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Recommendations for the design of timber structures to Eurocode 5: Design of timber structures

**Part 1: General – Common rules and
rules for buildings**

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Contents

0	Introduction	1
1	Scope	1
2	Normative references	1
3	Terms, definitions and abbreviations	1
4	Design responsibilities	2
5	Effective anchorages of timber floors to walls for buildings of Consequence Class 2a	2
6	Assignment of timbers to BS EN 338 strength classes [BS EN 1995-1-1:2004+A1:2008, 3.2]	3
7	Horizontally glued laminated hardwood members [BS EN 1995-1-1:2004+A1:2008, 3.3]	3
8	Factor $k_{c,90}$ for compression perpendicular to grain [BS EN 1995-1-1:2004+A1:2008, 6.1.5]	4
9	Effective lengths of compression members [BS EN 1995-1-1:2004+A1:2008, 6.3.2]	5
10	Limits on notches and circular holes in joists and studs for which no calculations are required	6
11	Design of beams with circular holes	6
12	Characteristic properties of fasteners	7
13	Yield moment of annular ring-shanked nails [BS EN 1995-1-1:2004+A1:2008, 8.3.1.1(4)]	7
14	Diameters for evaluating lateral load-carrying capacities of screws [BS EN 1995-1-1:2004+A1:2008, 8.7.1]	8
15	Axially loaded screws [BS EN 1995-1-1:2004+A1:2008, 8.7.2]	8
16	Connections made with punched metal plate fasteners [BS EN 1995-1-1:2004+A1:2008, 8.8.1]	9
17	Misalignment tolerances in punched metal plate fastener joints [BS EN 1995-1-1:2004+A1:2008, 8.8.5.1]	9
18	Contact pressure between timber members in punched metal plate fastener joints under compression [BS EN 1995-1-1:2004+A1:2008, 8.8.5.1(3)]	9
19	Trusses with punched metal plate fasteners [BS EN 1995-1-1:2004+A1:2008, 9.2.2]	10
20	Masonry shielding to wall diaphragms	11
21	Simplified analysis of wall diaphragms [BS EN 1995-1-1:2004+A1:2008, 9.2.4.3]	11
22	Contribution of plasterboard to racking resistance	22
23	Evaluation of design racking resistance of plasterboard-clad timber frame walls	23
24	Bracing to trussed rafter roofs [BS EN 1995-1-1:2004+A1:2008, 9.2.5.3]	23
25	Lateral load-carrying capacity of glued lap joints [BS EN 1995-1-1:2004+A1:2008, 10.3]	23

Annexes

Annex A (normative)	Exchange of information between building designer and component designer(s)	25
Annex B (informative)	Effective anchorage of floors to timber frame wall buildings of Consequence Class 2a	26
Annex C (informative)	Actions and combinations of actions that may be considered in the design of trussed rafters	28
Annex D (informative)	Masonry shielding to timber frame wall diaphragms	31
Annex E (normative)	Bracing of trussed rafter roofs	32
Annex F (informative)	Optional recommendations for the support of water tanks in trussed rafter roofs	57

Bibliography	59
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List of figures

- Figure 1 – Division of racking wall into wall diaphragms 13
- Figure 2 – Forces transmitted into underlying construction by bottom rail of wall diaphragm 16
- Figure 3 – Calculation of design stabilizing and destabilizing moments 19
- Figure 4 – Division of wall diaphragms into wall panels 21
- Figure B.1 – Details of effective anchorage of floors to timber frame walls in buildings of Consequence Class 2a 27
- Figure D.1 – Area of brickwork providing wind shield to timber frame structure 32
- Figure E.1 – Procedure for the design of roof bracing at rafter level 34
- Figure E.2 – Procedure for the design of roof bracing at ceiling level 35
- Figure E.3 – Standard bracing for rafter and web members of duopitch trussed rafters 36
- Figure E.4 – Standard bracing for rafter and web members of mono-pitch trussed rafters 39
- Figure E.5 – Limiting spans for standard bracing of trussed rafter roofs 42
- Figure E.6 – Basic wind zones for buildings at site altitudes ≤ 150 m 45
- Figure E.7 – Basic wind zones for buildings at site altitudes between 150 m and 300 m 46
- Figure E.8 – Wall plate splice joint 49
- Figure E.9 – Standard bracing for rafter members: detail C1 and D1 50
- Figure E.10 – Standard bracing for rafter members: detail C2 and D2 51
- Figure E.11 – Standard bracing for rafter members: detail C3 53
- Figure E.12 – Standard bracing for rafter members: detail D3 splice connection and D4 crossing connection 55
- Figure F.1 – Supports for water tanks 57

List of tables

- Table 1 – Assignment of temperate hardwoods to BS EN 338 strength classes 3
- Table 2 – Modification factors k_{lam} for characteristic strengths, stiffnesses and densities of hardwood glued laminated timber applicable for service classes 1 and 2 4
- Table 3 – Examples of hardwoods suitable for glue laminating 4
- Table 4 – Effective lengths of compression members 5
- Table 5 – Effective lengths of compression members in trussed rafters 5
- Table 6 – Maximum lengths of chord and internal members 10
- Table 7 – Modification factor, K_e , to account for loading eccentricities in girder trusses 11
- Table 8 – Values of sheathing combination factor, K_{comb} 17
- Table 9 – Total design shear capacities per unit length of the perimeter fasteners for various specifications of plasterboard 23
- Table C.1 – Summary of actions for duo-pitch and mono-pitch trussed rafters 28
- Table C.2 – Summary of action combinations for duo-pitch and mono-pitch trussed rafters 29
- Table E.1 – Thickness and fixing of sarking materials 33
- Table E.2 – Maximum truss spans for Figure E.5 43
- Table E.3 – Maximum design cumulative surface wind pressures (kN/m^2) on windward and leeward gable walls for roofs constructed using the details of Figure E.10 47
- Table E.4 – Maximum design cumulative surface wind pressures (kN/m^2) on windward and leeward gable walls for roofs constructed using the details of Figure E.11 and E.12 47
- Table E.5 – Maximum design horizontal wind force (kN/m) at bottom chord level on 12.5 mm thick plasterboard ceiling diaphragms 48

Table E.6 – Maximum design horizontal wind force (kN/m) at bottom chord level on 15 mm thick plasterboard ceiling diaphragms	48
Table F.1 – Sizes for support members for water tanks	58

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 60, an inside back cover and a back cover.

Foreword

This Published Document is published by BSI Standards Limited, under licence from The British Standards Institution and came into effect on 31 October 2012. It was prepared by Subcommittee B/525/5, *Structural use of timber*, under the authority of Technical Committee B/525, *Building and civil engineering structure*. A list of organizations represented on this committee can be obtained on request to its secretary.

Use of this document

The start and finish of text introduced or altered by Corrigendum No. 1 is indicated in the text by tags **[C1]** and **[C1]**. Minor editorial changes are not tagged.

As a guide, this Published Document takes the form of guidance and recommendations. It should not be quoted as if it were a specification or a code of practice and claims of compliance cannot be made to it.

Presentational conventions

The guidance in this standard is presented in roman (i.e. upright) type. Any recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a Published Document cannot confer immunity from legal obligations.

0 Introduction

When there is a need for guidance on a subject that is not covered by the Eurocode, a country can choose to publish documents that contain non-contradictory complementary information that supports the Eurocode. This Published Document, which has been prepared by BSI Subcommittee B/525/5, *Structural use of timber*, provides just such information and has been cited as a reference in the UK National Annex to BS EN 1995-1-1.

1 Scope

This Published Document gives non-contradictory complementary information for use with BS EN 1995-1-1 and NA to BS EN 1995-1-1.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 8103-1, *Structural design of low-rise buildings – Part 1: Code of practice for stability, site investigation, foundations, precast concrete floors and ground floor slabs for housing*

BS 8212, *Code of practice for dry lining and partitioning using gypsum plasterboard*

BS EN 301, *Adhesives, phenolic and aminoplastic, for loading bearing timber structures – Classification and performance requirements*

BS EN 338:2009, *Structural timber – Strength classes*

BS EN 520, *Gypsum plasterboards – Definitions, requirements and test methods*

BS EN 1995-1-1:2004+A1:2008, *Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings*

BS EN 13986, *Wood-based panels for use in construction – Characteristics, evaluation of conformity and marking*

BS EN 14592, *Timber structures – Dowel-type fasteners – Requirements*

BS EN 15425, *Adhesives – One component polyurethane for load bearing timber structures – Classification and performance requirements*

3 Terms, definitions and abbreviations

3.1 Terms and definitions [BS EN 1995-1-1:2004+A1:2008, 1.5]

For the purposes of this Published Document, the terms and definitions given in BS EN 1995-1-1:2004+A1:2008 and the following apply.

3.1.1 attic truss

trussed rafter that is designed to allow a habitable room within the roof space

3.1.2 chord member

member of the external profile of the truss (e.g. a rafter or a ceiling tie)

3.1.3 girder truss

multiple trussed rafter that:

- a) supports a width of roof greater than 2,5 times the normal truss spacing within the roof or greater than 1,5 m whichever is the lesser; or
- b) directly supports other trusses; or
- c) supports another girder

3.1.4 racking discontinuity

door opening or other large opening (e.g. window), the dimensions of which exceed stipulated limits

3.1.5 trussed rafter

structural assemblage of timber members of the same thickness, fastened together in one plane by punched metal plate fasteners

3.1.6 wind girder

structural element set in the plane of the chords of a roof truss construction to transmit external wind forces to the supporting structure and to the foundations

3.2 Abbreviations

For the purposes of this Published Document, the following abbreviations apply.

LVL	Laminated veneer lumber
OSB	Oriented strand board
UDL	Uniformly distributed load

4 Design responsibilities

On every project it is essential that one person assumes overall responsibility as building designer and is clearly defined as such. The building designer should be responsible for ensuring the integration of the design of the various building components including the detailing of suitable connections between the building components and their support structure. The building designer should be responsible for ensuring adequate provision is made for the stability of the building as a whole, as distinct from, and in addition to, the stability of individual components, including the detailing of all elements of bracing required in the building. The building designer should ensure that necessary information, including the information listed, for example, in Annex A is provided to all parties involved in the design of the building.

5 Effective anchorages of timber floors to walls for buildings of Consequence Class 2a

For timber-frame walls, the guidance for the effective anchorages of timber floors in buildings of Consequence Class 2a given in Annex B may be followed.

For timber floors supported on masonry walls, Figure 1 to Figure 13 in PD 6697:2010 may be used for the effective anchorages for buildings of Consequence Class 2a.

NOTE This guidance is in accordance with the NA to BS EN 1991-1-7:2006, NA.2.43.

6 Assignment of timbers to BS EN 338 strength classes [BS EN 1995-1-1:2004+A1:2008, 3.2]

6.1 Assignment of temperate hardwoods

The assignment of temperate hardwoods to BS EN 338 strength classes should be in accordance with Table 1.

Table 1 Assignment of temperate hardwoods to BS EN 338 strength classes

Strength class	Grade (in accordance with BS 5756)	Species commercial name	Source	Botanical identification
D30	TH1	Oak	UK	<i>Quercus spp.</i>
D24	TH2	Oak	UK	<i>Quercus spp.</i>
D40	THA ^{A)}	Oak	UK	<i>Quercus spp.</i>
D30	THB ^{A)}	Oak	UK	<i>Quercus spp.</i>
D24	TH1	Sweet chestnut	UK	<i>Castanea sativa</i>

^{A)} Grades THA and THB are only obtainable in cross-section sizes with no dimensions less than 100 mm and cross-sectional areas greater than 20 000 mm².

6.2 Assignment of large cross-section British grown Douglas fir

For sizes where the cross-sectional area exceeds 20 000 mm² and both cross-sectional dimensions are at least 100 mm, British grown Douglas fir, of SS grade in accordance with BS 4978, may be allocated to strength class C24.

7 Horizontally glued laminated hardwood members [BS EN 1995-1-1:2004+A1:2008, 3.3]

For horizontally glued laminated hardwood members having four or more laminations and produced in accordance with BS EN 386, the characteristic strengths, stiffnesses and densities should be taken as the products of the modification factors from Table 2 and the strength class data given in BS EN 338 for the relevant grade and species. The effect of member size on bending strength or tension strength should be taken into account in accordance with BS EN 1995-1-1:2004+A1:2008, 3.3 (3). Table 3 gives examples of hardwoods that are suitable for glue laminating.

NOTE Further information on hardwood species is given in the TRADA publication, Wood information sheet WIS 1–17 [1].

Table 2 **Modification factors k_{lam} for characteristic strengths, stiffnesses and densities of hardwood glued laminated timber applicable for service classes 1 and 2**

Strength property	Modification factor k_{lam}
Bending strength	1,25
Tension strength	1,25
Compression strength parallel to grain	1,10
Compression strength perpendicular to grain ^{A)}	1,50
Shear strength	1,50
Mean modulus of elasticity	1,07
Fifth percentile modulus of elasticity	1,07
Mean density	1,0
Fifth percentile density	1,05

^{A)} Compression perpendicular to grain value is based on the assumption that the laminations contain no wane.

Table 3 **Examples of hardwoods suitable for glue laminating**

Common name	Botanical identification
Kapur	<i>Dryobalanops aromatica</i> , <i>Dryobalanops oblongifolia</i>
American white oak	<i>Quercus alba</i>
Keruing	<i>Dipterocarpus spp.</i>
Opepe	<i>Nauclea diderrichii</i>
American red oak	<i>Quercus rubra</i>
Iroko	<i>Milicia excelsa</i> , <i>Milicia regia</i>
Jarrah	<i>Eucalyptus marginata</i>
American white ash	<i>Fraxinus americana</i>
Beech	<i>Fagus sylvatica</i>
European ash	<i>Fraxinus excelsior</i>
European oak	<i>Quercus spp.</i>
Sweet chestnut	<i>Castanea sativa</i>

8 Factor $k_{c,90}$ for compression perpendicular to grain [BS EN 1995-1-1:2004+A1:2008, 6.1.5]

In addition to the values for factor $k_{c,90}$ given in BS EN 1995-1-1:2004+A1:2008, 6.1.5, the following values for $k_{c,90}$ may be applied.

- For members on discrete supports supporting a uniformly distributed load, the value of $k_{c,90}$ may be taken as:
 - $k_{c,90} = 1,5$ for solid softwood timber;
 - $k_{c,90} = 1,75$ for glued laminated softwood timber.
- For bottom chords of trusses reinforced with punched metal plate fasteners over a support, the value of $k_{c,90}$ may be taken as:

$k_{c,90} = 1,5$ for solid softwood timber.

9 Effective lengths of compression members [BS EN 1995-1-1:2004+A1:2008, 6.3.2]

9.1 The effective length of a compression member should be either:

- in accordance with Table 4 for the particular end conditions; or
- derived from the deflected form of the compression member as affected by any restraint and/or fixing moment(s), the effective length being the distance between adjacent points of zero bending between which the member is in single curvature.

Table 4 Effective lengths of compression members

End condition	Ratio of effective length to actual length
Restrained at both ends in position and in direction	0,7
Restrained at both ends in position and at one end in direction	0,85
Restrained at both ends in position but not in direction	1,0
Restrained at one end in position and in direction and at the other end in direction but not in position	1,5
Restrained at one end in position and in direction and free at the other end	2,0

9.2 The effective lengths of compression members in trussed rafters should be in accordance with Table 5.

Table 5 Effective lengths of compression members in trussed rafters

Type of truss member	Direction of buckling	Effective length
Internal member	In plane	$0,9 \times$ Member length measured on its centreline
Internal member	Out-of-plane	$0,9 \times$ Member length measured on its centreline
Chord member	In plane	<i>At joint nodes:</i> $\min \begin{cases} \text{distance between adjacent points of contraflexure} \\ \text{max of adjacent bay lengths} \end{cases}$ <i>In bays:</i> $\min \begin{cases} \text{bay length} \\ \text{distance between adjacent points of contraflexure} \end{cases}$
Chord member	Out-of-plane	<i>At joint nodes:</i> Check not required where longitudinal binders are fixed and suitably restrained <i>In bays:</i> Distance between tiling battens, sheeting rails or other suitable restraint points

9.3 In the absence of a more detailed calculation, the out-of-plane slenderness ratio for a member of a multiple-ply truss may be based on an effective thickness of $k_r n b$ where:

n is the number of truss plies;

b is the thickness of each truss ply;

$$k_r = 0,9 - 0,1n. \quad \text{[C1]}$$

9.4 The effective length of storey height wall studs sheathed on one or both sides should be taken as 0,85 times the length of the stud when considering buckling out of the plane of the wall. Timber wall studs sheathed on one or both sides may be assumed to be fully laterally restrained in the plane of the wall.

10 Limits on notches and circular holes in joists and studs for which no calculations are required

10.1 For simply supported joists of solid timber, glued laminated timber or LVL of depth, h , less than 250 mm and at centres not exceeding 610 mm with a notch of depth, h_{notch} , the effect of notches need not be calculated where:

- a) $h_{\text{notch}} \leq 0,125h$; and
- b) the notch is located at the top edge of the joist between 0,07 and 0,25 of the span from the nearest support.

10.2 For simply supported joists of solid timber, glued laminated timber or LVL of depth, h , less than 250 mm and at centres not exceeding 610 mm with a hole of diameter, d_{hole} , the effect of holes need not be calculated where all of the following apply:

- a) $d_{\text{hole}} \leq 0,25h$
- b) the hole centre is equidistant from the top and bottom edges of the joist;
- c) the hole is located within 0,25 and 0,4 of the span from the nearest joist support;
- d) centres of adjacent holes are at least $3d_{\text{hole}}$ apart. Where adjacent holes are not of the same diameter, the maximum diameter should be used for d_{hole} .

10.3 The distance between the edge of a notch conforming to **10.1** and the edge of the nearest hole, conforming to **10.2** should be at least 200 mm.

10.4 For studs of solid timber, glued laminated timber or LVL of depth, h , less than 250 mm and at centres not exceeding 610 mm, having a hole diameter, d_{hole} , the effect of holes need not be calculated where all of the following apply:

- a) $d_{\text{hole}} \leq 0,25h$;
- b) the hole is central in the depth of the member;
- c) the distance between the end of the member and the closest hole is not less than the maximum of 150 mm and 0.05 times the length of the member between supports;
- d) the spacing between the holes is not less than $3d_{\text{hole}}$.

11 Design of beams with circular holes

11.1 For beams of solid timber of a minimum strength class C24 in accordance with BS EN 338, glued laminated timber or LVL of width, b , and depth, h , with a circular hole of diameter d_{hole} , where all of the following apply.

- a) The beam has a rectangular cross-section and the grain runs essentially parallel to the length of the member.
- b) The axis of the hole runs parallel to the width of the beam.
- c) $d_{\text{hole}} \leq 0,4h$
- d) The hole centre is equidistant from the top and bottom edges of the beam.

(If the hole centre is not equidistant from the top and bottom edges of the beam, then the calculation should be based on a larger hole whose centre is equidistant from the top and bottom edges of the beam and within which the actual hole is entirely contained.)

- e) The distance from the hole centre to the nearest end of the beam is a minimum of $4d_{\text{hole}}$ or h .
- f) The distance from the hole centre to an adjacent hole centre is a minimum of $4d_{\text{hole}}$ or h .

11.2 It should be verified that:

$$\sigma_{t,90,d} \leq f_{t,90,d}$$

where:

$\sigma_{t,90,d}$ is the design tensile stress perpendicular to the grain at the hole location;

$f_{t,90,d}$ is the design tensile strength perpendicular to grain.

11.3 The design tensile stress perpendicular to the grain at the hole location, $\sigma_{t,90,d}$ should be calculated as:

$$\sigma_{t,90,d} = \frac{1,8V_{\text{hole},d}d_{\text{hole}} + 0,07M_{\text{hole},d}}{bh^2} \quad (1)$$

where:

$V_{\text{hole},d}$ is the design shear force acting on the beam at the hole centre, in N;

$M_{\text{hole},d}$ is the design moment acting on the beam at the hole centre, in N·mm;

b , h , and d_{hole} are taken in mm.

11.4 The verification of the bending strength of a beam which contains a hole should be based on the properties of the residual cross-section at the hole position.

12 Characteristic properties of fasteners

BS EN 14592 enables manufacturers of C1 *Text deleted* C1 fasteners to declare characteristic properties for their products which may be used as an alternative to the default values derived from the expressions given in BS EN 1995-1-1:2004+A1:2008.

13 Yield moment of annular ring-shanked nails [BS EN 1995-1-1:2004+A1:2008, 8.3.1.1(4)]

For annular ring-shanked nails produced from wire having a minimum tensile strength of 600 N/mm², the following characteristic value for yield moment should be used:

$$M_{y,Rk} = 0,3f_u d^{2,6} \quad (2)$$

where:

f_u is the tensile strength of the wire, in N/mm²;

d is the nail diameter, being the minimum outer cross-sectional diameter of the unprofiled part, mm

NOTE Annular ring-shanked nails are not grooved nails as referred to in BS EN 1995-1-1:2004+A1:2008, 8.3.1.1(4).

14 Diameters for evaluating lateral load-carrying capacities of screws [BS EN 1995-1-1:2004+A1:2008, 8.7.1]

14.1 Screws should conform to BS EN 14592.

NOTE The current publication of BS EN 14592 requires that screws have a nominal diameter, d , equal to the outer thread diameter and may be:

- a) screws which have their threaded part turned down from the original rod diameter such that the outer thread diameter is equal to the smooth shank diameter; or
- b) screws produced by rolling or forging such that the outer thread diameter is greater than the smooth shank diameter.

14.2 For full-threaded and part-threaded screws the rules in BS EN 1995-1-1:2004+A1:2008, 8.2, should be applied using an effective diameter d_{ef} instead of d in the Johansen expressions.

14.3 Where the smooth shank extends $4d_{ef}$ either side of the shear plane, d_{ef} should be taken as the smooth shank diameter.

14.4 Where the threaded length extends $4d_{ef}$ either side of the shear plane, d_{ef} should be taken as 1,1 times the thread root diameter.

14.5 In all other cases, d_{ef} should be taken as the lower of the smooth shank diameter or 1,1 times the thread root diameter.

14.6 The embedment strength $f_{h,k}$ and the yield moment $M_{y,k}$ should be calculated using d_{ef} .

14.7 For screws with an effective diameter $d_{ef} > 6$ mm, the rules in BS EN 1995-1-1:2004+A1:2008, 8.5.1, should be applied except for spacing and edge distances, see 14.9.

14.8 For screws with an effective diameter $d_{ef} \leq 6$ mm, the rules in BS EN 1995-1-1:2004+A1:2008, 8.3.1, should be applied.

14.9 For screws of all diameters, the spacing, edge and end distances should be calculated using d and BS EN 1995-1-1:2004+A1:2008, Table 8.2.

15 Axially loaded screws [BS EN 1995-1-1:2004+A1:2008, 8.7.2]

15.1 For connections with screws in accordance with BS EN 14592, with $d < 6$ mm and $0,6 \leq d_1/d \leq 0,75$, the characteristic withdrawal capacity in solid timber may be calculated in accordance with expression 8.38 in BS EN 1995-1-1:2004+A1:2008, 8.7.2, using the following values for $f_{ax,k}$ and k_d :

$$f_{ax,k} = 0,032 \rho_k$$

$$k_d = 1,0$$

where:

$f_{ax,k}$ is the characteristic withdrawal strength perpendicular to grain, in N/mm²;

ρ_k is the characteristic timber density, in kg/m³;

d is outer thread diameter of screw, in mm;

d_1 is inner thread diameter (thread root diameter) of screw, in mm.

15.2 For connections with screws in accordance with BS EN 14592, with $d < 12$ mm and $d_h/d \leq 2,5$, the characteristic head pull-through capacity in solid timber may be calculated in accordance with the expression 8.40b) in BS EN 1995-1-1:2004+A1:2008, 8.7.2, using the following value for $f_{head,k}$:

$$f_{head,k} = 12(\rho_k/350)^{0,8} \quad (3)$$

where:

$f_{head,k}$ is the characteristic head pull-through strength perpendicular to the grain, in N/mm²;

ρ_k is the characteristic timber density, in kg/m³;

d is the outer thread diameter of the screw, in mm;

d_h is the head diameter of the screw, in mm.

16 Connections made with punched metal plate fasteners [BS EN 1995-1-1:2004+A1:2008, 8.8.1]

16.1 Unless the anchorage strength of punched metal plate fasteners, when placed directly over finger joints, has been determined in accordance with BS EN 1075, finger-jointed timber may not be used in trussed rafters.

16.2 Where k_{sys} is used for the member design of a truss made with punched metal plate fasteners, k_{sys} may also be used for the anchorage design, but not for the steel plate design of the punched metal plate fastener connections.

17 Misalignment tolerances in punched metal plate fastener joints [BS EN 1995-1-1:2004+A1:2008, 8.8.5.1]

The joint design should allow for a minimum misalignment of the punched metal plate fastener of 5 mm simultaneously in two directions parallel to the edges of the plate.

18 Contact pressure between timber members in punched metal plate fastener joints under compression [BS EN 1995-1-1:2004+A1:2008, 8.8.5.1(3)]

Contact pressure between timber members may be taken into account to reduce the value of F_{ED} in compression, provided that the gap between the members has an average value, which is not greater than 1,5 mm, and a maximum value of 3 mm. In the design of such connections in compression the component of F_{ED} perpendicular to the contact surfaces should be divided by two.

19 Trusses with punched metal plate fasteners [BS EN 1995-1-1:2004+A1:2008, 9.2.2]

19.1 The actions and combinations of actions that may be considered in the design of trussed rafters are given in Annex C.

NOTE Recommendations for the support of water tanks in trussed rafter roofs are given in Annex F.

19.2 To take account of practical factors such as handling, the size of members and span should be restricted in accordance with the following limits.

- a) Spans of trussed rafters manufactured in timber of target thickness between 35 mm and 47 mm should conform to the following:

$$\text{Span} \leq t \times 345$$

where:

t is the target thickness of timber.

NOTE 1 The use of mechanical handling equipment is particularly important for trussed rafters of more than 47 mm in thickness during their manufacture, loading, offloading and erection in order to prevent damage.

NOTE 2 Where a 2-ply truss is fixed together in the factory and mechanical handling equipment is utilized the member thickness t can be taken as two times the individual ply thickness.

- b) The maximum bay length of any top chord or bottom chord member, when measured on plan on the lower-edge position, should not exceed the appropriate value given in Table 6. These limits may be increased by 50% when applied to top chord extensions such as overhangs or hip rafters.
- c) The overall length of any internal member should not exceed the appropriate value given in Table 6. The overall length is the actual cut length as measured on the centre-line of the member.

Table 6 Maximum lengths of chord and internal members

Member width mm	35 mm thick timbers			47 mm thick timbers		
	Top chord	Bottom chord	Internal member	Top chord	Bottom chord	Internal member
60	—	—	2 400	—	—	3 500
72	1 900	2 500	3 600	3 300	3 300	5 200
84	2 090	2 740	4 030	3 440	3 780	5 580
97	2 300	3 000	4 500	3 600	4 300	6 000
120	2 600	3 400	—	3 900	5 000	—
145	2 800	3 700	—	4 100	5 300	—

19.3 To account for loading eccentricities in girder trusses, the design shear force $F_{v,Ed}$ perpendicular to grain at a joint should be multiplied by the modification factor, K_e , given in Table 7. Additionally, any joint in a girder truss which is subject to a tension force perpendicular to grain should have a minimum fastener bite in any chord member not less than 0,5 times the member depth.

Table 7 Modification factor, K_e , to account for loading eccentricities in girder trusses

Number of plies in girder truss	Modification factor, K_e
2	1,33
3	2,00
4	3,00

19.4 For trusses made with punched metal plate fasteners and supporting a floor imposed load $\geq 1,5 \text{ kN/m}^2$, all joints, except chord splices, should be designed using axial forces 50% greater than the axial forces utilized for the member design. Additionally, the minimum fastener bite on to chord members should be 0,5 times the member depth.

19.5 In splice joints the minimum length of the connector plate should be $0,8h + 80$ where h is the depth of the timber member, in mm.

20 Masonry shielding to wall diaphragms

Where timber frame walls are clad by masonry veneer, the external wind loading transferred to the timber structure may be determined using the information given in Annex D.

21 Simplified analysis of wall diaphragms [BS EN 1995-1-1:2004+A1:2008, 9.2.4.3]

COMMENTARY ON CLAUSE 21

Clause 21 gives a simplified method of analysis for wall diaphragms of platform framed timber buildings consisting of timber framing connected on one or both faces to a wood-based sheathing. The method applies to wall diaphragms that are connected to the underlying timber construction or foundations either by bottom rail connections or by a combination of bottom rail connections and tiedowns.

21.1 Construction of wall diaphragms

21.1.1 Timber frame

21.1.1.1 The timber frame consists of timber studs, not exceeding 610 mm centre to centre, between horizontal top and bottom timber rails. The timber framing members should be a minimum thickness of 38 mm, a minimum depth of 72 mm and a minimum strength class of C16, in accordance with BS EN 338.

21.1.1.2 The connection between horizontal rails and studs should comprise a minimum of two ring-shanked nails of diameter $\geq 3,1 \text{ mm}$ and having a penetration into the stud $\geq 45 \text{ mm}$, or equivalent.

21.1.2 Wood-based panel sheathing

21.1.2.1 The sheathing should conform to BS EN 13986.

NOTE For plasterboard see 22 and 23.

21.1.2.2 Shear buckling of the sheathing sheets may be disregarded, provided that:

$$b_{\text{net}}/t \leq 100$$

where:

b_{net} is the clear distance between studs;
 t is the thickness of the sheathing.

21.1.3 Fasteners connecting sheathing to timber framing

21.1.3.1 The diameter of the fasteners connecting the sheathing to timber framing should be no greater than 0,09 times the stud thickness. Additionally, where two sheathing sheets are connected to the same stud, the fastener edge distance for both the stud and sheathing sheet should be a minimum of three times the fastener diameter.

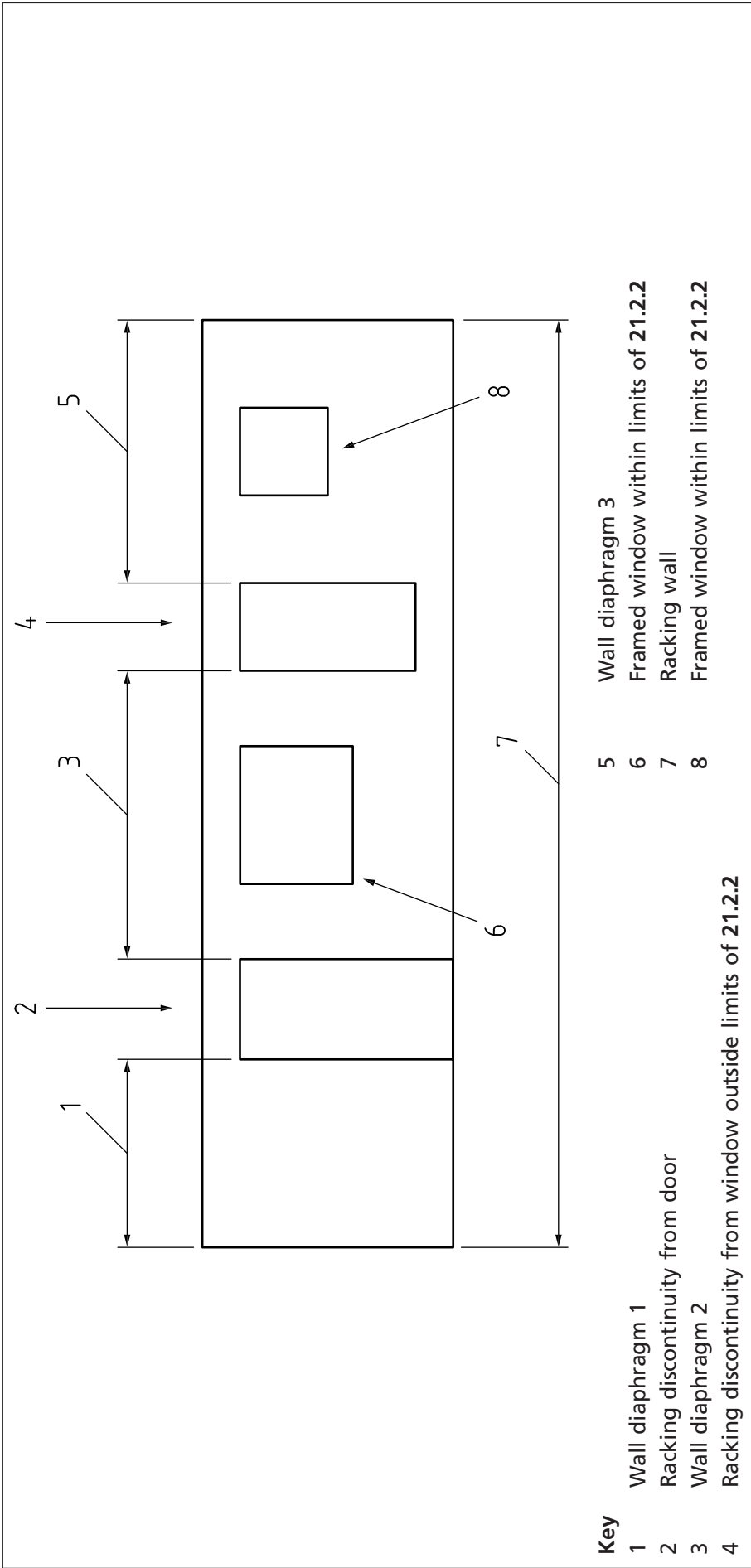
21.1.3.2 The fasteners fixing the sheathing to the framing should be equally spaced around the perimeter of each sheathing sheet at a maximum distance of 150 mm. Fasteners fixing the sheathing to the framing within the perimeter of a sheathing sheet should be equally spaced at a distance not more than twice the perimeter fastener spacing.

NOTE For further details on the construction of timber frame wall diaphragms refer to the TRADA publication, Timber frame construction [2].

21.2 General arrangements of wall diaphragms

21.2.1 A racking wall is a sheathed timber-frame wall, usually located in a direction parallel to the wind load, and often extending between return walls. A racking wall may comprise a single wall diaphragm or, if it contains racking discontinuities, it may comprise more than one wall diaphragm (see Figure 1).

Figure 1 Division of racking wall into wall diaphragms



21.2.2 Racking discontinuities are door openings or large openings such as windows that exceed any of the following limits.

- a) The vertical dimension of the opening is greater than 0,65 times the wall diaphragm height.
- b) The height to the underside of the opening is less than 0,25 times the wall diaphragm height.

21.2.3 A wall diaphragm may contain framed openings of dimensions within the limits given in **21.2.2a)** and **21.2.2b)**, provided that their effects on racking strength and stiffness are taken into account (see **21.5.2.8**).

21.2.4 Small openings within a length of wall diaphragm comprising only full height sheathing sheets may be allowed without reducing racking resistance if all of the following conditions are met.

- a) The opening does not exceed 300 mm in both length and height where the opening is framed.
- b) The opening does not exceed 150 mm in both length and height or 200 mm in diameter where the opening is unframed.
- c) The edge distance from the opening to any edge of a sheathing sheet is at least the maximum dimension of the opening.
- d) Only one such opening is allowed in a sheathing sheet and the spacing between such openings is at least 1 200 mm.

21.2.5 No more than two sheathing sheets of a length less than 600 mm should be used consecutively along the length of wall diaphragm.

21.2.6 A wall diaphragm with a framed opening of dimensions within the limits given in **21.2.2a)** and **21.2.2b)** may be designed to resist racking (see **21.5.2**) provided that the following conditions are met.

- a) Each full height sheathing sheet either side of the opening should have a minimum length of 0,25 times the width of the opening or one-eighth of the wall height, whichever is the larger. Alternatively, there should be a full width sheathing sheet (nominally 1 200 mm) within a distance of one-eighth of the wall height from the vertical edge of the opening.
- b) The connection between the edge stud of the panel below the opening and the cripple stud immediately adjacent to the opening should have a design shear capacity per unit length of no less than $f_{p,d,t}$ (see **21.5.2.2**).

21.3 Distribution of horizontal wind loads between wall diaphragms

For each orthogonal direction of wind load and at each floor level of the building, the following recommendations should be applied.

- a) The wall diaphragms should be identified on the building plan.
- b) Each wall diaphragm should be situated directly above an underlying load-bearing wall or alternatively a load transfer system of adequate strength and stiffness should be designed such that the loads are transferred to the nearby load-bearing walls in the underlying storey.
- c) Where the provisions of Annex E or BS EN 1995-1-1:2004+A1:2008, **9.2.3**, have been met, it may be assumed that the overlying roof or floor diaphragm is capable of distributing wind load to each wall diaphragm in proportion to its racking strength.
- d) Where there is eccentricity between the centroids of the wind load and

aggregated wall racking resistance, the forces resulting from the torsional moment on the floor diaphragm should be applied to orthogonal racking walls.

21.4 Design requirements for wall diaphragms under wind load

21.4.1 For each wall diaphragm, adequate racking, overturning and sliding resistance should be provided to resist the wind load determined in accordance with **21.3**.

NOTE In accordance with the note under Table NA.A1.2(A) of the NA to BS EN 1990:2002+A1:2005, as the verification of overturning equilibrium involves the racking resistance of the wall diaphragm, a combined verification uses the following partial factors for permanent load:

$$\gamma_{G,sup} = 1,35$$

$$\gamma_{G,inf} = 1,00$$

21.4.2 Sliding resistance may be provided by friction under design permanent load (reduced by any vertical component of design wind load) in conjunction with mechanical fasteners. The coefficient of friction between timber members or for the interface between the damp-proof course and the soleplate may be taken as 0,4 when calculating the design sliding resistance.

21.4.3 At each floor level, the overturning resistance of each wall diaphragm should be provided by ensuring that the design withdrawal capacity of its bottom rail-to-floor connection, $f_{v,d}$, see **20.5.2.5**, can be mobilized by the underlying construction (including at foundation level).

*NOTE Calculation of design racking strength in accordance with **21.5** ensures the overturning stability of the wall diaphragm (including taking into account the effects of design permanent load).*

21.4.4 For both overturning and racking calculations, additional permanent load may be utilized from the following sources.

- a) Return walls, provided that the connection between the return wall and the wall diaphragm is designed to transfer the additional permanent load mobilized.
- b) The weight of the underlying construction, mobilized via holding-down straps or tension fixings from the bottom rail of the wall diaphragm.

21.4.5 The design racking strength of wall diaphragms should be calculated in accordance with the method given in **21.5**.

21.5 Calculation of design racking strength

21.5.1 Design racking strength of racking walls

For a racking wall made up of more than one wall diaphragm, the design racking strength of the racking wall, $F_{v,Rd}$, should be calculated from:

$$F_{v,Rd} = \sum F_{i,v,Rd} \quad (4)$$

where:

$F_{i,v,Rd}$ is the design racking strength of each wall diaphragm in accordance with **21.5.2**.