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Incorporating Corrigendum No. 1

BSI Standards Publication

PUBLISHED DOCUMENT

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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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.

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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	*Quercus spp.*
D24	TH2	Oak	UK	*Quercus spp.*
D40	THA [A)]	Oak	UK	*Quercus spp.*
D30	THB [A)]	Oak	UK	*Quercus spp.*
D24	TH1	Sweet chestnut	UK	*Castanea sativa*

[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 mm^2.

6.2 Assignment of large cross-section British grown Douglas fir

For sizes where the cross-sectional area exceeds 20 000 mm^2 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	*Dryobalanops aromatica, Dryobalanops oblongifolia*
American white oak	*Quercus alba*
Keruing	*Dipterocarpus spp.*
Opepe	*Nauclea diderrichii*
American red oak	*Quercus rubra*
Iroko	*Milicia excelsa, Milicia regia*
Jarrah	*Eucalyptus marginata*
American white ash	*Fraxinus americana*
Beech	*Fagus sylvatica*
European ash	*Fraxinus excelsior*
European oak	*Quercus spp.*
Sweet chestnut	*Castanea sativa*

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.

a) For members on discrete supports supporting a uniformly distributed load, the value of $k_{c,90}$ may be taken as:

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

   2) $k_{c,90}$ = 1,75 for glued laminated softwood timber.

b) 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:

a) in accordance with Table 4 for the particular end conditions; or

b) 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 × Member length measured on its centreline
Internal member	Out-of-plane	0,9 × Member length measured on its centreline
Chord member	In plane	*At joint nodes:* $\min \begin{cases} \text{distance between adjacent points of contraflexure} \\ \text{max of adjacent bay lengths} \end{cases}$ *In bays:* $\min \begin{cases} \text{bay length} \\ \text{distance between adjacent points of contraflexure} \end{cases}$
Chord member	Out-of-plane	*At joint nodes:* Check not required where longitudinal binders are fixed and suitably restrained *In bays:* 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 $k_r nb$ where:

n is the number of truss plies;

b is the thickness of each truss ply;

[C1] $k_r = 0,9 - 0,1n$. [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) $h_{notch} \leq 0{,}125h$; 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) $d_{hole} \leq 0{,}25h$

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 $3d_{hole}$ 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) $d_{hole} \leq 0.25h$;

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 $3d_{hole}$.

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) $d_{hole} \leq 0{,}4h$

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:

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

where:

$\sigma_{t,90d}$ 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:

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

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:

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

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} ≤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 $0,6 \leq d_1/d \geq 0,75$, 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 $f_{\mathrm{ax,k}}$ and k_{d}:

$$f_{\mathrm{ax,k}} = 0,032\ \rho_{\mathrm{k}}$$

$$k_{\mathrm{d}} = 1,0$$

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	35 mm thick timbers			47 mm thick timbers		
mm	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 ≥ 1,5 kN/m², 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 0,8*h* + 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 ≥3,1 mm and having a penetration into the stud ≥45 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:

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

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.

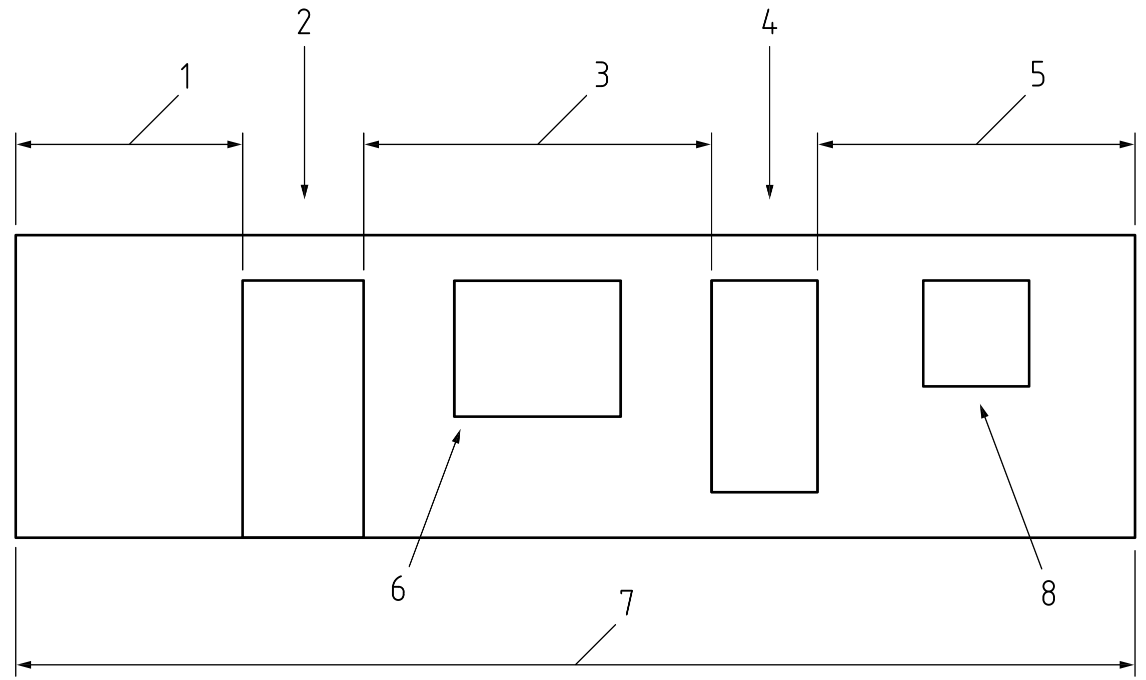
21.1.3.2 The fasteners fixing the sheathing to the framing should be equally spaced around the perimeter of each sheathing sheet at a maximum distance of 150 mm. Fasteners fixing the sheathing to the framing within the perimeter of a sheathing sheet should be equally spaced at a distance not more than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



Key

1 Wall diaphragm 1
2 Racking discontinuity from door
3 Wall diaphragm 2
4 Racking discontinuity from window outside limits of **21.2.2**
5 Wall diaphragm 3
6 Framed window within limits of **21.2.2**
7 Racking wall
8 Framed window within limits of **21.2.2**

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 **21.2.2**a) and **21.2.2**b), 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 **21.2.2**a) and **21.2.2**b) 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, $f_{w,d}$, 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:

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

where:

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

21.5.2 Design racking strength of wall diaphragms

21.5.2.1 The design racking strength of a wall diaphragm $F_{i,v,Rd}$ should be calculated as follows:

$$F_{i,v,Rd} = K_{opening} K_{i,w} f_{p,d,t} L \qquad (5)$$

where:

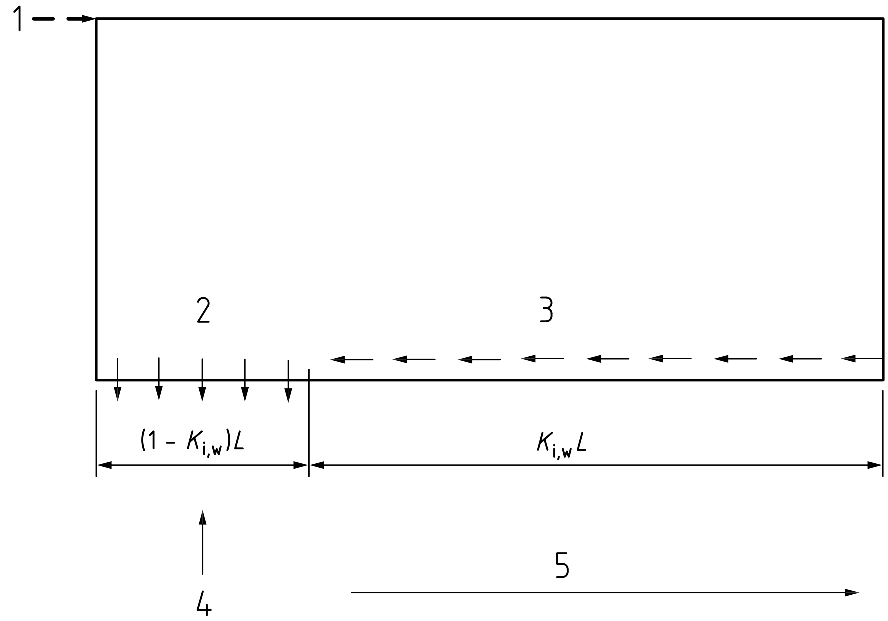
L is the length of the wall diaphragm, in m;

$f_{p,d,t}$ is the summation, in accordance with **21.5.2.2**, of the design shear capacities [C1⟩ per unit length of ⟨C1] the perimeter sheathing fasteners, in kN/m;

$K_{i,w}$ is a modification factor taking into account wall length, vertical load and holding-down arrangements, see Figure 2 and **21.5. 2.5**;

$K_{opening}$ is a modification factor, taking into account the effect of framed openings, see **21.5.2.8**.

Figure 2 **Forces transmitted into underlying construction by bottom rail of wall diaphragm**



Key

1 Design horizontal wind load

2 Sheathing-bottom rail fasteners providing uplift resistance per unit length of $\mu f_{p,d,t}$ (=$f_{w,d}$)

3 Sheathing-bottom rail fasteners providing horizontal shear resistance per unit length, $f_{p,d,t}$

4 Uplift resistance required from underlying construction (including at foundation level) of $(1 - K_{i,w}) f_{w,d} L$

5 Racking resistance transmitted to underlying construction of $K_{i,w} f_{p,d,t} L$

21.5.2.2 The total design shear capacity per unit length of the perimeter sheathing fasteners should be calculated as follows:

$$f_{p,d,t} = f_{p,d,1} + K_{comb} f_{p,d,2} \qquad (6)$$

with:

$f_{p,d,2} \leq f_{p,d,1}$

where:

$f_{p,d,1}$	is the design shear capacity per unit length of perimeter sheathing fasteners of the first or only sheathing layer, calculated in accordance with **21.5.2.4**, in kN/m;
$f_{p,d,2}$	is the design shear capacity per unit length of perimeter sheathing fasteners of the second sheathing layer, calculated in accordance with **21.5.2.4**, in kN/m;
K_{comb}	is the sheathing combination factor having the values in Table 8.

Table 8 **Values of sheathing combination factor, K_{comb}**

Details of second sheathing	K_{comb}
None	0
On opposite side of framing to first sheathing layer but having sheathing sheets and fasteners of the same type, dimensions and spacing	0,75
On opposite side of framing to first sheathing layer but having sheathing sheets and fasteners of the differing type, dimensions and spacing	0,5
On same side of framing to first sheathing layer	0,5

21.5.2.3 In order to limit racking deflection, the following condition should be applied:

$$K_{\mathrm{i,w}}\, f_{\mathrm{p,d,t}} \leq 8(1+K_{\mathrm{comb}})(L/H)$$

where:

H	is the height of the sheathed area of the wall diaphragm, in m, see Figure 4.

21.5.2.4 The design shear capacity per unit length of the perimeter fasteners to a sheathing sheet, $f_{\mathrm{p,d}}$, should be calculated from: C1⟩

$$f_{\mathrm{p,d}} = \frac{F_{\mathrm{f,Rd}}\left[1{,}15+s\right]}{s} \qquad (7)$$

⟨C1 where:

$F_{\mathrm{f,Rd}}$	is the design lateral capacity of an individual fastener, in kN;
s	is the sheathing perimeter fastener spacing, in m.

21.5.2.5 The modification factor $K_{i,w}$ should be calculated from equation (8). Where equation (8) gives a value of $K_{i,w} > 1$, $K_{i,w}$ should be taken as 1,0. Where equation (8) gives a value of $K_{i,w} < 0$, $K_{i,w}$ should be taken as 0. C1⟩

$$K_{\mathrm{i,w}} = \left[1+\left(\frac{H}{\mu L}\right)^2+\left(\frac{2M_{\mathrm{d,stb,n}}}{\mu f_{\mathrm{p,d,t}} L^2}\right)\right]^{0.5} - \left(\frac{H}{\mu L}\right) \qquad (8)$$

⟨C1

where:

$$M_{d,stb,n} = M_{d,stb} - M_{d,dst,top} \tag{9}$$

C1

$$\mu = min\left[1, f_{w,d}/f_{p,d,t}\right] \tag{10}$$

C1

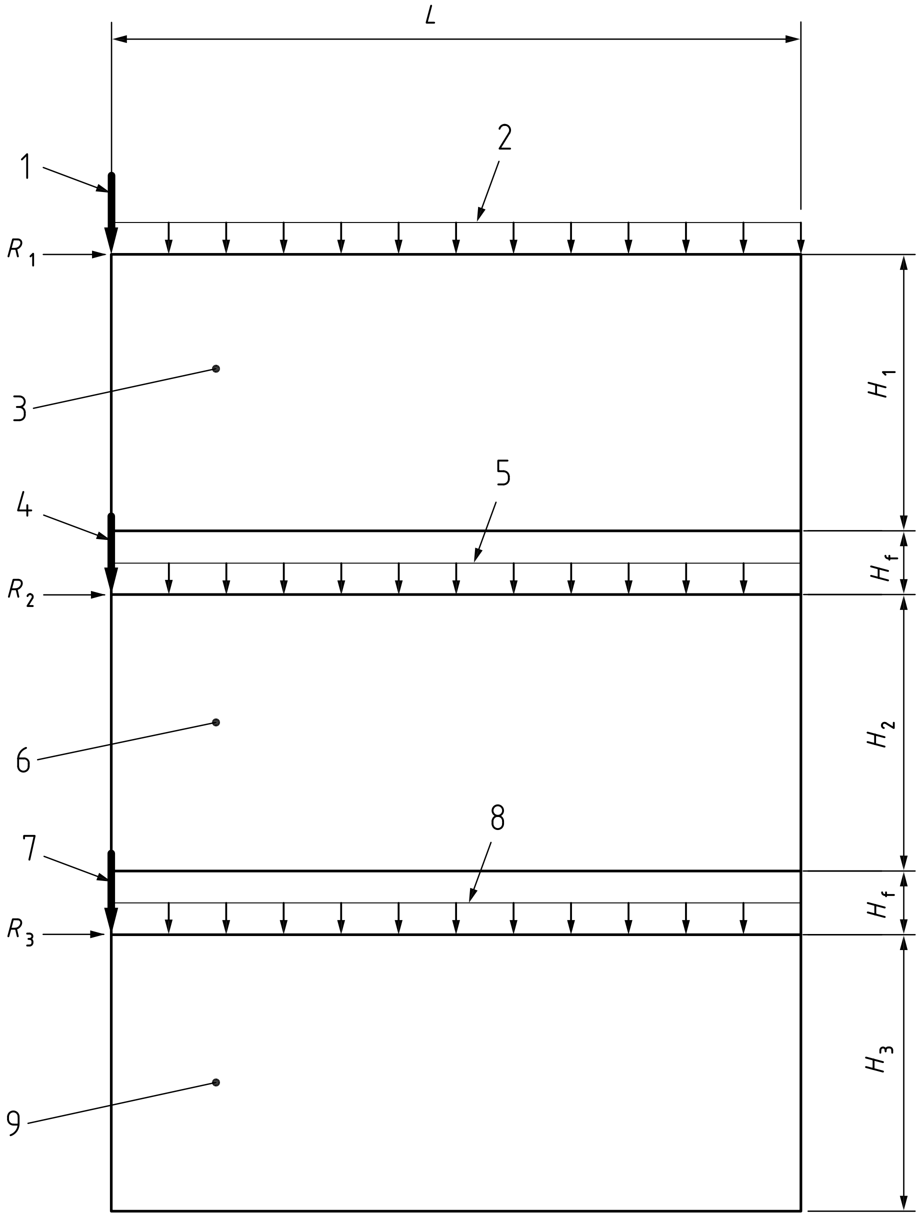
and where:

$f_{w,d}$ is the design withdrawal capacity of bottom rail-to-floor connection per unit length, in kN/m;

$M_{d,stb}$ is the design stabilizing moment, in kN·m, about the leeward end of the wall diaphragm (see Figure 3) from design permanent load (see note under **21.4.1**), reduced by any vertical component of design wind load;

$M_{d,dst,top}$ is the design destabilizing moment, in kN·m, about the top of the wall diaphragm from design wind load (see Figure 3).

Figure 3 **Calculation of design stabilizing and destabilizing moments**



Key

1	Design point load, V_1	H_1	Height of wall diaphragm 1
2	Design UDL, w_1	H_2	Height of wall diaphragm 2
3	Wall diaphragm 1	H_3	Height of wall diaphragm 3
4	Design point load, V_2	H_f	Depth of floor construction
5	Design UDL, w_2	R_1, R_2, R_3	Design horizontal windloads
6	Wall diaphragm 2	V_1, V_2, V_3	are design point loads (usually load from return wall)
7	Design point load, V_3		
8	Design UDL, w_3		
9	Wall diaphragm 3 [A), B)]		

[A)] Net design stabilizing moment ($M_{d,stb,n}$) for the calculation of the design racking strength of wall diaphragm 3 (see **20.5.2.5**).

$$M_{d,stb,n}=M_{d,stb}-M_{d,dst,top} \tag{11}$$

where:

C1 $M_{d,stb}=0.5w_1L^2+V_1L + 0.5w_2L^2+V_2L + 0.5w_3L^2+V_3L$ C1 (12)

Figure 3 **Calculation of design stabilizing and destabilizing moments**

$$M_{d,dst,top}=R_1(H_1+H_f + H_2+H_f)+R_2(H_2+H_f) \quad (13)$$

B) Leeward compression calculations (see **21.5.2.10**) for wall diaphragm 3:

$$W_{v,t,d}=w_1L + V_1+w_2L + V_2+w_3L + V_3 \quad (14)$$

$$M_{d,stb}=0.5w_1L^2+V_1L + 0.5w_2L^2+V_2L + 0.5w_3L^2+V_3L \quad (15)$$

$$M_{d,dst,base}=R_1(H_1+H_f + H_2+H_f + H_3)+R_2(H_2+H_f + H_3)+R_3H_3 \quad (16)$$

21.5.2.6 A check should be made that $f_{w,d}$ does not exceed the permanent load per unit length of the underlying construction. In particular, if the permanent load of the underlying wall storeys is being mobilized, it should be checked that the withdrawal capacity, $f_{w,d}$, can be achieved across all the horizontal interfaces present in the floor constructions.

21.5.2.7 For a wall diaphragm comprising only of full height sheathing sheets (see also **21.2.5)**, or containing only small openings in accordance with **21.2.4**, $K_{opening}$ should be taken as 1,0.

21.5.2.8 For a wall diaphragm with a framed opening, or openings, of dimensions within the limits given in **21.2.2** and meeting the provisions of **21.2.6**, $K_{opening}$ should be taken as:

$$K_{opening} = 1 - 1{,}9p \quad (17)$$

where:

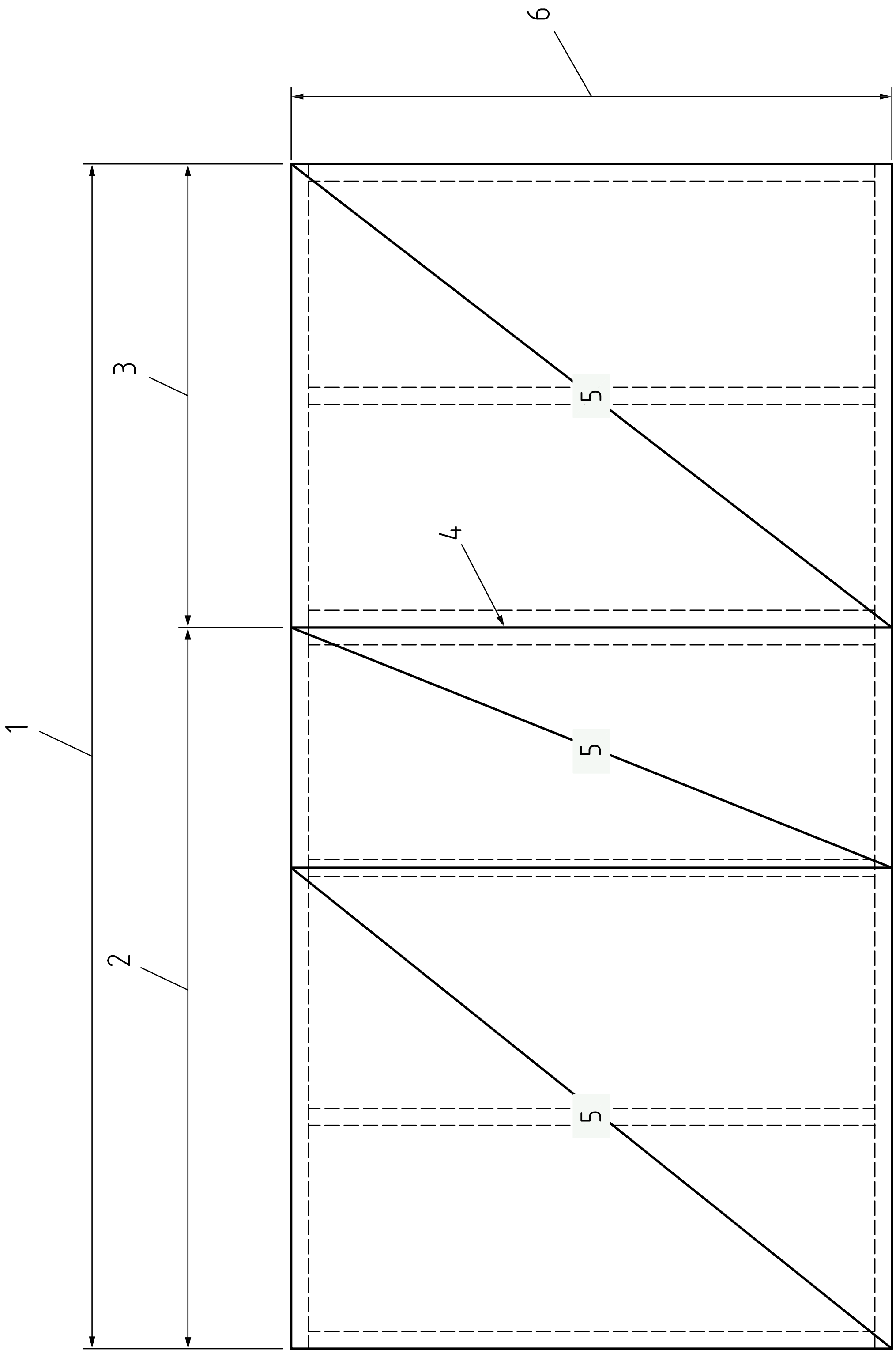
$$p = \frac{A}{HL} \quad (18)$$

and:

A is the aggregate area of openings in wall diaphragm, in m². Where the vertical dimension of an opening is less than half its horizontal dimension (L_{open}), the area of the opening is to be taken as $0.5(L_{open})^2$.

21.5.2.9 Where the wall diaphragm is made up of more than one wall panel (see Figure 4), the design shear capacity per unit length of the wall panel-to-wall panel connection should be greater than or equal to $f_{p,d,t}$.

Figure 4 **Division of wall diaphragms into wall panels**



Key

1 Wall diaphragm
2 Wall panel
3 Wall panel
4 Wall panel-to-wall panel connection having design shear capacity $\geq f_{p,d,t}$
5 Sheathing sheet
6 Panel height, *H*

21.5.2.10 Unless a special analysis is made to check that the compressive force at the leeward end of a wall diaphragm, $F_{c,d,leewd}$, does not cause either buckling of the wall studs or excessive bearing stresses on the horizontal framing members, the following condition should be observed:

$$F_{c,d,leewd} \leq F_{cR,d}$$

where:

$$F_{c,d,leewd} = 0,8W_{v,t,d}\left[\left(M_{d,dst,base}/M_{d,stb}\right) + \left(0,6/L\right)\right] \quad (19)$$

and:

$F_{cR,d}$ is the summation of the design compressive capacities of the studs in kN within 0,1L of the leeward end of the wall diaphragm. The design compressive capacity of each stud should be taken as the lesser of either its design buckling strength calculated in accordance with BS EN 1995-1-1:2004+A1:2008, **6.3.2**, or its design bearing capacity on the horizontal framing members calculated in accordance with BS EN 1995-1-1:2004+A1:2008, **6.1.5**;

$W_{v,t,d}$ is the total design vertical load acting on the wall diaphragm, in kN;

$M_{d,stb}$ is the design stabilizing moment, in kN·m, about the leeward end of the wall diaphragm (see Figure 3) from the design vertical load;

$M_{d,dst,base}$ is the design destabilizing moment, in kN·m, about the base of the wall diaphragm from design wind load (see Figure 3);

L is the length of the wall diaphragm, in m.

NOTE 1 The maximum design compressive force at the leeward end of a wall diaphragm is likely to occur under a combination of design permanent and design variable (i.e. floor imposed, snow) loads (refer to NA to BS EN 1990:2002+A1:2005, Table NA.A1.2(A), for details of design values of actions).

NOTE 2 For a wall diaphragm with a minimum of two studs within 0,1L of its leeward end in a dwelling of two or less storeys, the check on compressive force at the leeward end may be disregarded.

Where a return wall is located at the leeward end of a wall diaphragm, provided that the construction within the depth of the floor zone (h_f in Figure 3) and the connection between the wall diaphragm and return wall are designed to transfer the load, up to 50% of the design compressive force at the leeward end of the wall diaphragm, $F_{c,d,leewd}$, may be re-distributed to studs in the return wall a maximum of 1 m from the wall diaphragm.

22 Contribution of plasterboard to racking resistance

22.1 With the exception of separating walls comprising two or more built-up layers of plasterboard having a minimum thickness of 30 mm, plasterboard should provide not more than one-third of the total racking resistance.

22.2 In the case of separating walls comprising two or more built-up layers of plasterboard having a minimum thickness of 30 mm, plasterboard alone may be used to provide the racking resistance of a building.

22.3 Where plasterboard is combined with a wood-based sheathing on the same wall diaphragm, only the contribution of the wood-based sheathing should be taken into account in calculating the racking strength of the wall diaphragm in accordance with Clause **21**.

23 Evaluation of design racking resistance of plasterboard-clad timber frame walls

The design racking resistance of plasterboard-clad timber frame wall diaphragms, having timber framing conforming to **21.1.1.1**, may be calculated using the procedures of **21.5.2.1**, **21.5.2.5**, **21.5.2.7** and **21.5.2.8** and the appropriate value of total design shear capacity per unit length of the perimeter fasteners, $f_{p,d,t}$, given in Table 9.

Table 9 **Total design shear capacities per unit length of the perimeter fasteners for various specifications of plasterboard**

Plasterboard specification	$f_{p,d,t}$ **kN/m**
12,5 mm plasterboard fixed to one side of timber framing using plasterboard screws of 3,5 mm shank diameter at C1 300 m C1 spacing and penetrating at least 25 mm into timber framing.	1,27
15 mm plasterboard fixed to one side of timber framing using plasterboard screws of 3,5 mm shank diameter at 300 mm spacing and penetrating at least 25 mm into timber framing.	1,42
12,5 mm plasterboard fixed to both sides of timber framing using plasterboard screws of 3,5 mm shank diameter at 300 mm spacing and penetrating at least 25 mm into timber framing.	2,19
15 mm plasterboard fixed to both sides of timber framing using plasterboard screws of 3,5 mm shank diameter at 300 mm spacing and penetrating at least 25 mm into timber framing.	2,49
Separating wall of minimum 30 mm plasterboard (in two or more layers) with each layer individually fixed with plasterboard screws of 3,5 mm shank diameter at 300 mm spacing. The screws for each layer should penetrate at least 25 mm into timber framing.	2,19

24 Bracing to trussed rafter roofs [BS EN 1995-1-1:2004+A1:2008, 9.2.5.3]

Standard bracing details, suitable for fulfilling the functions of both roof and wall stability for spans up to 17 m should conform to Annex E. Provided that they conform to the design criteria and conditions of use, no further calculations are required.

NOTE ***E.3*** *describes principles and procedures by which bracing systems may be designed for trussed rafter roofs for which no standard bracing solution is available.*

25 Lateral load-carrying capacity of glued lap joints [BS EN 1995-1-1:2004+A1:2008, 10.3]

25.1 Clause **25** applies to lap joints in shear made from separate pieces of timber or other wood-based materials glued together and manufactured in accordance with BS 6446. Eccentricity in lap joints that induces tensile stress perpendicular to the glueline should not be permitted.

25.2 The following members may be bonded together:

a) softwood, of thickness measured normal to the plane of the glueline of up to 50 mm;

NOTE It is inadvisable to use excessively resinous pieces of softwood.

b) glued laminated timber, of thickness measured normal to the plane of the glueline of up to 50 mm;

c) plywood, OSB or particleboard, whose thickness only when it is connected to softwood or glued laminated timber, does not exceed 29 mm;

d) tempered hardboard conforming to BS EN 622-2 of not more than 8 mm thickness may be bonded to timber and glued laminated timber.

25.3 The following adhesives may be used:

1) phenolic and aminoplastic adhesives conforming to BS EN 301;

2) one-component polyurethane adhesives conforming to BS EN 15425.

25.4 It should be verified that:

$$\tau_d \leq K_{\mathrm{press}}\, f_{\mathrm{r,d}}$$

where:

τ_d is the design shear stress on the glueline;

$f_{\mathrm{r,d}}$ is the lowest design planar (rolling) shear strength of the connected members;

K_{press} is 0,9 when the close contact pressure on the glueline is generated by mechanical fasteners, otherwise 1,0 where pressure is applied in accordance with BS 6446.

25.5 For connected softwood members, the design planar (rolling) shear strength for the glueline should be calculated as follows:

$$f_{\mathrm{r,d}} = f_{\mathrm{v,d}}(1 - 0{,}67\sin\alpha) \tag{20}$$

where:

$f_{\mathrm{v.d}}$ is the design shear strength parallel to grain of the softwood;

α is the angle between the direction of load and the grain of the member.

25.6 When metal fasteners are present in the glued joint, they should not be considered as contributing to the strength of the joint.

NOTE It is advisable to consider the possibility of differential movements, distortion and stress concentrations at glued joints.

Annex A (normative) Exchange of information between building designer and component designer(s)

A.1 Exchange of information between building designer and trussed rafter designer

The building designer should provide the trussed rafter designer with the following information, preferably in the form of detailed technical drawings.

a) The height, orography and location of the building with reference to any unusual wind conditions.

b) The span and profile of the trussed rafter.

c) The spacing of the trussed rafters.

d) The pitch or pitches of the roof.

e) The method of support and position of load-bearing supports.

f) The type or weights of roof tiles or covering, including sarking, insulation and ceiling materials.

g) The size and position of water tanks or other ancillary equipment or loads (e.g. hoist tracks, mechanical and electrical equipment, dormer construction).

h) The overhang of rafters at eaves and other eaves details.

i) The positions and dimensions of hatches, chimneys and other openings.

j) The service use of the building with reference to any unusual environmental conditions.

k) Details of any special timber sizes where these are required to match existing construction.

l) The site snow load.

The trussed rafter designer should provide the building designer with the following information, preferably in the form of detailed technical drawings.

1) The sizes and strength classes of members.

2) The type, size and positions of all punched metal plate fasteners.

3) The positions and sizes of all bearings.

4) The position and spacing of all trussed rafters.

5) The loadings and other conditions for which the trussed rafters are designed.

6) The positions, fixings and sizes of any lateral supports necessary to prevent buckling of compression members of the trussed rafters.

7) The location and method of support for tanks or ancillary equipment.

8) The range of reactions (including for wind uplift) to be accommodated at the support positions.

9) Any special precautions for handling, storage or erection.

The building designer should use the information supplied by the trussed rafter designer to check that the trussed rafters supplied are suitable for their intended use.

*NOTE Further details on the information to be provided to site by the trussed rafter designer relating to transport, handling, storage, erection, positioning and fixing of multiple members together can be found in BS EN 14250:2010, **6.2**.*

A.2 Exchange of information between building designer and floor designer

The building designer should provide the floor designer with the following information, preferably in the form of detailed technical drawings.

a) The spans of the joists and beams.

b) The positions and types (i.e. built-in or on hangers) of all load bearing supports, including bearing lengths.

c) The design dead load or the type and weight of all floor and ceiling coverings.

d) Confirmation of the building occupancy and design imposed loads.

e) The position and weight of all internal non-load bearing walls.

f) The size of any additional loads (e.g. hoist tracks, dormer construction).

g) Details of rims for timber-frame construction.

h) Location and size of any services to be passed through the floor void.

i) Location and size of floor openings.

j) Any special features relating to floor construction.

The floor designer should provide the building designer with the following information, preferably in the form of detailed technical drawings.

1) The size and specification of all structural members.

2) The positions of all supports used in the design, including minimum required bearing lengths.

3) The loadings and other conditions for which the joists have been designed.

4) The positions and spacing of all the floor components.

5) The reactions at the support positions.

6) The design and specification for all joist to joist, and joist to timber beam, connections.

7) The design and specification for all joist and timber beam to masonry connections.

8) Details of where additional restraint may be required to the compression flange of the joists (e.g. near internal support of continuous joist where no ceiling is directly attached to the joist bottom flange).

9) Any special precautions for handling, storage or erection.

The building designer should use the information provided by the floor designer to check that the floor components supplied are suitable for their intended use.

Annex B (informative) Effective anchorage of floors to timber frame wall buildings of Consequence Class 2a

Effective anchorage of floors to timber frame walls in buildings of Consequence Class 2a (see BS EN 1991-1-7:2006) may be provided as shown in Figure B.1 provided that all of the conditions below are adhered to.

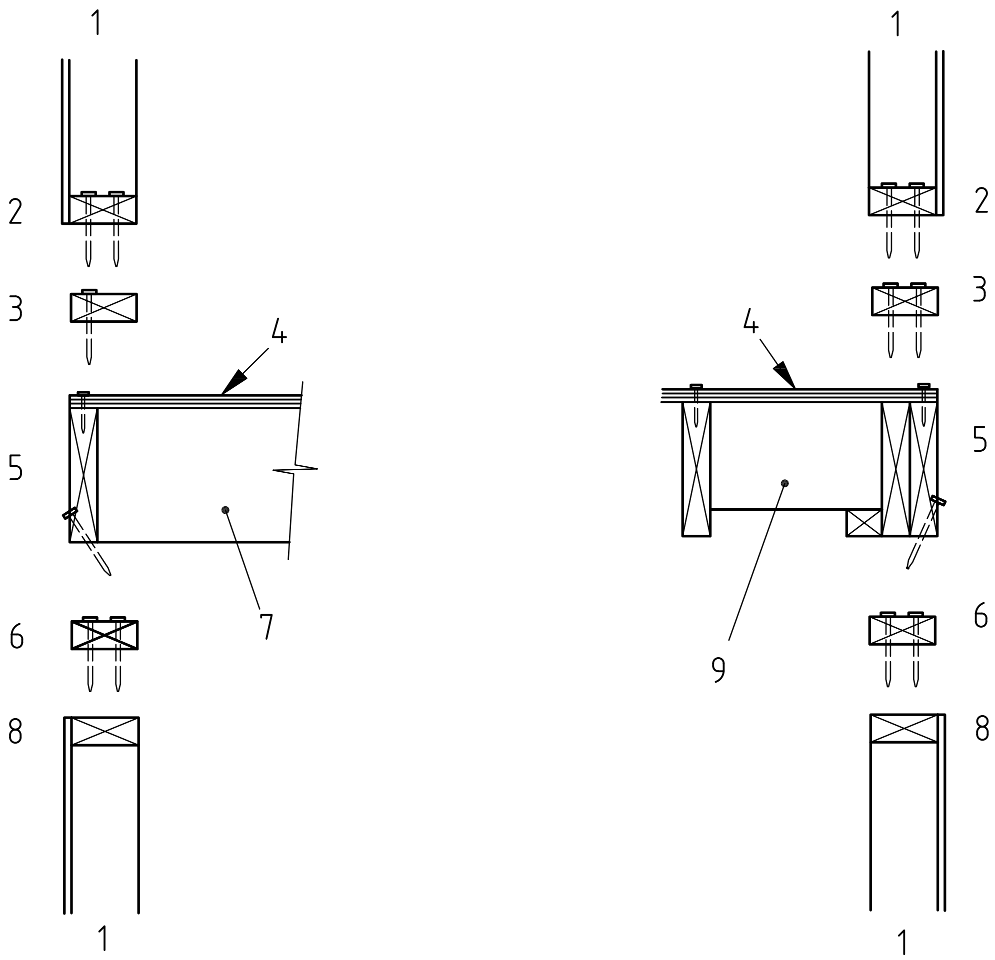
a) Floor joists to be at a maximum of 610 mm centres.

b) Structural deck is to be of minimum thickness 15 mm unless particleboard in which case minimum thickness to be 18 mm. Structural deck to extend across top surface of rim beam.

c) Minimum fixing specification for all horizontal interfaces is 3.1 mm diameter nails (minimum pointside penetration of 37 mm) at 300 mm centres.

d) For ceilings supported via resilient bars, blockings (of depth > 0.75 joist depth) are required between joists at a maximum of 2 000 mm centres. Blockings to be fixed at each end by a minimum of 2 no. 75 mm long × 3.1 mm diameter nails.

Figure B.1 **Details of effective anchorage of floors to timber frame walls in buildings of Consequence Class 2a**



a) Floor joists perpendicular to timber frame wall b) Floor joists parallel to timber frame wall

Key

1 Wall panel
2 Bottom rail
3 Sole plate
4 Structural deck
5 Rim beam
6 Head binder
7 Floor joists
8 Top rail
9 Blocking in end bay when ceiling supported via resilient bars

Annex C (informative) Actions and combinations of actions that may be considered in the design of trussed rafters

C.1 General

The actions that might be relevant to the design of trussed rafters that do not contain a habitable room are summarized in Table C.1.

C.2 Floor imposed loading for attic truss configuration

Trussed rafters having an attic truss configuration should, over the room area (even it is not intended that the room be habitable), be designed for at least Category A imposed loading, as given in the UK NA to BS EN 1991-1-1, Table NA.3, together with an allowance for partitions.

C.3 Action combinations

NOTE The action combinations given in BS EN 1990:2002+A1:2005, ***6.4.3.2****, can be considered for strength verification and BS EN 1995-1-1:2004+A1:2008,* ***2.2.3****, expression (2.2) can be considered for serviceability verification.*

For all duo-pitch or mono-pitch trussed rafters that do not contain a habitable room, the action combinations that may be considered in design as a minimum are summarized in Table C.2.

Table C.1 **Summary of actions for duo-pitch and mono-pitch trussed rafters**

Load type	**Symbol**	**Characteristic load**	**Position on trussed rafter**	**Load duration/ class**	Ψ_0
Truss self-weight	$G_{k,1}$	As appropriate	Full length of external and internal members	Permanent	n.a.
Tiles and battens	$G_{k,2}$	In absence of more specific information, $G_{k,2,sup}$ = 0,66 kN/m^2	Full length of top chord	Permanent	n.a.
Ceiling and insulation	$G_{k,3}$	In absence of more specific information, $G_{k,3,sup}$ = 0,25 kN/m^2	Full length of bottom chord	Permanent	n.a.
Water tank	$Q_{k,1}$	For tanks of capacity ≤300 l, $Q_{k,1}$ = 2 × 0,45 kN For tanks of capacity ≤450 l, $Q_{k,1}$ = 2 × 0,675 kN	For tanks supported as shown in Figure F.1, at two bottom chord nodes nearest water tank	Long term	$\psi_{0,1} = 1,0$
Plant and services	$Q_{k,2}$	As appropriate	As appropriate	Long term	$\psi_{0,2} = 1,0$
Storage load	$Q_{k,3}$	0,25 kN/m^2	Full length of bottom chord	Long term	$\psi_{0,3} = 1,0$
Snow	$Q_{k,4}$	As appropriate	Full length of top chord	Short term	$\psi_{0,4} = 0,5$
Snow (asymmetrical)	$Q_{k,5}$	As appropriate	Asymmetrically distributed to either top chord	Short term	$\psi_{0,5} = 0,5$

Table C.1 **Summary of actions for duo-pitch and mono-pitch trussed rafters**

Man load on top chord	$Q_{k,6}$	0,9 kN, only applicable for roof pitches ≤ 30°	Centre and each end of any top chord bay and 300 mm from end of unsupported overhangs > 600 mm	Short term	$\psi_{0,6} = 1,0$
Top chord imposed load in accordance with Table NA.7 of UK NA to BS EN 1991-1-1: 2002	$Q_{k,7}$	Pitch < 30°, 0,6 kN/m² 30°≤Pitch<60°, 0,6[(60-Pitch)/30] kN/m² Pitch > 60°, 0 kN/m²	Full length of top chord	Short term	$\psi_{0,7} = 0,7$
Man load, bottom chord	$Q_{k,8}$	0,9 kN	Centre and each end of any bottom chord bay where clearance >1,2 m	Short term	$\psi_{0,8} = 1,0$
Wind	$Q_{k,9}$	As appropriate	Full length of chord including end vertical members which are exposed to wind	Instantaneous	$\psi_{0,9} = 0,5$

Table C.2 **Summary of action combinations for duo-pitch and mono-pitch trussed rafters**

Action combination no.	**Combination of actions**		**Load duration**
AC01	**Description**	Permanent actions	Permanent
	Equation	$\Sigma\gamma_G G_{k,j} = \gamma_G G_{k,1} + \gamma_G G_{k,2} + \gamma_G G_{k,3}$	
AC02	**Description**	Permanent actions + water tank + plant + storage	Long term
	Equation	$\Sigma\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3}$	
AC03	**Description**	Permanent actions + water tank + plant + storage + snow (undrifted) + man (BC)	Short term
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,4} + \gamma_Q Q_{k,8}$	
AC04	**Description**	Permanent actions + water tank + plant + storage + service load + man (BC)	Short term
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,7} + \gamma_Q Q_{k,8}$	
AC05	**Description**	Permanent actions + water tank + plant + storage + snow (drifted left) + man (BC)	Short term
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,5} + \gamma_Q Q_{k,8}$	
AC06	**Description**	Permanent actions + water tank + plant + storage + snow (drifted right) + man (BC)	Short term
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,5} + \gamma_Q Q_{k,8}$	
AC07	**Description**	Permanent actions + water tank + plant + storage + man (TC)	Short term
	Equation	$\Sigma\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,6}$	

Table C.2 **Summary of action combinations for duo-pitch and mono-pitch trussed rafters**

AC08	**Description**	Permanent actions + water tank + plant + storage + wind left + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,9}$	
AC09	**Description**	Permanent actions + water tank + plant + storage + wind right + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,9}$	
AC10	**Description**	Permanent + water tank + plant + storage + snow (left) + wind (right) + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,5} + \psi_{0,9}\gamma_Q Q_{k,9}$	
AC11	**Description**	Permanent + water tank + plant + storage + snow (left) + wind (right) + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \psi_{0,5}\gamma_Q Q_{k,5} + \gamma_Q Q_{k,9}$	
AC12	**Description**	Permanent + water tank + plant + storage + snow (right) + wind (left) + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,5} + \psi_{0,9}\gamma_Q Q_{k,9}$	
AC13	**Description**	Permanent + water tank + plant + storage + snow (right) + wind (left) + internal suction	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \psi_{0,5}\gamma_Q Q_{k,5} + \gamma_Q Q_{k,9}$	
AC14	**Description**	Permanent actions + water tank + plant + storage + wind left + internal pressure	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,9}$	
AC15	**Description**	Permanent actions + water tank + plant + storage + wind right + internal pressure	Instantaneous
	Equation	$\Sigma\xi\gamma_G G_{k,j} + \gamma_Q Q_{k,1} + \gamma_Q Q_{k,2} + \gamma_Q Q_{k,3} + \gamma_Q Q_{k,9}$	
AC16	**Description**	Permanent actions + wind along ridge + internal pressure	Instantaneous
	Equation	$\Sigma\gamma_{G,inf} G_{k,j} + \gamma_Q Q_{k,9}$	

NOTE 1 $\gamma_G = 1{,}35$; $\gamma_{G,inf} = 1{,}0$; $\gamma_Q = 1{,}5$; $\xi = 0{,}925$

NOTE 2 ψ_0 *has only been included in the equations if the value for the associated variable load is less than 1,0.*

NOTE 3 Combination of actions including a man load have many sub-combination cases (i.e. man load positioned at centre and end of each bay).

NOTE 4 All of the above action combinations use expression 6.10b with the exception of AC07 and AC16 which use 6.10a.

Annex D (informative) Masonry shielding to timber frame wall diaphragms

For buildings up to three storeys high, it may be assumed that the shaded area of brickwork in Figure D.1 would shield the timber frame structure behind it from the wind pressure or wind suction exerted on that area of the gable, where the following statements are applicable.

a) The length, L, in m, is the shortest length of the following:

1) 1,3;

2) 1,3 A;

3) the distance from the corner of the wall to any windows or movement joints;

4) for walls under wind pressure, where $A < 0{,}28H$, $(A^2 + \sum B_i^2)/(0{,}56H - 2A)$;

5) for walls under wind suction, $(A^2 + \sum B_i^2)/0{,}46H$.

b) The number of walls, B_i, which might be considered to resist wind load on the gable, are limited by any movement joints. In addition, it is important that any wall, B_i, which is used to resist the wind load on one gable is not simultaneously used to resist wind load on the opposite gable.

c) The unfactored wind pressure/suction on the face (the peak velocity pressure multiplied by the external pressure coefficient on that face, both determined in accordance with BS EN 1991-1-4) does not exceed 0,6 kN/m² or 0,5 kN/m² in pressure and suction respectively.

d) The brickwork is fully bonded at the corner and laterally tied back to the timber frame in accordance with BS EN 1996-1-1.

e) The wall is a minimum of 100 mm thick and constructed with Group 1 clay masonry units of Category 2 and Class 2 execution control, having a density of 18 kN/m³, water absorption of 7%–12%, and a normalized mean compressive strength of 15 N/mm², in accordance with BS EN 771-1:2011 and BS EN 772-7:1998.

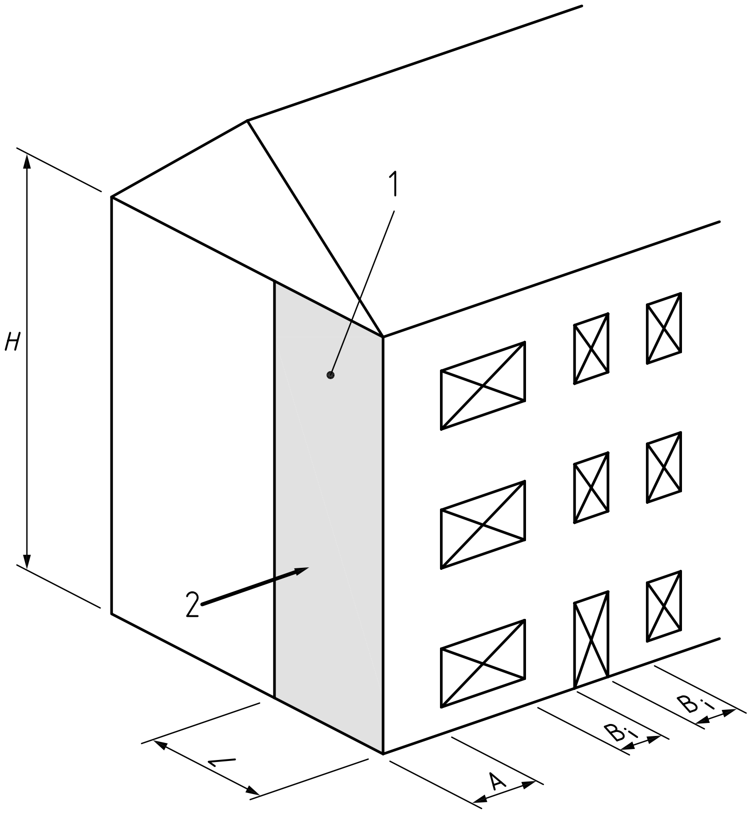
f) The wall is constructed with a prescribed masonry mortar Class M4, in accordance with BS EN 998-2: 2010.

g) Notwithstanding a) to f), the brickwork designer is still responsible for the full design of the brickwork, including checking the brickwork for resistance to the local external pressure coefficients while taking account of the pressures within the cavity.

h) The timber frame designer advises the brickwork designer of the assumptions which have been made in calculating the wind loads on the timber frame and therefore the loads which the brickwork is assumed to carry.

i) The timber frame designer checks that the timber frame is able to torsionally restrain the L x A angled section of brickwork (see Figure D.1).

Figure D.1 **Area of brickwork providing wind shield to timber frame structure**



Key

1 Shaded area may be assumed to be fully shielded from wind load in direction shown

2 Direction of applied wind load (±)

Annex E (normative) Bracing of trussed rafter roofs

E.1 General

Standard bracing arrangements should be selected in accordance with the flow diagrams in Figure E.1 and Figure E.2.

NOTE 1 A standard method for providing roof and wall stability has been evolved for spans up to 17 m from industry experience in the use of trussed rafters for domestic scale roofs.

NOTE 2 Figure E.1 enables the selection of suitable bracing details at rafter level for adequate roof stability at this level. Figure E.2 enables the plasterboard diaphragm at ceiling level to be appraised as a suitable means of maintaining wall stability at this level.

In addition to the bracing at rafter and ceiling level defined in Annex E, additional bracing may be required to stabilize the gable walls at intermediate levels. The building designer should be responsible for designing this additional bracing and verifying the overall stability of the masonry wall.

The standard bracing details derived in accordance with Annex E may be used without further calculation. If no standard bracing solution is available, then a specific bracing system should be designed by the building designer following the principles given in **E.3**.

Hipped ends on a trussed rafter roof may provide a satisfactory alternative to the bracing shown in Figure E.3 for the area contained by the hip end. Where the length of roof between the hip ends exceeds 1.8 m, this section should be braced, as shown in Figure E.3.

E.2 Sarking materials

NOTE 1 For the purposes of Annex E, sarking is defined as structural boarding used for the purposes of bracing the roof.

Where certain sarking materials are directly fixed to the top face of the rafter members, it is permissible to omit the rafter diagonal bracing, chevron bracing and longitudinal bracing at rafter level. This omission is acceptable where the sarking material is moisture resistant and provides an equivalent level of restraint to out-of-plane instability and wind forces.

NOTE 2 The minimum thicknesses of some sarking materials that are required to meet the bracing requirements only are given in Table E.1. Greater thicknesses might be necessary to meet imposed loading and durability requirements.

Sarking materials should be nailed to the rafters with an allowance being made for moisture related movement between boards. Joints between square-edged boards should be supported on timber noggings. Cantilevering or splicing of boards between rafters is not recommended.

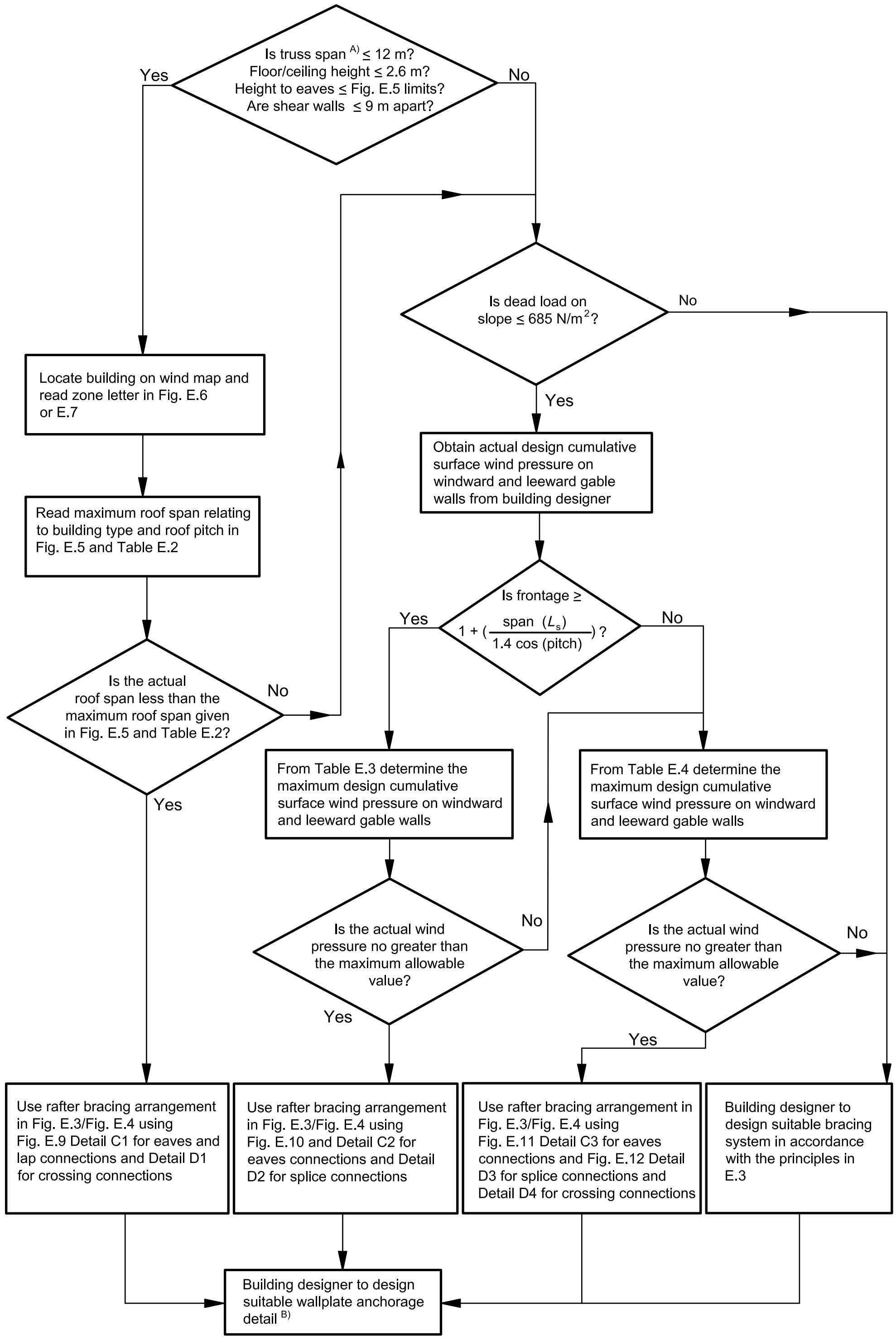
NOTE 3 It is important that the stability, verticality and straightness of the truss rafters is maintained during erection throughout the period of fixing of the sarking material.

Table E.1 **Thickness and fixing of sarking materials**

Material	**Minimum thickness** mm	**Fixing**
Plywood Oriented strand board (OSB) Chipboard	9 [A)] 9 [A)] 12 [A)]	3,0 mm diameter 50 mm long galvanized round wire nails fixed at 200 mm centres to every trussed rafter
Timber boarding (no more than one board in four may be jointed on any one rafter member)	16	Two 3,0 mm diameter 50 mm long galvanized round wire nails per board fixed to every trussed rafter

[A)] Suitable only where roof coverings (e.g. slates and tiles) are independently supported on battens, secured to counter battens. In all other cases, roof coverings may be attached directly to the board.

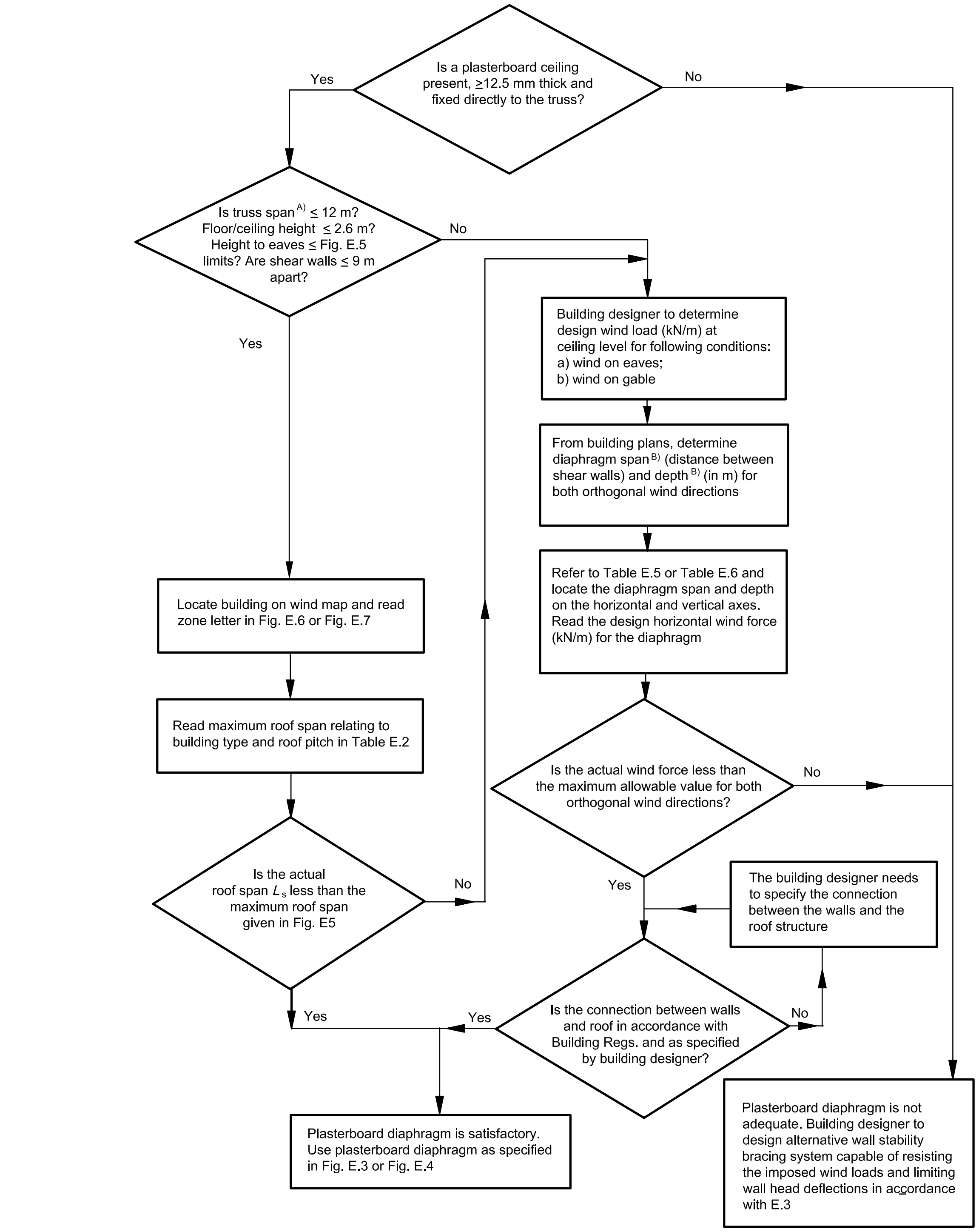
Figure E.1 **Procedure for the design of roof bracing at rafter level**



[A)]For monopitch roofs, use a span which is twice the actual span, Ls.

[B)]The wallplate anchorage detail should be designed to transfer the forces stated for the rafter diagonal braces in Figure E.9, Figure E.10 and Figure E.11 into the supporting structure.

Figure E.2 **Procedure for the design of roof bracing at ceiling level**

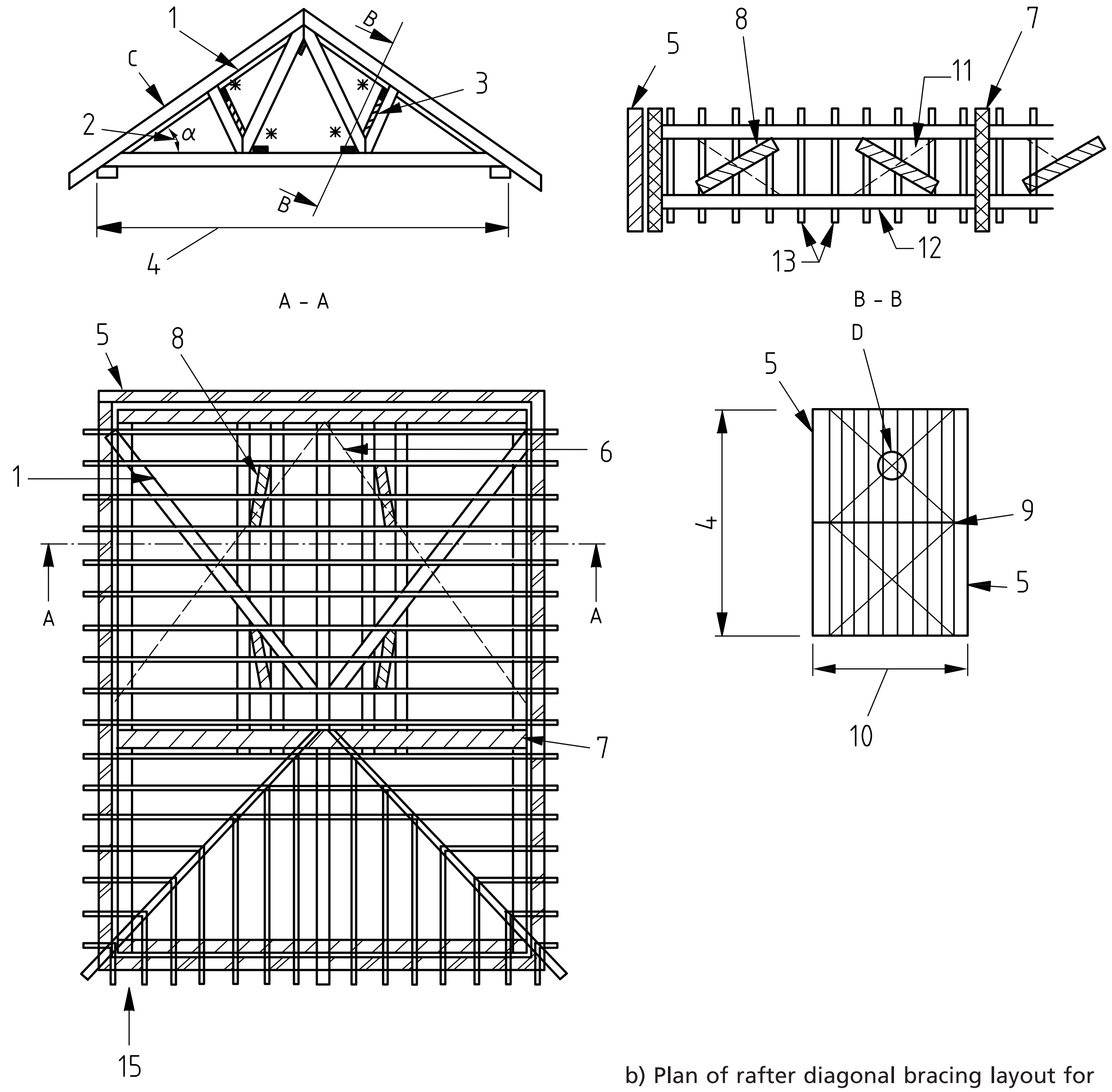


A) For monopitch roofs, use a span which is twice the actual span, L_s.

B) The diaphragm span and depth should be defined from any unbroken area of plasterboard bounded on each edge by timber wallplates, head plates or trusses which can be deemed to act as flanges to the diaphragm.

C) The wallplate anchorage detail should be designed to transfer the forces stated for the rafter diagonal braces given in Figure E.9 and Figure E.10.

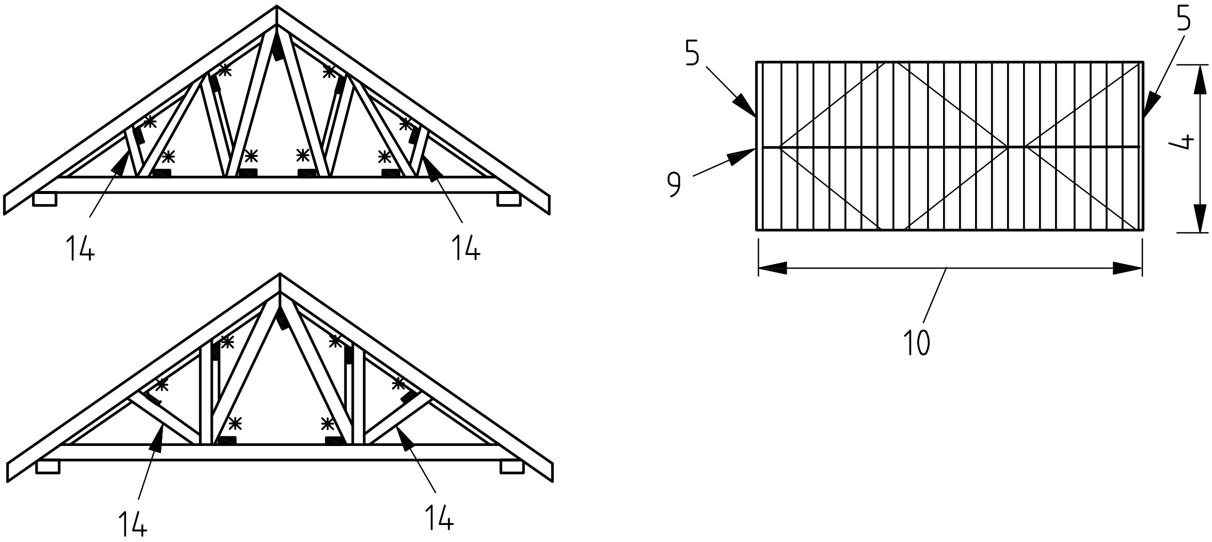
Figure E.3 **Standard bracing for rafter and web members of duopitch trussed rafters**



a) Plan of roof

b) Plan of rafter diagonal bracing layout for narrow fronted detached roofs (rafter braces cross)

Figure E.3 **Standard bracing for rafter and web members of duopitch trussed rafters**



c) Section A–A using WW truss (upper figure) and using fan truss (lower figure)

d) Plan of rafter diagonal bracing layout for wider-fronted detached roofs (rafter braces do not need to cross)

Key to Figure E.3

1	Rafter diagonal bracing nailed to wall plate	9	Ridge
2	Roof pitch, α	10	Frontage, *F*
3	Chevron bracing to webs	11	Alternative direction of chevron brace
4	Span of trussed rafter, L_s	12	Longitudinal brace
5	Gable end	13	Maximum of two trusses between chevron braces
6	Alternative direction of rafter diagonal bracing	14	For spans in excess of 11 m, additional chevron bracing is required
7	Party wall	C)	For details on view C [see Figure E.3a)], see Figure E.9, Figure E.10, Figure E.11 and Figure E.12
8	Chevron brace to be at approximately 45° and nailed to at least 3 trusses	D)	For details on view D [see Figure E.3b)], and refer to Figure E.9, detail D1 and Figure E.12, detail D4

Figure E.3 **Standard bracing for rafter and web members of duopitch trussed rafters**

Specification notes to Figure E.3

a) The maximum trussed rafter spacing is 600 mm.

b) Horizontal lateral connections between masonry walls and the roof structure are in accordance with the recommendations given in BS 8103-1 and are fixed at both rafter and ceiling tie level.

c) The ceiling is of plasterboard throughout (plasterboard conforming to BS EN 520 and that is of a minimum thickness of 12.5 mm and installed in accordance with BS 8212), or of similar rigid material fixed either directly to the bottom chords of the trussed rafters or to continuous counter battens which are fixed directly to the bottom chords of the trussed rafters. Where the ceiling is less rigid than plasterboard or is omitted, extra bracing will normally be required at ceiling level.

d) All bracing members are nailed to every trussed rafter they cross with two 3.35 mm diameter galvanized wire nails with a minimum length equal to the bracing thickness plus 32 mm. In all details 3.1 mm × 90 mm long, mechanically driven gun nails may be substituted for 3.35 mm × 65 mm long wire nails.

e) At least four rafter diagonal braces as shown in Figure E.3 (lap jointed, as required) are fixed to the undersides of the rafter members ideally at 45° to the rafters but not less than 35° or greater than 55° measured normal to the roof slope.

f) Longitudinal bracing members (lap jointed, if required) extend over the whole length of the roof and tightly abut the face of every gable and party wall.

g) A longitudinal bracing member is located at the apex and either:
   1) located at all other nodes (excluding support points); or
   2) where intermediate longitudinal bracing members are omitted, the resultant spacing between longitudinal braced nodes does not exceed 4.2 m measured along each rafter and 3.7 m measured along each ceiling tie, and temporary battens are installed to assist in the correct erection and alignment of the trussed rafters.

h) Internal compression members are provided with lateral restraint, where required by the trussed rafter designer.

i) On trussed rafter roofs with spans in excess of 8 m for duopitch roofs, chevron bracing as shown in Figure E.3 is required. For spans in excess of 11 m, additional chevron bracing is required.

j) Rafter diagonal bracing and chevron bracing members extend over the whole length of the roof except that this bracing may be omitted from no more than two trussed rafters between sets of bracing and single trussed rafters adjacent to the faces of gable and party walls.

NOTE 1 *Chevron bracing shown is not required on internal members of trusses with spans of 8 m or less.*

NOTE 2 * *denotes longitudinal bracing not required when the criteria described in specification note g)2) are met.*

NOTE 3 For the purposes of Annex E, a building may be defined as narrow fronted when the frontage is:

$$F < 1 + \left(\frac{L_s}{1.4\cos\alpha}\right)$$

E.3 Non-standard roof bracing

Where no standard bracing solution is available, a bracing system should be designed in accordance with BS EN 1995-1-1:2004+A1:2008, **9.2.5.3**. The bracing system, which may consist of a combination of elements (such as those given in Annex E or other suitable structural systems), should be capable of restraining all out-of-plane forces on the roof without excessive deflection (whether induced by external wind forces acting on the walls or by internal compression instability forces in the truss members themselves).

The bracing forces should be determined on the basis of the most unfavourable combinations of geometrical or structural imperfections and induced deflections under load.

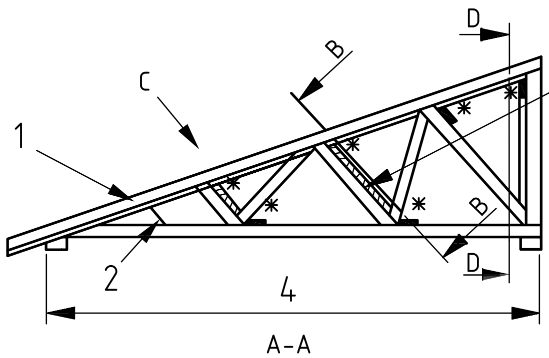
NOTE For detailed information about calculating the compression induced out-of-plane instability force to be resisted by the bracing system, see BS EN 1995-1-1:2004+A1:2008, ***9.2.5.3****[1]), and NA to BS EN 1995-1-1:2004+A1:2008, NA* ***2.11****.*

Where the bracing system is also required to serve the purpose of wall stability, out-of-plane wind forces should be combined with the appropriate compression induced instability force.

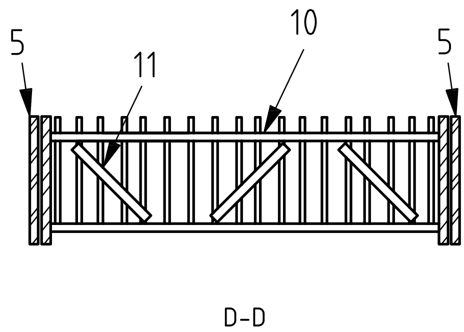
Whether the bracing system is either a purpose designed bracing system at rafter or ceiling level, or a prefabricated wind girder or ring beam, the bracing system chosen should be capable of resisting the imposed loads (wind and/or instability) and of limiting the horizontal deflection. The deflection limit should be agreed with the building designer; the values below may be used as guidance.

Limit on horizontal deflection = min 10 mm / *l*/500.

Figure E.4 **Standard bracing for rafter and web members of mono-pitch trussed rafters**

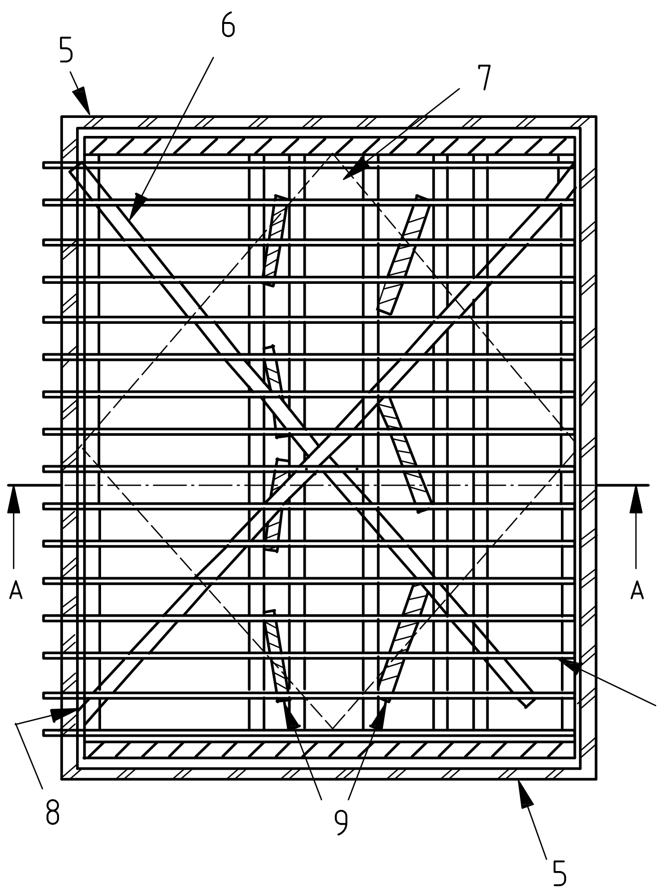


a) Section through roof

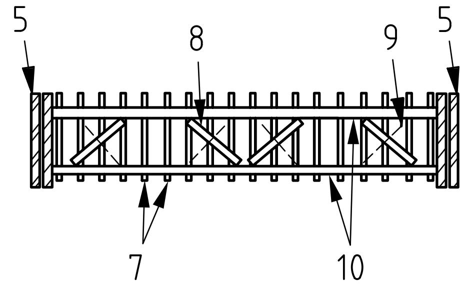


d) Chevron bracing on vertical end

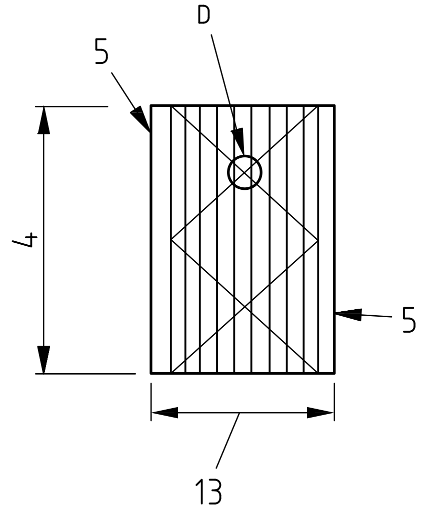
Figure E.4 **Standard bracing for rafter and web members of mono-pitch trussed rafters**



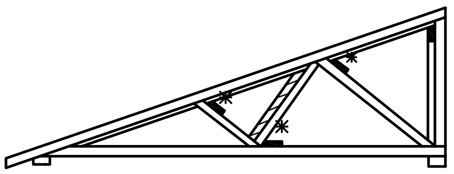
b) Plan of roof

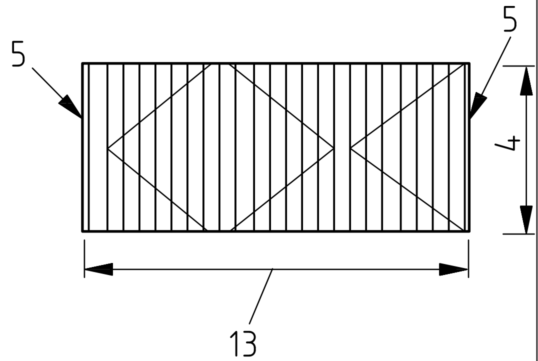


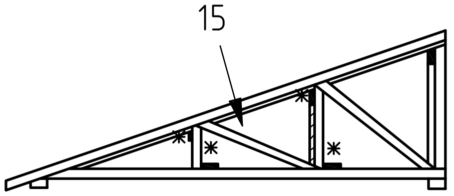
e) Chevron bracing on diagonal web



f) Plan of rafter diagonal bracing layout for narrow-fronted detached roofs (rafter braces cross)







c) Section on roof (using alternative mono trusses)

g) Plan of rafter diagonal bracing layout for wider fronted detached roofs (rafter braces do not need to cross)

Key to Figure E.4

1 Rafter diagonal bracing nailed to wall
2 Roof pitch, α
3 Chevron bracing to webs
4 Span of trussed rafter, L_s
5 Gable end
9 Alternative direction of chevron bracing
10 Longitudinal bracing
11 Web diagonal bracing
12 For spans in excess of 8,0 m, additional bracing is required

Figure E.4 **Standard bracing for rafter and web members of mono-pitch trussed rafters**

6 Alternative direction of rafter diagonal

7 Maximum of two trusses between chevron braces

8 Chevron brace to be approximately 45° and nailed to at least three trusses

13 Frontage *F*

C) For details on view C [see Figure E.4.a), section A–A)] see Figure E.9, Figure E.10, Figure E.11 and Figure E.12

D) For details on view D [see Figure E.4.f)], see Figure E.9, detail D1 and Figure E.12, detail 4

Specification notes to Figure E.4

a) The maximum trussed rafter spacing is 600 mm.

b) Horizontal lateral connections between masonry walls and the roof structure are in accordance with the recommendations given in BS 8103-1 and fixed at both rafter and ceiling tie level.

c) The ceiling is of plasterboard throughout (plasterboard which conforms to BS EN 520 and which is of a minimum thickness of 12.5 mm and installed in accordance with BS 8212), or of similar rigid material fixed either directly to the bottom chords of the trussed rafters or to continuous counter battens which are fixed directly to the bottom chords of the trussed rafters. Where the ceiling is less rigid than plasterboard or is omitted, extra bracing will normally be required at ceiling level.

d) All bracing members are nailed to every trussed rafter they cross with two 3.35 mm diameter galvanized wire nails with a minimum length equal to the bracing thickness plus 32 mm. In all details 3.1 mm × 90 mm long mechanically driven gun nails may be substituted for 3.35 mm × 65 mm long wire nails.

e) At least two rafter diagonal braces, as shown in Figure E.4, (lap jointed, as required) are fixed to the undersides of the rafter members ideally at 45° to the rafters but not less than 35° or greater than 55° measured normal to the roof slope.

f) Longitudinal bracing members (lap jointed, if required) extend over the whole length of the roof and tightly abut the face of every gable and party wall.

g) A longitudinal bracing member is located at the apex and either:
   1) is located at all other nodes (excluding support points); or
   2) where intermediate longitudinal bracing members are omitted, the resultant spacing between longitudinal braced nodes does not exceed 4.2 m measured along each rafter and 3.7 m measured along each ceiling tie, and temporary battens are installed to assist in the correct erection and alignment of the trussed rafters.

h) Internal compression members are provided with lateral restraint where required by the trussed rafter designer.

i) On trussed rafter roofs with spans in excess of 5 m for monopitch roofs, chevron bracing as shown in Figure E.4 is required. For spans in excess of 8 m additional chevron bracing is required.

j) Rafter diagonal bracing and chevron bracing members extend over the whole length of the roof except that this bracing may be omitted from no more than two trussed rafters between sets of bracing and single trussed rafters adjacent to the faces of gable and party walls.

k) Monopitch trussed rafters also require diagonal bracing fixed to the inside face of the end vertical if this member is not laterally restrained at its top or bottom by connection to a wall, beam or by being clad in plywood or similar rigid sheet material adequately fixed to the truss.

NOTE 1 Chevron bracing shown is not required on internal members of trusses having spans of 5 m or less.

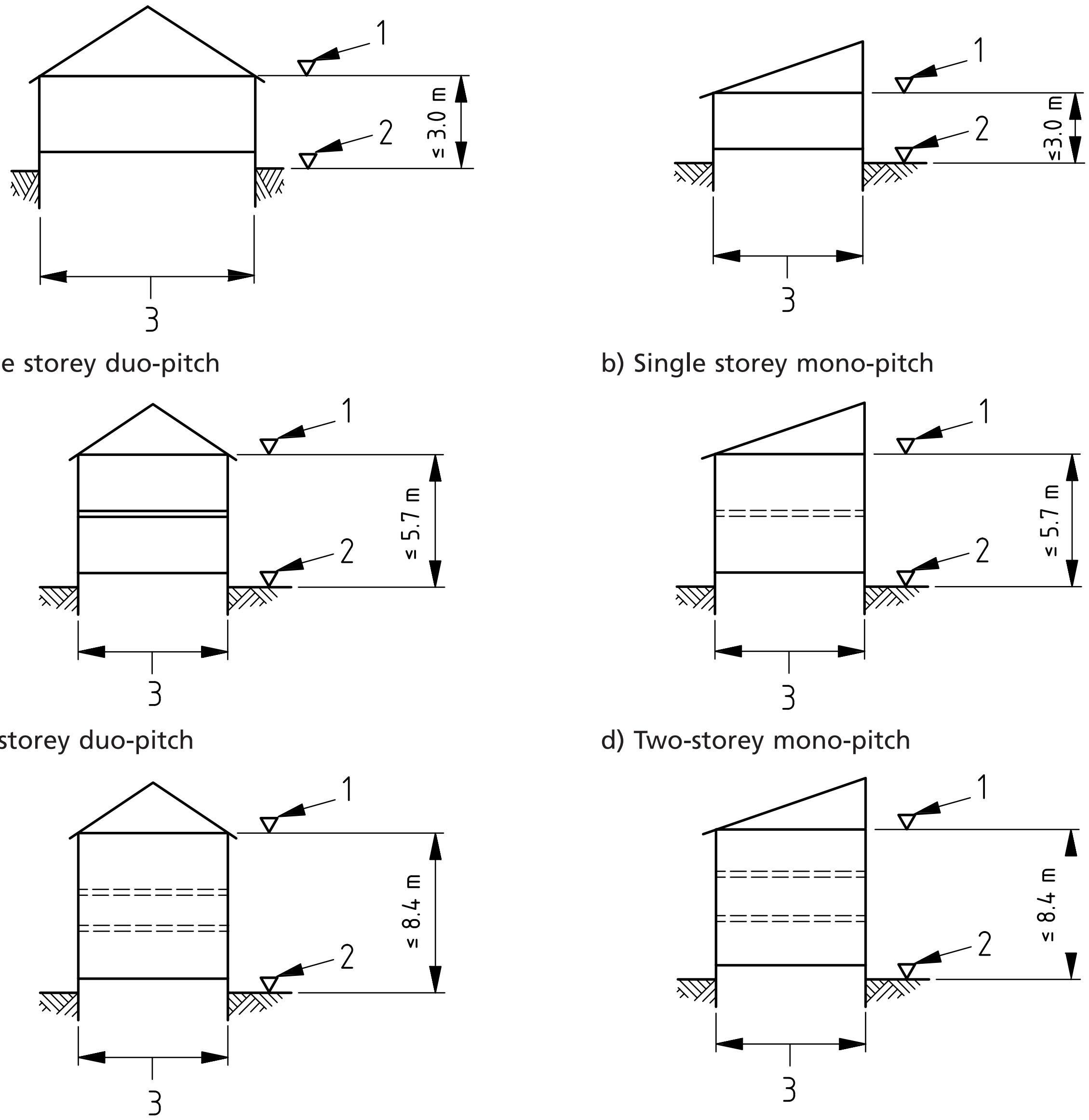
*NOTE 2 * denotes longitudinal bracing not required when the criteria described in specification note g)2) are met.*

NOTE 3 For the purposes of this annex, a building may be defined as narrow fronted when the frontage is:

Figure E.4 **Standard bracing for rafter and web members of mono-pitch trussed rafters**

$$F < 1 + \left(\frac{L_s}{1.4\cos\alpha}\right)$$

Figure E.5 **Limiting spans for standard bracing of trussed rafter roofs** [A)]



a) Single storey duo-pitch

b) Single storey mono-pitch

c) Two-storey duo-pitch

d) Two-storey mono-pitch

e) Three-storey duo-pitch

f) Three-storey mono-pitch

Key

1 Underside of ceiling tie

2 Ground level

3 Truss span L_s

[A)] For maximum truss spans, see Table E.2.

Table E.2 **Maximum truss spans for Figure E.5**

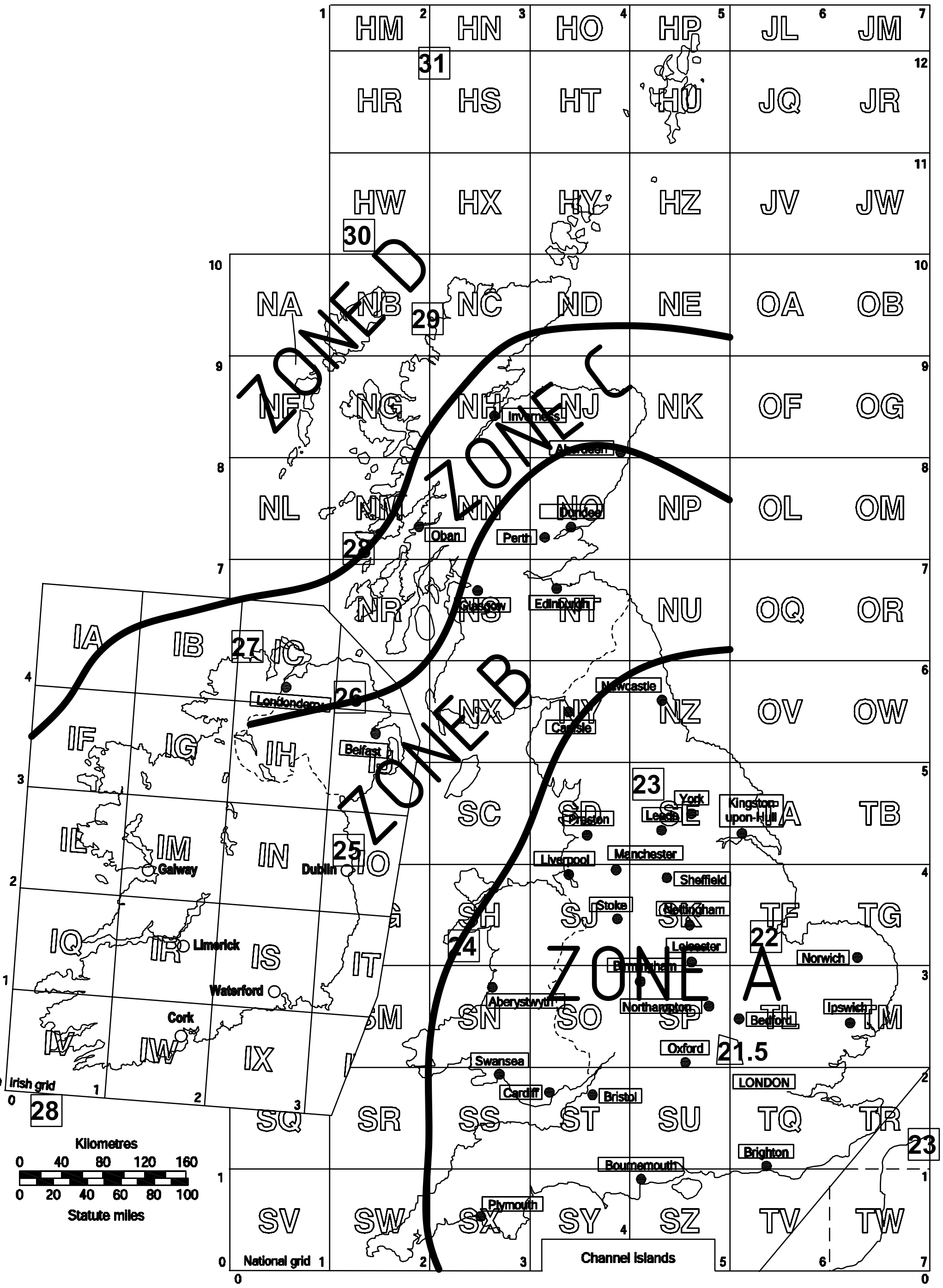
Building profile (see Figure E.5)	Maximum truss span, L_s				
	Roof pitch	Zone A A)	Zone B A)	Zone C A)	Zone D A)
	°	m	m	m	m
Single storey duo-pitch					
	22,5	12,0	12,0	12,0	12,0
	25,0	12,0	12,0	12,0	11,3
	27,5	12,0	12,0	11,4	10,2
	30,0	12,0	11,6	10,3	9,1
	32,5	12,0	10,5	9,3	8,0
	35,0	11,3	9,4	8,7	7,7
	37,5	10,2	8,9	7,8	6,7
	40,0	9,2	7,9	6,8	6,3
	42,5	8,1	7,6	6,5	5,4
	45,0	7,7	6,6	5,6	5,2
Single-storey mono-pitch					
	22,5	9,6	8,4	7,2	6,5
	25,0	8,5	7,2	6,5	5,8
	27,5	7,7	6,5	5,8	5,2
	30,0	6,9	5,9	5,2	4,5
	32,5	6,3	5,3	4,6	4,0
	35,0	5,7	5,0	4,4	3,8
	37,5	5,2	4,5	3,9	3,4
	40,0	4,6	4,0	3,4	3,2
	42,5	4,4	3,8	3,2	2,8
	45,0	3,9	3,3	2,8	2,7
Two-storey duo-pitch					
	22,5	12,0	12,0	12,0	10,7
	25,0	12,0	12,0	10,7	9,3
	27,5	12,0	11,4	9,4	8,8
	30,0	11,6	10,2	8,9	7,8
	32,5	10,5	9,1	7,9	6,8
	35,0	9,3	8,1	6,9	6,4
	37,5	8,9	7,7	6,6	5,5
	40,0	7,9	6,7	5,6	5,2
	42,5	7,5	6,4	5,3	4,4
	45,0	6,6	5,5	5,1	4,2
Two-storey mono-pitch					
	22,5	8,3	7,1	6,3	5,6
	25,0	7,2	6,4	5,7	5,0
	27,5	6,5	5,7	5,1	4,5
	30,0	5,9	5,2	4,5	3,9
	32,5	5,3	4,5	4,0	3,4
	35,0	5,0	4,0	3,7	3,2
	37,5	4,5	3,8	3,3	2,8
	40,0	3,9	3,4	3,1	2,7
	42,5	3,7	3,2	2,7	2,7
	45,0	3,3	2,8	2,7	2,7

Table E.2 **Maximum truss spans for Figure E.5**

Building profile (see Figure E.5)	**Roof pitch** °	**Maximum truss span, L_s** Zone A [A)] m	Zone B [A)] m	Zone C [A)] m	Zone D [A)] m
Three-storey duo-pitch					
	22,5	12,0	12,0	10,7	9,3
	25,0	12,0	11,4	9,3	8,1
	27,5	11,6	10,2	8,9	7,7
	30,0	10,5	9,1	7,8	6,7
	32,5	9,3	8,0	6,8	6,3
	35,0	8,9	7,6	6,4	5,4
	37,5	7,9	6,7	5,5	5,2
	40,0	6,9	6,3	5,2	4,3
	42,5	6,6	5,4	4,4	4,1
	45,0	5,6	5,2	4,2	3,9
Three-storey mono-pitch					
	22,5	7,6	6,4	5,6	4,6
	25,0	6,6	5,7	5,0	4,0
	27,5	5,9	5,1	4,5	3,8
	30,0	5,3	4,5	3,9	3,3
	32,5	5,0	4,0	3,4	3,1
	35,0	4,5	3,8	3,2	2,7
	37,5	3,9	3,3	2,8	2,7
	40,0	3,7	3,2	2,7	2,7
	42,5	3,3	2,7	2,7	2,7
	45,0	3,1	2,7	2,7	2,7

[A)] Wind zones are shown in Figure E.6 and Figure E.7.

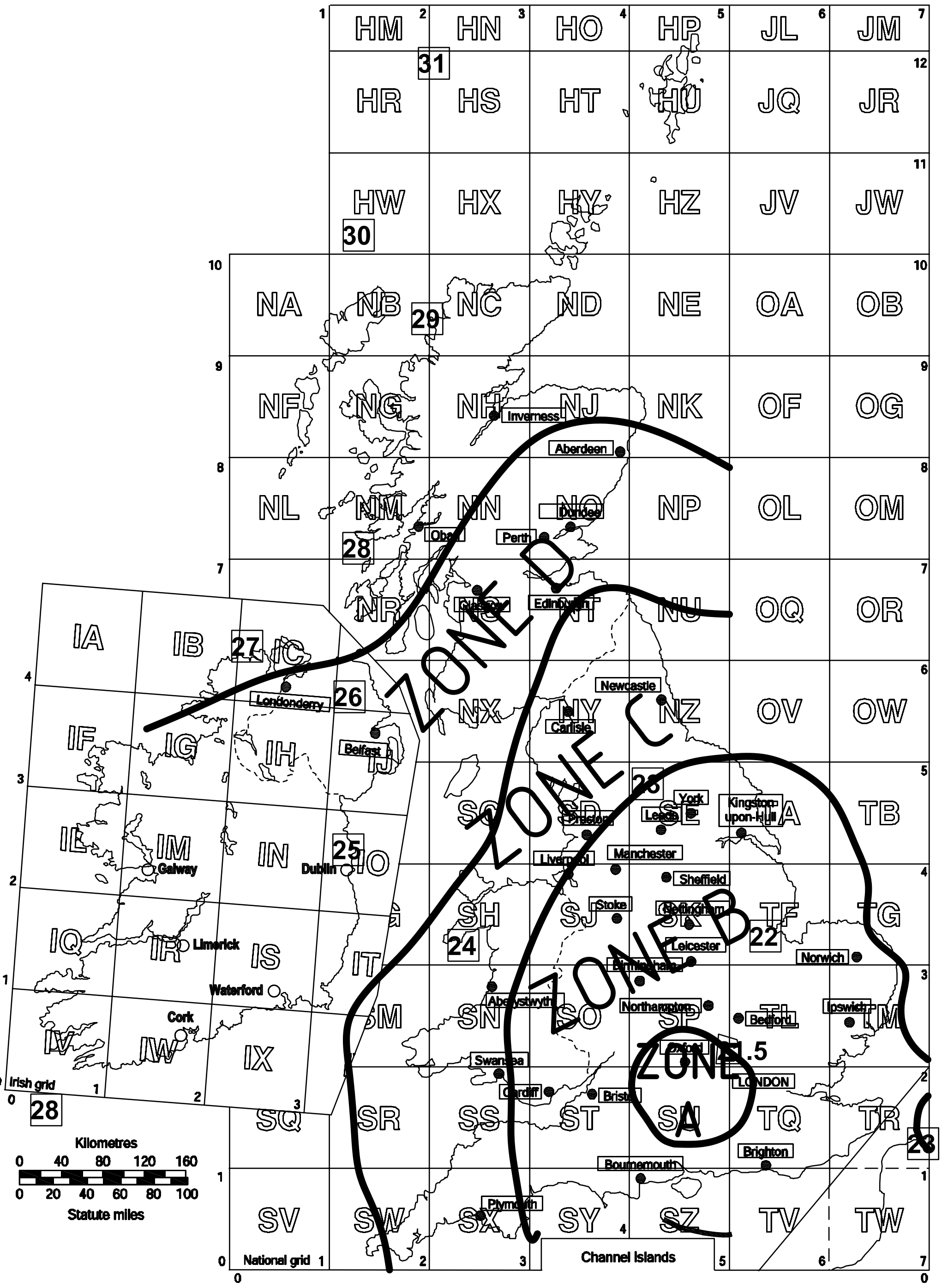
Figure E.6 **Basic wind zones for buildings at site altitudes ≤150 m**



NOTE 1 This map is intended for sites in the United Kingdom, Isle of Man and Channel Islands only.

NOTE 2 The isopleths in the Irish Republic are shown for purposes of interpolation only.

Figure E.7 **Basic wind zones for buildings at site altitudes between 150 m and 300 m**



NOTE 1 This map is intended for sites in the United Kingdom, Isle of Man and Channel Islands only.

NOTE 2 The isopleths in the Irish Republic are shown for purposes of interpolation only.

Table E.3 **Maximum design cumulative surface wind pressures (kN/m²) on windward and leeward gable walls for roofs constructed using the details of Figure E.10**

Roof pitch (°)	Maximum design cumulative surface wind pressure (kN/m²)on windward and leeward gable walls for roof of span (m)							
	8	9	10	11	12	13	14	15
22,5	S.B. [A)]	S.B. [A)]	S.B. [A)]	3,26	2,60	2,09	1,76	1,43
25,0	S.B. [A)]	S.B. [A)]	3,69	2,93	2,36	1,91	1,55	1,31
27,5	S.B. [A)]	4,17	3,27	2,66	2,13	1,73	1,41	1,16
30,0	4,94	3,78	2,97	2,36	1,95	1,58	1,29	1,07
32,5	4,49	3,45	2,70	2,15	1,74	1,41	1,19	0,98
35,0	4,08	3,14	2,46	1,97	1,59	1,29	1,05	0,86
37,5	3,68	2,82	2,21	1,80	1,46	1,19	0,96	0,80
40,0	3,36	2,58	2,03	1,61	1,29	1,05	0,86	N.A
42,5	3,08	2,36	1,85	1,47	1,19	0,96	N.A	N.A
45,0	2,78	2,12	1,65	1,31	1,05	N.A	N.A	N.A

NOTE Intermediate values may be obtained by linear interpolation.

[A)] S.B. refers to minimum standard bracing. See Figure E.3 or Figure E.4 together with Figure E.9.

Table E.4 **Maximum design cumulative surface wind pressures (kN/m2) on windward and leeward gable walls for roofs constructed using the details of Figure E.11 and E.12**

Roof pitch (°)	Maximum design cumulative surface wind pressure (kN/m²)on windward and leeward gable walls for roof of span (m)							
	10	11	12	13	14	15	16	17
22.5	S.B. [A)]	4,98	4,05	3,33	2,82	2,36	1,97	1,65
25.0	5,55	4,47	3,65	3,00	2,49	2,13	1,79	1,50
27.5	4,94	4,04	3,29	2,72	2,27	1,89	1,64	1,38
30.0	4,47	3,60	2,99	2,48	2,06	1,73	1,46	1,23
32.5	4,07	3,27	2,67	2,21	1,88	1,58	1,34	1,13
35.0	3,71	2,99	2,45	2,03	1,68	1,41	1,22	1,04
37.5	3,35	2,73	2,24	1,85	1,55	1,29	N.A	N.A
40.0	3,06	2,46	2,01	1,67	1,38	N.A	N.A	N.A
42.5	2,79	2,25	1,83	1,52	N.A	N.A	N.A	N.A
45.0	2,52	2,03	1,65	N.A	N.A	N.A	N.A	N.A

NOTE Intermediate values may be obtained by linear interpolation.

[A)] S.B. refers to minimum standard bracing. See Figure E.3 or Figure E.4 together with Figure E.9.

Table E.5 **Maximum design horizontal wind force (kN/m) at bottom chord level on 12.5 mm thick plasterboard ceiling diaphragms**

Diaphragm depth m	Diaphragm span A) B) m								
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
6	2,12	1,91	1,73	1,58	1,47	1,37	1,26	1,19	1,11
7	2,46	2,22	2,01	1,85	1,70	1,59	1,49	1,38	1,31
8	2,82	2,54	2,30	2,12	1,95	1,80	1,68	1,58	1,50
9	3,17	2,85	2,60	2,37	2,16	2,04	1,91	1,77	1,67
10	3,51	3,17	2,88	2,64	2,43	2,25	2,12	1,98	1,86
11	3,87	3,48	3,17	2,90	2,69	2,49	2,31	2,18	2,06
12	4,22	3,80	3,45	3,17	2,91	2,72	2,54	2,37	2,22
13	4,58	4,11	3,74	3,42	3,17	2,94	2,75	2,57	2,42
14	4,94	4,43	4,04	3,69	3,41	3,17	2,96	2,76	2,58
15	5,28	4,76	4,32	3,96	3,65	3,39	3,17	2,96	2,81
16	5,63	5,07	4,61	4,22	3,90	3,62	3,38	3,17	2,97
17	6,14	5,39	4,91	4,49	4,13	3,84	3,68	3,36	3,17

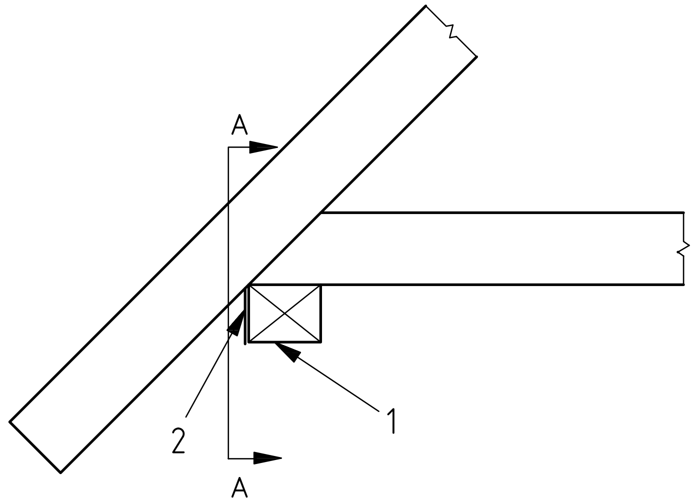
A) Intermediate values may be obtained by linear interpolation.
B) For plasterboard ceiling diaphragms, wallplates should be spliced in accordance with Figure E.8 or using an alternative splice detail capable of resisting a design instantaneous axial action of 4,9 kN.

Table E.6 **Maximum design horizontal wind force (kN/m) at bottom chord level on 15 mm thick plasterboard ceiling diaphragms**

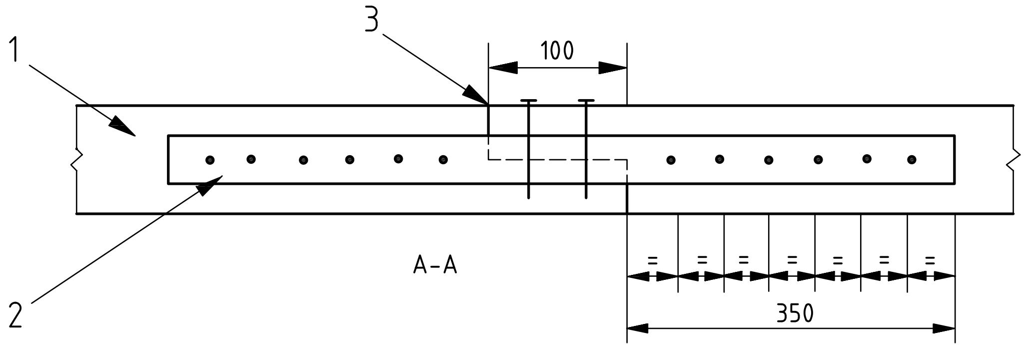
Diaphragm depth m	**Diaphragm span A) B) m**								
	9	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
6	2,37	2,13	1,94	1,77	1,65	1,53	1,41	1,32	1,25
7	2,76	2,49	2,25	2,07	1,91	1,79	1,67	1,55	1,46
8	3,17	2,84	2,57	2,37	2,19	2,01	1,88	1,77	1,68
9	3,54	3,20	2,91	2,66	2,42	2,28	2,13	1,98	1,86
10	3,93	3,54	3,23	2,96	2,72	2,52	2,37	2,22	2,09
11	4,34	3,90	3,54	3,24	3,00	2,79	2,58	2,43	2,30
12	4,73	4,25	3,87	3,54	3,26	3,05	2,84	2,66	2,49
13	5,13	4,61	4,19	3,83	3,54	3,30	3,08	2,87	2,70
14	5,52	4,95	4,52	4,14	3,81	3,54	3,32	3,09	2,90
15	5,91	5,33	4,83	4,44	4,08	3,80	3,54	3,32	3,14
16	6,30	5,69	5,16	4,73	4,37	4,05	3,78	3,54	3,33
17	6,87	6,03	5,49	5,03	4,62	4,31	4,11	3,77	3,54

A) Intermediate values may be obtained by linear interpolation.
B) For plasterboard ceiling diaphragms, wallplates should be spliced in accordance with Figure E.8 or using an alternative splice detail capable of resisting a design instantaneous axial action of 4,9 kN.

Figure E.8 **Wall plate splice joint**



a) Heel detail at bearing



b) Elevation of wall plate splice

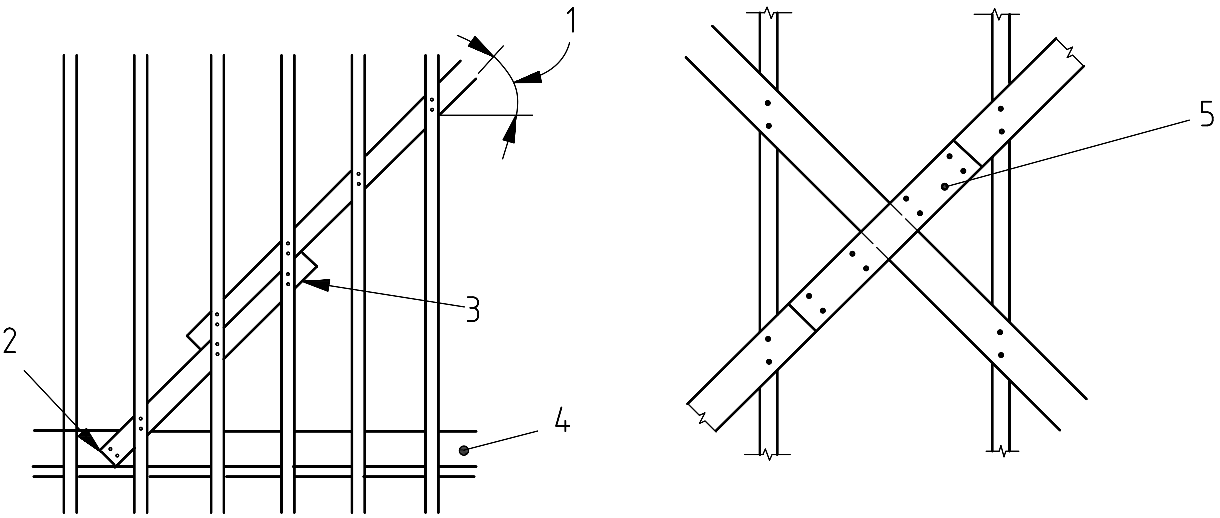
Key

1 Wall plate

2 Steel strap fixed using 6 no. 3,75 mm × 30 mm square twist nails into each member

3 100 mm long half lap joint with no. 2 nail fixings

Figure E.9 **Standard bracing for rafter members: detail C1 and D1**



a) Detail C1: Eaves and lap connection – view on slope on topside of rafters (see arrow C section A-A on Figure E.3 or Figure E.4)

b) Detail D1: Crossing connection for narrow fronted buildings (see arrow D on plan view of narrow fronted roof rafter diagonal bracing in Figure E.3 or E.4)

Key

1 Ideally 45° but not less than 35° or greater than 55°

2 Nail to wall plate

3 Rafter diagonal brace lap jointed, where required

4 Wall plate

5 600 mm long timber splice plate the same size as bracing timbers

Timber splice plate should be fixed using minimum of 4 no. 3,35 diameter galvanized nails each side × minimum nail length equal to bracing thickness + 32 mm. Protruding nails should be clenched over as required.

a) All bracing members are of minimum width 89 mm and minimum depth 22 mm and free from major strength reducing defects. Additionally the cross-sectional area should not be less than 2 134 mm^2. Provided these conditions are met, no further strength grading is necessary for bracing members.

b) All bracing members are nailed to every trussed rafter they cross with two 3,35 mm diameter wire nails with a minimum length equal to the bracing thickness plus 32 mm.

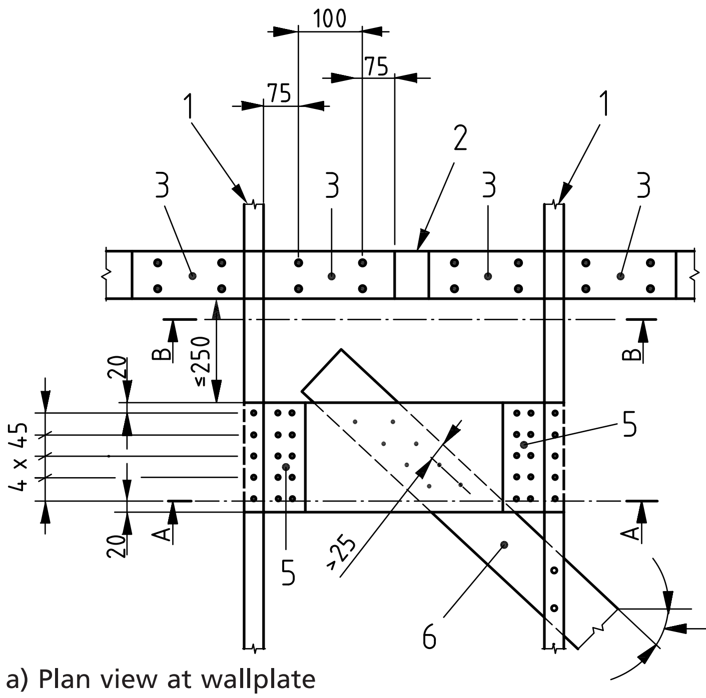
c) In all details 3,1 mm × 90 mm long mechanically driven gun nails may be substituted for 3,35 mm × 65 mm long wire nails.

d) Where bracing members are provided in two pieces, they are lap jointed over at least two trussed rafters and are nailed as described in b) or c).

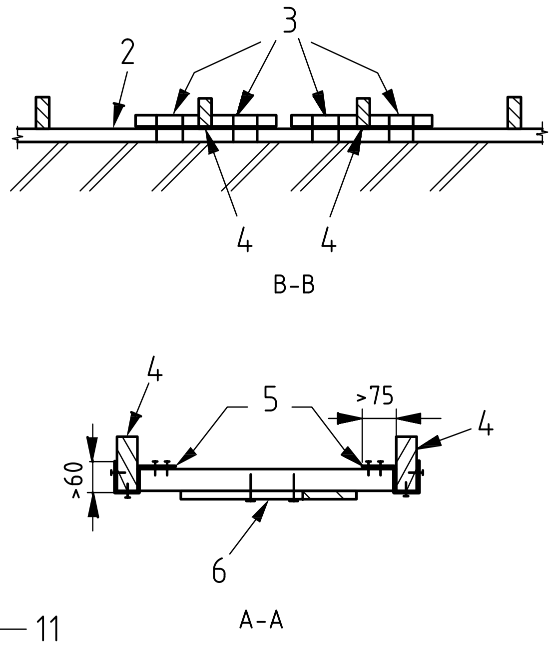
e) Where nails exceed the thickness of bracing members they should either be clenched, or a packing piece added behind the joint.

f) For the purpose of designing a suitable wallplate anchorage detail a design instantaneous axial action of 4,9 kN should be assumed to exist in the rafter diagonal braces.

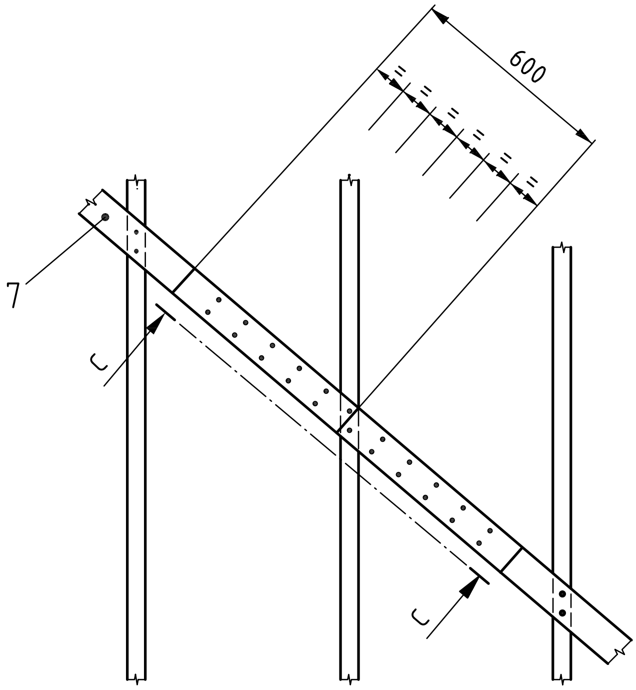
Figure E.10 **Standard bracing for rafter members: detail C2 and D2**



a) Plan view at wallplate

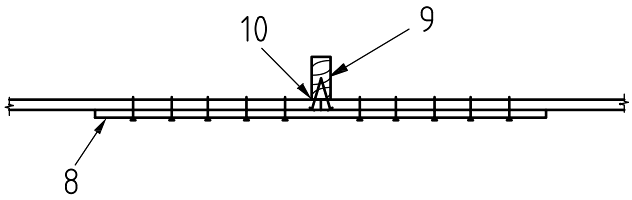


b) Detail C2:Eaves connection – view on slope on topside of rafters [see C, Figure E.3 or Figure E.4]



c) Detail D2: Splice connection – view on slope on underside of rafters [A)]

Figure E.10 **Standard bracing for rafter members: detail C2 and D2**



d) Rafter diagonal bracing splice

[A] Cross-over of diagonal rafter braces in the slope of the roof is not permitted. The bracing system depicted in Figure E.10 is only applicable to wide-fronted buildings.

Key

1 Trussed rafters at ≤ 600 mm centres
2 Wall plate
3 47 mm × 100 mm × 250 mm long timber blocks to be fixed hard against each side of truss and nailed to wallplate using 4 no. 4 diameter × 90 mm long nails
4 Trussed rafters at ≤ 600 mm centres
5 1 mm thick steel bracket fixed to both rafter and timber shelf using minimum of 10 no. 3,35 mm diameter × 30 mm long square twist nails
6 22 mm × 97 mm diagonal rafter brace fixed to 47 mm × 220 mm timber shelf (minimum grade C16) using 8 no. 3,35 mm diameter × 65 mm long nails
7 22 mm × 97 mm rafter brace
8 Timber splice plate (22 mm × 97 mm × 1 200 mm) fixed to each diagonal rafter brace using 10 no. 3,35 mm diameter × 65 mm long nails
9 Rafter
10 Rafter braces butt-jointed over a truss
11 Ideally 45° but not less than 35° or greater than 55°

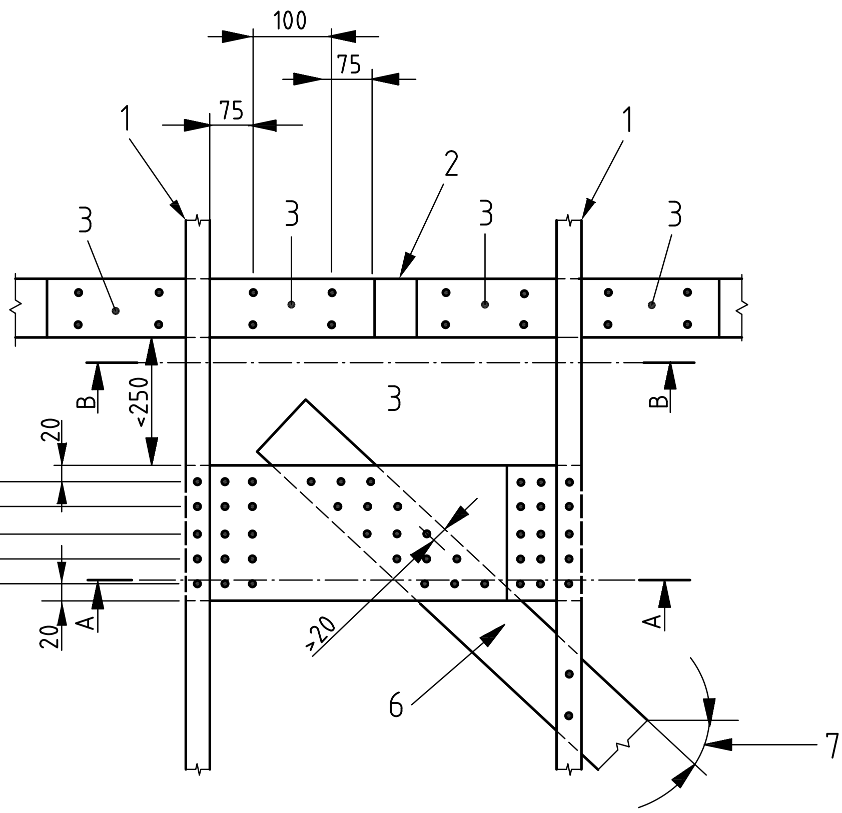
a) All bracing members are nailed to every trussed rafter they cross with two 3,35 mm diameter wire nails with a minimum length equal to the bracing thickness plus 32 mm.

b) In all details 3,1 mm × 90 mm long mechanically driven gun nails may be substituted for 3,35 mm × 65 mm long wire nails.

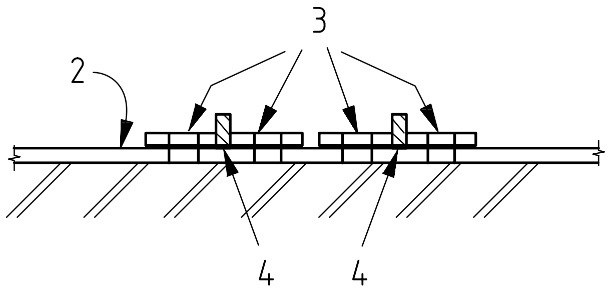
c) Where nails exceed the thickness of bracing members, they should either be clenched or a packing piece added behind the joint.

d) Other connection details may be used as alternative to those shown provided they are designed to provide equivalent stiffness and can resist a design instantaneous axial action in the rafter brace of 8.3 kN in the case of details shown in Figure E.10.

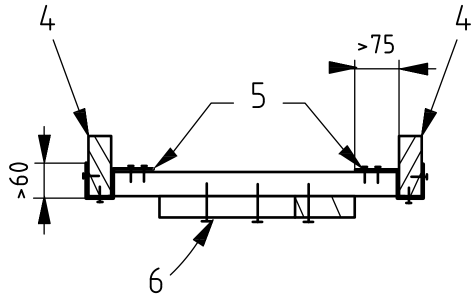
Figure E.11 **Standard bracing for rafter members: detail C3**



a) Plan view at wall plate



B-B



A–A

b) Detail C3: Eaves connection: view on slope on topside of rafters [see C, Figure E.3c) or Figure E.4a)]

Key

1	Trussed rafters at ≤ 600 mm centres
2	Wall plate

Figure E.11 **Standard bracing for rafter members: detail C3**

3	47 mm × 100 mm × 250 mm long timber blocks to be fixed hard against each side of truss and nailed to wallplate using 4 no. 4 mm diameter × 90 mm long nails
4	1 mm thick steel bracket fixed to both rafter and timber shelf using minimum of 10 no. 3,75 mm diameter × 30 mm long square twist nails
5	35 mm × 120 mm diagonal rafter brace (minimum grade C16) fixed to 47 mm × 220 mm timber shelf (minimum grade C16) using 15 no. 3,35 mm diameter × 75 mm long nails
6	Ideally 45° but not less than 35° or greater than 55°

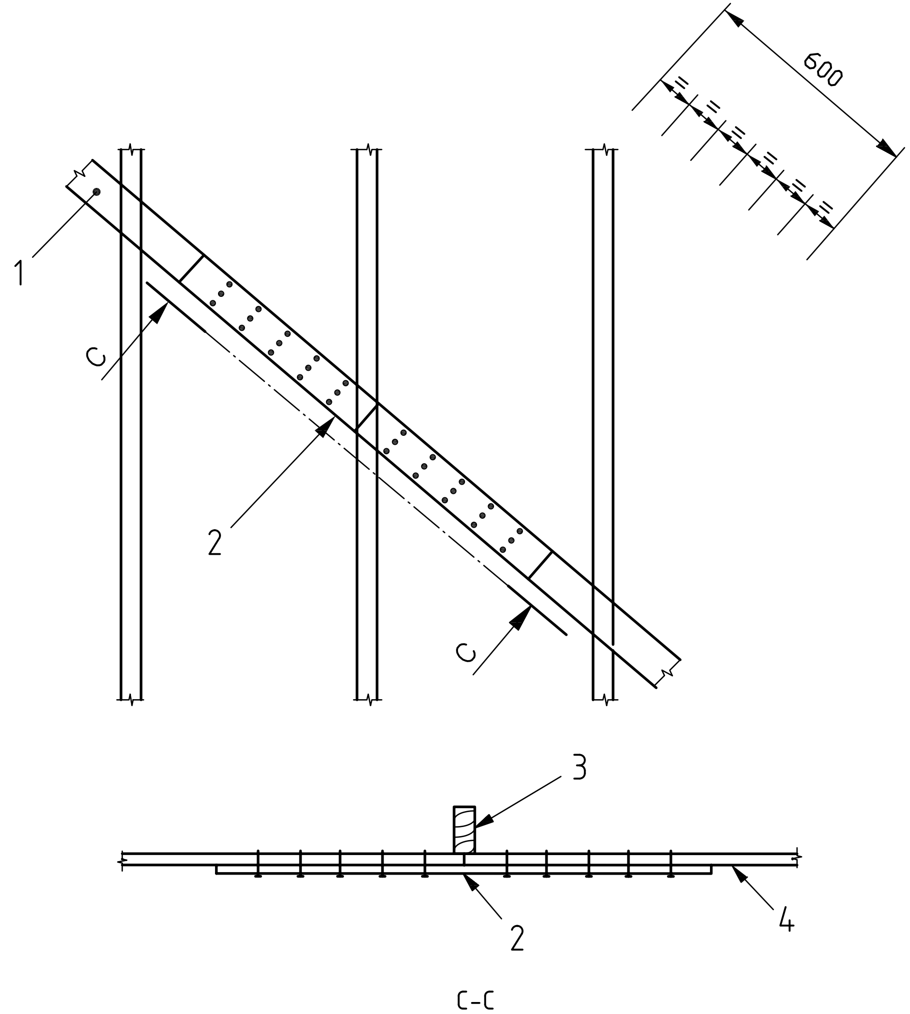
a) All bracing members are nailed to every trussed rafter they cross with two 3,35 mm diameter wire nails with a minimum length equal to the bracing thickness plus 32 mm.

b) In all details 3,1 mm × 90 mm long mechanically driven gun nails may be substituted for 3,35 mm × 75 mm long wire nails.

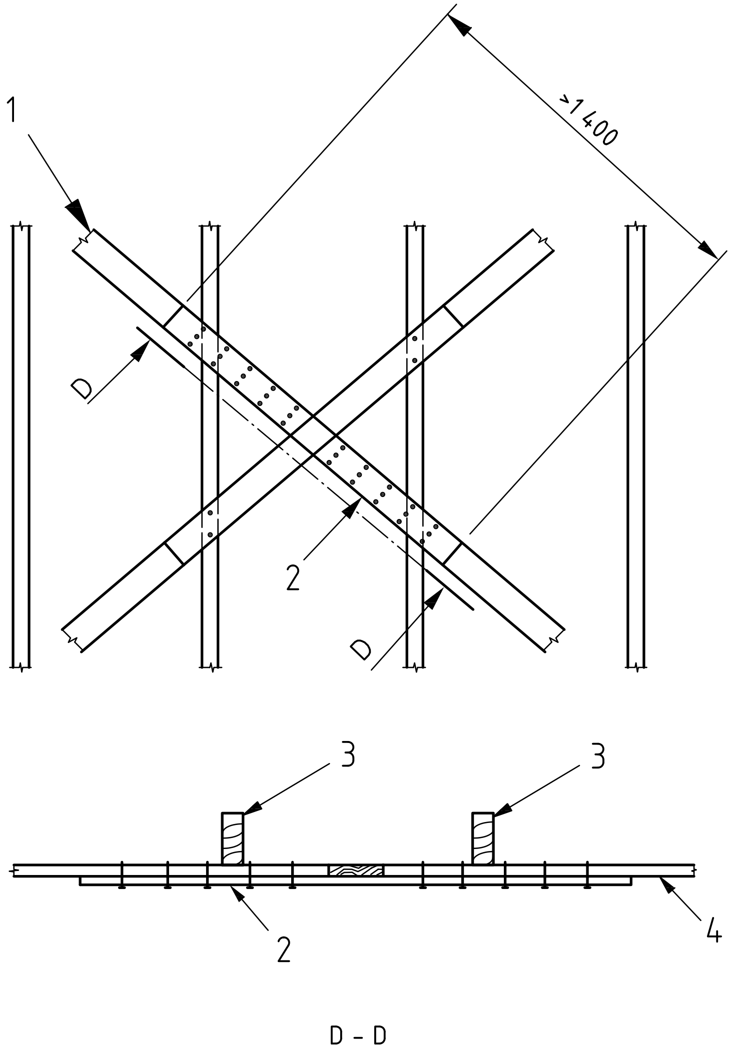
c) Where nails exceed the thickness of bracing members, they should either be clenched or a packing piece added behind the joint.

d) Other connection details may be used as alternative to those shown provided they are designed to provide equivalent stiffness and can resist a design instantaneous axial action in the rafter brace of 14,5 kN in the case of details shown in Figure E.11.

Figure E.12 **Standard bracing for rafter members: detail D3 splice connection and D4 crossing connection**



a) Detail D3: Splice connection: view on slope on underside of rafters



b) Detail D4: Crossing connection for narrow fronted buildings [see D, Figure E.3b) or Figure E.4f)]

Key

1	35 mm × 120 mm rafter brace
3	Rafter

Figure E.12 **Standard bracing for rafter members: detail D3 splice connection and D4 crossing connection**

2	Timber splice plate (35 mm × 120 mm) fixed to each diagonal rafter brace using 15 no 3,35 mm diameter × 75 mm long nails	4	Rafter braces butt-jointed over a truss

a) All bracing members are nailed to every trussed rafter they cross with two 3,35 mm diameter wire nails with a minimum length equal to the bracing thickness plus 32 mm.

b) In all details 3,1 mm × 90 mm long mechanically driven gun nails may be substituted for 3,35 mm × 75 mm long wire nails.

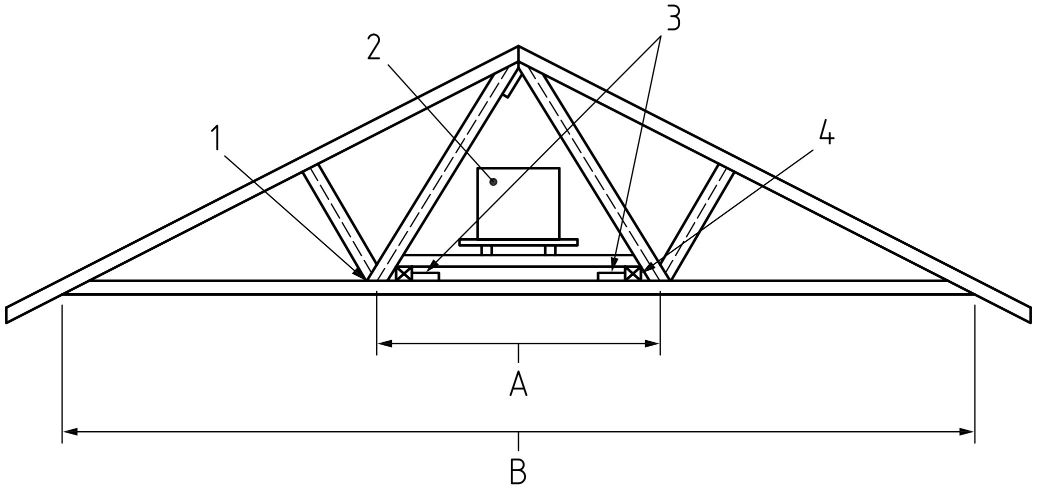
c) Where nails exceed the thickness of bracing members they should either be clenched or a packing piece added behind the joint.

d) Other connection details may be used as alternatives to those shown provided they are designed to provide equivalent stiffness and can resist a design instantaneous axial action in the rafter brace of 14,5 kN in the case of details shown in Figure E.12.

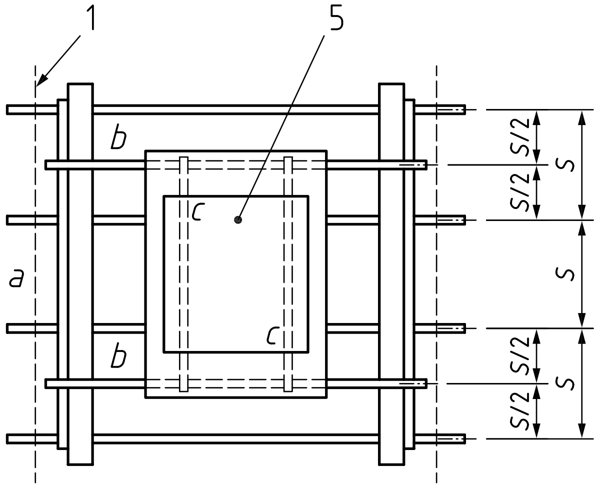
Annex F (informative) Optional recommendations for the support of water tanks in trussed rafter roofs

Water tanks may be supported as shown in Figure F.1, using the timber sizes given in Table F.1. The platform at the base of the cold water storage tank should fully support the tank over at least its entire plan area. The platform material should be a minimum of 18 mm thick timber boarding or moisture-resistant wood based material.

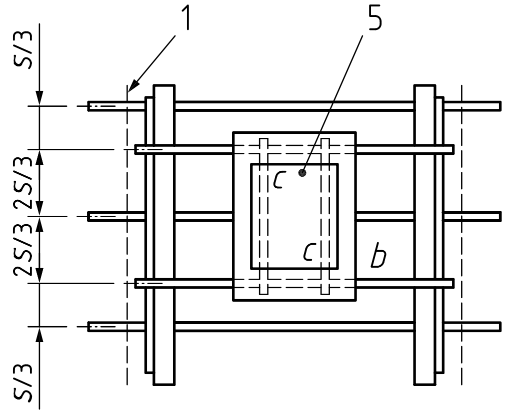
Figure F.1 **Supports for water tanks**



a) Section through roof



b) Plan view of tank: detail A



c) Plan view of tank: detail B

Key

1 Node point
2 Tank placed centrally
3 Offset bracing to clear tank bearers
4 Bearer “A” placed as close to the node point as possible
5 Tank

A Bay size
B Trussed rafter span, L_s
S Trussed rafter spacing

NOTE See Table F.1 for minimum sizes and strength class for support members.

Table F.1 **Sizes for support members for water tanks** [A)]

Tank capacity to marked waterline	Minimum member sizes			Maximum trussed rafter span for Fink configuration m	Maximum bay size for other configuration m
	a mm	*b* mm	*c* mm		
Detail A Not more than 450 L supported on four trussed rafters	47 × 72	2 × 35 × 145 or 1 × 47 × 169	47 × 120	6,5	2,2
	47 × 72	2 × 35 × 169	47 × 120	9,0	2,8
	47 × 72	2 × 47 × 169	47 × 120	12,0	3,8
Detail A Not more than 300 L supported on four trussed rafters	47 × 72	2 × 35 × 97 or 1 × 47 × 120	47 × 72	6,5	2,2
	47 × 72	2 × 35 × 120 or 1 × 47 × 145	47 × 72	9,0	2,8
	47 × 72	2 × 35 × 145	47 × 72	12,0	3,8
Detail B Not more than 230 L supported on three trussed rafters	47 × 72	1 × 47 × 97	47 × 72	6,5	2,2
	47 × 72	2 × 35 × 97 or 1× 47 × 120	47 × 72	9,0	2,8
	47 × 72	2 × 35 × 120 or 1 × 47 × 145	47 × 72	12,0	3,8

A) Support members of minimum strength class C16, in accordance with BS EN 338:2009.

Bibliography

Standards publications

For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 4978, *Visual strength grading of softwood – Specification*

BS 5756, *Visual strength grading of hardwood – Specification*

BS 6446, *Specification for manufacture of glued structural components of timber and wood based panels*

BS EN 386, *Glued laminated timber – Performance requirements and minimum production requirements*

BS EN 622-2, *Fibreboards – Specifications – Part 2: Requirements for hardboards*

BS EN 771-1:2011, *Specification for masonry units – Part 1: Clay masonry units*

BS EN 772-7:1998, *Methods of test for masonry units – Part 7: Determination of water absorption of clay masonry damp proof course units by boiling in water*

BS EN 998-2:2010, *Specification for mortar for masonry – Part 2: Masonry mortar*

BS EN 1075, *Timber structures – Test methods – Joints made with punched metal plate fasteners*

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