3.75	4.24	4.48	8.83	10.44
20	35	70	7.70	7.98	8.53	8.80	10.87	12.34	5.59	5.97	6.53	6.72	9.74	11.01
20	44	88	7.92	8.24	8.87	9.18	11.61	13.37	5.98	6.20	6.62	6.83	10.28	11.78
20	47	94	8.03	8.36	9.02	9.35	11.90	13.77	6.01	6.24	6.68	6.89	10.50	12.08
20	60	120	8.61	9.02	9.82	10.21	13.36	15.69	6.24	6.50	7.01	7.26	11.63	13.59
20	72	144	9.30	9.77	10.71	11.18	14.90	17.67	6.55	6.85	7.44	7.73	12.85	15.17
20	97	194	11.00	11.62	12.86	13.35	15.98	17.70	7.41	7.79	8.56	8.94	14.59	16.15
20	147	294	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15
24	16	32	4.80	5.14	5.80	6.13	8.78	10.77	2.81	3.00	3.39	3.58	6.97	8.55
24	22	44	6.61	7.06	7.97	8.43	12.07	14.81	3.86	4.13	4.66	4.93	9.58	11.75
24	35	70	10.33	10.69	11.39	11.72	14.27	16.06	6.15	6.57	7.42	7.84	12.56	14.05
24	44	88	10.47	10.86	11.63	12.01	14.91	16.99	7.73	8.17	8.69	8.95	12.94	14.66
24	47	94	10.55	10.95	11.75	12.14	15.18	17.37	7.91	8.18	8.72	8.98	13.12	14.92
24	60	120	11.05	11.53	12.47	12.94	16.60	19.30	8.04	8.35	8.95	9.25	14.11	16.28

24	72	144	11.70	12.25	13.34	13.89	18.18	21.37	8.28	8.63	9.30	9.64	15.26	17.81
24	97	194	13.43	14.14	15.57	16.29	21.62	23.95	9.05	9.48	10.34	10.77	18.09	21.33
24	147	294	16.00	16.54	17.57	18.07	21.62	23.95	11.21	11.84	13.10	13.72	19.26	21.33

Lateral design loads for one 4.6 grade steel dowel in a 2-member softwood glulam joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Dowel diameter	Minimum member thickness	Direction of loading															
		Parallel to the grain								Perpendicular to the grain							
		Strength classes															
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	65	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	90	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	115	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	140	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	165	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	190	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	215	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
10	65	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.21	3.50	3.36	3.58	3.50	3.66	3.58
10	90	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	115	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	140	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	165	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	190	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	215	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
12	65	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.01	3.69	4.33	4.01	4.54	4.33	4.75	4.54
12	90	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	115	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	140	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87

12	165	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	190	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	215	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
16	65	7.81	7.19	8.42	7.81	8.83	8.42	9.24	8.83	4.91	4.52	5.30	4.91	5.56	5.30	5.81	5.56
16	90	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	6.80	6.26	7.33	6.80	7.69	7.33	8.01	7.69
16	115	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	140	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	165	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	190	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	215	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
20	65	9.29	8.56	10.03	9.29	10.52	10.03	11.00	10.52	5.63	5.19	6.08	5.63	6.37	6.08	6.67	6.37
20	90	12.87	11.85	13.88	12.87	14.39	13.88	14.72	14.39	7.80	7.18	8.41	7.80	8.82	8.41	9.23	8.82
20	115	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	9.96	9.18	10.75	9.96	11.20	10.75	11.46	11.20
20	140	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	165	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	190	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	215	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
24	65	10.59	9.76	11.43	10.59	11.99	11.43	12.55	11.99	6.20	5.71	6.68	6.20	7.01	6.68	7.34	7.01
24	90	14.67	13.51	15.83	14.67	16.60	15.83	17.37	16.60	8.58	7.90	9.26	8.58	9.71	9.26	10.16	9.71
24	115	18.31	17.26	19.02	18.31	19.48	19.02	19.93	19.48	10.96	10.10	11.83	10.96	12.40	11.83	12.98	12.40
24	140	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	13.34	12.29	14.40	13.34	14.89	14.40	15.24	14.89
24	165	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
24	190	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
24	215	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89

Lateral design loads per shear plane for one 4.6 grade steel dowel in a 3-member softwood glulam joint

Service Class	1	k_{mod} =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Dowel diameter	Minimum member thickness		Direction of loading															
			Parallel to the grain								Perpendicular to the grain							
			Strength classes															
	Outer members	Inner member	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
8	65	130	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	90	180	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	115	230	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	140	280	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	165	330	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	190	380	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
8	215	430	2.79	2.68	2.90	2.79	2.97	2.90	3.03	2.97	2.30	2.21	2.39	2.30	2.45	2.39	2.50	2.45
10	65	130	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.22	3.50	3.36	3.58	3.50	3.66	3.58
10	90	180	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	115	230	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	140	280	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	165	330	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	190	380	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
10	215	430	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
12	65	130	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.20	3.93	4.47	4.20	4.65	4.47	4.83	4.65
12	90	180	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	115	230	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	140	280	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	165	330	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
12	190	380	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87

12	215	430	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
16	65	130	8.30	7.77	8.83	8.30	9.18	8.83	9.53	9.18	5.80	5.46	6.14	5.80	6.36	6.14	6.58	6.36
16	90	180	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	6.99	6.53	7.44	6.99	7.75	7.44	8.01	7.75
16	115	230	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	140	280	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	165	330	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	190	380	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
16	215	430	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
20	65	130	10.81	10.17	11.45	10.81	11.87	11.45	12.30	11.87	7.58	7.18	7.98	7.58	8.24	7.98	8.51	8.24
20	90	180	13.10	12.23	13.96	13.10	14.39	13.96	14.72	14.39	8.75	8.22	9.28	8.75	9.64	9.28	9.99	9.64
20	115	230	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.16	9.49	10.83	10.16	11.20	10.83	11.46	11.20
20	140	280	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	165	330	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	190	380	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
20	215	430	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
24	65	130	13.56	12.82	14.30	13.56	14.80	14.30	15.29	14.80	9.55	9.08	10.01	9.55	10.31	10.01	10.62	10.31
24	90	180	15.90	14.91	16.90	15.90	17.56	16.90	18.22	17.56	10.62	10.02	11.21	10.62	11.61	11.21	12.00	11.61
24	115	230	18.31	17.39	19.02	18.31	19.48	19.02	19.93	19.48	12.00	11.25	12.74	12.00	13.24	12.74	13.73	13.24
24	140	280	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	13.57	12.67	14.47	13.57	14.89	14.47	15.24	14.89
24	165	330	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
24	190	380	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
24	215	430	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89

Lateral design loads for one 4.6 grade steel dowel in a 2-member LVL joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

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Dowel diameter	Minimum member thickness	Direction of loading			
		Parallel to the grain		Perp. to the grain	
		LVL			
		KERTO Q	KERTO S	KERTO Q	KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	1.73	1.73	1.18	1.18
8	33	2.11	2.11	1.44	1.44
8	39	2.50	2.50	1.70	1.70
8	45	2.75	2.75	1.96	1.96
8	51	2.75	2.75	2.22	2.22
8	57	2.75	2.75	2.27	2.27
8	63	2.75	2.75	2.27	2.27
8	75	2.75	2.75	2.27	2.27
10	27	2.11	2.11	1.41	1.41
10	33	2.58	2.58	1.72	1.72
10	39	3.05	3.05	2.04	2.04
10	45	3.52	3.52	2.35	2.35
10	51	3.99	3.99	2.66	2.66
10	57	4.07	4.07	2.98	2.98
10	63	4.07	4.07	3.29	3.29
10	75	4.07	4.07	3.32	3.32
12	27	2.48	2.48	1.62	1.62
12	33	3.03	3.03	1.98	1.98

CD2.5
Note this is a snapshot of an interactive spreadsheet

12	39	3.58	3.58	2.34	2.34
12	45	4.13	4.13	2.70	2.70
12	51	4.69	4.69	3.06	3.06
12	57	5.24	5.24	3.42	3.42
12	63	5.58	5.58	3.78	3.78
12	75	5.58	5.58	4.50	4.50
16	27	3.16	3.16	1.99	1.99
16	33	3.86	3.86	2.43	2.43
16	39	4.56	4.56	2.87	2.87
16	45	5.26	5.26	3.31	3.31
16	51	5.96	5.96	3.75	3.75
16	57	6.67	6.67	4.19	4.19
16	63	7.37	7.37	4.63	4.63
16	75	8.77	8.77	5.52	5.52
20	27	3.76	3.76	2.28	2.28
20	33	4.59	4.59	2.78	2.78
20	39	5.43	5.43	3.29	3.29
20	45	6.26	6.26	3.80	3.80
20	51	7.10	7.10	4.30	4.30
20	57	7.93	7.93	4.81	4.81
20	63	8.77	8.77	5.32	5.32
20	75	10.44	10.44	6.33	6.33
24	27	4.28	4.28	2.51	2.51
24	33	5.24	5.24	3.06	3.06
24	39	6.19	6.19	3.62	3.62
24	45	7.14	7.14	4.18	4.18
24	51	8.09	8.09	4.73	4.73
24	57	9.05	9.05	5.29	5.29
24	63	10.00	10.00	5.85	5.85
24	75	11.90	11.90	6.96	6.96

Lateral design loads per shear plane for one 4.6 grade steel dowel in a 3-member LVL joint

Service Class: 1

Load duration: Short term

Design situation: Normal

k_{mod} = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Dowel diameter	Minimum member thickness		Direction of loading			
			Parallel to the grain		Perp. to the grain	
			LVL			
	Outer members	Inner member	KERTO Q	KERTO S	KERTO Q	KERTO S
(mm)	(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	54	2.11	2.11	1.62	1.62
8	33	66	2.33	2.33	1.75	1.75
8	39	78	2.58	2.58	1.90	1.90
8	45	90	2.75	2.75	2.06	2.06
8	51	102	2.75	2.75	2.24	2.24
8	57	114	2.75	2.75	2.27	2.27
8	63	126	2.75	2.75	2.27	2.27
8	75	150	2.75	2.75	2.27	2.27
10	27	54	2.90	2.90	2.24	2.24
10	33	66	3.13	3.13	2.37	2.37
10	39	78	3.40	3.40	2.52	2.52
10	45	90	3.70	3.70	2.69	2.69
10	51	102	4.02	4.02	2.89	2.89
10	57	114	4.07	4.07	3.09	3.09
10	63	126	4.07	4.07	3.31	3.31

10	75	150	4.07	4.07	3.32	3.32
12	27	54	3.81	3.81	2.97	2.97
12	33	66	4.04	4.04	3.07	3.07
12	39	78	4.32	4.32	3.22	3.22
12	45	90	4.64	4.64	3.39	3.39
12	51	102	4.99	4.99	3.59	3.59
12	57	114	5.35	5.35	3.80	3.80
12	63	126	5.58	5.58	4.03	4.03
12	75	150	5.58	5.58	4.51	4.51
16	27	54	6.00	6.00	4.69	4.69
16	33	66	6.18	6.18	4.74	4.74
16	39	78	6.44	6.44	4.84	4.84
16	45	90	6.76	6.76	4.99	4.99
16	51	102	7.13	7.13	5.17	5.17
16	57	114	7.54	7.54	5.37	5.37
16	63	126	7.97	7.97	5.60	5.60
16	75	150	8.91	8.91	6.12	6.12
20	27	54	8.63	8.63	5.50	5.50
20	33	66	8.74	8.74	6.71	6.71
20	39	78	8.95	8.95	6.76	6.76
20	45	90	9.24	9.24	6.85	6.85
20	51	102	9.59	9.59	6.99	6.99
20	57	114	9.99	9.99	7.17	7.17
20	63	126	10.44	10.44	7.37	7.37
20	75	150	11.44	11.44	7.86	7.86
24	27	54	10.34	10.34	6.05	6.05
24	33	66	11.69	11.69	7.39	7.39
24	39	78	11.82	11.82	8.74	8.74
24	45	90	12.05	12.05	8.96	8.96
24	51	102	12.36	12.36	9.04	9.04
24	57	114	12.73	12.73	9.17	9.17
24	63	126	13.16	13.16	9.33	9.33
24	75	150	14.15	14.15	9.75	9.75

Lateral design loads per shear plane for one 4.6 grade steel dowel in a 3-member timber-steel-timber joint

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Dowel diameter	Minimum timber thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	22	3.13	3.24	3.46	3.56	4.38	4.97	2.17	2.32	2.62	2.77	4.25	4.82
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	25	3.15	3.27	3.51	3.62	4.52	5.18	2.47	2.64	2.81	2.89	4.38	5.01
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	38	3.48	3.65	3.98	4.14	5.45	6.42	2.67	2.78	3.01	3.12	5.24	6.17
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	47	3.83	4.04	4.44	4.64	6.24	7.44	2.85	2.99	3.26	3.40	5.99	7.13
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	63	4.59	4.86	5.39	5.66	6.88	7.62	3.29	3.47	3.83	4.01	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	75	5.09	5.26	5.59	5.75	6.88	7.62	3.67	3.89	4.31	4.52	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	100	5.09	5.26	5.59	5.75	6.88	7.62	4.16	4.30	4.57	4.70	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	150	5.09	5.26	5.59	5.75	6.88	7.62	4.16	4.30	4.57	4.70	6.72	7.44

Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	200	5.09	5.26	5.59	5.75	6.88	7.62	4.16	4.30	4.57	4.70	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	250	5.09	5.26	5.59	5.75	6.88	7.62	4.16	4.30	4.57	4.70	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	300	5.09	5.26	5.59	5.75	6.88	7.62	4.16	4.30	4.57	4.70	6.72	7.44
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
12	22	3.82	4.09	4.62	4.85	5.86	6.59	2.50	2.67	3.02	3.19	5.62	6.30
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	25	4.30	4.45	4.74	4.88	5.97	6.77	2.84	3.04	3.43	3.62	5.71	6.45
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	38	4.54	4.74	5.13	5.33	6.88	8.03	3.52	3.66	3.92	4.06	6.50	7.56
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	47	4.89	5.12	5.60	5.84	7.73	9.14	3.67	3.83	4.15	4.31	7.27	8.57
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	63	5.69	6.00	6.63	6.95	9.45	10.47	4.09	4.30	4.72	4.93	8.84	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	75	6.38	6.75	7.50	7.87	9.45	10.47	4.49	4.74	5.23	5.47	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	100	6.99	7.23	7.68	7.90	9.45	10.47	5.44	5.77	6.21	6.38	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	150	6.99	7.23	7.68	7.90	9.45	10.47	5.65	5.84	6.21	6.38	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	200	6.99	7.23	7.68	7.90	9.45	10.47	5.65	5.84	6.21	6.38	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	250	6.99	7.23	7.68	7.90	9.45	10.47	5.65	5.84	6.21	6.38	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	300	6.99	7.23	7.68	7.90	9.45	10.47	5.65	5.84	6.21	6.38	9.09	10.07
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
16	22	4.87	5.20	5.87	6.21	8.90	10.55	3.06	3.27	3.69	3.91	7.80	9.57
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	25	5.53	5.91	6.68	7.06	9.54	10.63	3.48	3.72	4.20	4.44	8.87	9.91
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	38	7.11	7.38	7.90	8.16	10.20	11.70	5.29	5.65	6.14	6.32	9.38	10.71
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	47	7.37	7.68	8.30	8.61	11.06	12.88	5.63	5.84	6.25	6.45	10.07	11.67
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	63	8.15	8.56	9.37	9.77	12.99	15.39	5.94	6.20	6.72	6.98	11.68	13.80
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8

16	75	8.90	9.38	10.34	10.82	14.63	17.16	6.31	6.61	7.22	7.53	13.08	15.58
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	100	10.71	11.35	12.59	12.95	15.50	17.16	7.29	7.69	8.49	8.89	14.51	16.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	150	11.46	11.85	12.59	12.95	15.50	17.16	9.09	9.40	9.99	10.27	14.51	16.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	200	11.46	11.85	12.59	12.95	15.50	17.16	9.09	9.40	9.99	10.27	14.51	16.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	250	11.46	11.85	12.59	12.95	15.50	17.16	9.09	9.40	9.99	10.27	14.51	16.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	300	11.46	11.85	12.59	12.95	15.50	17.16	9.09	9.40	9.99	10.27	14.51	16.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	22	5.80	6.19	6.99	7.39	10.59	12.99	3.51	3.75	4.24	4.48	8.83	10.82
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	25	6.59	7.04	7.95	8.40	12.04	14.76	3.99	4.27	4.82	5.09	10.03	12.30
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	38	10.01	10.63	11.30	11.64	14.19	16.04	6.07	6.48	7.32	7.74	12.80	14.38
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	47	10.38	10.77	11.54	11.93	14.92	17.12	7.50	8.02	8.79	9.05	13.28	15.14
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	63	11.01	11.50	12.49	12.98	16.85	19.73	8.12	8.43	9.06	9.36	14.72	17.13
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	75	11.74	12.32	13.47	14.04	18.61	22.01	8.39	8.75	9.47	9.83	16.09	18.94
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	100	13.65	14.40	15.92	16.68	22.60	25.03	9.29	9.75	10.68	11.15	19.39	22.84
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	150	16.72	17.28	18.36	18.88	22.60	25.03	11.76	12.45	13.82	14.51	20.63	22.84
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	200	16.72	17.28	18.36	18.88	22.60	25.03	13.01	13.45	14.30	14.70	20.63	22.84
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	250	16.72	17.28	18.36	18.88	22.60	25.03	13.01	13.45	14.30	14.70	20.63	22.84
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	300	16.72	17.28	18.36	18.88	22.60	25.03	13.01	13.45	14.30	14.70	20.63	22.84
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
24	22	6.61	7.06	7.97	8.43	12.07	14.81	3.86	4.13	4.66	4.93	9.58	11.75
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	25	7.51	8.02	9.06	9.58	13.72	16.83	4.39	4.69	5.30	5.60	10.89	13.35
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	38	11.41	12.20	13.77	14.56	18.86	21.07	6.67	7.13	8.05	8.51	16.55	18.59

Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	47	13.91	14.39	15.33	15.79	19.35	21.93	8.25	8.82	9.96	10.53	16.94	19.04
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	63	14.29	14.87	16.02	16.59	21.08	24.40	10.64	11.01	11.74	12.10	18.03	20.69
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	75	14.91	15.58	16.92	17.58	22.84	26.74	10.77	11.19	12.00	12.41	19.25	22.37
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	100	16.77	17.64	19.38	20.25	27.16	32.33	11.47	12.00	13.03	13.55	22.44	26.55
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	150	21.63	22.92	24.85	25.55	30.58	33.87	13.83	14.59	16.11	16.87	27.24	30.17
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	200	22.62	23.39	24.85	25.55	30.58	33.87	16.78	17.78	19.00	19.54	27.24	30.17
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	250	22.62	23.39	24.85	25.55	30.58	33.87	17.30	17.89	19.00	19.54	27.24	30.17
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	300	22.62	23.39	24.85	25.55	30.58	33.87	17.30	17.89	19.00	19.54	27.24	30.17
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for
one 4.6 grade steel dowel in a 3-member steel-timber-steel joint
(For use in multiple shear plane connections. The steel plates should not be on the outside.)

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

Design values | Characteristic values | Reset table | Return to menu

Dowel diameter	Minimum timber thickness	Direction of loading											
		Parallel to the grain						Perpendicular to the grain					
		Strength classes											
		C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70	C14	C16, C18, C20 & C22	C24	C27, C30, C35 & C40	D30, D35 & D40	D50, D60 & D70
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	22	1.63	1.74	1.97	2.08	2.98	3.65	1.09	1.16	1.31	1.39	2.84	3.48
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	25	1.85	1.98	2.24	2.36	3.38	4.15	1.23	1.32	1.49	1.58	3.22	3.95
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	38	2.82	3.01	3.40	3.59	4.87	5.39	1.88	2.01	2.27	2.39	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	47	3.48	3.72	3.96	4.07	4.87	5.39	2.32	2.48	2.80	2.96	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	63	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	75	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	100	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	150	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26

Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	200	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	250	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
10	300	3.60	3.72	3.96	4.07	4.87	5.39	2.94	3.04	3.23	3.32	4.75	5.26
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3
12	22	1.91	2.04	2.31	2.44	3.49	4.29	1.25	1.34	1.51	1.59	3.24	3.97
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	25	2.17	2.32	2.62	2.77	3.97	4.87	1.42	1.52	1.71	1.81	3.68	4.51
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	38	3.30	3.53	3.99	4.21	6.04	7.40	2.16	2.31	2.61	2.75	5.59	6.86
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	47	4.09	4.37	4.93	5.21	6.68	7.40	2.67	2.85	3.22	3.41	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	63	4.94	5.11	5.43	5.58	6.68	7.40	3.58	3.83	4.32	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	75	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	100	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	150	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	200	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	250	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	300	4.94	5.11	5.43	5.58	6.68	7.40	4.00	4.13	4.39	4.51	6.43	7.12
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6

16	22	2.43	2.60	2.94	3.11	4.45	5.46	1.53	1.64	1.85	1.95	3.90	4.79
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	25	2.77	2.96	3.34	3.53	5.05	6.20	1.74	1.86	2.10	2.22	4.43	5.44
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	38	4.20	4.49	5.07	5.36	7.68	9.42	2.64	2.83	3.19	3.37	6.74	8.27
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	47	5.20	5.56	6.28	6.63	9.50	11.65	3.27	3.50	3.95	4.17	8.34	10.22
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	63	6.97	7.45	8.41	8.89	10.96	12.13	4.38	4.69	5.29	5.59	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	75	8.11	8.38	8.90	9.16	10.96	12.13	5.22	5.58	6.30	6.66	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	100	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	150	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	200	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	250	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	300	8.11	8.38	8.90	9.16	10.96	12.13	6.43	6.65	7.06	7.26	10.26	11.36
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	22	2.90	3.10	3.50	3.70	5.30	6.49	1.76	1.88	2.12	2.24	4.41	5.41
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	25	3.29	3.52	3.97	4.20	6.02	7.38	2.00	2.13	2.41	2.55	5.01	6.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	38	5.00	5.35	6.04	6.39	9.15	11.22	3.03	3.24	3.66	3.87	7.62	9.35
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	47	6.19	6.62	7.47	7.90	11.31	13.87	3.75	4.01	4.53	4.79	9.43	11.56
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	63	8.30	8.87	10.01	10.59	15.16	17.70	5.03	5.38	6.07	6.42	12.64	15.50
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	75	9.88	10.56	11.92	12.60	15.98	17.70	5.99	6.40	7.23	7.64	14.59	16.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	100	11.82	12.22	12.99	13.35	15.98	17.70	7.98	8.53	9.63	10.18	14.59	16.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	150	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	200	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15

Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	250	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
20	300	11.82	12.22	12.99	13.35	15.98	17.70	9.20	9.51	10.11	10.39	14.59	16.15
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6
24	22	3.30	3.53	3.99	4.21	6.04	7.40	1.93	2.06	2.33	2.46	4.79	5.88
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	25	3.75	4.01	4.53	4.79	6.86	8.41	2.20	2.35	2.65	2.80	5.44	6.68
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	38	5.71	6.10	6.89	7.28	10.43	12.79	3.34	3.57	4.03	4.26	8.28	10.15
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	47	7.06	7.54	8.52	9.00	12.90	15.82	4.13	4.41	4.98	5.27	10.24	12.55
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	63	9.46	10.11	11.42	12.07	17.29	21.20	5.53	5.91	6.68	7.06	13.72	16.83
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	75	11.26	12.04	13.59	14.37	20.58	23.95	6.59	7.04	7.95	8.40	16.33	20.03
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	100	15.01	16.05	17.57	18.07	21.62	23.95	8.78	9.39	10.60	11.20	19.26	21.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	150	16.00	16.54	17.57	18.07	21.62	23.95	12.23	12.65	13.44	13.82	19.26	21.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	200	16.00	16.54	17.57	18.07	21.62	23.95	12.23	12.65	13.44	13.82	19.26	21.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	250	16.00	16.54	17.57	18.07	21.62	23.95	12.23	12.65	13.44	13.82	19.26	21.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	300	16.00	16.54	17.57	18.07	21.62	23.95	12.23	12.65	13.44	13.82	19.26	21.33
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for
one 4.6 grade steel dowel in a 3-member glulam-steel-glulam joint

Service Class: 1

Load duration: Short term

Design situation: Normal

$k_{mod,combined}$ = 0.90

γ_M = 1.30

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Dowel diameter	Minimum glulam thickness	Direction of loading															
		Parallel to the grain								Perpendicular to the grain							
		Strength classes															
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	65	5.83	5.52	6.05	5.83	6.20	6.05	6.34	6.20	4.18	3.91	4.46	4.18	4.64	4.46	4.83	4.64
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	90	5.83	5.59	6.05	5.83	6.20	6.05	6.34	6.20	4.76	4.57	4.94	4.76	5.06	4.94	5.18	5.06
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	115	5.83	5.59	6.05	5.83	6.20	6.05	6.34	6.20	4.76	4.57	4.94	4.76	5.06	4.94	5.18	5.06
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	140	5.83	5.59	6.05	5.83	6.20	6.05	6.34	6.20	4.76	4.57	4.94	4.76	5.06	4.94	5.18	5.06
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	165	5.83	5.59	6.05	5.83	6.20	6.05	6.34	6.20	4.76	4.57	4.94	4.76	5.06	4.94	5.18	5.06
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	190	5.83	5.59	6.05	5.83	6.20	6.05	6.34	6.20	4.76	4.57	4.94	4.76	5.06	4.94	5.18	5.06
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
12	65	7.26	6.77	7.75	7.26	8.07	7.75	8.39	8.07	5.12	4.80	5.44	5.12	5.65	5.44	5.86	5.65
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	90	8.00	7.68	8.31	8.00	8.51	8.31	8.71	8.51	6.37	5.93	6.72	6.37	6.88	6.72	7.04	6.88
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6

12	115	8.00	7.68	8.31	8.00	8.51	8.31	8.71	8.51	6.47	6.21	6.72	6.47	6.88	6.72	7.04	6.88
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	140	8.00	7.68	8.31	8.00	8.51	8.31	8.71	8.51	6.47	6.21	6.72	6.47	6.88	6.72	7.04	6.88
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	165	8.00	7.68	8.31	8.00	8.51	8.31	8.71	8.51	6.47	6.21	6.72	6.47	6.88	6.72	7.04	6.88
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	190	8.00	7.68	8.31	8.00	8.51	8.31	8.71	8.51	6.47	6.21	6.72	6.47	6.88	6.72	7.04	6.88
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
16	65	10.15	9.52	10.77	10.15	11.18	10.77	11.60	11.18	7.20	6.80	7.59	7.20	7.86	7.59	8.12	7.86
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	90	12.53	11.68	13.39	12.53	13.96	13.39	14.28	13.96	8.50	7.96	9.04	8.50	9.40	9.04	9.76	9.40
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	115	13.12	12.59	13.63	13.12	13.96	13.63	14.28	13.96	10.04	9.35	10.72	10.04	11.07	10.72	11.32	11.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	140	13.12	12.59	13.63	13.12	13.96	13.63	14.28	13.96	10.41	9.99	10.81	10.41	11.07	10.81	11.32	11.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	165	13.12	12.59	13.63	13.12	13.96	13.63	14.28	13.96	10.41	9.99	10.81	10.41	11.07	10.81	11.32	11.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	190	13.12	12.59	13.63	13.12	13.96	13.63	14.28	13.96	10.41	9.99	10.81	10.41	11.07	10.81	11.32	11.07
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	65	13.39	12.64	14.14	13.39	14.64	14.14	15.14	14.64	9.59	9.11	10.06	9.59	10.37	10.06	10.68	10.37
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	90	15.91	14.89	16.94	15.91	17.62	16.94	18.30	17.62	10.78	10.15	11.41	10.78	11.83	11.41	12.24	11.83
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	115	18.86	17.55	19.88	18.86	20.35	19.88	20.82	20.35	12.35	11.55	13.14	12.35	13.67	13.14	14.19	13.67
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	140	19.13	18.36	19.88	19.13	20.35	19.88	20.82	20.35	14.11	13.15	15.07	14.11	15.71	15.07	16.21	15.71
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	165	19.13	18.36	19.88	19.13	20.35	19.88	20.82	20.35	14.90	14.30	15.47	14.90	15.85	15.47	16.21	15.85
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	190	19.13	18.36	19.88	19.13	20.35	19.88	20.82	20.35	14.90	14.30	15.47	14.90	15.85	15.47	16.21	15.85
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6

24	65	17.03	16.15	17.90	17.03	18.48	17.90	19.06	18.48	12.32	11.77	12.86	12.32	13.21	12.86	13.57	13.21
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	90	19.50	18.32	20.67	19.50	21.45	20.67	22.23	21.45	13.26	12.56	13.97	13.26	14.43	13.97	14.90	14.43
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	115	22.59	21.10	24.08	22.59	25.07	24.08	26.06	25.07	14.73	13.85	15.61	14.73	16.19	15.61	16.78	16.19
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	140	25.89	24.20	26.90	25.89	27.55	26.90	28.18	27.55	16.49	15.43	17.55	16.49	18.26	17.55	18.96	18.26
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	165	25.89	24.85	26.90	25.89	27.55	26.90	28.18	27.55	18.42	17.17	19.67	18.42	20.50	19.67	21.33	20.50
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	190	25.89	24.85	26.90	25.89	27.55	26.90	28.18	27.55	19.80	19.00	20.57	19.80	21.06	20.57	21.55	21.06
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for
one 4.6 grade steel dowel in a 3-member steel-glulam-steel joint

(For use in multiple shear plane connections. The steel plates should not be on the outside.)

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

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Dowel diameter	Minimum glulam thickness	Direction of loading															
		Parallel to the grain								Perpendicular to the grain							
		Strength classes															
		GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c	GL24h	GL24c	GL28h	GL28c	GL32h	GL32c	GL36h	GL36c
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
10	65	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	90	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	115	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	140	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	165	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
10	190	4.12	3.96	4.28	4.12	4.38	4.28	4.48	4.38	3.36	3.23	3.50	3.36	3.58	3.50	3.66	3.58
Steel thickness (mm) >		3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
12	65	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	90	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	115	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	140	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87

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Note this is a snapshot of an interactive spreadsheet

Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	165	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
12	190	5.66	5.43	5.88	5.66	6.02	5.88	6.16	6.02	4.57	4.39	4.75	4.57	4.87	4.75	4.98	4.87
Steel thickness (mm) >		3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
16	65	9.28	8.68	9.64	9.28	9.87	9.64	10.10	9.87	5.93	5.46	6.39	5.93	6.71	6.39	7.02	6.71
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	90	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	115	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	140	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	165	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
16	190	9.28	8.90	9.64	9.28	9.87	9.64	10.10	9.87	7.36	7.06	7.64	7.36	7.83	7.64	8.01	7.83
Steel thickness (mm) >		4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
20	65	11.22	10.33	12.10	11.22	12.69	12.10	13.28	12.69	6.80	6.26	7.34	6.80	7.69	7.34	8.05	7.69
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	90	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	9.41	8.67	10.16	9.41	10.65	10.16	11.15	10.65
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	115	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	140	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	165	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
20	190	13.53	12.99	14.05	13.53	14.39	14.05	14.72	14.39	10.53	10.11	10.94	10.53	11.20	10.94	11.46	11.20
Steel thickness (mm) >		6	6	6	6	6	6	6	6	6	6	6	6	6	6	6	6
24	65	12.79	11.78	13.80	12.79	14.47	13.80	15.14	14.47	7.48	6.89	8.07	7.48	8.46	8.07	8.86	8.46
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	90	17.71	16.31	19.02	17.71	19.48	19.02	19.93	19.48	10.35	9.54	11.17	10.35	11.72	11.17	12.26	11.72
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	115	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	13.23	12.19	14.28	13.23	14.89	14.28	15.24	14.89
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	140	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	165	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
24	190	18.31	17.57	19.02	18.31	19.48	19.02	19.93	19.48	14.00	13.44	14.54	14.00	14.89	14.54	15.24	14.89
Steel thickness (mm) >		7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2

Lateral design loads per shear plane for one 4.6 grade steel dowel in a 3-member LVL-steel-LVL joint

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

Design values | Characteristic values | Reset table | Return to menu

Dowel diameter	Minimum LVL thickness	Direction of loading: Parallel to the grain, LVL, KERTO Q	Parallel to the grain, LVL, KERTO S	Perp. to the grain, LVL, KERTO Q	Perp. to the grain, LVL, KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	2.63	2.63	2.05	2.05
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	33	2.87	2.87	2.18	2.18
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	39	3.14	3.14	2.34	2.34
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	45	3.45	3.45	2.52	2.52
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	51	3.77	3.77	2.72	2.72
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	57	3.89	3.89	2.93	2.93
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	63	3.89	3.89	3.15	3.15
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	75	3.89	3.89	3.21	3.21
Steel thickness (mm) >		2.5	2.5	2.5	2.5

CD2.5
Note this is a snapshot of an interactive spreadsheet

10	27	3.68	3.68	2.91	2.91
Steel thickness (mm) >		3	3	3	3
10	33	3.90	3.90	3.00	3.00
Steel thickness (mm) >		3	3	3	3
10	39	4.19	4.19	3.15	3.15
Steel thickness (mm) >		3	3	3	3
10	45	4.52	4.52	3.33	3.33
Steel thickness (mm) >		3	3	3	3
10	51	4.88	4.88	3.54	3.54
Steel thickness (mm) >		3	3	3	3
10	57	5.26	5.26	3.77	3.77
Steel thickness (mm) >		3	3	3	3
10	63	5.66	5.66	4.01	4.01
Steel thickness (mm) >		3	3	3	3
10	75	5.75	5.75	4.52	4.52
Steel thickness (mm) >		3	3	3	3
12	27	4.91	4.91	3.91	3.91
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	33	5.10	5.10	3.97	3.97
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	39	5.38	5.38	4.08	4.08
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	45	5.72	5.72	4.24	4.24
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	51	6.10	6.10	4.44	4.44
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	57	6.51	6.51	4.67	4.67
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	63	6.95	6.95	4.93	4.93
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	75	7.87	7.87	5.47	5.47
Steel thickness (mm) >		3.6	3.6	3.6	3.6
16	27	7.62	7.62	4.79	4.79
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	33	8.01	8.01	5.86	5.86
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	39	8.20	8.20	6.33	6.33
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	45	8.50	8.50	6.41	6.41
Steel thickness (mm) >		4.8	4.8	4.8	4.8

16	51	8.87	8.87	6.56	6.56
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	57	9.30	9.30	6.75	6.75
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	63	9.77	9.77	6.98	6.98
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	75	10.82	10.82	7.53	7.53
Steel thickness (mm) >		4.8	4.8	4.8	4.8
20	27	9.07	9.07	5.50	5.50
Steel thickness (mm) >		6	6	6	6
20	33	11.09	11.09	6.72	6.72
Steel thickness (mm) >		6	6	6	6
20	39	11.65	11.65	7.94	7.94
Steel thickness (mm) >		6	6	6	6
20	45	11.84	11.84	9.04	9.04
Steel thickness (mm) >		6	6	6	6
20	51	12.14	12.14	9.08	9.08
Steel thickness (mm) >		6	6	6	6
20	57	12.52	12.52	9.20	9.20
Steel thickness (mm) >		6	6	6	6
20	63	12.98	12.98	9.36	9.36
Steel thickness (mm) >		6	6	6	6
20	75	14.04	14.04	9.83	9.83
Steel thickness (mm) >		6	6	6	6
24	27	10.34	10.34	6.05	6.05
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	33	12.64	12.64	7.39	7.39
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	39	14.94	14.94	8.74	8.74
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	45	15.75	15.75	10.08	10.08
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	51	15.92	15.92	11.43	11.43
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	57	16.21	16.21	12.03	12.03
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	63	16.59	16.59	12.10	12.10
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	75	17.58	17.58	12.41	12.41
Steel thickness (mm) >		7.2	7.2	7.2	7.2

Lateral design loads per shear plane for
one 4.6 grade steel dowel in a 3-member steel-LVL-steel joint

(For use in multiple shear plane connections. The steel plates should not be on the outside.)

Service Class	1	$k_{mod,combined}$ =	0.90
Load duration	Short term	γ_M =	1.30
Design situation	Normal		

Design values | Characteristic values | Reset table | Return to menu

Dowel diameter	Minimum LVL thickness	Direction of loading: Parallel to the grain, LVL, KERTO Q	Parallel to the grain, LVL, KERTO S	Perp. to the grain, LVL, KERTO Q	Perp. to the grain, LVL, KERTO S
(mm)	(mm)	(kN)	(kN)	(kN)	(kN)
8	27	2.09	2.09	1.42	1.42
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	33	2.55	2.55	1.74	1.74
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	39	2.75	2.75	2.05	2.05
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	45	2.75	2.75	2.27	2.27
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	51	2.75	2.75	2.27	2.27
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	57	2.75	2.75	2.27	2.27
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	63	2.75	2.75	2.27	2.27
Steel thickness (mm) >		2.5	2.5	2.5	2.5
8	75	2.75	2.75	2.27	2.27

Steel thickness (mm) >		2.5	2.5	2.5	2.5
10	27	2.55	2.55	1.70	1.70
Steel thickness (mm) >		3	3	3	3
10	33	3.12	3.12	2.08	2.08
Steel thickness (mm) >		3	3	3	3
10	39	3.69	3.69	2.46	2.46
Steel thickness (mm) >		3	3	3	3
10	45	4.07	4.07	2.84	2.84
Steel thickness (mm) >		3	3	3	3
10	51	4.07	4.07	3.21	3.21
Steel thickness (mm) >		3	3	3	3
10	57	4.07	4.07	3.32	3.32
Steel thickness (mm) >		3	3	3	3
10	63	4.07	4.07	3.32	3.32
Steel thickness (mm) >		3	3	3	3
10	75	4.07	4.07	3.32	3.32
Steel thickness (mm) >		3	3	3	3
12	27	2.99	2.99	1.96	1.96
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	33	3.66	3.66	2.39	2.39
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	39	4.33	4.33	2.83	2.83
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	45	4.99	4.99	3.26	3.26
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	51	5.58	5.58	3.70	3.70
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	57	5.58	5.58	4.13	4.13
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	63	5.58	5.58	4.51	4.51
Steel thickness (mm) >		3.6	3.6	3.6	3.6
12	75	5.58	5.58	4.51	4.51
Steel thickness (mm) >		3.6	3.6	3.6	3.6
16	27	3.81	3.81	2.40	2.40
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	33	4.66	4.66	2.93	2.93
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	39	5.50	5.50	3.46	3.46
Steel thickness (mm) >		4.8	4.8	4.8	4.8

16	45	6.35	6.35	3.99	3.99
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	51	7.20	7.20	4.53	4.53
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	57	8.05	8.05	5.06	5.06
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	63	8.89	8.89	5.59	5.59
Steel thickness (mm) >		4.8	4.8	4.8	4.8
16	75	9.16	9.16	6.66	6.66
Steel thickness (mm) >		4.8	4.8	4.8	4.8
20	27	4.54	4.54	2.75	2.75
Steel thickness (mm) >		6	6	6	6
20	33	5.55	5.55	3.36	3.36
Steel thickness (mm) >		6	6	6	6
20	39	6.55	6.55	3.97	3.97
Steel thickness (mm) >		6	6	6	6
20	45	7.56	7.56	4.58	4.58
Steel thickness (mm) >		6	6	6	6
20	51	8.57	8.57	5.19	5.19
Steel thickness (mm) >		6	6	6	6
20	57	9.58	9.58	5.80	5.80
Steel thickness (mm) >		6	6	6	6
20	63	10.59	10.59	6.42	6.42
Steel thickness (mm) >		6	6	6	6
20	75	12.60	12.60	7.64	7.64
Steel thickness (mm) >		6	6	6	6
24	27	5.17	5.17	3.02	3.02
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	33	6.32	6.32	3.70	3.70
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	39	7.47	7.47	4.37	4.37
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	45	8.62	8.62	5.04	5.04
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	51	9.77	9.77	5.71	5.71
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	57	10.92	10.92	6.39	6.39
Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	63	12.07	12.07	7.06	7.06

Steel thickness (mm) >		7.2	7.2	7.2	7.2
24	75	14.37	14.37	8.40	8.40
Steel thickness (mm) >		7.2	7.2	7.2	7.2

CD3 Provision of restraint against the rotation of individual timber frame walls

CD3.1 Introduction

There are two ways to ensure that a timber frame building can resist the overturning moments applied to it by wind forces. The upper bound approach in Section 10.5.2 of the *Manual* checks the overturning resistance of the building as a whole, assuming that it acts as a rigid box, and where necessary the building is fastened to the foundation along the base of the windward wall. The lower bound approach in Section 10.8.1.5 checks the overturning resistance of the individual walls, and ensures their stability by requiring tie-downs, where necessary, at the end of each individual racking wall. If the continuity of floor and ceiling diaphragms across openings can make the walls act as if they were continuous across the building to the hinge line, and if all the connections between the vertical and horizontal diaphragms are strong enough to mobilise the entire weight of the building in overturning, then the building will indeed act as a rigid box and the two approaches will give the same result.

EC0, however, states that the stability of both the whole building and the individual elements of it should be demonstrated. In particular, the design method for timber frame walls implies that individual walls should be restrained against the overturning forces acting on them. This document therefore shows how the rotational stability of individual racking walls can be ensured.

As stated in Section 10.8.1.5 of the *Manual*, a properly constructed timber frame wall can be restrained against rotation by means of a vertical restraint force applied to its windward end, by an equivalent vertical load applied to the top of the wall by the structure above it, by the connection of its bottom rail to the foundation or wall beneath it, or by a combination of these.

Any of the following may be utilised.

- *Connection of an end wall to an adjacent return wall.* The weight of a limited length of return wall and any vertical load on it and its attachment to the foundation may all be utilised to hold down the end of a wall.
- *Connection of an intermediate wall to a stud supporting the lintel over a doorway or window opening.* Half the vertical load on the lintel and the weight of a limited length of wall panel beneath a window and its attachment to the foundation may all be utilised.
- *Connection of bottom rail to the foundation or the wall beneath it.*
- *Straps and brackets.* Vertical restraint straps or bolted brackets with adequate strength and stiffness may be specified to attach the wall to the foundation or the wall beneath.

The designer is at liberty to choose which of the various elements are utilised.

Where intermediate walls end at openings for doors or windows, the designer may be able to demonstrate that the continuity provided by the superstructure (floor or ceiling diaphragm) will provide the necessary residual constraint against overturning.

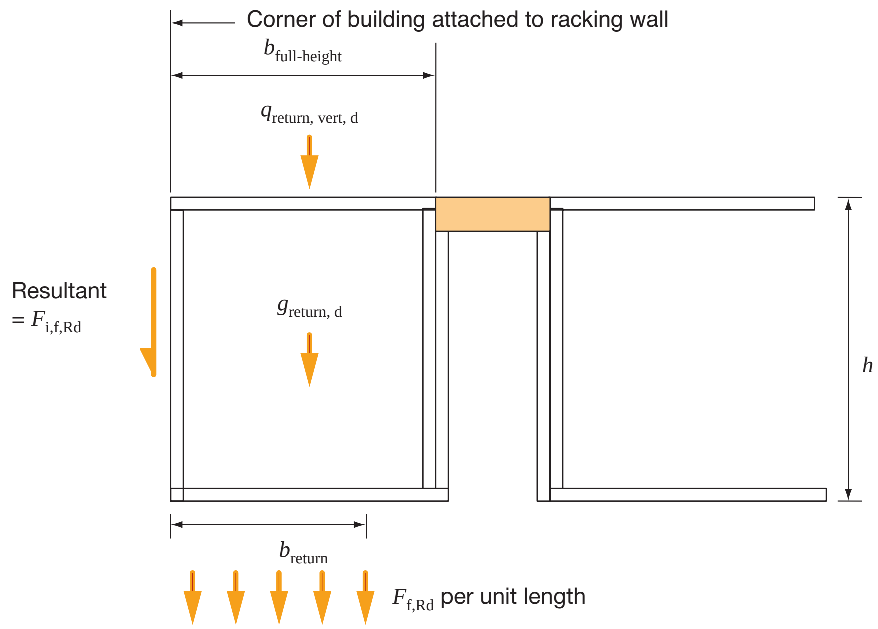


Figure CD3.1 Dimensions for return wall

CD3.2 Connection to a return wall panel

The vertical restraint, $F_{i,f,Rd}$, which can be applied to the end of wall via its connection to a return wall, can be calculated as:

$$F_{i,f,Rd} = b_{return}(q_{return,vert,d} + g_{return,d} + F_{f,Rd}) \text{ (kN)}$$

Where $b_{return} = \min.\begin{cases} h \\ b_{full-height} \end{cases}$ (m)

h = height of wall panel (m)

$b_{full\text{-}height}$ = actual length of uninterrupted full height return wall adjacent to corner (m)

$q_{return,vert,d}$ = design value of uniformly distributed vertical load per unit length onto the return wall comprising permanent loads minus the vertical component of any wind uplift (kN/m)

$g_{return,d}$ = design value of dead weight of timber frame return wall per unit length (kN/m)

$F_{f,Rd}$ = effective fastener resistance per unit length

$= \dfrac{F_{f,Rd,1}}{s_1} + \dfrac{0.5F_{f,Rd,2}}{s_2}$ (kN/m)

$F_{f,Rd,1}$, $F_{f,Rd,2}$ = lateral design capacity of one fastener in sheathings 1 and 2 respectively in the return wall (kN)

s_1, s_2 = fastener spacings in sheathings 1 and 2 respectively in the return wall (m)

Values of $F_{f,Rd}$ for certain standard wall configurations can be obtained directly from the *Manual*, Table 10.9. If the term $F_{f,Rd}$ is utilised, the fasteners attaching the bottom rail of the return panel to the substrate should be either ringed shank nails or screws, with a design withdrawal resistance equal to at least $F_{f,Rd}$ kN/m (see the *Manual*, Table 10.8).

The connections between the end of the return wall and the end of the racking wall should have a design resistance of at least $F_{i,f,Rd}$ kN/m. Values for some standard fixings are given in the *Manual*, Tables 10.5 and 10.6.

CD3.3 Connection to a stud supporting an opening

The vertical restraint, $F_{i,f,Rd}$, which can be applied to the end of wall via its connection to a stud supporting a lintel over an opening, can be calculated as:

$$F_{i,f,Rd} = 0.5b_{opening}q_{i,vert,d} + b_{restraint}(g_{wall,d} + F_{f,Rd}) \text{ (kN)}$$

Where $b_{opening}$ = width of opening (m)

$q_{i,vert,d}$ = design value of uniformly distributed vertical load per unit length onto the wall comprising permanent loads minus the vertical component of any wind uplift (kN/m)

$b_{restraint} = \min.\begin{cases} b_{restraint,v} \\ b_{restraint,h} \end{cases}$ for a window opening (m)

= 0.0 for a door opening

$b_{restraint,v}$ = height of panel to bottom of window (m) (see Figure CD3.2)

$b_{restraint,h}$ = width of window (m) (see Figure CD3.2)

$g_{wall,d}$ = design value of dead weight of reduced height timber frame wall per unit length (kN/m)

$F_{f,Rd}$ = effective fastener resistance per unit length (kN/m)

If the term involving $F_{f,Rd}$ is utilised for a window opening, the fasteners attaching the bottom rail of the panel to the substrate should be either ringed shank nails or screws, with a design withdrawal resistance equal to at least $F_{f,Rd}$ kN/m (see the *Manual*, Table 10.8).

The connections between the end of the return wall and the end of the racking wall should have a design resistance of at least $F_{i,f,Rd}$ kN/m. Values for some standard fixings are given in the *Manual*, Tables 10.5 and 10.6.

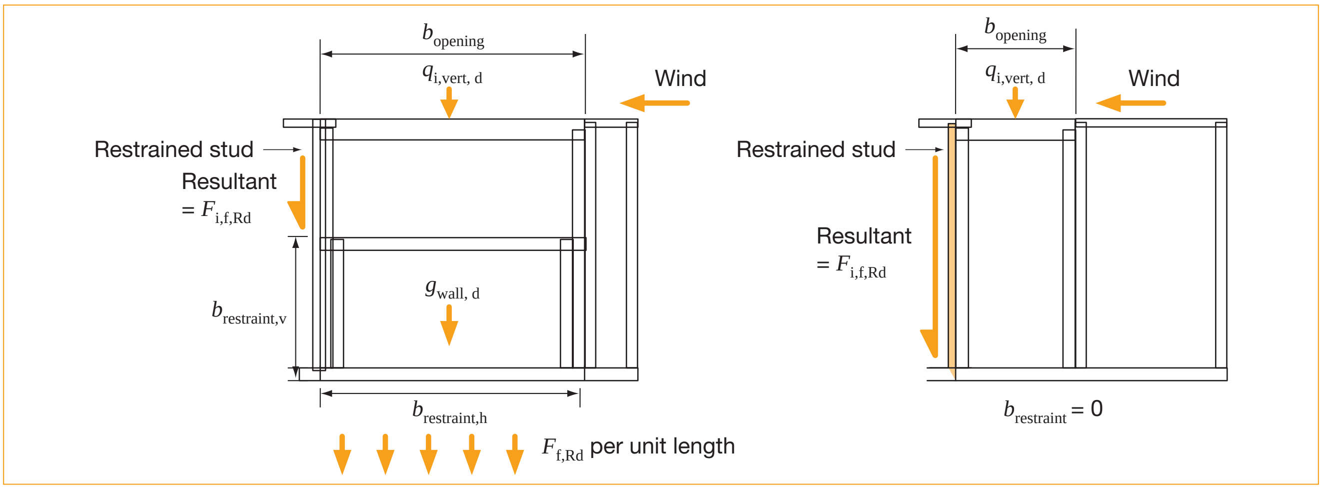


Figure CD3.2 Dimensions for openings

CD3.4 Connection of bottom rail to the foundation or wall beneath

Some overturning resistance will inevitably be provided by the attachment of the sheathing to the bottom rail of the timber framing, and by the attachment of the bottom rail to the foundation of the building or the top of the wall beneath. In the test method on which the racking resistance expressions given in EC5 Method B were based, the bottom rail was bolted down and the nails attaching the sheathing to the bottom rail and leading stud resisted all the overturning moment. It may therefore be assumed that any overturning moment which does not exceed the calculated shear capacity of the wall can be resisted by the sheathing-to-frame connection, but the connection between the bottom rail and the substructure must be designed to provide the required resistance.

Assuming that the wall acts as a rigid assembly rotating about one corner, fasteners distributed along the bottom rail can provide a design resisting moment of $R_{\mathrm{dist,d}}\, b^2/3$ Nmm, where $R_{\mathrm{dist,d}}$ is the design resistance of the fasteners to vertical load in N/mm and b is the length of the wall (between openings or returns) in mm.

Table 10.8 in the *Manual* gives the design axial load capacities of some fasteners commonly used to connect the bottom rail of a wall panel to a timber wall plate. Figure CD3.3 shows some brackets used to attach wall plates to the foundation. If such brackets are relied on to restrain the rotation of individual walls they should be spaced no more than 600mm apart.

CD3.5 Restraint straps and brackets

Vertical restraint straps or bolted brackets with adequate strength and stiffness may be specified on the end studs at openings and corners to attach the wall to the foundation or the wall beneath (see Figures CD3.4 and CD3.5). Consideration should be given to the effects of shrinkage across intermediate floors and to the true stiffness of foundation straps as commonly installed. Tables 10.5 and 10.6 in the *Manual* give load capacities for nails in steel straps. Typical straps are made from steel to BS EN 10142; Fe PO2 G 1.5mm to 3mm thick and from 20mm to 75mm wide. Angle brackets may be up to about 6mm thick.

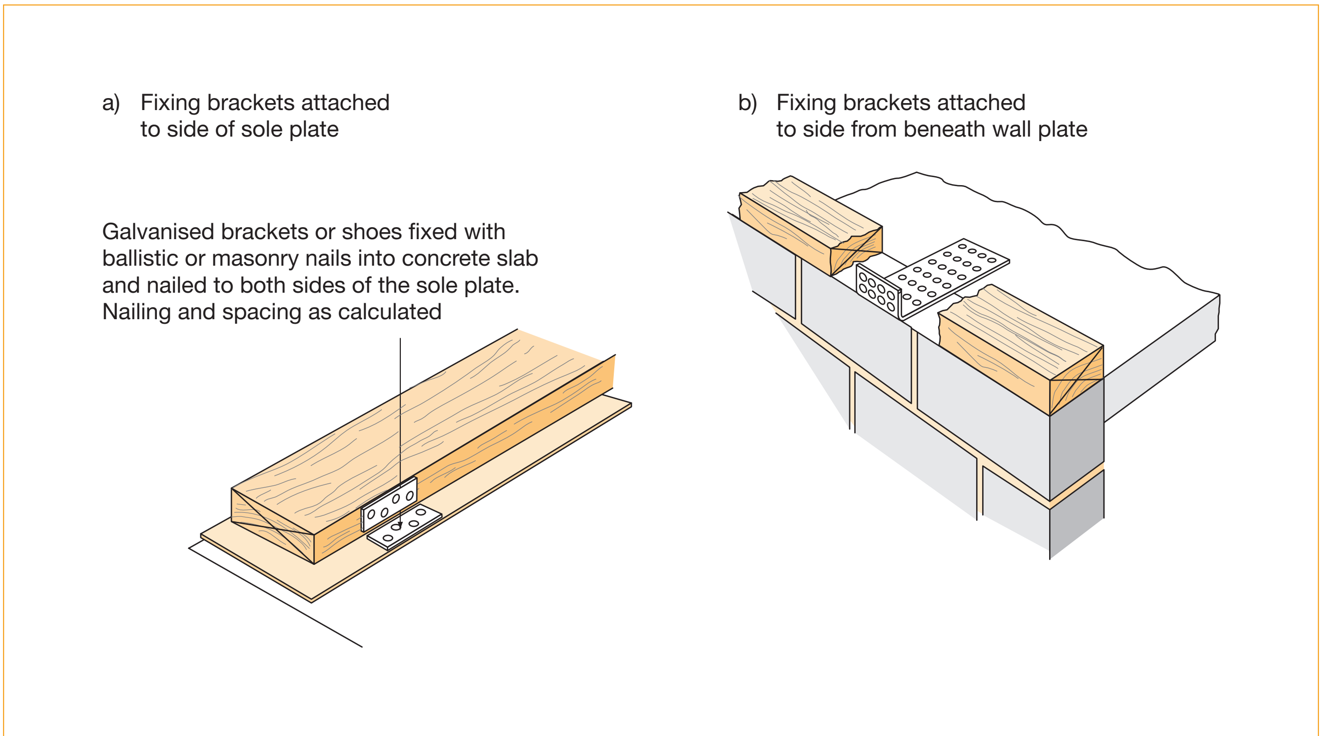
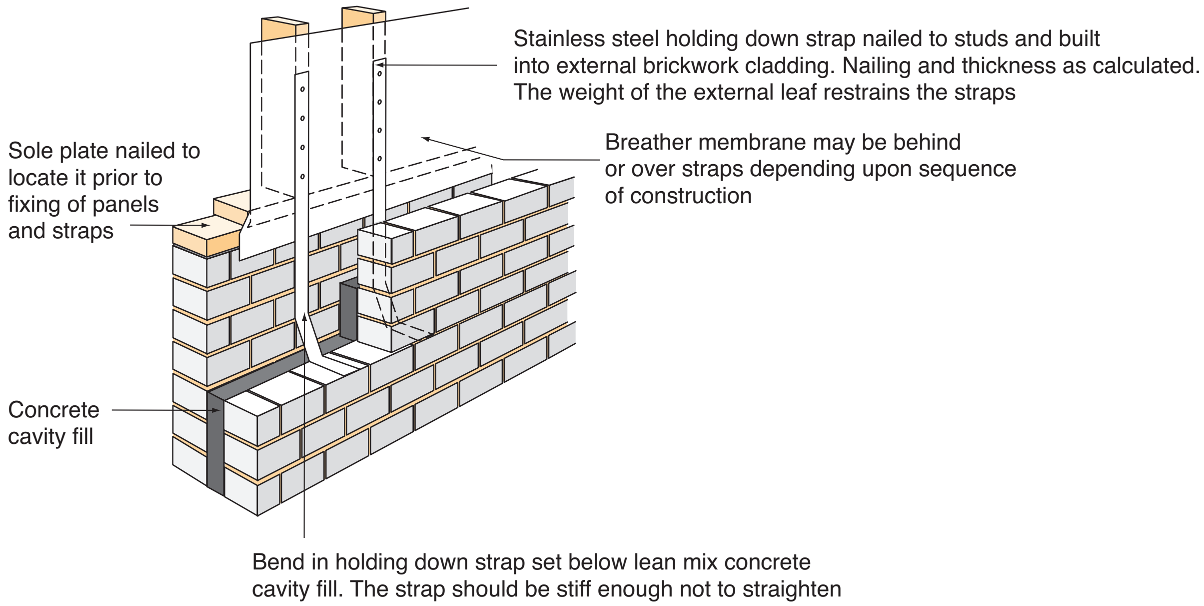


Figure CD3.3 Some methods of attaching a wall plate to the foundation

a) Holding down straps showing correct installation method



b) Holding down bolts and brackets

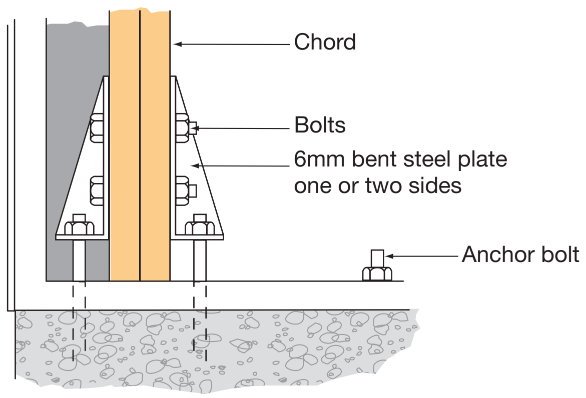


Figure CD3.4 Some methods of connecting the end of a wall to the foundation

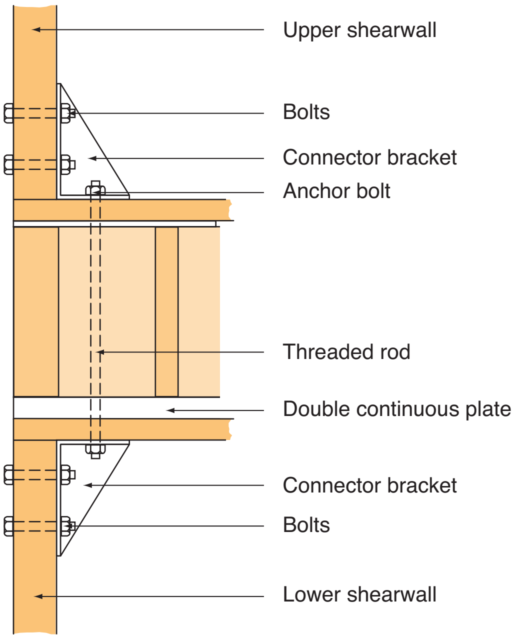


Figure CD3.5 A suggested method for connecting the end of a wall to the floor beneath

CD4 Design values for timber connectors

Timber connectors are used in conjunction with bolts to transfer shear load between two or more members placed side by side. The design values for timber connectors given by EC5 are unproven for the types of connector used in the UK, and a comparison of the design values given by EC5 and BS 5268-2:2002 indicates that in many cases the values given by EC5 are unsafe. It is recommended that until the EC5 rules have been reviewed, design values for connectors should continue to be based on information from BS 5268-2. This document gives a method for converting the basic connector loads given in BS 5268 to design values for use in a building which is otherwise designed to EC5.

BS 5268-2 tabulates 'basic loads' for one connector unit. A basic load has to be modified by various factors to obtain a BS 5268 permissible load value. A connector unit is defined as one of:

- a single-sided sided toothed plate and bolt, in a steel-to-timber joint
- a double-sided toothed plate and bolt or two single-sided toothed plates back-to-back on a single bolt, in a timber-to-timber joint
- a split ring and bolt, in a timber-to-timber joint
- a single-sided shear plate and bolt, in a steel-to-timber joint
- two single-sided shear plates plates back-to-back on a single bolt, in a timber-to-timber joint.

BS 5268-2 gives rules to allow for the effects on the basic load of service class, spacing, edge and end distances, angle of load to grain and the number of connectors in a line. These rules should be applied to the appropriate basic load tabulated in BS 5268-2 to obtain a 'basic design load', $R_{basic,d}$, for one connector unit. Provided that the BS 5268 rules for minimum spacings and distances are complied with and the appropriate reductions for sub-standard dimensions are applied, it is unnecessary to consider the corresponding rules for spacings and distances given in EC5.

The Eurocode 5 design load-carrying capacity for the connection is then calculated as:

$$R_d = nk_c R_{basic,d}$$

Where n = total number of connector units

k_c = factor from Table CD4.1 to take account of the differing load duration and safety factors used in the two codes

$R_{basic,d}$ = BS 5268 basic load per connector unit modified for sub-standard spacing, end and edge distance, service class, angle of load to grain and number of connectors in a line, according to BS 5268-2 rules

Table CD4.2 gives some indicative values of R_d for a three-member timber-to-timber connection made with two connector units on one bolt, to assist the designer in choosing a suitable connector.

Table CD4.1 Recommended k_c factors to convert BS 5268-2 basic design loads for timber connectors used in service classes 1 and 2 to Eurocode design capacities

Connector type	EC5 load duration			
	Long-term (e.g. storage)	Medium-term (e.g. floor imposed)	Short-term (e.g. snow)	Instantaneous (e.g. wind)
Toothed plates	1.35	1.35	1.51	1.69
Shear plates and split rings	1.35	1.35	1.69	2.03

Table CD4.2 Examples of short-term design capacities for a single 3-member C16 timber connection[a]

Connector type	Nominal size (mm)	Design load capacity under short-term loading in service classes 1 and 2 (kN)	
		Load parallel to grain	Load perpendicular to grain
Round toothed plate	38	6.80	4.65
	50	10.3	7.10
	63	12.2	9.06
	75	13.6	10.5
Split ring	64	26.7	18.7
	102	49.8	34.7
Shear plate	67	28.9	20.2
	102	40.2	28.2

Note

a The values shown are for a 3-member connection in C16 timber, with the central member 72mm thick and the two outer members each 44mm thick. The values are for the complete connection, i.e. for two shear planes or connector units, and are derived from BS 5268-2, factored in accordance with Table CD4.1.

CD5 Contact details for the manufacturers of some timber construction products

This list of manufacturers is included for the convenience of designers and specifiers. Its purpose is to provide them with rapid access to sources of additional design information for commonly used timber construction products, such as span tables for prefabricated timber joists. The list is not comprehensive, and inclusion of a manufacturer in the list does not imply any endorsement or recommendation of that manufacturer's products.

Prefabricated timber joists

Boise Cascade Corporation
Boise Cascade Sales Ltd
Lexham House
Hill Avenue
Amersham
Buckinghamshire
HP6 5BW
UK
T: +44 (0)1494 434222
F: +44 (0)1494 431557

Finnforest UK Ltd
The Heights
59/65 Lowlands Road
Harrow on the Hill
Middlesex
HA1 3AE
UK
T: +44 (0)20 8420 0777
F: +44 (0)20 8422 9369
W: www.finnforest.co.uk
For all general product and stockist information please call
T: +44 (0)20 8437 8369

Gang-Nail Systems Ltd
Christy Estate
Ivy Road
Aldershot
GU12 4XG
UK
W: www.eleco.com/gang-nail
T: +44 (0)1252 334 691
F: +44 (0)1252 334 562

James Jones & Sons Limited
Timber Systems Division
Greshop Industrial Estate
Forres
Scotland
IV36 2GW
UK
W: www.jji-joists.com
T: +44 (0)1309671111
F: +44 (0)1309 671720

MiTek Industries Ltd
MiTek House
Grazebrook Industrial Park
Peartree Lane
Dudley
DY2 0XW
UK
W: www.mitek.co.uk
T: +44 (0)1384 451 400
F: +44 (0)1384 451 411

Swelite AB (Masonite Beams)
P.O. Box 5
SE-914 29
Rundvik
Sweden
T: +46 930 142 00
F: +46 930 307 86
W: www.masonite-beams.se

TrusJoist
East Barn
Perry Mill Farm
Birmingham Road
Hopwood
Worcestershire
B48 7AL
W: www.trusjoist.co.uk
T: +44 (0)121 445 6666
F: +44 (0)121 445 6677

Wolf Systems Ltd
Shilton Industrial Estate
Bulkington Road
Shilton
Coventry
CV7 9QL
UK
W: www.wolf-system.co.uk
T: +44 (0)24 7660 2303
F: +44 (0)24 7660 2243

Trussed rafter systems

Alpine Automation Ltd
Threemilestone
Truro
TR4 9LD
UK
W: www.alpineuk.com
T: +44 (0)1872 245 450
F: +44 (0)1872 245 451

Gang-Nail Systems Ltd
Christy Estate
Ivy Road
Aldershot
GU12 4XG
UK
W: www.eleco.com/gang-nail
T: +44 (0)1252 334 691
F: +44 (0)1252 334 562

MiTek Industries Ltd
MiTek House
Grazebrook Industrial Park
Peartree Lane
Dudley
DY2 0XW
UK
W: www.mitek.co.uk
T: +44 (0)1384 451 400
F: +44 (0)1384 451 411

Wolf Systems Ltd
Shilton Industrial Estate
Bulkington Road
Shilton
Coventry
CV7 9QL
UK
W: www.wolf-system.co.uk
T: +44 (0)24 7660 2303
F: +44 (0)24 7660 2243

Timber engineering hardware

Cowley Structural Timberwork Ltd
The Quarry
Grantham Road
Waddington
Lincoln
Lincolnshire
LN5 9NT
UK
W: www.cowleytimberwork.co.uk
T: +44 (0)1522 720913
F: +44 (0)1522 722778

Cullen Building Products Ltd
1 Wheatstone Place
Southfield Industrial Estate
Glenrothes
Fife KY6 2SW
UK
W: www.cullen-bp.co.uk
T: +44 (0)1592 771 132
F: +44 (0)1592 771 182

Expamet Building Products Ltd (*BAT*)
Greatham Street
Longhill Industrial Estate (North),
Hartlepool
TS25 1PR
UK
W: www.expamet.co.uk
T: Sales: +44 (0)1429 866611
T: Technical: +44 (0)1429 866655
F: +44 (0)1429 866633

Simpson Strong-Tie Ltd
Winchester Road
Cardinal Point
Tamworth
Staffordshire B78 3HG
UK
W: www.strongtie.co.uk
T: +44 (0)1827 255 600
F: +44 (0)1827 255 629

Teco Building Products
Wellington Road
Portslade, Brighton
East Sussex
BN41 1DN
UK
T: +44 (0)1273 416617
F: +44 (0)1273 410074

CD6 Contact information of sponsor organisations.

British Woodworking Federation

The British Woodworking Federation (BWF) is recognised as the leading representative body for the woodworking industry and builders' joinery and represents leading manufacturers, distributors and installers of doors, windows, conservatories, staircases, architectural joinery, timber frame buildings and engineered timber components. It provides a complete range of services to its Members including expert advice on technical issues, employment and contractual law, health and safety, tax issues, environmental matters.

British Woodworking Federation
55 Tufton Street
London
United Kingdom
SW1P 3QL
Tel: + 44 (0)870 458 6939
Fax: + 44 (0)870 458 6949
Email: bwf@bwf.org.uk
Web: www.bwf.org.uk

Canada Wood UK

Canada Wood is a collaboration between Canadian wood products industry associations and government for the creation of a single global representation. Industry partners include the Canadian Lumbermen's Association, Canadian Plywood Association, Council of Forest Industries, Forest Products Association of Canada, Quebec Wood Export Bureau and Western Red Cedar Export Association. The Maritime Lumber Bureau, National Lumber Grades Authority and Ontario Lumbermen's Association are represented also by Canada Wood in the UK.

Canada Wood serves as a link to approximately 10% of the world's forest area and to Canada's wealth of timber and wood products, and in the UK aims to provide existing and new trade partners with the support that is available through our information resources and marketing initiatives

Canada Wood UK
PO Box 1
Farnborough, Hants
GU14 6WE
United Kingdom

Tel: + 44 (0)1252 522 545
Fax: + 44 (0)1252 522 546
Email: office@canadawooduk.org
Web: www.canadawood.info

James Donaldson & Sons Ltd
Suite A, Haig House
Haig Business Centre
Balgonie Road
Markinch
United Kingdom
KY7 6AQ
Tel: + 44 (0)1592 752 244
Fax: + 44 (0)1592 752 277
Email: info@donaldsontimber.com
Web: www.donaldsontimber.com

MiTek Industries
Mitek House
Peartree Lane
Grazebrook Industrial Park
Dudley
United Kingdom
DY2 0XW
Tel: + 44 (0)1384 451 400
Fax: + 44 (0)1384 451 411
Web: www.mii.com

Sinclair Knight Merz
SKM Anthony Hunt
OneSixty
160 Dundee Street
Edinburgh
United Kingdom
EH11 1DQ
Tel: +44 (0)131 222 3530
Fax: +44 (0)131 222 3531
Web: www.skmconsulting.com

TrusJoist
East Barn
Perry Mill Farm
Birmingham Road
Hopwood
Worcestershire
United Kingdom
B48 7AJ
Tel: + 44 (0)121 445 6666
Fax: + 44 (0)121 445 6677
Web: www.trusjoist.co.uk