Use of Structural Eurocodes – EN 1995 (Design of Timber Structures)
Companion Document to EN1995-1-1

BD 2405
Whilst this document provides practical guidance on the use of Eurocode BS EN 1995-1-1 for the design of buildings, it shall only be applied in conjunction with both the Eurocode and its National Annex published by the British Standards Institution.

It should be noted that this guidance has been based on the published Eurocode, BS EN 1995-1-1:2004, together with the draft of its National Annex, as available at the time of writing (February 2004). Since then there have been a few amendments to both EN 1995-1-1 and the National Annex which users are strongly advised to take into consideration when using this document. The changes have been in the following sections of EN 1995-1-1 and National Annex:

**EN 1995-1-1:**
- Section 1.2: Normative references
- Section 1.6: Symbols used in EN 1995-1-1
- Section 6.1.5: Compression perpendicular to the grain
- Section 6.1.7: Shear
- Section 8.3.1: Laterally loaded nails
- Section 8.3.1.1: General
- Section 8.3.2: Axially loaded nails
- Section 8.7.2: Axially loaded screws

**National Annex:**
- The whole National Annex

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EXECUTIVE SUMMARY

This work has been carried out for Communities and Local Government Buildings Regulation Division under research contract BD2405 ‘Use of Structural Eurocodes – EN 1995 (Design of Timber Structures)’, a Framework project under the Structural Integrity Business Plan. This report forms the output D5 (213772) of the contract.

This report serves mainly as a companion document to EN 1995-1-1. In addition this companion document includes the differences between the current BS 5268: Part 2 and EN 1995-1-1. This is the penultimate document of the project, having already been peer reviewed.

The following documents have been created within the programme, covering EN 1995-1-1 and related standards:

- Comparison document i.e. Chapter 1 of this document. The comparison document given in Chapter 1 can be a stand-alone document and may be published as a handbook for people who are interested in the differences

- Companion document i.e. Chapter 2 gives a commentary on EN 1995-1-1 and links to EN 1990 and EN 1991. This document concentrates on explaining each section/part of EN 1995-1-1 in a much more user-friendly language and for better understanding of the Code

- Design guide (practical step-by-step guide, helping designers to use and get familiar with EN 1995-1-1 in their designs):
  - A flow chart is produced to lead designers through the practical design process
  - Offers a software tool to interface with EN 1990, the loading code
  - An electronic route map is also produced to enable designers to identify the location of key passages and sources of information given in EN 1995-1-1
  - The design guide is set out in an order related to the design process, rather than matching the layout of EN 1995-1-1.

It is also recommended that similar documents to those above will be needed for the fire and bridges parts of EN 1995. However, it is believed that EN 1995-1-2 covering structural fire design for timber structures, will be vital (**please note that this is outside the scope of the current BRE contract with Communities and Local Government**). The part on timber bridges, EN 1995-2, will be used in the UK primarily by the highways and railways authorities, who should be capable of generating their own guidance as required.
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INTRODUCTION

0.1 General

In the UK, the design of civil and structural engineering works has generally been based on a series of Codes of Practice. These Codes of Practice were drafted by committees within the British Standards Institution (BSI), and published by BSI. The BSI structural Codes of Practice are widely respected, and used in many other countries. However, most major countries have their own codes or equivalent documents. As a result, various different design procedures have been required for structures according to which country they are built in.

As part of the European Union’s (EU) initiative to facilitate trade within the construction sector, the Construction Products Directive (CPD), the EU has commissioned a common set of Codes to be used for construction throughout the EU. Drafting of these Codes was initially overseen directly by the EU, but later it was passed to the European Standards Organisation (CEN). Membership of CEN includes the national standards organisations of countries belonging to the European Union, EFTA, and some applicant countries to the European Union. Hence, a complete suite of Codes for civil and structural engineering design is being developed, termed the Eurocodes. After a period of coexistence, these Codes will replace the existing national Codes.

The Eurocodes aim to:

- provide a common design basis for the design of structures within EU Member States;
- facilitate the exchange of construction services between Member States;
- facilitate the marketing and use of structural components;
- provide a means to demonstrate compliance of building and civil engineering works with the requirements of the CPD;
- improve the competitiveness of the European construction industry in countries outside the European Union.

The Eurocodes cover ten main subjects:

- EN 1990 Basis of structural design
- EN 1991 Actions on structures
- EN 1992 Design of concrete structures
• EN 1993 Design of steel structures
• EN 1994 Design of composite steel and concrete structures
• EN 1995 Design of timber structures
• EN 1996 Design of masonry structures
• EN 1997 Geotechnical design
• EN 1998 Design of structures for earthquake resistance
• EN 1999 Design of aluminium structures.

Many of these Codes are subdivided into a series of parts. EN 1995 (Eurocode 5), Design of timber structures, has three parts:

• EN 1995-1-1, General – Common rules for buildings
• EN 1995-1-2 General rules – Structural fire design
• EN 1995-2 Bridges.

The introduction of the key Eurocode for the design of timber structures, EN 1995-1-1 Design of timber structures – General – Common rules for buildings, will have a major impact on the design of future timber structures in the UK. The Eurocodes are based on limit state design, whilst timber is the only principal construction material for which the UK Codes are not limit state design codes but permissible stress design. Owing to this, the Building Regulations Division of Communities and Local Government commissioned a consortium led by BRE and including Buro Happold, the Institution of Structural Engineers, the University of Surrey and the National House Building Council to produce supporting documentation to assist designers in using this Eurocode.

Chapter 1 reviews the major differences between EN 1995-1-1 and the equivalent UK Codes, to provide a reference document in the companion document (Chapter 2) and design guide (Chapter 3).

0.2 Scope

This document serves mainly as a companion document to EN 1995-1-1 giving a commentary on EN 1995-1-1 and its links to EN 1990 and EN 1991. This document concentrates on explaining each section/part of EN 1995-1-1 in a much more user-friendly language and for better understanding of the Code.

Chapter 1 comprises the main differences between BS 5268: Part 2 and EN 1995-1-1 for information only, while Chapter 2 concentrates on explaining each section/part of EN 1995-1-1. It includes simple explanations of various parts of EN 1995-1-1 parallel with its contents. It is recommended that the users of this document try to put aside momentarily their knowledge and experience of
using BS 5268 when reading Chapter 2 of this document because BS 5268: Part 2 has been based on permissible stress design, while EN 1995-1-1 is an ultimate limit state design which is explained in this document.

Chapter 3 includes design guides via a practical step-by-step guide, helping designers to use EN 1995-1-1 in their designs. A flow chart is given to lead designers through the practical design process. In addition, an electronic route map is also included to enable designers to identify the location of key passages and sources of information. This part of the document is set out in an order related to the design process rather than matching the layout of EN 1995-1-1.

0.3 The Eurocodes

0.3.1 BACKGROUND

The Commission of the European Community decided on an action programme in the field of construction based on Article 95 of the Treaty of Rome. Within this action programme the Commission took the initiative to establish a set of harmonised technical rules for the structural design of construction works, with the following European Commission objective:

‘The Eurocodes to establish a set of common technical rules for the design of buildings and civil engineering works which will ultimately replace the differing rules in the various Member States’.

The Commission established, in the mid 1970’s, a Steering Committee consisting of representatives of Member States whose work on the Eurocodes programme led to the publication of a set of first generation Eurocodes after fifteen years.

In 1989, a Special Agreement was made between CEN and the European Commission transferring the responsibility of producing the structural Eurocodes to CEN. The agreement also specified that the Eurocodes are to serve as reference documents to be recognised by authorities of the Member States for the following purposes:


b) as a basis for specifying contracts for the execution of construction works and related engineering services in the area of public works. This relates to Council Procurement Directives for:

- Works, which covers procurement by public authorities of civil engineering and building works, with a current (2004) threshold of about 5m Euros for an individual project, and
• Services, which cover procurement of services by public authorities, with current (2004) thresholds for Government Departments of 130k Euros and others, including local authorities, of 200k Euros.

c) as a framework for drawing up harmonised technical specifications for construction products.

0.3.2 RELATIONSHIP BETWEEN THE EUROCODES AND NATIONAL REGULATIONS/PUBLIC AUTHORITY REQUIREMENTS

There is a clear and vital distinction between design codes and National Regulations/Public Authority Requirements. Harmonisation of National requirements is outside the scope of Eurocode development. It is the objective however that the Eurocodes, together with their appropriate National Annexes, should be recognised in National Regulations as one of the routes for meeting compliance. The legal status of the Eurocodes under the Building Regulations will be exactly the same as that of the current National Codes of Practice. In accordance with normal rules following the introduction of European Standards, Eurocodes will be called up in public procurement specifications and to be used for the design of products for the purpose of obtaining a CE (Conformité Européen) mark.

0.3.3 EUROCODE PROGRAMME AND THE RELATIONSHIP BETWEEN VARIOUS EUROCODES

The structural Eurocodes are shown in Table 0.1. Each, generally consists of a number of parts, which cover the technical aspects of the structural and fire design of buildings and civil engineering structures, with specific parts relating to bridges. A list of the various parts and the publication date of each EN is continuously being updated on the Thomas Telford website www.eurocodes.co.uk.

<table>
<thead>
<tr>
<th>Table 0.1 The structural Eurocodes</th>
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<tbody>
<tr>
<td><strong>EN Number</strong></td>
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<tr>
<td>EN 1990</td>
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<td>EN 1999</td>
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The Eurocodes are a harmonised set of documents that have to be used together. Their linked relationship is shown in Figure 0.1.
In accordance with Figure 0.1, EN 1995-1-1 has to be used with EN 1990 the head key Eurocode and the appropriate parts of EN 1991: Actions on structures and EN 1997: Geotechnical design.

**Figure 0.1 Linked relationship between the Eurocodes**

0.3.4 **DIFFERENCES IN PHILOSOPHY BETWEEN EXISTING BRITISH STANDARDS AND EUROCODES**

The principal differences in philosophy between existing British Standards and EN 1995-1-1 are explained in Chapter 1.

0.3.5 **SUPPORTING AND RELATED DOCUMENTS (PRODUCT STANDARDS, ETC): REQUIRED AND AVAILABLE**

The following standards are required for the use of EN 1995-1-1

**General reference Eurocodes:**
EN 1990: Eurocode: Basis of structural design

EN 1991 (All parts): Eurocode: Actions on structures


EN 1997: Eurocode 7 Geotechnical design

EN 1998: Eurocode 8 Design of structures for earthquake resistance
Other reference standards:

Harmonised standards (hENs): hENs are Harmonised Product Standards (i.e. head standards or an umbrella covering different materials. They give the rules by which products can meet the requirements for CE marking to be placed on the market. hENs reference (call-up) all other relevant CEN standards covering a particular product:

- Fire requirements
- Test methods
- Classification standards
- Production standards
- Material specifications
- Grading standards
- Etc.

hENs are written (or being written) for:

- Solid timber (EN 14081 – Parts 1-4). This is approved and publication is expected in 2006
- All panel products (EN 13986). This was published in June 2002
- Glulam (EN 14080). This is published in June 2005
- Laminated Veneer Lumber, LVL, (EN 14374). This was published in November 2004 for structural LVL
- Fasteners/connectors (EN 14592/EN 14545). These are about to go for Formal Vote by European Countries
- Trusses/trussed rafters (EN 14250). This was published in November 2004
- Timber frame walls/floors/roofs (EN 14732). This is expected in late 2006.

You will also probably need (depending on your sector) the following CEN Standards:

- EN338 Timber strength classes
- EN 1912 Visual grades (assignment to strength classes)
- EN 336 Timber size tolerances
- EN 1194 Glulam strength classes
- EN 12369: Part 1 Strength properties of OSB/chipboard/fibreboards
- EN 12369: Part 2 Strength properties of plywood.
0.3.6 ROLE OF NATIONAL ANNEX – USING EN EUROCODES AT A NATIONAL LEVEL

It is the responsibility of each national standards body (e.g. the British Standards Institute (BSI) in the UK) to implement Eurocodes as national standards.

The national standard implementing each Eurocode part will comprise, without any alterations, the full text of the Eurocode and its annexes as published by the CEN (Figure 0.2, c, d and e). This may be preceded by a National Title Page (a) and National Foreword (b), and may be followed by a National Annex (f).

0.3.6.1 Rules and contents of National Annexes for Eurocodes

The European Commission, recognising the responsibility of regulatory and national competent authorities in each EU Member State, has safeguarded their right to determine values related to safety matters at national level through a national annex. These safety matters include different levels of protection that may prevail at national, regional or local level, and ways of life.

A National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned. Where a Eurocode Clause allows choice, a recommended value or method is given.
0.3.6.2 Nationally Determined Parameters (NDPs)

NDPs will allow Member States to choose the level of safety applicable to their territory. The values, classes or methods to be chosen or determined at national level, are:

- values and/or classes where alternatives are given in the Eurocode (e.g. levels of safety)
- values to be used where only a symbol is given in the Eurocode (e.g. partial factors)
- country-specific data (geographical, climatic, etc) (e.g. snow maps)
- procedures to be used where alternative procedures are given in the Eurocodes.

0.3.6.3 National Annexes

The National Standards Bodies (i.e. BSI in the UK) should publish the NDPs in a National Annex. A National Annex is not required if a Eurocode part is not relevant for the Member State (e.g. seismic design for some countries).

In addition to NDPs, a National Annex may also contain

- decisions on the application of informative annexes
- references to non-contradictory complementary information (NCCI) to assist the user in applying the Eurocode. The NCCI is sometimes referred to as ‘Rump Standards’ or ‘Residual Standards’

It should be noted that in EN 1995-1-1, NDPs are used for situations other than just to safeguard Member States’ rights to define safety. They have been used to cover situations where there is no possibility of a consensus view being reached on an issue (e.g. for most of the serviceability section and the sections on detailing rules in EN 1995-1-1).

0.4 Basis of structural design (The use of EN 1990 for timber structure design)

It is recommended that EN 1990 is studied separately in detail. However, this section introduces the principles and describes the objectives of EN 1990, lists the requirements and provides information on the representative values of the loads to be used in the combination of actions for use with the design and detailing clauses of EN 1995-1-1. It also gives the values adopted by the BSI National Annex to EN 1990.

The principal differences between EN 1990 and UK practice are all listed and explained in a publication by Gulvanessian, Calgaro and Holicky who provide a comprehensive description, background and commentary to EN 1990. Chapter 2 of the BRE Handbook on Actions on Structures also provides guidance on EN 1990, which describes the background to the selections made in the BSI National Annex to EN 1990.
0.4.1 DESIGN LINK TO EN 1990 CREATED BY THIS PROJECT

EN 1990 is the head key Eurocode for the harmonised Structural Eurocodes. It establishes and provides comprehensive information and guidance for all the Eurocodes on the principles and requirements for safety and serviceability, describes the basis of their design and verification and gives guidelines for related aspects of structural reliability and durability of structures. It is based on the limit state design concept and used in conjunction with the partial factor method. EN 1995 does not give the material-independent clauses required for design. These are only included in EN 1990. Hence, it is very important that EN 1990 is used with all the Eurocode parts. EN 1990 is required for the verification of both ultimate and serviceability limit states as it provides the information for safety factors for actions and a combination of superimposed actions.

The limit state design concept and the partial safety factor method used in EN 1995 constitutes a major change from the traditional BS approach. In the Eurocodes system the safety and reliability concept is to be chosen and developed by the designer. Under BS 5268-2 combining loads is more simplistic. The direct value of potential loads is used without any partial factor to allow for uncertainty in the possible value of the loads. Furthermore, loads are combined by the direct arithmetical summation of relevant loads over any particular duration.

This project has developed a design tool for EN 1990 to assist the designer in determining their loading cases and incorporating the various safety factors. The tool interface is shown below:
The EN 1990 step-by-step process

Output help in ‘Limit state’ function
Interactive tool to help combine variable loads

A summary page overviews all choices and implications
The support software tool is intended to help the designer choose the appropriate safety and combination factors and also decide on the design working life of the structure. The software tool is interactive, asking the designer to answer a set of questions which then are translated into the appropriate safety factor cluster required for a particular load case and loading scenario. It then prints a summary of all settings for a better overview. The various steps are laid out clearly, also giving guidance and explanation and summarising the output from the choices.

0.4.2 REQUIREMENTS OF EN 1990

The requirements of EN 1990 which need to be adhered to by EN 1995 are:

(i) Fundamental requirements: These relate to safety, serviceability and robustness requirements

(ii) Reliability differentiation

(iii) Design situations: EN 1990 stipulates that a relevant design situation is selected taking account of the circumstances in which the structure may be required to fulfil its function. EN 1990 classifies design situations for ultimate limit state verification as follows:

• persistent situations (conditions of normal use);
• transient situations (temporary conditions e.g. during execution);
• accidental situations; and
• seismic situations.

(iv) Design working life: For buildings and other common structures the recommended design working (i.e. the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without major repair being necessary) is 50 years. For concrete, design working life needs to be considered for material property deterioration, life-cycle costing and evolving maintenance strategies.

(v) Durability

(vi) Quality assurance.

0.4.3 PRINCIPLES OF LIMIT STATE DESIGN

Ultimate and serviceability limit states

(a) Ultimate limit states are those associated with collapse or with other forms of structural failure and concern:

• the safety of people; and
• the safety of the structure and its contents.
(b) Serviceability limit states correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met and concern:

- the functioning of the construction works or parts of them;
- the comfort of people; and
- the appearance.

There are differences between the concept of design situations approach in EN 1990 and the approach of the BSI codes. In the verification of serviceability limit states in EN 1990, separate load combination expressions are used depending on the design situation being considered. For each of the particular design situations an appropriate representative value for an action is used.

0.4.3.1 The representative values of the actions to be used for the different design situations

The representative values of the actions

In addition to the characteristic values of actions which are similar to the BSI definition, other representative values are specified in EN 1990 for variable and accidental actions. Three representative values commonly used for variable actions are the combination value $\psi_0 Q_k$, the frequent value $\psi_1 Q_k$ and the quasi-permanent value $\psi_2 Q_k$. The factors are reduction factors of the characteristic values of variable actions, and each is defined below.

The attached programme has been designed and produced in this project to take designers through EN 1990 step by step in order to obtain the appropriate $\psi_0$, $\psi_1$ and $\psi_2$ values for buildings.

The combination value $\psi_0 Q_k$, the frequent value $\psi_1 Q_k$, and the quasi-permanent value $\psi_2 Q_k$ are explained below:

The combination value $\psi_0 Q_k$ is associated with the combination of actions for ultimate and irreversible serviceability limit states (the serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed) in order to take account of the reduced probability of the simultaneous occurrence of the most unfavourable values of several independent actions.

The frequent value $\psi_1 Q_k$ is primarily associated with the frequent combination in the serviceability limit states and it is also assumed to be appropriate for verification of the accidental design situation of the ultimate limit states. In both cases, the reduction factor $\psi_1$ is applied as a multiplier of the leading variable action.

The quasi-permanent value $\psi_2 Q_k$ is mainly used for the assessment of long-term effects, for example in checking cracking or deflection. But it is also used for the representation of variable actions in accidental and seismic combinations.
of actions (ultimate limit states) and for the verification of frequent and quasi-
permanent combinations (long-term effects) of serviceability limit states.

0.4.4 VERIFICATION BY THE PARTIAL FACTOR METHOD

0.4.4.1 Ultimate limit states verification

For the ultimate limit state verification, EN 1990 stipulates that the effects of
design actions do not exceed the design resistance of the structure at the
ultimate limit state; and the following ultimate limit states need to be verified.

a) For the limit state verification for static equilibrium (EQU)

\[ E_{d, \text{dst}} \leq E_{d, \text{stb}} \]  

(6.7 of EN 1990)

where:

- \( E_{d, \text{dst}} \) is the design value of the effect of destabilising actions;
- \( E_{d, \text{stb}} \) is the design value of the effect of stabilising actions.

b) For internal failure or excessive deformation of the structure or structural
members, including footings, piles, basement walls, etc., where the strength
of construction materials of the structure governs (STR); and for failure or
excessive deformation of the ground where the strengths of soil or rock are
significant in providing resistance (GEO);

\[ E_{d} \leq R_{d} \]  

(6.8 of EN 1990)

where:

- \( E_{d} \) is the design value of the effect of actions such as internal force, moment
  or a vector representing several internal forces or moments.
- \( R_{d} \) is the design value of the corresponding resistance.

Combination of actions for ultimate limit states

The fundamental (persistent and transient) design situations for ultimate limit
state verifications, other than those relating to fatigue, are symbolically
represented as follows:

\[ \sum_{j=1}^{G_{k,i}} \gamma_{j} G_{k,i} + \gamma_{f} P + \gamma_{Q_{k,1}} Q_{k,1} + \sum_{i>1}^{\gamma_{Q_{i,1}}} \psi_{i,1} Q_{k,i} \geq 0 \]  

(6.10 of EN 1990)

This combination assumes that a number of variable actions are acting
simultaneously, \( Q_{k,1} \) is the dominant variable action and this is combined with
the combination value of the accompanying variable actions \( Q_{k,i} \).

\( P \) is a relevant representative value for prestressing actions which in normal
timber structures is zero.
Alternatively, EN 1990 allows the use of the following equations together.

\[
\sum_{j=1}^{n} \gamma_{G,j} G_{k,j} + \gamma_{P} P + \gamma_{Q,j} Q_{k,j} + \sum_{i=1}^{m} \gamma_{Q,i} Q_{k,i} \quad (6.10a \text{ of EN 1990})
\]

\[
\sum_{j=1}^{n} \xi_{j} \gamma_{G,j} G_{k,j} + \gamma_{P} P + \gamma_{Q,j} Q_{k,j} + \sum_{i=1}^{m} \gamma_{Q,i} Q_{k,i} \quad (6.10b \text{ of EN 1990})
\]

where \( \xi \) is a reduction factor for \( \gamma_{G,j} \) within the range 0.85 to 1.

In the case of (6.10a) and (6.10b) the National Annex may additionally modify expression 6.10a to include permanent actions only. (i.e. The variable actions are not included in (6.10a)).

The more unfavourable of expressions (6.10a) and (6.10b) may be applied instead of expression 6.10, but only under conditions defined by the National Annex.

EN 1990 also provides expressions for verifying both the accidental and seismic design situations.

**Partial factors for the ultimate limit states**

For buildings, the recommended partial factors for the persistent and transient situation in EN 1990 are \( \gamma_G = 1.35 \) and \( \gamma_Q = 1.5 \), but these may be altered by the National Annex. Values of combination coefficient \( \psi \) are given in EN 1990 but for simplicity the attached programme can be used for step-by-step design.

### 0.4.4.2 Serviceability limit states verification

For the serviceability limit states verification EN 1990 stipulates that:

\[
E_d \leq C_d \quad (6.10\text{ of EN 1990})
\]

where:

- \( C_d \) is the limiting design value of the relevant serviceability criterion.
- \( E_d \) is the design value of the effects of actions specified in the serviceability criterion, determined on the basis of the relevant combination.

**Combination of actions for the serviceability limit states**

For serviceability limit states verification, EN 1990 requires the three combinations below to be investigated. EN 1990 gives three expressions for serviceability design: characteristic, frequent and quasi-permanent.

a) The characteristic (rare) combination used mainly in those cases when exceedance of a limit state causes permanent local damage or permanent unacceptable deformation.

\[
\sum_{j=1}^{n} G_{k,j} + \sum_{i=1}^{m} \psi_{0,i} Q_{k,i} \quad (6.14b \text{ of EN 1990})
\]
b) The frequent combinations used mainly in those cases when exceedance of a limit state causes local damage, large deformations or vibrations which are temporary.

\[
\sum_{j=1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \psi_{2,1} Q_{k,1} \geq \text{(6.15b of EN 1990)}
\]

c) The quasi-permanent combinations used mainly when long-term effects are of importance.

\[
\sum_{j=1} G_{k,j} + P + \psi_{2,1} Q_{k,1} \geq \text{(6.16b of EN 1990)}
\]

Partial factors for serviceability limit states

Unless otherwise stated (e.g. in EN 1991 to EN 1999), the partial factors for serviceability limit states are equal to 1.0. \(\psi\) factors as given in EN 1990. Alternatively, the attached programme can be used for determining the \(\psi\) factors.

0.4.5 RECOMMENDATIONS FOR COMBINATION AND PARTIAL FACTORS TO BE ADOPTED IN THE BSI NATIONAL ANNEX

0.4.5.1 Choice of NDPs for the BSI National Annex to EN 1990 for serviceability limit state verification.

Based on the considerations of:

- levels of reliability ‘enjoyed’ in the UK; and
- usability, both for the super-structure and the sub-structure.

The UK National Annex has adopted the use of either:

- Expression 6.10 with \(\gamma_G = 1.35\) and \(\gamma_Q = 1.5\), or
- Expression 6.10a and 6.10b with \(\gamma_G = 1.35\) and \(\gamma_Q = 1.5\) and \(\xi = 0.925\).

for the persistent and transient design situations with the EN 1990 recommended \(\psi\) values, except the \(\psi_0\) value for wind is reduced from 0.6 to 0.5.

For the accidental design situations, expression (6.11b) of EN 1990 is adopted in the BSI National Annex and \(\psi_{1,1}\) is chosen for the loading variable action.

0.4.5.2 Choice of NDPs for the BSI National Annex to EN 1990 for serviceability limit state verification.

The BSI National Annex adopts the expression (6.14b) and (6.15b) and (6.16b) with \(\gamma = 1\), and the \(\psi\) value as for the ultimate limit state verifications.

0.4.6 RESISTANCE PARTIAL FACTORS

The material partial safety factors \(\gamma_c\) and \(\gamma_s\) are given in EN 1990 for ultimate limit state verifications and for serviceability limit state verifications.
CHAPTER 1

Differences between BS 5268: Part 2 and EN 1995-1-1

1.1 Structures of the Codes

1.1.1 CURRENT BRITISH CODES FOR DESIGN IN TIMBER

The current British Codes for design in timber are the parts of BS 5268. These are as follows:

BS 5268: Structural use of timber
- Part 2: 2002 Code of Practice for permissible stress design, materials and workmanship
- Part 3: 1998 Code of Practice for trussed rafter roofs
- Part 5: 1989 Code of Practice for the preservative treatment of structural timber
- Part 6-1: 1996 Code of Practice for timber frame walls. Dwellings not exceeding four storeys
- Part 6.2: 2001 Code of Practice for timber frame walls. Buildings other than dwellings not exceeding four storeys
- Parts 7.1 to 7.7 Recommendations for the calculation basis for span tables for various elements.

There is no current BS 5268 Part 1. BS 5268-2 is the key code in the series, covering structural design in timber in general. The other parts provide subsidiary information. BS 5268-2 specifies how timber structures may be designed to withstand applied loads. However, the appropriate loads to be used for design purposes are given in BS 6399: Loading for buildings, with three parts covering dead and imposed loads, wind loads and imposed roof loads. All
standards relevant to structural design in timber are set out in Appendix A, arranged by topic with British national standards matched to equivalent British adoptions of European standards where appropriate.

1.1.2 CURRENT AND ANTICIPATED EUROPEAN CODES FOR DESIGN IN TIMBER

European standards must be published in every country whose national standards body is a member of CEN, and conflicting national standards must be withdrawn according to an agreed timetable. In the UK, BSI publishes CEN standards with a number that includes a BS prefix, for instance EN 1990: 2002 is published as BS EN 1990: 2002. The main text is the agreed English language version of the EN, but BSI adds a National Foreword, and when appropriate a National Annex. The BSI standard may, on occasion, be published in a later year than the CEN standard (for instance, BS EN 912: 2000 ‘Timber fasteners – Specifications for connectors for timber’ is the BSI adoption of EN 912: 1999). In this report, adopted European standards are referred to with the prefix EN rather than BS EN to distinguish them from British national standards, and for consistency with the numbers of draft and prospective standards, which do not yet have a BS prefix.

The Eurocodes form an integrated suite for design. Hence, the design in timber will be based on EN 1990: 2002 ‘Basis of structural design’. Loads, which are termed ‘actions’ in the Eurocodes, will be determined according to the appropriate parts of EN 1991 ‘Actions on structures’. Detailed design for a timber building will be undertaken to EN 1995-1-1, with aspects of structural fire design to EN 1995-1-2. It will be necessary to design the foundations to EN 1997. In relevant areas, it would also be necessary to use EN 1998 ‘Design of structures for earthquake resistance’.

EN 1995-1-1 has not yet been published but its publication is anticipated in early 2005. The UK National Annex has been prepared only in draft form and is to go out for public comments before its publication. This review is based on the pre-standard issued for public comment, EN 1995-1-1: 2003, and the 4th draft, September 2003, of the UK National Annex, with comments on differences expected in the published EN 1995-1-1.

1.2 Design basis

1.2.1 DESIGN BASIS FOR THE EUROCODES

Most of the basic Eurocode design principles that apply to timber structures also apply to structures of other materials, and are established in EN 1990.

Basic requirements given in EN 1990 include adequate durability, and the ability to resist disproportionate collapse as a result of impact, explosion, etc.

The Eurocodes are based on limit state (ultimate and serviceability limits) design. The requirements for structural reliability are assessed against defined states, beyond which the structure no longer satisfies its performance criteria.
The principles of limit state design used in the Eurocodes are set out in EN 1990, Section 3, with a more descriptive explanation in STEP 1 lecture A2 ‘Limit state design and safety format’(d). Two types of limit state are considered for the Eurocodes. These are ultimate limit states and serviceability limit states. Limit states that concern the safety of people and of the structure are classified as ultimate limit states. There may also be circumstances in which an ultimate limit state concerns the protection of the contents. Ultimate limit states are thus states where the structure has reached the point of collapse or other major disruption. Limit states that concern the functioning of the structure under normal use, cracking, deflections sufficiently great to cause concern, and the comfort of people are classified as serviceability limit states. Serviceability requirements should be agreed for each individual design project.

In the design of structures, relevant design situations must be considered by verifying the design to demonstrate that no limit state will be exceeded when appropriate design values are used. Design situations are classified as:

- persistent (the conditions of normal use)
- transient (eg during construction or repair)
- accidental (fire, explosion, impact, or the consequences of local failure)
- seismic.

Verification is based on the application of partial factors to actions and material properties which are selected to give an appropriate level of certainty that the design values will not be exceeded in practice. This is termed reliability. Different partial factors are selected for particular ultimate limit states, whilst they are generally set at 1.0 for serviceability limit states.

Actions applying to structures are classified as:

- permanent \((G)\) eg self-weight, fixed equipment, indirect actions (eg from shrinkage)
- variable \((Q)\) eg imposed loads, wind and snow
- accidental \((A)\) eg explosions, impact from vehicles.

EN 1995-1-1 explains how some aspects of the design basis should be implemented for timber structures, and sets out certain aspects of design that are important for timber but are not relevant to many other structural materials, particularly how to allow for the effects of duration of load and moisture content.

1.2.2 DESIGN BASIS FOR BS 5268

BS 5268 is based on permissible stress design, and this phrase appears in the title of Part 2, the key part. However, it does not include a description of the design principles equivalent to that in EN 1990. Grade stress values for
properties of materials, most of which are given in tabular form in BS 5268-2 itself, incorporate safety factors so that they represent the stress that the material is considered able to bear over the life of a building with a reasonable level of safety. The grade stresses are considered for a 50-year load duration. A series of modification factors are then applied to the mechanical properties, generally increasing the load-bearing capacity for shorter load durations. The permissible stress in service for a particular type and direction of load to the element is thus established, and verified against the stress applied by the design loads. These design loads are normally obtained from the relevant parts of BS 6399.

In addition to supporting normal loads, BS 5268-2 requires timber structures to withstand accidental damage without catastrophic collapse. Specific robustness design requirements were not usually necessary for buildings up to four storeys until recently. However, the recent Approved Document A gives guidance for the new requirements.

1.3 Methods of determining material properties

1.3.1 GENERAL PROCEDURES

1.3.1.1 General procedure for material properties under the Eurocodes
EN 1990 requires that material properties in general, and particularly strength properties, should be represented by characteristic values. The less favourable tail of the assumed distribution (usually normal distribution) of the property (5 percentile) should be used. Thus, where a low value of a mechanical property is unfavourable (for instance bending strength for a beam stressed in bending by a load), the 5% fractile value of the property should be used. This means that the characteristic value for bending strength will be that which 95% of the hypothetical population of timber of that strength class would be expected to exceed, whilst 5% would be expected to be weaker than that value. As timber is a material that exhibits a high degree of variability, it is important that determinations of property values take such variability into account. Such unfavourable characteristic values are used to verify ultimate limit states. However, EN 1990 requires that the structural stiffness parameters and thermal expansion coefficients should normally be represented by mean values. These parameters are generally used to verify serviceability limit states. EN 1990 also requires that different values of these parameters should be used to take into account the duration of the load. This again is a key issue for timber.

1.3.1.2 General procedure for material properties under BS 5268-2
The foreword to BS 5268-2 states that from BS 5268-2: 1984, stresses have been estimated from the 5% lower exclusion values for strength and stiffness (previously 1% values had been used). This change was made to align with the general use of 5% characteristic values for strength of materials. In fact, the tables present both mean and minimum values for stiffness, and either value may be used depending on the circumstances.
1.3.2 MATERIAL REQUIREMENTS

1.3.2.1 Material requirements of EN 1995-1-1

The characteristic values for materials that are used in design to EN 1995-1-1 are taken from subsidiary standards for the particular types of material. Clauses 3.2 to 3.7 specify the harmonized standards with which materials must comply. In general, these standards also give the source of property values for these materials suitable for use in structural design to EN 1995-1-1. Measurements of strength properties in the Eurocode scheme are based on a test duration of five minutes ± two minutes. The strength properties of timber and timber-based members can be subject to size effects (e). EN 1995-1-1 sets a reference depth for solid timber members of 150mm, and a formula is given by which the characteristic bending and tensile strength parallel to the grain can be increased for members of less depth. However, EN 1995-1-1 does not require a reduction in characteristic strength for members of greater depth. Similarly, the characteristic bending and tensile strength of glulam may be increased for members less than 600 mm depth, and the bending strength of LVL (laminated veneer lumber) less than 300 mm deep. LVL has a reference length in tension of 3000 mm; the tensile strength of LVL should be adjusted for length whether it is longer or shorter than the reference length.

A particular feature of timber-based materials is their response to load duration and moisture influences. The load that a member is able to withstand without failure is greater if that load is only applied for a short duration than if it is loaded for a long period. Also, if the member has a high moisture content, as a result of exposure to high humidity or to water (i.e. rain, condensation, etc.), it will fail at lower loads than if it has a low moisture content. In addition, if a member is subject to a constant bending stress, its deflection will continue to increase with time, the process being known as creep (f). Creep in members with a high moisture content will be appreciably greater than in those with a low moisture content. These duration and moisture influences are substantially greater for board materials in which wood has been broken down into small particles and then reconstituted than for solid timber.

Load-duration classes are given in EN 1995-1-1. Three service classes are established:

- Service class 1: is characterised by the moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65% for a few weeks per year. In such moisture conditions most timber will attain an average moisture content not exceeding 12%.

- Service class 2: is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85% for a few weeks per year. In such moisture conditions most timber will attain an average moisture content not exceeding 20%.

- Service class 3: due to climatic conditions, is characterised by higher moisture contents than service class 2.
EN 1995-1-1 accounts for these effects by a factor termed \( k_{\text{mod}} \) to modify strength values according to a service class and a load-duration. Values of \( k_{\text{mod}} \) for material types, service classes and load durations are presented in Table 3.1. They range from 1.1 for instantaneous actions such as impact in service class 1 to low values of 0.5 for solid timber under permanent load in service class 3 and 0.2 for certain board materials under permanent load – fibreboards are not approved for long-term loads under the more severe service classes.

Creep is determined by using a factor termed \( k_{\text{def}} \) with the modulus of elasticity of a material in relation to each individual action. \( k_{\text{def}} \) factors are given in Table 3.2 for material types and service classes, for quasi-permanent actions (defined in EN 1990). The final mean modulus in response to each action is the mean modulus divided by \( (1 + \psi_2 k_{\text{def}}) \), \( \psi_2 \) being the quasi-permanent factor for that type of action (\( \psi \) factors are explained below in the section on combining loads under the Eurocodes). The smallest \( k_{\text{def}} \) factors are for solid timber, glulam and LVL which range from 0.6 in service class 1 to 2.0 in service class 3. \( k_{\text{def}} \) factors for board materials range up to 4.0 for materials limited to the drier service classes. For timber which is installed at or near its fibre saturation point (i.e. the cells of timber are saturated with water) and is likely to dry out under load, \( k_{\text{def}} \) should be increased by 1.0. The effects of moisture on the mechanical properties of wood samples are given in ‘Wood as a building material’.

1.3.2.2 Material requirements of BS 5268-2

BS 5268-2 has much more extensive sections on specific materials than EN 1995-1-1. Property values for the materials are tabulated within BS 5268-2, and many of the design rules are included with the material.

BS 5268-2 sets a common reference depth of 300mm for solid timber and for glulam (LVL is not covered in BS 5268-2 yet). Factor \( K_7 \) modifies bending stress according to depth, and factor \( K_{14} \) tensile stress. The approach of BS 5268-2 is the opposite of that of EN 1995-1-1 in that the mechanical properties of materials in BS 5268-2 already take into account the reduction factors.

Factors \( K_7 \) and \( K_{14} \) are identical to each other below the reference depth of 300 mm, but use different formulae to EN 1995-1-1.

Whilst BS 5268-2 addresses the influences of load duration and moisture content, its approach appears simple compared to the conversions applied by EN 1995-1-1. BS 5268-2 uses three service classes identical to those of EN 1995-1-1. It is important to note that the EN 1995-1-1 and BS 5268-2 load duration classes do not have equivalent durations even when called by the same name. Grade stresses are based on the ability of members to bear ‘long-term’ loads (for the design life of the building which is 50 years). For solid timber and glulam, a factor \( K_3 \) is used to increase stresses for members subject to shorter term loading; this factor modifies values to give a similar duration effect to \( k_{\text{mod}} \) for the same materials. Also, for solid timber and glulam, a factor \( K_2 \) is used to reduce stresses of service classes 1 & 2 in order to be used for service class 3; this is rather more detailed than EN 1995-1-1, as different values of the factor are used for different stresses (EN 1995-1-1 uses only one factor for all the stresses). Creep of solid timber and glulam is generally ignored, as the design limit of 0.003 span, and also 14 mm for domestic floor joists, under maximum load, is considered adequate to accommodate creep without needing to take
into account the intermittent nature of most true imposed loading. Annex K has a more detailed procedure for calculating and verifying deflections, which takes creep, load duration and service class into account. Annex K retains the \( K_2 \) factor to modify the modulus of elasticity, but in other respects the calculation of deflections is similar to the Eurocode system except that BS 5268-2 Annex A gives much smaller values for creep in response to variable loads, as the creep modulus is modified by both the quasi-permanent load factor \( \psi_2 \) and by \( k_{\text{def}} \) factors which reduce for shorter load durations.

BS 5268-2 has two approaches to design with plywood. Grade stress values are tabulated for some types of plywood, known as established plywoods (experience in their use is available in the UK). These values are modified for service class and load duration by factor \( K_{36} \), which has one set of values for stresses and another for moduli. Alternatively, design may use characteristic values for plywood. The formulae for converting characteristic strength values to grade stress values include a \( k_{\text{mod}} \) factor equivalent to that in EN 1995-1-1, except that the factor values are adjusted for the different load durations used in BS 5268-2. The formula for converting characteristic moduli into grade moduli divides the characteristic modulus by \((1 + k_{\text{def}})\); the BS 5268-2 \( k_{\text{def}} \) values account for both service class and load-duration (unlike EN 1995-1-1 where \( k_{\text{def}} \) values only reflect service class, and quasi-permanent \( \psi_2 \) factors are used for load duration).

For board materials other than plywood, BS 5268-2 uses characteristic values, with \( k_{\text{mod}} \) and \( k_{\text{def}} \) factors used in the same way as with characteristic values for plywood.

BS 5268-2 allows any new board materials to be used/designed provided that they follow the requirements given in sections 4 (for plywoods) and 5 (for any boards other than plywood).

### 1.3.3 SOLID TIMBER

#### 1.3.3.1 Solid timber under EN 1995-1-1

Timber is a highly variable material. Its properties depend on the species of timber, but also vary greatly in response to differences in density of the fibre material, and to the presence of knots and other defects. In order to obtain reliable performance, structural timber is sorted into strength classes (sometimes called grades) with more closely defined properties. Grading may be done visually, or by machine. The European strength classes for timber are based on a unified system set out in EN 338 ‘Structural timber – Strength classes’. Strength classes are annotated by an alphabetical letter followed by a numerical number (ie C16, C24, D30, etc.). The letter ‘C’ represents softwood (coniferous) species and poplar, whilst ‘D’ represents hardwood (deciduous) species. In addition, the numerical number represents the characteristic lower 5th percentile value of the bending strength of 150 mm deep timber in N/mm².

EN 1995-1-1 requires that timber members comply with EN 14081-1. This is currently available as EN 14081-1: 2000 ‘Timber structures – Strength graded structural timber with rectangular cross section. Part 1: General requirements’. The EN 14081 suite of standards has been rejected by the CEN committee TC...
124 ‘Timber structures’, but it is expected that they will be passed at a future vote without major changes. Round timber must comply with EN 14544. This is currently available as EN 14544: 2002 ‘Timber structures – Strength graded structural timber with round cross-section – Requirements’. Timber to EN 14544 must be graded in accordance with an appropriate standard, but currently no such standards are published.

EN 14081-1 requires timber to be strength graded either to an approved visual grading standard, or by machine using settings determined in accordance with EN 14081-2. This is currently available as EN 14081-2: 2000 ‘Timber structures – Strength graded structural timber with rectangular cross section. Part 2: Machine grading – Additional requirements for initial type testing’. EN 14081-3 deals with ‘Timber structures – Strength graded structural timber with rectangular cross section. Part 3: Machine grading – Additional requirements for factory production control’ whilst EN 14081-4 deals with ‘Timber structures – Strength graded structural timber with rectangular cross section. Part 4: Machine grading – Grading machine settings for machine controlled systems’.

The strength classes of EN 338, the grades into which timber would normally be graded, are based on measured values of bending strength, mean modulus of elasticity in bending and density. Strength and modulus are measured to EN 408 and density to ISO 3131: 1975 ‘Wood – Determination of density for physical and mechanical tests’, and these are evaluated to EN 384. For allocation to a strength class, the requirements for characteristic bending strength, mean modulus, and both characteristic and mean density must be met.

EN 408 and EN 384 are currently available as 1995 editions. However, draft revisions, EN 408: 2000 ‘Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties’ and EN 384: 2000 ‘Structural timber – Determination of characteristic values of mechanical properties’, have been prepared. The principal effect of adopting these revisions would be to change the method of measuring modulus of elasticity in bending to demonstrate compliance with EN 338. Currently the local modulus of elasticity (MOE) measurement is a true shear-free determination of the modulus. The revised standards would require measurement of the global modulus of elasticity (i.e., including shear deflection in addition to bending deflection), with results then adjusted to subtract the estimated shear deflection. Direct measurement of the true modulus requires measurement along the neutral axis over a relatively short span. A note included with EN 384 points out that it has been shown that these test results tend to be inconsistent both within and between laboratories. Global MOE is measured from the tension face over a larger span. Measurements have been shown to be far more consistent, and can be made more conveniently than the local modulus, particularly in conjunction with determining bending strength. The conversion formula to adjust values to determine the pure bending modulus was developed from comparative testing work undertaken by several countries (including work by BRE), and so there should be no systematic bias that might cause a substantial change in the structural performance of timber allocated to grades as a result of changing the method of measurement. If measuring global MOE does indeed give more consistent results, this should bring a slight improvement to the systems for defining and confirming strength grades for timber.
For visual strength grading, CEN accepts certain national grading schemes including those used in the UK. EN 14081-1 gives requirements for visual grading schemes in an annex, and refers to EN 1912 ‘Structural timber – Strength classes – Assignment of visual grades and species’ which assigns grades and species to strength classes. Once a particular source and species grade has been accepted as meeting a strength class, timber is graded according to visual rules, and any requirement for testing to demonstrate that the timber continues to meet the EN 338 strength class requirements will depend on the grading scheme used.

The principles of machine strength grading of timber are set out in BRE Digest 476(o). There are two alternative approaches to machine strength grading. The output-controlled system is relatively straightforward and very efficient as samples from the graded timber are proof tested in accordance with EN 408 at regular intervals, and the results may be used to optimise the machine settings determining the grades. However, this process requires long runs and high throughputs of timber of a consistent type, and is rarely feasible in Europe. The machine-controlled system requires detailed calibration of the machine’s output with any input source and species of timber to be graded. These settings are published, and accurate grading relies on always using the appropriate settings for the input timber and output grades on a correctly calibrated machine. As with visual grading, once appropriate machine settings have been accepted for grading a particular source of timber to specific strength classes, there is no requirement for further testing to EN 408 to demonstrate that the timber continues to meet the EN 338 strength class.

Machine strength grading is currently dominated by a few brands of grading machine that work by bending the timber across its smallest dimension to measure its stiffness as it passes between rollers. Methods of grading using alternative scanning systems have been investigated, and may be introduced on a significant scale. Systems based on X-rays or stress wave technology have the operational advantage of being able to grade virtually the full length of a piece timber with a single pass and reduced or no mechanical contact with the timber. Bending-type machines are unable to grade about 0.5 m at each end of a piece, and these ends must be assessed visually. Hence alternative systems offer faster throughput with lower labour costs. However, alternative systems have not yet achieved as accurate a prediction of the strength and other grades properties as bending-type machines. Thus, to meet the grade requirements, such grading machines must be set to allocate less timber to the higher grades and more to the lower or reject grades. Neural networks are also being evaluated, to sort timber into grades by evaluating a number of indicating parameters instead of using a single regression parameter. Such assessment has the potential to grade more efficiently than would be possible from just one of the parameters, but complex systems are harder to calibrate accurately and confidently than single-parameter algorithms. EN 14081-2 specifies a method for evaluating the relationship between indicating property and timber grade for machine-controlled grading, describing size and global cost matrices that shall be used to determine the appropriate machine settings.

Historically, strength grading schemes in the UK require third-party certification. It is currently unclear whether this will continue with the introduction of EN 14081-1 but, in any case, it will be possible to CE mark timber to EN 14081-1 and
import it into the UK for structural use without third-party certification. Such timber may also have been visually strength graded to a system that is currently unfamiliar in the UK, and may thus produce a slightly different distribution of grade properties.

Whilst bending strength, modulus of elasticity and density must be measured to establish that a grade of timber meets a particular EN 338 strength class, the other properties tabulated in EN 338 are derived from the bending strength values for the measured properties. However, methods of measurement for these properties are currently given in EN 408 and EN 1193, whilst all of these tests are brought together in EN 408: 2000. In principle, direct measurement might demonstrate that a particular type of timber had superior properties to the derived values. However, neither BS 5268-2 nor EN 1995-1-1 encourage this by stating explicitly that such values are permissible for design, whilst the extensive test programme that would be required makes it highly unlikely that such an approach would be adopted for structural timber due to the cost involved in testing.

1.3.3.2 Solid timber under BS 5268-2

In BS 5268-2, the properties of each grade of timber are published as grade stresses. These are design stresses based the maximum long-term loading that should be applied to the material, with safety factors included within these values, prior to adjustments for service class, etc. These stresses are based on a reference depth of 300mm and a life duration of 50 years. BS 5268-2 has a legacy of various series of grades. Several series of visual strength grades cover timber of particular combinations of source and species. Subsequently, machine grades were added. These developments gave rise to a complex series of grades, all with different properties. By selecting a particular grade, it is possible to utilize the mechanical advantages of different species and grades of timber. However, designing in this way meant that timber of a very specific type had to be supplied for construction. To alleviate this, an additional series of strength classes was added; the existing grades could be specified directly or accepted as meeting a designated strength class for which their range of properties was adequate.

When the European standard strength classes were introduced, as published in EN 338: 1995, the intention in the UK had been to replace the existing strength class grades in BS 5268-2 with those of EN 338. The grades of EN 338 were designed to give a smooth progression of levels for bending strength, modulus of elasticity and density, all set at rounded values. Detailed data on the structural timber currently in use in the UK (primarily home-grown and Northern European Scots pine and Sitka spruce) were considered. These data suggested that EN 338 did not have an ideal match of strength and stiffness values for structural timber as used in the UK, which would result in an appreciable loss of yield during the grading process. Accordingly, the density values and grade stresses corresponding to the strength classes of EN 338 were accepted, but alternative moduli of elasticity were established in the BS 5268-2 table of strength class properties. In addition, two further grades intended primarily for trussed rafters were created, TR20 and TR26, to substitute for existing BS 5268-2 machine strength grades that would not be matched closely by EN 338. These trussed rafter grades have relatively high permissible strengths in relation to their stiffness.

Owing to their evolution over time, the BS 5268-2 grades of timber are not based on a straightforward measurement and evaluation process as are the EN 338 strength classes. Furthermore, the tabulated values are permissible stresses rather than characteristic values that could be confirmed by direct tests. Many of the grades in BS 5268-2 were transferred from the preceding code, CP 112: Part 2. Most of the test work to establish those values was based on small clear specimens, which give much higher test results than structural sizes complete with defects. However, since its publication in 1995, EN 408 would have been the appropriate test method for investigating the performance of grades of structural timber. The development of permissible stresses for strength grade timber is discussed further under safety factors.

Like EN 14081-1, EN 519 allows either output-controlled or machine-controlled machine grading. For a machine-controlled system, EN 519 requires proof from a large sample that the output grades will meet their required properties. Unlike the global cost matrix of EN 14081-2, EN 519 does not specify a particular method for demonstrating what is achieved; the global cost matrix method could be considered as an appropriate method of meeting the requirements of EN 519. A significant difference between the standards is that EN 519 requires the regression equation between the bending strength of the timber and the machine’s indicating property to have a coefficient of determination of not less than 0.45, and between the bending modulus of elasticity and the indicating property of 0.5. This requirement excludes imprecise grading machines, such as X-ray and stress wave machines have tended to be. Excluding imprecise machines ensures that the structural potential of timber is used more effectively, with greater yields in the higher grades and probably a slightly reduced proportion of members appreciably below the grade lower 5th percentile lower limit.

The derivation of the additional properties tabulated for the BS 5268-2 grades of timber is even less clear than for bending strength, bending modulus and density, the properties which are linked to direct measurement in the BS 5268-2 adoption of the EN 338 strength classes. In general, permissible stresses under BS 5268-2 are in the region of one third of the characteristic strengths under EN 338 because safety factors are built in, whilst moduli are similar as deflections and serviceability limits are generally applied without safety factors. Key comparisons between the two sets of values for strength class timber (a package of timber should grade almost identically under either system) are given below.

Compression parallel to the grain bears the expected relationship for softwood, but for the highest strength hardwoods the BS 5268-2 values are about two thirds of those for EN 338. For shear strength, the formula in EN 338 Annex A sets a maximum value of 3.8 N/mm²; in Table 1 this limit is applied for
softwoods but not for hardwoods. BS 5268-2 values for shear of hardwood are almost half of those tabulated for EN 338.

EN 338 strength properties for tension and compression perpendicular to the grain are derived from density values. The 2003 version of EN 338 introduced a major change in the values used for compression perpendicular to the grain in softwoods; values are less than half of those in EN 338: 1995. The density values of BS 5268-2 and EN 338 are identical. BS 5268-2 has values for compression perpendicular to the grain both with and without wane. With wane permitted, these values are about 0.8 of those of EN 338: 2003 for softwoods, but only about 0.35 of those for hardwoods. For BS 5268-2, tension perpendicular to the grain is not tabulated, but may be taken as one third of shear parallel to the grain. However, for EN 338: 2003 it is tabulated as a function of density at low densities, but capped at a maximum of 0.6 N/mm² for the higher strength classes. As a result, BS 5268-2 values for tension perpendicular to the grain are less than half those of EN 338: 2003 for the low and moderate strength classes of softwood, but are greater than EN 338: 2003 for high strength hardwoods.

Mean moduli of elasticity given in EN 338: 2003 and BS 5268-2 are very similar. Characteristic moduli in EN 338: 2003 and minimum moduli in BS 5268-2 are a fixed proportion of the mean moduli, except that EN 338 uses a higher ratio for hardwoods than for softwoods, whilst BS 5268-2 only uses the higher ratio for the higher strength hardwoods.

The simplification of the grade system to strength class grades that will result from the introduction of EN 1995-1-1 will lead in principle to a slight reduction in the ability to use timber to the full extent of its available properties. The loss of the ability to utilize the full capacity of visual and North American grades will probably be less significant than the loss of the TR strength class grades. However, the benefits of stocking and supplying timber to a unified system may well outweigh any costs from the reduction of grades. A clear economic cost is expected from reduced yields, resulting from machine grading to true EN 338 strength class grades rather than the BS 5268-2 interpretation of those grades. Further changes could potentially result from the introduction of alternative types of strength grading machines.

After allowing for the use of grade stresses rather than characteristic strengths in BS 5268-2, the major properties (those applying parallel to the grain) of the EN 338 strength class grades differ very little from those of the strength class grades as used in BS 5268-2. The one exception is the relatively higher compression stresses permitted by BS 5268-2. There are considerable differences in the values derived for some other properties, but there are also substantial differences between BS 5268-2 and EN 1995-1-1 in the formulae used for design with some of these properties, for example bearings at supports utilizing compressive strength perpendicular to the grain. Overall, it seems likely that changes in the design process and levels of safety factors may have more influence on the volume and quality of timber required for construction than changes in the measurement and classification of timber.
1.3.4 GLULAM AND LVL

1.3.4.1 Glulam and LVL under the Eurocodes
EN 1995-1-1 requires glued laminated timber members to comply with EN 14080. This is currently available as EN 14080: 2000 ‘Timber structures – Glued laminated timber – Requirements’. This specifies glulam, whose properties may either be calculated from the timber used to form it, or by testing of an adequate sample of the manufactured glulam. Timber used to form glulam must conform to EN 14081, the standard for solid timber for structural use. Glulam to EN 14080 may conform to one of the strength classes with characteristic values tabulated in EN 1194: 1999 ‘Timber structures – Glued laminated timber – Strength classes and determination of characteristic values’, except that mean density is omitted. Alternatively, characteristics for the particular type may be evaluated directly, in accordance with EN 1194. Test methods for evaluating the characteristic values of glulam are identical to those for solid timber, but additional tests are required to check the quality of glued joints within the material. The scope of EN 1194 is limited to horizontally laminated glulam, and so there is no clear route for vertically laminated glulam to be used under EN 1995-1-1.

EN 1995-1-1 requires LVL structural members to comply with EN 14374. This is currently available as EN 14374: 2002 ‘Timber structures – Structural laminated veneer lumber – Requirements’. Characteristic values for types of LVL must be evaluated by the methods used for solid timber, and some further tests devised for wood based panels. EN 14374 also gives additional information on the material, such as embedding strengths, that are given directly in EN 1995-1-1 for solid timber and some other materials.

1.3.4.2 Glulam and LVL under BS 5268-2
Under BS 5268-2, both horizontally and vertically glued laminated timber are included. However, LVL is not covered by the standard. Properties for glulam are determined by calculating the properties of laminates of strength grade timber combined together. A series of conversion factors for various properties are given, depending on the strength class of laminate timber and the number of laminates. Other requirements for the quality of finger joints, etc, also affect the resulting properties calculated for glulam. The net result of these factors seems to be that BS 5268-2 permits a greater increase in bending stress for glulam compared to its constituent solid timber than occurs under European standards if strength class timber is laminated to the formula given in EN 1194.

1.3.5 WOOD-BASED PANELS

1.3.5.1 Wood-based panels under the Eurocodes
EN 1995-1-1 requires wood-based panels to comply with EN 13896 and LVL used as panels to comply with EN 14279. EN 13896: 2002 ‘Wood-based panels for construction – characteristics, evaluation of conformity and marking’ is already published, whilst ‘Laminated Veneer Lumber (LVL) – Specifications, definitions, classification and requirements’ is currently available as EN 14279: 2001. Unlike the standards referenced in EN 1995-1-1 for solid timber, glulam and LVL, these standards cover panels for general use in addition to those suitable for structural use, and it is essential to select suitable grades. Even the structural
grades range from those suitable for service class 1 only through to those suitable for all service classes. Extensive information on wood-based panels, their selection and use is given in the BRE Digest 477 series (h-m).

The key test methods for determining the mechanical properties of panels are given in EN 789 ‘Timber structures – Test methods – Determination of mechanical properties of wood-based panels’. The current version of this standard was published in 1996. Revision is under way and EN 789: 2003 will be published in due course; the principal effect of this revision will be on the tests for panel and planar shear, which are currently in Annexes. Following experience of using these tests, the revision will introduce modified test pieces and the tests will be moved into the main body of the standard. Characteristic values are derived from test results according to EN 1058 ‘Wood-based panels – Determination of characteristic values of mechanical properties and density’.

Characteristic values for particular structural grades of panel, other than plywood and IVL, are given in EN 12369-1: 2001 ‘Wood-based panels – Characteristic values for structural design – Part 1: OSB, particleboards and fibreboards’. Alternatively, values for proprietary panels may be derived directly from testing. Also, EN 1995-1-1 requires that the use of softboards to EN 622-4 should be restricted to non-structural uses and should be designed by testing for wind bracing (there are no characteristic values for softboards given in EN 12369-1). One panel product that does not appear to be covered within the EN 1995-1-1 system is cement-bonded particleboard. This product is included in EN 13896, but not in EN 12369-1. It would be possible to evaluate its properties directly to comply with EN 13896, but EN 1995-1-1 does not give the modification factors for cement-bonded particleboard that it gives for other materials.

For plywood, a second part for EN 12369, EN 12369-2: 2003 ‘Wood-based panels – Characteristic values for structural design – Part 2: Plywood’, is in draft. EN 12369-2 sets out strength and modulus classes, and plywood would be characterised by four classes, covering bending strength and modulus along and across the panel. This draft also states that some other properties may be derived from bending strength, modulus and density data as an alternative to direct determination to EN 789 and EN 1058. Hence, publication of EN 12369-2 will formalise the presentation of design data for specific plywoods rather than providing grade data directly. Also, as EN 13896 was published prior to agreement on EN 12369-2, it does not refer to the latter standard, and so until EN 13896 is revised, there is unlikely to be a direct chain of reference from EN 1995-1-1 to EN 12369-2.

No tabulated characteristic values are currently available for IVL to EN 14279, and so determination from test results for specific products will be required.

1.3.5.2 Wood-based panels under BS 5268-2

BS 5268-2 has two sections covering panel products. Section 4 covers plywood, whilst section 5 covers panel products other than plywood. It also has two approaches to deriving design data for plywood. For types of plywood with a long history of structural use in the UK, tables of grade stresses are published, similar to those for the various grades of solid timber. The tabulated values are based on test data that was generated in the past, and are supported by
longstanding satisfactory experience with constructions designed to BS 5268-2
and incorporating those grades of plywood. Alternatively, design for plywood or
other wood-based panel products may be based on characteristic values for
panel materials. The sources for characteristic values are the same product or
test standards that are used for design according to the Eurocodes, with
formulae to convert these values to permissible stresses or moduli that are
similar to the formulae used under the Eurocode. Panel products for roofs,
floors and walls, including cement-bonded particleboard (for which standard
characteristic values are not tabulated) may also be specified in accordance with
EN 12871: ‘Wood-based panels – Performance, specification and requirements
for load-bearing boards for use in floors, walls and roofs’ and EN 12872: ‘Wood-
based panels – Guidance on the use of load-bearing boards in floors, walls and
roofs.’ Note: BS 7916: 1998 ‘Code of practice for the selection and application of
particleboard, oriented strand board (OSB), cement bonded particleboard and
wood fibreboards for specific purposes’ is no longer valid and is withdrawn.
BS7916 must not be used.

In order to continue using structural plywood from traditional sources when
designing to the Eurocodes, characteristic values in accordance with EN 789 and
EN 1058 will need to be determined in place of the BS 5268-2 grade stress
tables. For other panel products, this procedure is already in place under BS
5268-2: 2002, section 5. Plywood or other board materials will be affected by the
changes to the design process, safety factors involved and their production
specifications.

1.3.6 ADHESIVES

1.3.6.1 Adhesives under the Eurocodes
Adhesive joints form crucial parts of many timber structures, particularly within
components such as glulam and IVL. Unfortunately, specification is limited by
the paucity of standards specifying structural adhesives. EN 1995-1-1 sets a
general requirement for strength and durability. It then covers the appropriate
types of adhesives to EN 301 ‘Adhesives, phenolic and aminoplast, for load-
bearing timber structures: Classification and performance requirements’ for the
different service classes. However, EN 1995-1-1 does not refer to EN 12436
‘Adhesives for load-bearing timber structures – Casein adhesives – Classification
and performance requirements’. Other types of adhesive are in commercial use,
for instance for finger jointing green timber, but performance specifications or
test methods are not yet available within European (or British) standards. A
summary of structural wood adhesives is given in STEP 1 ‘Adhesives’ (n).

1.3.6.2 Adhesives under BS 5268-2
BS 5268-2 gives a table of permitted adhesives. However, these are either to EN
301 or to a British Standard that has now been withdrawn. Hence, the
introduction of EN 1995-1-1 will have little effect on the already poor position
for specifying adhesives for use with structural timber. However, BS 5268-2 does
have extensive advice on the production of sound adhesive joints that is not
replicated in EN 1995-1-1 or any document referenced by it. It has been
suggested that similar advice should be added to a UK National Annex for EN
1995-1-1.
1.3.7 **METAL FASTENERS**

1.3.7.1 **Metal fasteners under the Eurocodes**

Metal fasteners form a prime means of connecting timber members. A range of different type of fastener and connector are used. There are two key European harmonised standards for specifying these; EN 14592 ‘Timber structures – Fasteners – Requirements’, which so far is only available as a committee draft, and EN 14545, currently available as EN 14545: 2002 ‘Timber structures – Connectors – Requirements’. EN 14592 specifies requirements for nails, staples, screws, dowels, bolts, connectors, punched metal fasteners and nailing plates. Some of these fasteners are available in a range of different types (for instance the performance of nails depends on whether they are round or square, smooth-shanked or enhanced, etc), and these types are not always clearly defined in EN 1995-1-1. However, their definitions in the draft EN 14592 are generally less ambiguous.

One group of fastener types, comprising nails, staples, bolts, dowels and screws, are referred to as dowel-type fasteners. The only European product standard applying to any individual type of such fastener is EN 10230-1 ‘Steel wire nails – Loose nails for general applications’, but draft EN 14592 specifies the steel to be used for most such fasteners, with suitable types of corrosion protection. Key properties for the performance of these fasteners are the yield moment (bending of the fastener), embedment strength (failure of embedment occurs as a strong rigid fastener crushes the area of the timber member it bears on) and withdrawal capacity (a process termed the rope effect adds to the strength of a connection by pulling the members together via a tensile load in the fastener as the members start to slide across each other, and is thus dependent on the axial hold of the fastener in the members). Yield moment is a property of the fastener alone; it may be derived from a EN 1995-1-1 rule. Alternatively, EN 1995-1-1 permits the yield moment of any dowel-type fastener to be measured to EN 409 (although the scope of EN 409 is limited to nails), and characteristic values calculated to EN 14358. Embedment strength is a property of the combination of fastener and substrate. It may be derived from a EN 1995-1-1 rule or measured to EN 383 ‘Timber structures – Test methods – Determination of embedding strength and foundation values for dowel type fasteners’ and characteristic values calculated to EN 14358. Withdrawal capacity is also a property of the combination of fastener and substrate. Values may be derived from rules in EN 1995-1-1, or measured to EN 1382 ‘Timber structures – Test methods – Withdrawal capacity of timber fasteners’ and EN 1383 ‘Timber structures – Test methods – Pull-through resistance of timber fasteners’, and characteristic values calculated to EN 14358. Alternatively, the rope effect may be omitted from calculations. If the rules in EN 1995-1-1 do not apply to particular nailed or stapled joints, the load-carrying capacity and stiffness of the joints may be determined according to EN 1380 ‘Timber structures – Test methods – Load bearing nailed joints’ or EN 1381 ‘Timber structures – Test methods - Load bearing stapled joints’.

Punched metal plate fasteners are steel sheets with an array of teeth punched out from the plate. They are forced into the faces of adjacent timber members to connect them. Punched metal plate fasteners are specified in draft EN 14592, and also in EN 14545. Overlapping specification may be resolved before the standards are published. Requirements for the steel and corrosion protection...
are included. Anchorage strengths may be obtained from EN 1995-1-1 or measured by testing to EN 1075 ‘Timber structures – Test methods – Joints made with punched metal plate fasteners’, with characteristic values derived according to EN 14545; performance depends on the combination of fastener and timber.

Split ring and shear plate connectors operate by locating in annular grooves machined into timber members. A split ring connector is placed between two timber members, engaging in a groove in each member, with a bolt along the axis of the ring to hold the members together in contact with the ring. Shear forces are transferred directly from one member to the adjacent member through the ring. Shear plate connectors are circular plates with annular flanges and central bolt holes; shear forces are transferred from the member to the annular flange, via the connector to the central bolt, and through the bolt to the connector in the adjacent member. As with punched metal plate fasteners, current drafts of both EN 14592 and EN 14545 include brief harmonized specifications for split ring and shear plate connectors. Detailed specifications for size and shape for these connectors are given in EN 912 ‘Timber fasteners – Specifications for connectors for timber’. Under EN 1995-1-1, performance is calculated according to a series of factors including the dimensions of the connectors and the density of the timber – no test methods are referred to in EN 1995-1-1.

Toothed-plate connectors are metal plates, often circular but available in other shapes, with teeth projecting out perpendicular to the plane of the plate, in one or both directions. They act in the same way as split ring or shear plate connectors, depending on whether the teeth project in one or both directions, but instead of grooves being machined into the timber members, the connector teeth are forced into the timber. As with split ring and shear plate connectors, current drafts of both EN 14592 and EN 14545 include brief harmonized specifications for toothed-plate connectors, whilst detailed specifications of size and shape are given in EN 912. Under EN 1995-1-1, performance is calculated according to a series of factors including the dimensions of the connectors and the density of the timber – no test methods are referred to in EN 1995-1-1.

Note, formulae for calculating the load-carrying capacity and slip-moduli for connector joints that are similar but not necessarily identical to those in EN 1995-1-1 are given in EN 13271 ‘Timber fasteners – Characteristic load-carrying capacities and slip-moduli for connector joints’.

Principles for evaluating joints that do not conform to the design rules given in EN 1995-1-1 are given in EN 26891 ‘Timber structures – Joints made with mechanical fasteners – General principles for the determination of strength and deformation characteristics’ and EN 28970 ‘Timber structures – Testing of joints made with mechanical fasteners – Requirements for wood density’.

1.3.7.2 Metal fasteners under BS 5268-2
Tabulated values are given for deriving permissible loads for joints with other dowel-type fasteners, applying to fasteners that meet British standards where available. The standard for nails is BS 1202-1 ‘Specification for nails – Steel nails’ (this standard was revised in 2002 to refer to the European standard EN 10230-1 ‘Steel wire nails - Loose nails for general applications’ for types included in EN
The standard for screws is BS 1210 ‘Specification for wood screws’. The standard listed for bolts is for black bolts to EN 20898-1, which has been revised to EN ISO 898-1 ‘Mechanical properties of fasteners made of carbon steel and alloy steel – Bolts, screws and studs’, and for washers BS 4320 ‘Specification for metal washers for general engineering purposes – Metric series’. There is no standard for dowels, which are required to have a tensile strength \( \geq 400 \, \text{N/mm}^2 \). These tabulated values were derived according to Annex G, and may alternatively be calculated directly according to the Annex; its formulae are taken (with slight modification) from DD ENV 1995-1-1: 1994, the draft for development version of EN 1995-1-1, and are thus similar to EN 1995-1-1 except that the rope effect is omitted.

Under BS 5268-2, split ring and shear plate connectors are specified to BS 1579, but that standard has been replaced by EN 912 ‘Timber fasteners – Specifications for connectors for timber’. In BS 5268-2, permissible loads for joints are obtained from tabulated values.

As with split ring and shear plate connectors, under BS 5268-2 toothed-plate connectors are specified to BS 1579 (replaced by EN 912), and permissible loads for joints are obtained from tabulated values.

BS 5268-2 does not refer to test methods for establishing load capacities for any joints made with fasteners or connectors nor does it address joints with staples.

### 1.3.8 DURABILITY

#### 1.3.8.1 Durability under EN 1995-1-1

There are two key aspects to durability; durability of timber-based materials and durability of associated items such as adhesives, fasteners and connectors. EN 1995-1-1 section 4 is devoted to durability but is very brief.

EN 1995-1-1 section 4.1 covers resistance to biological organisms. This simply requires that timber and wood-based materials shall either have adequate natural durability in accordance with EN 350-2 for the particular hazard classes defined in EN 335-1, EN 335-2 and EN 335-3 or be given a preservative treatment in accordance with EN 351-1 and EN 460. EN 460 relates to satisfying durability requirements by natural durability, or identifying a need for preservative treatment. The titles of the above standards are:

- EN 335-1 ‘Hazard classes of wood and wood-based products against biological attack – Classification of hazard classes’
- EN 335-2 ‘Hazard classes of wood and wood-based products against biological attack – Guide to the application of hazard classes to solid wood’
- BS EN 335-3 ‘Hazard classes of wood and wood-based products against biological attack – Application to wood-based panels’
Meeting the above requirements should result in adequate durability of the timber-based products.

EN 1995-1-1 requires metal connections to be inherently corrosion-resistant or protected against corrosion, and examples of suitable minimum specifications are given. Although, the durability of adhesives is not mentioned in this section, it is however required as stated in the section specifying adhesives as materials.

1.3.8.2 Durability under BS 5268

BS 5268-2 includes requirements for the durability of timber and timber-based materials in the relevant sections on timber and plywood. However, it also refers to BS 5268-5, which is a part of the standard devoted specifically to the durability of structural timber (BS 5268-5 ‘Structural use of timber – Code of practice for the preservative treatment of structural timber’). BS 5268-5 is a helpful guide covering the need for preservative protection, risks from high moisture content, how design can reduce risks of degradation, and the potential of natural durability. Assessment of the risks of degradation and the severity of the consequences is covered, with selection and application of preservative treatment and properties of treated timber. The key supporting standard for preservation was BS 5589 ‘Code of practice for preservation of timber’. The BS 5268 approach of having a code for durability of timber-based structures gives the designer much more support in developing the best approach to achieving durability, encouraging design features that will reduce the risk of decay. However, both BS 5268-5 and BS 5589 have recently been declared obsolete as the National British Standards they called up are replaced by European Standards. Obsolete standards remain available for reference, but are no longer maintained and updated by the relevant BS committee.

The method by which durability was specified through preservative treatment under British Standards is far more prescriptive than the Eurocode approach, using process-based specifications. The CEN system is based on specifications for end results such as levels of penetration and retention of preservative achieved. BS 8417 ‘Preservation of timber – Recommendations’ has been published as a guide to assist users in the transition to the CEN system.

In addition to the radically different approaches to specifying durability of timber adopted by EN 1995-1-1 and the BS 5268 series, ongoing European Union legislation is curtailing the use of many of traditional preservative treatments. Therefore, durability through design becomes increasingly important. Durability in the future may require more careful detailing, selection of preservative and application to achieve the required level of durability than has been the case in the past.
The BS 5268-2 approach to specifying the durability of metal fasteners, etc, and of adhesives, is similar to that of EN 1995-1-1.

1.4 Design and material safety factors

1.4.1 ACTIONS (LOADS)

1.4.1.1 Actions (Loads) under the Eurocodes

‘Actions’ means ‘Loads’ in Eurocodes. Actions under the Eurocodes are represented by their characteristic values. The self-weight of the structure, fixed equipment, and actions caused by shrinkage or uneven settlement are classed as permanent actions. Imposed loads, wind actions and snow loads are termed variable actions, whilst actions from explosions or impact by vehicles are termed accidental actions.

Self-weight of the structure is represented by nominal values, which are combined to form a single action. For timber, EN 1991-1-1 gives values which correspond to the mean density of timber in a given strength class to EN 338, or the appropriate standard for other timber-based materials.

EN 1991-1-1 gives imposed loads for parts of buildings for specific types of use. Both concentrated and distributed loads are given. The values presented cover a range with a recommended value but the option for a National Annex to select a particular value within the range. The UK National Annex to EN 1991-1-1 has not yet been published. EN 1991-1-1 also gives information to allow imposed loads resulting from storage of materials to be calculated. EN 1991-1-3: gives snow loads, whilst EN 1991-1-4, still to be published, covers wind actions. The characteristic values of these variable actions correspond to an upper value with an intended probability of being achieved in a given return period.

Further parts of EN 1991, not all of which are yet published, cover other features such as actions on structures exposed to fires, actions during execution (construction), accidental actions, etc.

1.4.1.2 Loads (Actions) under BS 5268

BS 5268-2 requires design loads to be in accordance with the parts of BS 6399 and other relevant standards. The self-weight of structures can be determined to within fairly close tolerances and similar to those under EN 1991-1-1.

The magnitude assigned to imposed, snow and wind loads in particular circumstances are derived on similar bases for both BS 6399 and the Eurocodes, and it is expected that Nationally Determined Parameters in the relevant National Annexes to the Eurocodes will lead to loads determined for the UK under the Eurocodes being very similar to loads currently used by BS 5268-2 and provided by some parts of BS 6399.
1.4.2 PARTIAL FACTORS (SAFETY FACTORS)

1.4.2.1 Partial factors under the Eurocodes

Under the limit state design process as used by the Eurocodes, partial factors are applied to both actions and material properties. The purpose of these factors is to allow for uncertainties such as:

- There may be situations that the applied action in service is more severe than the characteristic values used in the design.
- The strength of the structure might be less adequate than calculated from the material characteristic values.

The individual partial factors are shown in EN 1990 Annex C Figure C3. In principle, two partial factors are applied to each action or material property. All partial factors are given the symbol $\gamma$. Overall, there are two main partial factors:

- $\gamma_F$ Partial factor for actions, also accounting for model uncertainties and dimensional variations
- $\gamma_M$ Partial factor for a material property, also accounting for model uncertainties and dimensional variations.

Recommended values for partial factors for actions are given in EN 1990 Annex A, although alternative values may be set by a National Annex. Different partial factors are used depending on the limit state being assessed, whether the action is permanent or variable, and whether the effects of the action are favourable or unfavourable to the stability of the design. For the design of structural members for ultimate limit states that do not involve geotechnical actions, the recommended partial factor for unfavourable permanent action $j$, $\gamma_{G_i,\text{sup}} = 1.35$, and the recommended partial factor for unfavourable variable action $i$, $\gamma_{Q_i} = 1.5$. For serviceability limit states, the partial factors for actions are normally taken as 1.0.

EN 1990 states, that for serviceability limit states, the partial factors for properties of materials should be taken as 1.0. For ultimate limit states, recommended values for partial factors for properties and resistances of timber and associated materials are given in EN 1995-1-1 Table 2.3, although alternative values may be set by a National Annex. For fundamental combinations of actions (these are the persistent and transient design situations described earlier) EN 1995-1-1 Table 2.3 recommends a value of $\gamma_M$ of 1.3 for solid timber and most board materials, but this is reduced to 1.25 for glulam and 1.2 for IVL, plywood and OSB, which are considered to maintain their performance more reliably than other timber-based materials. For accidental design situations, a partial factor of $\gamma_M = 1.0$ is recommended.

If design is assisted by testing, this should be according to EN 1990 Annex D which is for all materials (ie all Eurocodes). However, it is not believed that Annex D is appropriate for timber structures.
1.4.2.2 Safety factors under BS 5268-2

For the longer-standing materials of timber, glulam and certain grades of plywood, BS 5268-2 publishes tables of permissible grade stresses. These grade stresses are used directly (after modification for effects such as duration of load) for verification against the loads expected in service. Hence, the safety factors for the design process are included in the grade stress values, and their magnitude is not revealed in the code. The first edition of BS 5268-2, which replaced CP 112-2, was published in 1984. The derivation of the strength properties for softwood and laminated timber tabulated in that edition is described in a BRE report. Lower 5th percentile stress values for the bending strength of structural sized specimens of key structural grades of solid timber were determined from extensive testing, for short duration loads and adjusted to a reference depth of 200mm. The BS 5268-2 committee subsequently decided to use a reference size of 300mm rather than 200mm. To derive permissible grade bending stresses, the characteristic strengths were reduced by the following factors:

Section depth 0.849
Duration of load 0.563
General safety 0.724

Inverting the safety factor of 0.724 gives a factor of 1.38. However, the characteristic values were converted from 200 mm to 300 mm depth by the formula \( K = (\text{ratio of depths})^{0.403} \). If the grade stress values are subsequently adjusted within BS 5268-2 for reduced member depth, from 300mm to 75mm the formula for the factor \( K_7 = (300/h)^{0.11} \). This latter, smaller factor was justified as due to variability in the test data, and lack of data for hardwoods and machine graded softwoods. However, if converted back to the 200mm depth at which the characteristic values for the grades were determined, the values will have been reduced from the originals by a factor of 0.888. This could be considered as an additional part of the safety factor, extending the safety factor to 0.643, or 1.555 if the inverse is considered.

The same explicit safety factor of 0.724 or 1.38 was applied to derive permissible stresses for tension and compression parallel to the grain, but a smaller section width adjustment factor was used to convert tension results from 200mm to 300 mm sections, whilst no adjustment was made for compression. Permissible stresses for glulam are calculated from the values for the component timber, and so the safety factors applying to glulam under BS 5268-2 will be the same as those for solid timber.

BRE Report 79\(^{(q)}\) describes the determination of design stresses for plywood for the 1984 edition of BS 5268-2 (grade stresses for plywood in the current edition have been modified from those in the 1984 edition). The level of any safety factors applied when setting the permissible grade stresses for plywoods that are tabulated in BS 5268-2 are using the following:

\[
\text{Stress} = f_k \times 0.855 \times 0.864/1.5
\]

\[
E_{\text{mean}} = E_{k,\text{mean}} \times (0.9/0.9)/(1 + K_{\text{def}})
\]

\[
E_{\text{min}} (5\%) = 0.8 \times E_{\text{mean}}
\]
For service class 2:
The modification factors for $E$ are obtained using:

- For long term: $(1+k_{\text{def}} \text{ for long term})/(1 + k_{\text{def}} \text{ for long term})$
- For medium term: $(1+k_{\text{def}} \text{ for long term})/(1 + k_{\text{def}} \text{ for medium term})$
- For short and very short term: $(1+k_{\text{def}} \text{ for long term})/(1 + k_{\text{def}} \text{ for short term})$

The modification factors for stresses are obtained using:

- For long term: $(k_{\text{mod}} \text{ for long term})/(k_{\text{mod}} \text{ for long term})$
- For medium term: $(k_{\text{mod}} \text{ for medium term})/(k_{\text{mod}} \text{ for long term})$
- For short and very short term: $(1+k_{\text{def}} \text{ for short term})/(1 + k_{\text{def}} \text{ for long term})$

For service class 3:
The modification factors for $E$ are obtained using:

- For long term: dry modification factor for long term/[(1+k_{\text{def}} \text{ for long term, wet})/(1 + k_{\text{def}} \text{ for long-term, dry})]
- For medium term: dry modification factor for medium term/[(1+k_{\text{def}} \text{ for medium term, wet})/(1 + k_{\text{def}} \text{ for medium-term, dry})]
- For short and very short term: dry modification factor for short term/[(1+k_{\text{def}} \text{ for short term, wet})/(1 + k_{\text{def}} \text{ for short-term, dry})]

The modification factors for stresses are obtained using:

- For long term: dry modification factor for long term x [(k_{\text{mod}} \text{ for long term, wet})/(k_{\text{mod}} \text{ for long-term, dry})]
- For medium term: dry modification factor for medium term x [(k_{\text{mod}} \text{ for medium term, wet})/(k_{\text{mod}} \text{ for medium-term, dry})]
- For short and very short term: dry modification factor for long term x [(k_{\text{mod}} \text{ for short term, wet})/(k_{\text{mod}} \text{ for short-term, dry})]

For other types of plywood, and for panel products other than plywood, BS 5268-2 gives a procedure for design data to be derived from characteristic values. These characteristic values are drawn from the same sources as would be used under EN 1995-1-1 (for instance EN 12369-1 for panel products other than plywood) or the actual test results (5 percentiles). Furthermore, the formula for converting characteristic strength into permissible stress is $X_d = k_{\text{mod}}X_k/(1.35\gamma_M)$. The $k_{\text{mod}}$ and $\gamma_M$ factors are identical to EN 1995-1-1 ($\gamma_M$ is given as 1.2 for plywood and 1.3 for other panel products), whilst 1.35 matches $\gamma_G$, the partial factor for permanent loads, used in the Eurocode design process.
Hence, except for the use of $\gamma_Q = 1.5$ as the partial factor for variable loads with the Eurocodes, the safety factors used by BS 5268-2 are identical to those used in the Eurocode system. The reason for not using $\gamma_Q = 1.5$ is due to the different load durations in both standards.

For joints made with dowel-type fasteners, BS 5268-2 gives tabulated values for load-bearing capacity. However, for most material and fastener combinations for which the values are tabulated, the formulae used to calculate the tabulated values are given in Annex G, which may be used to calculate the joint load capacity with alternative sizes of member, etc. The formulae are taken from DD ENV 1995-1-1, with slight modifications. There is no direct reference to safety factors, and the underlying values are difficult to compare because of the BS 5268-2 approach of giving values for long-term loading and modification factors for shorter terms, whilst EN 1995-1-1 adopts the opposite approach. However, the formula includes two reduction factors, $K_d$ and $F_d$. When these are included, the permissible stresses for a single fastener in a joint are broadly similar to those under EN 1995-1-1 when $\gamma_M$ and a typical value of $\gamma_F$ are included. Hence, it appears that safety factors for dowel-type fasteners are broadly similar in BS 5268-2 and EN 1995-1-1.

BS 5268-2 does not give design formulae for joints made with punched metal plate, toothed-plate, split-ring or shear-plate connectors. Hence, it is not possible to identify the magnitude of any safety factors incorporated in the tabulated basic load values that are used to calculate permissible stress values other than those given in relevant certificates (ie BBA, BSI, BM TRADA & BRE Certifications).

In common with EN 1995-1-1, BS 5268-2 does not include safety factors for calculating design values for deflections.

When using design by testing for solid timber, BS 5268-2 seems to include only a small safety factor as part of the factor for duration of loading. The test load should exceed the design load by a duration-based modification factor $K_{85}$, the shorter duration values for which are 1.30 for very short term-loads and 1.52 for short-term loads. These durations are shorter than the test duration, and so the $K_{85}$ modification factor could be seen as including a safety factor, although the other factor in modifying test loads, $K_{73}$ which depends on the number of structures tested, falls to 1.0 if five are tested. If the factor was intended to generate characteristic values, twenty structures would need to be tested to reduce $K_{73}$ to 1.0. If roof or floor decking is tested, the $K_{85}$ factor is based on Eurocode 5, having a value of $1.35 - \gamma_M - k_{mod, test}/k_{mod, design}$, but again, testing five structures would yield the lower 20th percentile rather than the lower 5th percentile strength.

### 1.4.3 LOAD FACTORS

#### 1.4.3.1 Combining loads under the Eurocodes

Many of the actions on buildings are variable. Extreme wind gusts are very infrequent, with very short durations. Snow loads are occasional, with short durations, under most UK climates. Loads imposed by occupants and items they store vary as people congregate at functions, etc. This has two important
consequences for timber structures. Firstly, it is highly unlikely, or in many cases virtually impossible, for each action to be at its most severe at the same time. This, together with the partial factor of $\gamma_Q = 1.5$ commonly applied to variable actions under Eurocode design to account for the uncertainty over their maximum value, means that designing a building to withstand the full sum of all actions at once would result in gross over-design. Secondly, because timber is strongly affected by duration of loading, it is important to be able to design for the length or proportion of the time that particular actions apply. To achieve this, actions are combined in various combinations, and high loads over long periods may be more severe than the highest loads over a very short duration. Procedures for combining loads are set out in EN 1990.

Psi ($\psi$) factors are defined to determine the proportion of each variable action to be included in particular circumstances:

$\psi_0Q_k$ – combination value of a variable action

$\psi_1Q_k$ – frequent value of a variable action

$\psi_2Q_k$ – quasi-permanent value of a variable action.

Recommended values of $\psi$ factors for buildings are given in EN 1990 Table A1.1, although these values may be set by a National Annex.

The expressions in EN 1990 include a term for prestressing, which is not usually relevant for timber structures. Ignoring this, for fundamental combinations for verifying ultimate limit states for equilibrium, structural and geotechnical design situations, the principle is to combine the permanent actions multiplied by their partial factor, a leading variable action multiplied by its partial factor, and the combination value ($\psi_0Q_k$) of each of the other variable actions multiplied by their partial factors. In principle, this should be verified with each variable action in turn set as the leading variable action, but in practice it is often obvious that particular combinations will be more severe. The values for partial factors are selected according to whether the loads are favourable or unfavourable to the design situation. For timber-based materials, which are affected by duration of load, it will also be necessary to determine the design loads for longer durations; for any load duration, only the relevant actions that apply for that and longer durations are combined. In principle it is necessary to verify that the structure can withstand the design load at each duration, as resistance will be greater at the shorter durations. In practice, it is often obvious at an early stage which is the most critical duration.

For accidental or seismic design situations, partial factors for the actions are omitted, the design value for the accidental or seismic action is included in the combination, and the quasi-permanent values of the variable actions ($\psi_2Q_k$) are used (for some accidental design situations, the frequent value ($\psi_1Q_k$) of the leading variable action might be appropriate).

There are three methods of combining actions for verifying serviceability limit states. Partial factors for actions are taken as 1.0 for all serviceability limit states. Irreversible limit states are those where function would be impaired or, for instance, damage might occur to partitions as a result of floor deflection. To
verify irreversible limit states, the characteristic combination of actions should be used. Permanent actions are combined with the leading variable action, and the sum of the combination values of the other variable actions ($\psi_{0,i}Q_{k,i}$).

Reversible limit states are those, such as deflections, which do not have permanent consequences when actions and deflections have returned to within the design limits. To verify reversible limit states, the frequent combination of actions should be used. Permanent actions are combined with the frequent value of the leading variable action ($\psi_{1,1}Q_{k,1}$), and the sum of the quasi-permanent values of the other variable actions ($\psi_{2,i}Q_{k,i}$). Long-term effects such as creep, and the effect of deflections on the appearance of the building, are verified against the quasi-permanent combination of actions. To verify quasi-permanent serviceability limit states, permanent actions are combined with the quasi-permanent values of all variable actions ($\psi_{2,i}Q_{k,i}$).

1.4.3.2 Combining loads under BS 5268-2

Under BS 5268-2, the approach to combining loads is more simplistic than in the Eurocodes. The direct value of potential loads is used, without any partial factor to allow for uncertainty in their possible value, and loads are combined by direct arithmetical summation of the loads that are relevant over any particular duration. Examples of the loads to be considered for each duration are given in Table 17 of the Code, which sets the values of $K_3$, the modification factor for the strength of timber with load duration that should be used to modify strength properties.

One exception to this simplistic approach is Annex K, which gives an alternative, more sophisticated method for calculating and assessing deflections than that in body of the standard. Annex K uses the same process as EN 1990, with the characteristic combination of loads, and calculation of creep using quasi-permanent values of the loads. A table of values for $\psi$ factors is printed in the Code. Another provision of BS 5268-2 is that, when considering the effects of accidental damage, the designer should normally multiply the values recommended for long-term permissible stresses by a factor of two. Although this is not related directly to combining loads, it has the effect of making designing to cope with accidental situations much less onerous, as does the omission of partial factors and the use of quasi-permanent or frequent values of variable actions when combining actions for accidental design situations under EN 1990.

It can be expected that, provided the magnitude of the $\psi$ factors used is appropriate, the more sophisticated procedure used for combining loads under the Eurocodes should be able to achieve more consistent levels of structural safety and reliability than the approach used in BS 5268-2. Despite this, BS 5268-2 has a good record of delivering structurally sound buildings.

1.5 Other major differences

- Notations in EN 1995-1-1 are different to those commonly used in the UK and in BS 5268. This is one of the major differences and, if care is not taken, drastic failures of structures can take place due to design using wrong notations (ie major and minor axes, etc.) in the analysis
- EN 1995-1-1 is not easy for engineers and designers to understand, while BS 5268 is more descriptive.

- EN 1995-1-1 assumes a level of technical knowledge and competence

- EN 1995-1-1 does not cover timber design to the same extent as BS 5268

- Timber frame design part of EN 1995-1-1 does not include the contribution of plasterboard, brick cladding, effects of openings, etc in the design

- Effective length definition in compression members is not included in EN 1995-1-1

- Durations of loads in EN 1995-1-1 are different to those in BS 5268

- Safety factors in EN 1995-1-1 are different to those in BS 5268: Part 2

- The information in EN 1995-1-1 is basic and is in need of Non-Contradictory Complementary Information

- Knowledge of other Eurocodes is needed for the use of EN 1995-1-1 which creates many problems for the designers both in time consumption and enormous cost

- Certain parts of EN 1995-1-1 will definitely require IT capability on the part of the designer

- EN 1995-1-1 does not include Design by Testing while BS 5268 does. The Design by Testing of EC0 is not appropriate for timber structures or timber components

- EN 1995-1-1 includes serviceability design in more detail than BS 5268: Part 2

- EN 1995-1-1 uses different limits for deflection for different parts of the structure, while BS 5268 uses either 0.003 L or 14 mm overall, except for trussed rafters

- Vibration design is included in EN 1995-1-1, while BS 5268 considers the deflection limitation deals with the vibration automatically

- EN 1995-1-1 does not deal with the design of panel products extensively by itself and refers to other supporting standards, while BS 5268 does

- Design using adhesives and adhesive information are generally inadequate in both standards

- Appendix A lists all the comparative British Standards with CEN standards and their status at the time of writing this report.
1.6 List of comparative British Standards with CEN Standards and their status

**BS AND EN STANDARDS FOR TIMBER STRUCTURES**

**LOADING CODES**

<table>
<thead>
<tr>
<th>BS Standard</th>
<th>BS EN Standard</th>
<th>Comments</th>
</tr>
</thead>
</table>

ISO 2631-2: 2003 Mechanical vibration and shock – Evaluation of human exposure to whole-body vibration – Part 2: Vibration in buildings (1 Hz to 80 Hz)
### Timber Structural Design – Specific elements

| BS 5268-3: 1998 Structural use of timber. Code of practice for trussed rafter roofs. | The BS is being revised with the intention of offering it for publication as a European standard. The date is unknown. | Draft EN 1995-1-1 has structural design of trusses and refers to EN 14250 (currently EN 14250: 2001 Timber structures. Product requirements for prefabricated trusses using punched metal plate fasteners) but lacks the general design guidance of the BS. |
| BS 5268-6-1: 1996 Structural use of timber. Code of practice for timber frame walls. Dwellings not exceeding four storeys | EN 1995-1-1 has two methods (methods A & B), of which the UK recommends method B only. | Draft EN 1995-1-1 has structural design of wall diaphragms, and refers to EN 594: 1996 ‘Timber structures. Test methods. Racking strength and stiffness of timber frame wall panels’. However, no guidance is given for converting the test values to design values. |
| BS 5268-7.1: 1989 Structural use of timber. Recommendations for the calculation basis for span tables. Domestic floor joists. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.2: 1989 Structural use of timber. Recommendations for the calculation basis for span tables. Joists for flat roofs. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.3: 1989 Structural use of timber. Recommendations for the calculation basis for span tables. Ceiling joists. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.4: 1989 Structural use of timber. Ceiling binders. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.5: 1990 Structural use of timber. Recommendations for the calculation basis for span tables. Domestic rafters. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.6: 1990 Structural use of timber. Recommendations for the calculation basis for span tables. Purlins supporting rafters. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |
| BS 5268-7.7: 1990 Structural use of timber. Recommendations for the calculation basis for span tables. Purlins supporting sheeting or decking. | EN 1995-1-1 can be used to come up with design formulae. | No equivalent European standard available. |

**TIMBER STANDARDS**

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 5291: 1984 Specification for manufacture of finger joints of structural softwood.</td>
<td>BS EN 385: 2001 Finger jointed structural timber. Performance requirements and minimum production requirements.</td>
<td>BS is withdrawn for BS EN.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>BS 5756: 1997 Specification for visual strength grading of hardwood.</td>
<td></td>
<td>Remains as UK grading standard accepted as a method of meeting EN grading standards (see below).</td>
</tr>
<tr>
<td>EN 14081-2: 2000 Timber structures. Strength graded structural timber with rectangular cross section. Part 2: Machine grading. Additional requirements for initial type testing.</td>
<td></td>
<td>Not yet a full EN standard. It is intended that the parts of EN 14081 will replace EN 518 and EN 519. However, EN 14081-2 has just been rejected and the parts of EN 14081 are being considered as a package.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>BS 373: 1957 Methods of testing small clear specimens of timber.</td>
<td></td>
<td>No equivalent European standard available.</td>
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<tr>
<td>BS 5820: 1979 Methods of testing for determination of certain physical and mechanical properties of timber in structural sizes.</td>
<td>BS EN 408: 1995 Timber structures. Structural timber and glued laminated timber. Determination of some physical and mechanical properties.</td>
<td>BS withdrawn for BS EN.</td>
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<td></td>
<td>BS EN 384: 1995 Structural timber. Determination of characteristic values of mechanical properties and density.</td>
<td>Revised version issued as EN.</td>
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## Timber – General test standards

<table>
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<tr>
<th>BS standard</th>
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<th>Comments</th>
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<tbody>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
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| BS 4471: 1987  
Specification for sizes of sawn and processed softwood. | BS EN 1313-1: 1997  
Round and sawn timber. Permitted deviations and preferred sizes. Softwood sawn timber. | BS withdrawn for BS EN. |
| BS 5450: 1977  
Specification for sizes of hardwoods and methods of measurement. | BS EN 1313-2: 1999  
Round and sawn timber. Permitted deviations and preferred sizes. Hardwood sawn timber. | BS withdrawn for BS EN. |
| BS EN 336: 2003  
Structural timber. Sizes, permitted deviations. | | |
| BS EN 1309-1: 1997  
| BS EN 1310: 1997  
Round and sawn timber. Method of measurement of features. | | |
| BS EN 1311: 1997  
Round and sawn timber. Method of measurement of biological degrade. | | |
| BS EN 1312: 1997  
Round and sawn timber. Determination of the batch volume of sawn timber. | | |
| BS EN 1315-1: 1997  
Dimensional classification. Hardwood round timber. | | |
| BS EN 1315-2: 1997  
Dimensional classification. Softwood round timber. | | |
<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
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<tbody>
<tr>
<td>BS 6100-4-1: 1992</td>
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<td>No equivalent European standard available.</td>
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<td>BS 6100-4-2: 1984</td>
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<tr>
<td>BS 6100-4-4: 1992</td>
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<td>No equivalent European standard available.</td>
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<tr>
<td>BS 7359: 1991</td>
<td>BS EN 13556: 2003</td>
<td>BS withdrawn for BS EN.</td>
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<tr>
<td>Nomenclature of commercial timbers including sources of supply.</td>
<td>Round and sawn timber. Nomenclature of timbers used in Europe.</td>
<td></td>
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<tr>
<td></td>
<td>BS EN 844-1: 1995</td>
<td></td>
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<tr>
<td></td>
<td>Round and sawn timber. Terminology. General terms common to round timber and sawn timber.</td>
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<td>BS EN 844-2: 1997</td>
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<td>BS EN 844-3: 1995</td>
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<td></td>
<td>BS EN 844-4: 1997</td>
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<td></td>
<td>Round and sawn timber. Terminology. Terms relating to moisture content.</td>
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<td>BS EN 844-5: 1997</td>
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<td></td>
<td>Round and sawn timber. Terminology. Terms relating to dimensions of round timber.</td>
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<tr>
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<td>BS EN 844-6: 1997</td>
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<tr>
<td></td>
<td>Round and sawn timber. Terminology. Terms relating to dimensions of sawn timber.</td>
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<tr>
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<td>BS EN 844-7: 1997</td>
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<tr>
<td></td>
<td>Round and sawn timber. Terminology. Terms relating to anatomical structure of timber.</td>
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<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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</table>
### Glued laminated timber standards

<table>
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<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>BS EN 387: 2001 Glued laminated timber. Large finger joints. Performance requirements and minimum production requirements.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Note: Polyurethane adhesives are used for structural laminated timber in some EU countries. However, the only adhesive type currently referred to by EN 386 and EN 10480 are to EN 301 (phenolic and aminoplastic). Other adhesives are not precluded, but it is hoped to introduce polyurethane adhesives more proactively in EN 14080. Drafting of an EN standard for polyurethane adhesives is at an early stage.</td>
<td></td>
</tr>
</tbody>
</table>
## Panel product standards

Formaldehyde has been declared as carcinogenic to humans by the International Agency for Research on Cancer, part of the World Health Organisation. This will affect wood-based panels and the panel industry.

## All panel products – General standards

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
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</thead>
</table>

## All panel products – Test standards

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<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS EN 120: 1992 Wood-based panels. Determination of formaldehyde content. Extraction method called the perforator method.</td>
<td></td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 310: 1993 Wood-based panels. Determination of modulus of elasticity in bending and of bending strength.</td>
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<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 311: 2002 Wood-based panels. Surface soundness. Test method.</td>
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<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS standard</td>
<td>BS EN standard</td>
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<tr>
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<tr>
<td>BS EN 318: 2002</td>
<td>Wood-based panels. Determination of dimensional changes associated with changes in relative humidity.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 321: 2002</td>
<td>Wood-based panels. Determination of moisture resistance under cyclic test conditions.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 322: 1993</td>
<td>Wood-based panels. Determination of moisture content.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 323: 1993</td>
<td>Wood-based panels. Determination of density.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 324-1: 1993</td>
<td>Wood-based panels. Determination of dimensions of boards. Determination of thickness, width and length.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 324-2: 1993</td>
<td>Wood-based panels. Determination of dimensions of boards. Determination of squarness and edge straightness.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS EN 325: 1993</td>
<td>Wood-based panels. Determination of dimensions of test pieces.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 326-1: 1994</td>
<td>Wood-based panels. Sampling, cutting and inspection. Sampling and cutting of test pieces and expression of test results.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 326-2: 2000</td>
<td>Wood-based panels. Sampling, cutting and inspection. Quality control in the factory.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>DD ENV 717-1: 1999</td>
<td>Wood-based panels. Determination of formaldehyde release. Formaldehyde emission by the chamber method.</td>
<td>DD ENV rather than full EN standard; revised version issued as EN.</td>
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<tr>
<td>BS EN 717-2: 1995</td>
<td>Wood-based panels. Determination of formaldehyde release. Formaldehyde release by the gas analysis method.</td>
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<td>BS EN 717-3: 1996</td>
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<td>BS EN 1058: 1996</td>
<td>Wood-based panels. Determination of characteristic values of mechanical properties and density.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS EN 13446: 2002</td>
<td>Wood-based panels. Determination of withdrawal capacity of fasteners.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS EN 13879: 2002</td>
<td>Wood-based panels. Determination of edgewise bending properties.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<td>BS 1088-1: 2003</td>
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<td>There are no plans for EN standards for marine plywood.</td>
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<tr>
<td>Marine plywood. Requirements.</td>
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<td>BS 1088-2: 2003</td>
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<td>Marine plywood. Determination of bonding quality using the knife test.</td>
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<td>BS 4512: 1969</td>
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<td>Withdrawn as no longer used. Replaced by various ENs.</td>
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<tr>
<td>Methods of testing for clear plywood.</td>
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<td>BS 6566-1: 1985</td>
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<td>Withdrawn; no direct replacement.</td>
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<tr>
<td>Plywood. Specification for construction of panels and characteristics of plies including marking.</td>
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<td>BS 6566-2: 1985</td>
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<td>BS 6566-3: 1985</td>
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<td>Withdrawn; no direct replacement.</td>
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<tr>
<td>Plywood. Specification for acceptance levels for post-manufacture batch testing including sampling.</td>
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<td>BS 6566-4: 1985</td>
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<td>Plywood. Specification for tolerances on the dimensions of plywood panels.</td>
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<td>BS 6566-5: 1985</td>
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<td>Plywood. Specification for moisture content.</td>
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<td>BS 6566-6: 1985</td>
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<tr>
<td>Plywood. Specification for limits of defects for the classification of plywood by appearance.</td>
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<td>BS 6566-8: 1985</td>
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<td>BS EN 313-1: 1996</td>
<td></td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 313-2: 2000</td>
<td></td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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<tr>
<td>BS EN 314-1: 1993</td>
<td></td>
<td>Revised version issued as EN.</td>
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</table>
### Panel products – Plywood standards (continued)

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS EN 314-2: 1993</td>
<td>Plywood. Bonding quality. Requirements.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 315: 2000</td>
<td>Plywood. Tolerances for dimensions.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 635-1: 1995</td>
<td>Plywood. Classification by surface appearance. General.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 635-2: 1995</td>
<td>Plywood. Classification by surface appearance. Hardwood.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 635-3: 1995</td>
<td>Plywood. Classification by surface appearance. Softwood.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>DD ENV 635-4: 1997</td>
<td>Plywood. Classification by surface appearance. Plywood. Classification by surface appearance. Parameters of ability finishing. Guidelines.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 635-5: 1999</td>
<td>Plywood. Classification by surface appearance. Methods for measuring and expressing characteristics and defects.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 636: 2003</td>
<td>Plywood. Specifications.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 1072: 1995</td>
<td>Plywood. Description of bending properties for structural plywood.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 1084: 1995</td>
<td>Plywood. Formaldehyde release classes determined by the gas analysis method.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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</table>
## Panel products – Standards for both particleboards and fibreboards

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<thead>
<tr>
<th>BS standard</th>
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<tr>
<td>BS EN 317: 1993</td>
<td>Specification for fibre building boards.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 319: 1993</td>
<td>Particleboards and fibreboards. Determination of swelling in thickness after immersion in water.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 12369-1: 2001</td>
<td>Wood-based panels. Characteristic values for structural design. OSB, particleboards and fireboards.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
</tr>
<tr>
<td>BS EN 317: 1993</td>
<td>Particleboards and fibreboards. Determination of tensile strength perpendicular to the plane of the board.</td>
<td>All previous BS standards are either withdrawn or updated to BS ENs.</td>
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## Panel products – Fibreboard standards

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<th>BS standard</th>
<th>BS EN standard</th>
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<tbody>
<tr>
<td>BS 1142: 1989</td>
<td>Specification for fibre building boards.</td>
<td>Withdrawn; replaced by many BS ENs.</td>
</tr>
<tr>
<td>BS EN 316: 1999</td>
<td>Wood fibreboards. Definition, classification and symbols.</td>
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<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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Panel products – Fibreboard standards (continued)

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<th>Comments</th>
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Panel products – Standards for all particleboards

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<th>BS EN standard</th>
<th>Comments</th>
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Panel products – Oriented strand board standards

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<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
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<tbody>
<tr>
<td>BS EN 300: 1997</td>
<td>BS EN 622</td>
<td>Oriented strand boards (OSB). Definitions, classification and specifications.</td>
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</table>
### Panel products – Cement-bonded particleboard standards

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### Panel products – Flooring standards

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<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 12869-1: 1997 Wood-based panels. Structural floor decking on joists. Part 1. Performance specifications.</td>
<td>There does not yet seem to be a logical reference chain from Draft EN 1995-1-1 to this EN, although it appears important for structural design.</td>
<td></td>
</tr>
<tr>
<td>EN 12869-2: 1997 Wood-based panels. Structural floor decking on joists. Part 2. Performance requirements.</td>
<td>There does not yet seem to be a logical reference chain from Draft EN 1995-1-1 to this EN, although it appears important for structural design.</td>
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### Panel products – Wall sheathing standards

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<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
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</thead>
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<tr>
<td>EN 12870-1: 1997</td>
<td>Wood-based panels. Structural wall sheathing on studs. Part 1. Performance specifications.</td>
<td>There does not yet seem to be a logical reference chain from Draft EN 1995-1-1 to this EN, although it appears important for structural design.</td>
</tr>
<tr>
<td>EN 12870-2: 1997</td>
<td>Wood-based panels. Structural wall sheathing on studs. Part 2. Performance requirements.</td>
<td>There does not yet seem to be a logical reference chain from Draft EN 1995-1-1 to this EN, although it appears important for structural design.</td>
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### Standards for Timber-Based Assemblies

#### Timber-based assemblies – Codes

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<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 8103-1: 1995 Structural design of low-rise buildings. Code of practice for stability, site investigation, foundations and ground floor slabs for housing.</td>
<td></td>
<td>Despite its title, this code includes useful design information, for instance on structural connections between elements to resist wind uplift, and hence is referenced by BS 5268-3. However, BS 8103-1: 1995 is already outdated due to the publication of BS 6399-2: 1996 ‘Loading for buildings. Code of practice for wind loads’. There is no reference to equivalent information through EN codes.</td>
</tr>
<tr>
<td>BS 8103-3: 1996 Structural design of low-rise buildings. Code of practice for timber floors and roofs for housing.</td>
<td></td>
<td>This code includes useful design information, although much of it is equivalent to that in Building Regulations tables. However, this code is in need of revision and update. There is no reference to equivalent information through EN codes.</td>
</tr>
<tr>
<td>BS 8201: 1987 Code of practice for flooring of timber, timber products and wood-based panel products.</td>
<td></td>
<td>This code includes useful design information. There is no reference to equivalent information through EN codes.</td>
</tr>
</tbody>
</table>

#### Timber-based assemblies – Specification standards

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 585-1: 1989 Wood stairs. Specification for stairs with closed risers for domestic use, including straight and winder flights and quarter or half landings.</td>
<td></td>
<td>Declared obsolescent.</td>
</tr>
</tbody>
</table>
### Timber-based assemblies – Specification standards (continued)

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
<th>Reference Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 14732-1: 2003</td>
<td>Timber structures. Prefabricated wall, floor and roof elements. Part 1. Product requirements.</td>
<td>There does not yet seem to be a logical reference chain from EN 1995-1-1 to this standard. This standard is still being circulated by CEN. There is an overlap with ETAG 007 ‘Timber frame building kits’ and with draft ETAG ‘Prefabricated wood-based loadbearing stressed skin panels’ (ref NTC 14/03/41).</td>
</tr>
<tr>
<td>EN 14732-2: 2003</td>
<td>Timber structures. Prefabricated wall, floor and roof elements. Part 2. Performance requirements and minimum production requirements.</td>
<td>There does not yet seem to be a logical reference chain from EN 1995-1-1 to this standard. This standard is still being circulated by CEN. There is an overlap with ETAG 007 ‘Timber frame building kits’ and with draft ETAG ‘Prefabricated wood-based loadbearing stressed skin panels’ (ref NTC 14/03/41).</td>
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</table>

### Timber-based assemblies – Test standards

<table>
<thead>
<tr>
<th>Standard</th>
<th>Description</th>
<th>Reference Notes</th>
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</thead>
<tbody>
<tr>
<td>BS EN 380: 1993</td>
<td>Timber structures. Test methods. General principles for static load testing.</td>
<td>There does not yet seem to be a logical reference chain from EN 1995-1-1 to this standard, although it appears important for generating structural design data.</td>
</tr>
<tr>
<td>BS EN 595: 1995</td>
<td>Timber structures. Test methods. Test of trusses for the determination of strength and deformation behaviour.</td>
<td>There does not yet seem to be a logical reference chain from EN 1995-1-1 to this standard, although it appears important for generating structural design data.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
</tr>
<tr>
<td>-------------</td>
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</tr>
<tr>
<td>BS 1202-1: 2002 Specification for nails. Steel nails.</td>
<td>BS EN 10230-1: 2000 Steel wire nails. Loose nails for general applications.</td>
<td>BS 1202-1 revised to refer to BS EN 10230-1 where appropriate.</td>
</tr>
<tr>
<td>BS 1210: 1963 Specification for wood screws.</td>
<td></td>
<td>BS 1210 has been declared obsolete; no European standard for wood screws has been identified.</td>
</tr>
<tr>
<td>BS 4320: 1968 Specification for metal washers for general engineering purposes. Metric series.</td>
<td>BS EN ISO 898-1: 1999 Mechanical properties of fasteners made of carbon steel and alloy steel. Bolts, screws and studs.</td>
<td>BS 4320 has been declared obsolete; BS EN ISO standards for washers are now available but are not considered to be direct replacements.</td>
</tr>
</tbody>
</table>

**STANDARDS FOR JOINTS IN TIMBER-BASED STRUCTURES**

<table>
<thead>
<tr>
<th>Joints in timber-based structures – Specification standards</th>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>BS standard</strong></td>
<td><strong>BS EN standard</strong></td>
<td><strong>Comments</strong></td>
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<tr>
<td>BS 1202-1: 2002 Specification for nails. Steel nails.</td>
<td>BS EN 10230-1: 2000 Steel wire nails. Loose nails for general applications.</td>
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<tr>
<td>BS 4320: 1968 Specification for metal washers for general engineering purposes. Metric series.</td>
<td>BS EN ISO 898-1: 1999 Mechanical properties of fasteners made of carbon steel and alloy steel. Bolts, screws and studs.</td>
<td>BS 4320 has been declared obsolete; BS EN ISO standards for washers are now available but are not considered to be direct replacements.</td>
<td></td>
</tr>
</tbody>
</table>
## Joints in timber-based structures – Test standards

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 6948: 1989</td>
<td>BS EN 1075: 2000</td>
<td>BS remains, only partially replaced by BS ENs.</td>
</tr>
</tbody>
</table>
# Adhesives Standards for Timber-Based Structures

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 1203: 2001</td>
<td>Hot-setting phenolic and aminoplastic wood adhesives. Classification and test method.</td>
<td></td>
</tr>
<tr>
<td>BS 1204: 1993</td>
<td>Specification for type MR phenolic and aminoplastic synthetic resin adhesives for wood.</td>
<td>Withdrawn; said to be replaced by BS EN 12765: 2001 but the ‘replacement’ standard is a classification for non-structural applications.</td>
</tr>
<tr>
<td>BS EN 301: 1992</td>
<td>Adhesives, phenolic and aminoplastic, for load-bearing timber structures: classification and performance requirements.</td>
<td></td>
</tr>
</tbody>
</table>

Note: Polyurethane adhesives are used for structural timber joints, including glulam and finger joints, in some EU countries. These adhesives allow successful green gluing. There is currently no EN standard for polyurethane adhesives, but development of such a standard has been included in the adhesives committee programme and is at an early stage.
DURABILITY STANDARDS FOR TIMBER

Durability – Standards for specifying durability and treatment of timber

Note, the basis for achieving durability in timber through CEN standards is substantially different to that used historically with British Standards. Whilst British Standards had an advisory style (recommending formulations and methods of treatment), the CEN approach is to define hazard classes and indicate timber species or loadings of preservative that will meet those classes. As a result of this different approach, many of the current British Standards in this area will remain in place, offering guidance for the foreseeable future.
<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 5268-5: 1989</td>
<td></td>
<td>This standard is also listed amongst the other parts of BS 5268 under loading codes. No equivalent overall BS EN for durability of timber in structures. It has been declared obsolete.</td>
</tr>
<tr>
<td>BS 8417: 2003 Preservation of timber. Recommendations</td>
<td></td>
<td>Useful advisory standard giving guidance for preservative treatment of timber through the EN standards. As a BS it cannot be referenced within the body of EN 1995-1-1 when it is published, but could be referenced in a National Annex or national application document. In the future it may be put forward as the basis of a European standard.</td>
</tr>
<tr>
<td>BS EN 335-1: 1992 Hazard classes of wood and wood-based products against biological attack. Classification of hazard classes.</td>
<td></td>
<td>Revision under way.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>-----------------------------</td>
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</tr>
<tr>
<td>BS EN 599-1: 1997</td>
<td>Durability of wood and wood-based products. Performance of preservatives as determined by biological tests. Specification according to hazard class.</td>
<td>Revision under way.</td>
</tr>
</tbody>
</table>
Durability – Standards specifying wood preservative formulations

Note, creosote may now only be used in poles and railway sleepers, whilst from July 2004, limitations on the use of arsenic are expected to preclude the use of CCA preservative in all domestic buildings and various other situations.

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 144: 1997 Specification for coal tar creosote for wood preservation.</td>
<td></td>
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<tr>
<td>BS 4072: 1999 Copper/chromium/arsenic preparations for wood preservation.</td>
<td></td>
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</tr>
</tbody>
</table>
## Durability – Test standards for preservatives and preservative treatments

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS 838: 1961</td>
<td>BS EN standard</td>
<td>Methods of test for toxicity of wood preservatives to fungi. Withdrawn; no direct replacement.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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</tr>
<tr>
<td>BS 7282: 1990</td>
<td></td>
<td>Field test method for determining the relative protective effectiveness of a wood preservative in ground contact.</td>
</tr>
<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>------------------</td>
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</tr>
<tr>
<td>BS EN 84: 1997</td>
<td>Wood preservatives. Accelerated ageing of treated wood prior to biological testing. Leaching procedure.</td>
<td></td>
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<tr>
<td>BS EN 330: 1993</td>
<td>Wood preservatives. Field test method for determining the relative protective effectiveness of a wood preservative for use under a coating and exposed out-of-ground contact. L-joint method.</td>
<td></td>
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<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td>BS 1982-0: 1990 Fungal resistance of panel</td>
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<tr>
<td>products made of or containing materials of</td>
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<tr>
<td>organic origin. Guide to methods for</td>
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<tr>
<td>determination.</td>
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<td>organic origin. Method for determination of</td>
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<tr>
<td>resistance to wood-rotting Basidiomycetes.</td>
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<td>BS 1982-2: 1990 Fungal resistance of panel</td>
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<td>resistance to cellulose-decomposing microfungi.</td>
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<tr>
<td>resistance to mould or mildew.</td>
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<td>Determination of the preventive action against</td>
<td>of the protective effectiveness against Lyctus</td>
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</tr>
<tr>
<td>Lyctus brunneus (Stephens) (laboratory method).</td>
<td>brunneus (Stephens). Application by surface</td>
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<tr>
<td>Determination of the toxic values against</td>
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<td></td>
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<tr>
<td>Anobium punctatum (De Geer) by larval</td>
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<tr>
<td>transfer (laboratory method).</td>
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<tr>
<td>Determination of eradicant action against</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hylotrupes bajulus (Linnaeus) larvae</td>
<td></td>
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<tr>
<td>(laboratory method).</td>
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<tr>
<td>action against recently hatched</td>
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<tr>
<td>larvae of Hylotrupes bajulus</td>
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<td>against larvae of Anobium punctatum</td>
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<td>Determination of the protective</td>
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<tr>
<td>(laboratory method).</td>
<td>Application by impregnation (laboratory</td>
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<td>Wood preservatives.</td>
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<td>Determination of toxic values</td>
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<tr>
<td>against Reticulitermes santonenis</td>
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<td>de Feytaud (Laboratory method).</td>
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<td>BS 6240: 1990, EN 118: 1990</td>
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<td>Wood preservatives.</td>
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<td>Determination of preventive action</td>
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<tr>
<td>against Reticulitermes santonensis</td>
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<td>de Feytaud (laboratory method).</td>
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<td>Wood preservatives.</td>
<td>Wood preservatives.</td>
<td>as BS EN</td>
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<td>Laboratory method for determining</td>
<td>Determination of the protective</td>
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<td>the effectiveness of a preservative</td>
<td>effectiveness against blue stain in</td>
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<td>treatment against blue stain in</td>
<td>service. Brushing procedure.</td>
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<td>service. Brushing procedure.</td>
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<tr>
<td>BS standard</td>
<td>BS EN standard</td>
<td>Comments</td>
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<tr>
<td></td>
<td>BS EN 49-1: 1992 Wood preservatives. Determination of the protective effectiveness against Anobium punctatum (De Geer) by egg-laying and larval survival. Application by surface treatment (laboratory method).</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS EN 113: 1997 Wood preservatives. Test method for determining the protective effectiveness against wood destroying basidiomycetes. Determination of the toxic values.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS EN 273: 1992 Wood preservatives. Determination of the curative action against Lyctus brunneus (Stephens) (Laboratory method).</td>
<td></td>
</tr>
<tr>
<td></td>
<td>BS EN 370: 1993 Wood preservatives. Determination of eradicant efficacy in preventing emergence of Anobium punctatum (De Geer).</td>
<td></td>
</tr>
</tbody>
</table>
### Durability – Test methods for susceptibility to degrading organisms (continued)

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
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</thead>
</table>

### Durability – Terminology standards

<table>
<thead>
<tr>
<th>BS standard</th>
<th>BS EN standard</th>
<th>Comments</th>
</tr>
</thead>
</table>
CHAPTER 2

Companion to EN 1995-1-1

2.1 Introduction

This chapter is mainly aimed at designers. However, the contents can be used for education, training and CPD courses. It deals with a commentary of different sections of EN 1995-1-1 page by page and explains and highlights important issues that one should consider in design. For clarity and ease of reference, each page of EN 1995-1-1 is considered separately so that readers can identify important issues within each page as they use it.

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

2.2 Page 1 of BS EN 1995-1-1

The only thing which is missing and you need to be aware of on this page is that ‘you cannot mix your design using this code and our national standards BS 5268 during the co-existence period. You need to base your design on either of these codes completely separately’.

This page is self-explanatory and gives general information regarding the title, publishers and the availability of the standard in three different languages which are English, German and French. It also gives the statutory requirements with which UK has to comply. The title of EN 1995-1-1 is presented as BS EN 1995-1-1 with BS implying that it is the UK Standard which is not any different to other countries’ versions.

Please note that the document can be used for the following countries (and any future members of the CEN) in conjunction with their National Annex. The following CEN members are obliged to implement this code:

Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungry, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

2.3 Pages 2, 3, 4 & 5 of BS EN 1995-1-1

These are contents pages only.
2.4 Page 6 of BS EN 1995-1-1

This page is blank.

2.5 Page 7 of BS EN 1995-1-1

Foreword
You do not need this part. The only thing you need to remember is that we are obliged to implement the standard by 2010 and withdraw our national standards then. Therefore, the sooner you use and familiar yourself with this code the better it would be.

Background of the Eurocode programme
Again you do not need this part except that it is useful to know the reference numbers of other Eurocodes in case you need them.

[Note: More explanation and guidance are given in Chapter 1 of this document regarding other Eurocodes and their interaction with this Eurocode].

2.6 Page 8 of BS EN 1995-1-1

Status and field of application of Eurocodes
Nothing is different in this part from that which we currently use and have to comply with regarding the means of proof of compliance of building and civil engineering works, the basis for specifying contracts for construction works and related engineering services.

National standards implementing Eurocodes
This part is just for general information.

2.7 Page 9 of BS EN 1995-1-1

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products
This part points out that you need to use the appropriate EN standards as supporting documents for this code. You need to be careful when using imported products with the CE marking because you need to know that all the information accompanying the CE marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account. So, if you are going to use an imported product with CE marking, ask for the above information because the National Determined Parameters used may be different to those for the UK.

Additional information specific to EN 1995-1-1
All you need to know is that this code is based on the limit state concept, while our BS 5268 is based on the permissible stress design concept. These are two completely different concepts which cannot be combined and used in design.
You must use this code in conjunction with EN 1990 (Basis of design) and the relevant parts of EN 1991 (actions). Please note ‘Actions’ means ‘Loads’.

**National Annex for EN 1995-1-1**

You do not need this part as this document includes all the Nationally Determined Parameters given in our National Annex.

### 2.8 ‘Section 1: General of BS EN 1995-1-1’ (pages 10 – 18)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

### 1.1 Scope

#### 1.1.1 SCOPE OF EN 1995

This clause identifies the requirements. There are six requirements given here with two of them being ‘Principle’ requirements and the rest ‘Application Rules’.

**‘Principle’ requirements** are annotated by suffix ‘P’. This means that you MUST comply with the requirement. They are general statements and definitions for which there is no alternative, as well as requirements and analytical models for which no alternative is permitted unless specifically stated.

**‘Application Rules’** are generally recognised rules which comply with and satisfy the ‘Principle requirements’.

**(1)P:** All this says is that the code applies to the design of structures in timber or wood-based panel products. It says that if you use this code, you would comply with the principles and requirements for the safety and serviceability of structures and the basis of design and verification given in EN 1990: 2002.

**(2)P:** This highlights that the code is only for mechanical resistance, serviceability, durability and fire resistance. It does not cover other issues such as thermal and acoustics which our Approved Documents should be used for. The requirements and verification should be provided by a Technical Approval Certification carried out by a Notified Body (ie Certification organisations which have been approved by UKAS).

**(3), (4) & (5):** These are ‘Application Rules’ which say that EN 1990 (basis of design), EN 1991 (loads) and EN 1997 (geotechnical and foundation design) should be used with this code. You should consider using EN 1998 if you are also designing for earthquake zones.
It also says that EN 1995 has two major parts (EN 1995-1 for structures and EN 1995-2 for timber bridges). EN 1995-1 is divided into further two parts EN 1995-1-1 for timber structures and EN 1995-1-2 for fire).

### 1.1.2 SCOPE OF EN 1995-1-1

This clause is like a contents page for the code highlighting the subjects covered by each section of the code.

The only thing that is ‘Principle’ requirement and you must remember is that the code does not cover the design of structures in an environment where the temperature exceeds 60°C which is not usual in UK.

### 1.2 Normative references

These are important references which you will need depending on what structure your are designing. The only thing to remember is that the references which you are using should be the updated versions (including amendments or revisions).

### 1.3 Assumptions

The ‘Principle’ requirement is that you must use the assumptions given in EN 1990 (basis of design code)

### 1.4 Distinction between Principles and Application Rules

‘Principle’ requirements are annotated by suffix ‘P’. This means that you MUST comply with the requirement. They are general statements and definitions for which there is no alternative as well as requirements and analytical models for which no alternative is permitted unless specifically stated.

‘Application Rules’ are generally recognised rules which comply with and satisfy the ‘Principle requirements’.

### 1.5 Terms and definitions

Sub-clauses 1.5.1 to 1.5.2.10: You need to familiarise yourself with the terms and definitions given in these sub-clauses and those listed in EN 1990 clause 1.5.

The three major ones which need to be explained are ‘Characteristic values’, ‘Equilibrium moisture content’ and ‘Stiffness property’.

‘Characteristic values’ is the lower 5 percentile value of a population (for example, a batch of timber pieces or products). This means that 5% of the
timber pieces or products in the batch have properties that are lower than the value of the 5 percentile.

‘Equilibrium moisture content’ is the moisture content of timber or product which reaches a value that does not fluctuate.

‘Stiffness properties’ are the modulus of elasticity, shear modulus and slip modulus. Please note these are not the product of moduli to the second moment of area (ie EI, etc.).

1.6 Symbols used in EN 1995-1-1

These symbols and notations are very important as we in the UK have been used to using different symbols and notations in the past. Many major errors can occur if you mix these symbols with those that we have previously used.

**Major failures can occur if we forget two major and crucial symbols and notations:**

(a) **Member axes:** We have been used to x-x, y-y and z-z being the major, minor and out of plane axes respectively. These have been changed into y-y, z-z and x-x respectively in this code:

![X-Z being out of plane axis](image1)

![Z-X being out of plane axis](image2)

In BS 5268

(b) **The use of ‘,’ and ‘.’ in numbers:** We have been used to using ‘.’ as decimal places which has been changed to ‘,’ and vice-versa in EN 1995-1-1. For example:

**In BS 5268**

1,000.00 means one thousand

**In EN 1995-1-1**

1.000,00 means one thousand

(c) **Definition of element and component:** There is great confusion over the definition of elements and components at European level. The UK’s definition of elements and components are reversed in the Eurocode and the rest of Europe. For clarity the following are definitions which we must adopt:

‘Element’ is a structure or part of the structure which consists of members or components. For example, a truss rafter is an ‘Element’ because it includes members/components which are rafters, internal web, ceiling tie and punched
metal plate connectors. Another example would be a wall panel which is called ‘Element’ because it consists of elements or components such as studs and sheathing.

‘Component’ is a member or part of an element.

2.9 ‘Section 2: Basis of design’ of BS EN 1995-1-1 (pages 19 – 25)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

2.1 Requirements

2.1.1 BASIC REQUIREMENTS

You would automatically comply with the Principles and Applications Rules stipulated in this clause when using (which you must) the EN 1990 (basis of design), EN 1991 (Actions, ie loads) and others with this code.

2.1.2 RELIABILITY MANAGEMENT

You must familiar yourself with EN 1990 (basis of design) because the UK in the past has not distinguished between different levels of reliability in design which would depend on the type of structure and its use. However, EN 1990 now provides a vehicle for the designers to assess and select different levels of reliability e.g. for structural safety or for serviceability, taking into account:

- the cause and/or mode of attaining a limit state
- the possible consequences of failure in terms of risk to life, injury, potential economical losses
- public aversion to failure
- expense and procedures necessary to reduce the risk of failure
- design supervision differentiation
- inspection during execution
- quality levels of the design
- etc.
You need to know about at least Section 6, Annexes B, C and D of EN 1990. EN 1990 at the start seems to be a daunting task but as you get familiar with it, it is a very useful code which allows you to manage risks and choose how safe, economical and good quality your structure will be.

**Design working life and durability**

The ‘Applications Rules’ are given in clause 2.3 of EN 1990. Design working life is an important part of design which is the assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without repair being necessary. Design working life is useful for:

- Selection of design actions (e.g. wind, earthquake, etc.)
- Consideration of material property deterioration (e.g. creep, fatigue, etc.)
- Life-cycle costing
- Evolving maintenance strategies

Table 2.1 of EN 1990 gives examples of indicative design working lives as follows:

<table>
<thead>
<tr>
<th>Category</th>
<th>Indicative design working life (Years)</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10</td>
<td>Temporary structures</td>
</tr>
<tr>
<td>2</td>
<td>10 to 25</td>
<td>Replaceable structural parts</td>
</tr>
<tr>
<td>3</td>
<td>15 to 30</td>
<td>Agricultural and similar structures</td>
</tr>
<tr>
<td>4</td>
<td>50</td>
<td>Buildings and other common structures.</td>
</tr>
<tr>
<td>5</td>
<td>100</td>
<td>Monumental buildings, bridges and other civil engineering structures</td>
</tr>
</tbody>
</table>

Durability is another important factor to be considered in design. Design must consider the following interrelated factors:

- Design of a structure or part of it in its environment is such that it remains fit for use during the design working life, given appropriate maintenance
- Design should be in such a way that deterioration should not impair the durability and performance of the structure, having due regard to the anticipated level of maintenance
- The intended and future use of the structure
- The required performance criteria
- The expected environmental influences
- The composition, properties and performance of materials
- The choice of structural system
- The shape of members and structural detailing
- The quality of workmanship measures
- The maintenance during the intended life
- The performance of a structure with time is shown diagrammatically below:

![Durability - Performance of a structure with time](image-url)

The above diagram shows that the performance of a structure during its life reduces in time to the level that after the SLS (Serviceability Limit State) is reached there are visible signs of damage or malfunctioning (e.g., excessive deflections, cracks, etc.) and if no repairs are carried out, the performance of the structure will reduce further to the point of ULS (Ultimate Limit State), after which failure may occur.

### 2.2 Principles of limit state design

#### 2.2.1 GENERAL

The ‘Principle’ requirement is that design should consider:

- Different material properties
Different time-dependent behaviour of materials (ie creep, load-duration, etc.) [Note, this is more pronounced for timber]

Different climatic conditions (e.g. moisture variations, temperatures, relative humidity, etc.)

Different design situations (e.g. stages of construction, change of support conditions, etc.)

There are two main parts of limit state design namely ‘ultimate limit state’ and ‘serviceability limit state’.

### 2.2.2 Ultimate Limit State

The ultimate limit states shall be those that concern:

- the safety of the structure
- the safety of people
- in special circumstances the protection of the contents

The ‘Principle’ requirement is that you use the following stiffness properties:

**For elastic analysis:**

\[
E_{\text{mean}}, G_{\text{mean}} \text{, and } K_u \text{ should be used if the structure/element consists of the same material components (or with the same creep properties).}
\]

\[
E_{\text{mean,fin}}, G_{\text{mean,fin}} \text{, adjusted to the load-duration causing the largest stress, should be used if the structure/element consists of different material components (i.e. different creep properties such as a composite element) where:}
\]

- \[
E_{\text{mean,fin}} = E_{\text{mean}}/(1 + \psi^2 K_{\text{def}})
\]
- \[
G_{\text{mean,fin}} = G_{\text{mean}}/(1 + \psi^2 K_{\text{def}})
\]
- \[
K_u = 2/3 K_{\text{ser}}
\]

**For plastic analysis:**

\[
E_d, G_d \text{ design values can be used without any adjustment for duration of load.}
\]

- \[
E_d = E_{\text{mean}}/Y_M
\]
- \[
G_d = G_{\text{mean}}/Y_M
\]
Where:

\( E_{\text{mean}} \) is mean modulus of elasticity

\( E_{\text{mean,fin}} \) is final mean modulus of elasticity

\( E_d \) is design modulus of elasticity

\( G_{\text{mean}} \) is mean shear modulus

\( G_{\text{mean,fin}} \) is final mean shear modulus

\( G_d \) is design shear modulus

\( K_u \) is instantaneous slip modulus of a connection for the ultimate limit state

\( K_{\text{ser}} \) is slip modulus of a connection

\( \psi_2 \) is factor for quasi-permanent action causing the largest stress in relation to the strength (Note: if the action is a permanent action, \( \psi_2 \) should be replaced by 1)

\( k_{\text{def}} \) is a factor for the evaluation of creep deformation taking into account the relevant service class

\( Y_M \) is partial factor for material property

### 2.2.3 SERVICEABILITY LIMIT STATE

The serviceability limit states shall be those that concern:

- the functioning of the structure or structural elements under normal use.
- the comfort of people
- the appearance of the construction works

The ‘Principle’ requirement [(1)P] is:

- Deformation should not damage surfacing materials, ceilings, floors, partitions and finishes
- Deformation should not affect the functional needs and appearance requirements

Five ‘Application rules’ are listed which are important to apply by referring to the diagram below:
Instantaneous deformation \((u_{\text{inst}})\) should be calculated using \(E_{\text{mean}}\), \(G_{\text{mean}}\) and \(K_{\text{ser}}\) for combination of loads (EN 1990, clause 6.5.3(2)a).

Final deformation \((u_{\text{fin}})\) should be calculated using quasi-permanent combination of loads [EN 1990, clause 6.5.3(2)c)]. When a structure has components and elements with the same creep behaviour, \(u_{\text{fin}}\) should be:

\[
u_{\text{fin}} = u_{\text{fin},G} + u_{\text{fin},Q1} + u_{\text{fin},Qi}
\]

Where,

\[
u_{\text{fin},G} = u_{\text{inst},G} (1+k_{\text{def}}) \text{ for permanent loads G}
\]

\[
u_{\text{fin},Q1} = u_{\text{inst},Q1} (1+\psi_{2,1} k_{\text{def}}) \text{ for the main imposed load Q1}
\]

\[
u_{\text{fin},Qi} = u_{\text{inst},Qi} (\psi_{0,i} + \psi_{2,i} k_{\text{def}}) \text{ for the other imposed loads Qi (i>1)}
\]

*It is important that \(\psi_{2}\) factors are omitted from formulae 6.16a and 6.16b of EN 1990 when using the above formulae.*

\(u_{\text{inst},G}\), \(u_{\text{inst},Q1}\) and \(u_{\text{inst},Qi}\) are instantaneous deformations for actions G, Q1 and Qi.

\(\psi_{2,1}\) and \(\psi_{2,i}\) are factors for the quasi-permanent value of imposed loads.

\(\psi_{0,i}\) is the factor for the combination value of imposed loads.

\(k_{\text{def}}\) for timber and wood-based materials, is ‘deformation factor’, better known as ‘creep factor’, given in Table 3.2 of the code which is a factor for the evaluation of creep deformation taking into account the relevant service class.

\(k_{\text{def}}\) for connections, is ‘deformation factor’ better known as ‘creep factor’ given in 2.3.2.2(3) and 2.3.2.2(4) of the code which is a factor for the evaluation of creep deformation of connections taking into account the relevant service class.

\(k_{\text{def}}\) should be doubled when the connections have timber members with the same creep behaviour. \(k_{\text{def}} = 2 \sqrt{k_{\text{def},1} k_{\text{def},2}}\) when the connections have timber members with different creep behaviour where \(k_{\text{def},1}\) and \(k_{\text{def},2}\) are factors for the two members.
(4): Final deformation should be calculated using final mean values of the appropriate moduli given below \((E_{\text{mean,fin}}, G_{\text{mean,fin}}\text{ and } K_{\text{ser,fin}})\) when the structure has components or elements with different materials (ie different creep behaviour):

\[
E_{\text{mean,fin}} = E_{\text{mean}}(1+k_{\text{def}})
\]

\[
G_{\text{mean,fin}} = G_{\text{mean}}(1+k_{\text{def}})
\]

\[
K_{\text{ser,fin}} = K_{\text{ser}}(1+k_{\text{def}})
\]

(6): \(E_{\text{mean}}, G_{\text{mean}}\text{ and } K_{\text{ser}}\) should be used for vibrations

### 2.3 Basic variables

#### 2.3.1 ACTIONS AND ENVIRONMENTAL INFLUENCES

**2.3.1.1 General**

(1): This is an ‘Application rule’ which gives relevant parts of EN 1991 for different loads to be used in design.

(2)P and (3)P are ‘Principle requirements’ which say that the effect of moisture content should be considered in design. Moisture content affects the strength and stiffness of timber and wood-based materials. It also affects the loads.

**2.3.1.2 Load-duration classes**

Timber and wood-based products are materials that, if put under a load for a duration and with varying moisture content, their strength and stiffness performance reduce as time goes by. Therefore, (1)P and (2)P are two ‘Principle requirements’ which simply require you to class your loads into one of the classes given in the table below. This is so that the effect of duration of load on strength (ie \(k_{\text{mod}}\)) and stiffness (ie \(k_{\text{def}}\)) of the structure can be determined for different materials.

It should be noted that (2)P is one of the ‘Principle requirements’ in which national choice is allowed for assignment of load-duration classes. The table below includes the variations given in the UK National Annex which replaces Tables 2.1 and 2.2 of the code:
It is worth mentioning that BS 5268: Part 2’s load durations are very different to those mentioned above and must not be confused or mixed with those above. For example, the long-term category of BS 5268: part 2 is for 50 years or more while medium-term is for accumulated imposed loads of 30 days which are very different to those mentioned in the Eurocode given in the table above. **FORGET ABOUT BS 5268 LOAD-DURATIONS.**

### 2.3.1.3 Service classes

Service classes are important as they represent moisture content in timber and wood-based products which affects the strength and stiffness of the materials, thereby the structure.

It should be noted that all the ‘Principle requirements’ (1)P, (2)P, (3)P & (4)P are allowed to be changed by the national choice. Table below summarises this section and includes the variations given in the UK National Annex:

<table>
<thead>
<tr>
<th>Service class</th>
<th>Definitions</th>
<th>Typical moisture content (m.c.)</th>
<th>Type of construction</th>
</tr>
</thead>
</table>
| 1             | Moisture content (m.c.) resulting from 20°C and Relative Humidity (RH) of surrounding air only exceeding 65% for a few weeks per year. | • Timber ≤ 12%.  
• Panels ≤ 8% | Warm roofs, intermediate floors, timber frame walls (internal and party walls). |
| 2             | Moisture content (m.c.) resulting from 20°C and Relative Humidity (RH) of surrounding air only exceeding 85% for a few weeks per year. | • Timber ≤ 20%.  
• Panels ≤ 15% | Cold roofs, ground floors, timber frame walls (external walls), external uses where member is protected from direct wetting.  
It is worth noting that this service class is the usual and safe choice for the UK unless you can guarantee that Service Class 1 is applicable during the life of the structure. |
| 3             | Conditions leading to higher moisture content than 1 and 2 above. | • Timber > 20%.  
• Panels > 15% | External uses, fully exposed, outdoor structures or situations with constant high humidity and moisture content environment. |
2.3.2 MATERIALS AND PRODUCT PROPERTIES

2.3.2.1 Load-duration and moisture influences on strength
As previously said, the duration of load and moisture content affects strength and stiffness. This section relates to strength and highlights that you should use $k_{\text{mod}}$. The only important thing that needs to be remembered is that when designing a connection (ie a joint) consisting of two timber members with different materials (ie different time-dependent behaviour), the design should use:

$$k_{\text{mod}} = \sqrt{k_{\text{mod},1} \cdot k_{\text{mod},2}}$$

where $k_{\text{mod},1}$ & $k_{\text{mod},2}$ are modification factors for the two different materials in the joint.

2.3.2.2 Load-duration and moisture influences on deformations
There are three ‘Application rules’ in this section:

1. For serviceability limit state:
   
   Final deformation should be calculated using final mean values of the appropriate moduli given below ($E_{\text{mean,fin}}$, $G_{\text{mean,fin}}$ and $K_{\text{ser,fin}}$) when the structure has components or elements with different materials (ie different creep behaviour):
   
   $$E_{\text{mean,fin}} = E_{\text{mean}}/(1+k_{\text{def}})$$
   $$G_{\text{mean,fin}} = G_{\text{mean}}/(1+k_{\text{def}})$$
   $$K_{\text{ser,fin}} = K_{\text{ser}}/(1+k_{\text{def}})$$

2. For ultimate limit state:
   
   Final deformation should be calculated using final mean values of the appropriate moduli given below ($E_{\text{mean,fin}}$, $G_{\text{mean,fin}}$ and $K_{\text{ser,fin}}$) when the distribution of forces and moments are affected by the stiffness distribution:
   
   $$E_{\text{mean,fin}} = E_{\text{mean}}/(1+\psi_2 k_{\text{def}})$$
   $$G_{\text{mean,fin}} = G_{\text{mean}}/(1+\psi_2 k_{\text{def}})$$
   $$K_{\text{ser,fin}} = K_{\text{ser}}/(1+\psi_2 k_{\text{def}})$$

3. $k_{\text{def}}$ should be doubled when the connections have timber members with the same creep behaviour.

4. $k_{\text{def}} = 2 \sqrt{k_{\text{def},1} \cdot k_{\text{def},2}}$ when the connections have timber members with different creep behaviour where $k_{\text{def},1}$ and $k_{\text{def},2}$ are factors for the two members.
2.4 Verification by partial factor method

This section is the most important part of design which must be remembered.

2.4.1 DESIGN VALUE OF MATERIAL PROPERTY

It should be noted that (1)P is one of the ‘Principle requirements’ in which national choice is allowed for partial factors. The UK N.A. recommendations are already included below:

Before any mechanical properties are used in design, they must be converted into ‘design properties’ by:

For strength: \[ X_d = k_{\text{mod}} \cdot \frac{X_k}{Y_M} \]

For stiffness: \[ E_d = \frac{E_{\text{mean}}}{Y_M} \]
\[ G_d = \frac{G_{\text{mean}}}{Y_M} \]

Where

\( X_d \) is design value of characteristic strength property

\( E_d \) is design value of modulus of elasticity

\( G_d \) is design value of shear modulus

\( k_{\text{mod}} \) is modification factor taking into account the duration of load and moisture content

\( X_k \) is characteristic value of strength property

\( Y_M \) is the factor for material properties given in table below
2.4.2 DESIGN VALUES OF GEOMETRICAL DATA

All this section says is that you can use nominal dimensions.

2.4.3 DESIGN RESISTANCE

Design resistance is ‘load-carrying capacity’ which must be calculated by:

$$R_d = k_{\text{mod}} \cdot R_k/Y_M$$

Where

- $R_d$ is design resistance (ie design load-carrying capacity)
- $R_k$ is characteristic value of load-carrying capacity
- $k_{\text{mod}}$ is modification factor taking into account the duration of load and moisture content
- $Y_M$ is the factor for material properties given in table above

<table>
<thead>
<tr>
<th>Type of material</th>
<th>$Y_M$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For ‘Fundamental combinations’</td>
<td></td>
</tr>
<tr>
<td>Solid timber, untreated</td>
<td>1.3</td>
</tr>
<tr>
<td>Solid timber, preservative treated</td>
<td>1.3</td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>1.25</td>
</tr>
<tr>
<td>LVL, plywood, OSB</td>
<td>1.2</td>
</tr>
<tr>
<td>Particleboards</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, hard</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, medium</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, MDF</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, soft</td>
<td>1.3</td>
</tr>
<tr>
<td>Connections (except for punched metal plate fasteners)</td>
<td>1.3</td>
</tr>
<tr>
<td>Punched metal plate fasteners, anchorage strength</td>
<td>1.3</td>
</tr>
<tr>
<td>Punched metal plate fasteners, plate (steel) strength</td>
<td>1.15</td>
</tr>
<tr>
<td>For ‘Accidental combinations’</td>
<td></td>
</tr>
<tr>
<td>‘Accidental combinations’</td>
<td>1.0</td>
</tr>
</tbody>
</table>
2.4.4 VERIFICATION OF EQUILIBRIUM (EQU)

This section is dealt with by default in any design by a good designer making sure that the design load-carrying capacity is equal to or more than the applied design loads.

2.10 ‘Section 3: Material properties’
(pages 26 – 29)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

3.1 General

If Sections 1 and 2 of the code are understood well, then this section will mostly be acting as a specification. The most important information is given, such as $k_{\text{mod}}$ and $k_{\text{def}}$ values in addition to depth-, width- and length-factors which are explained below.

3.1.1 STRENGTH AND STIFFNESS PARAMETERS

You automatically comply with (1)P provided that the information given in this section is used as ‘design specification’.

3.1.2 STRESS-STRAIN RELATIONSHIP

All this section is saying that all the analyses are based on elastic (i.e. linear relation of stress and strain) theory. However, it allows the elasto-plastic analysis be used for compression members.
3.1.3 STRENGTH MODIFICATION FACTORS ($K_{\text{mod}}$) FOR SERVICE CLASSES AND LOAD-DURATION CLASSES

There is nothing here that you have not learnt so far except that if a structure is loaded by loads which have different load-durations, the $k_{\text{mod}}$ factor for the shortest load-duration should be used. For example, if the structure is loaded by dead (permanent load-duration) and imposed loads (medium term load-duration), then $k_{\text{mod}}$ factor should be chosen from medium-term load-duration.

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
<th>Load-duration class</th>
<th>Permanent action</th>
<th>Long term action</th>
<th>Medium term action</th>
<th>Short term action</th>
<th>Instantaneous action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
<td>EN 14081-1</td>
<td>1</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>EN 14080</td>
<td>1</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>LVL</td>
<td>EN 14374, EN 14279</td>
<td>1</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td></td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td></td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 638</td>
<td>Part 1, Part 2, Part 3</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 2, Part 3</td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 3</td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>OSB</td>
<td>EN 300</td>
<td>EN 312</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 4, Part 5</td>
<td>2</td>
<td>0.30</td>
<td>0.40</td>
<td>0.55</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 5</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 6, Part 7</td>
<td>2</td>
<td>0.30</td>
<td>0.40</td>
<td>0.55</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Fibreboard, hard</td>
<td>EN 622-2</td>
<td>HB.LA, HB.HLA 1 or 2</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HB.HLA 1 or 2</td>
<td>2</td>
<td>0.20</td>
<td>0.30</td>
<td>0.45</td>
<td>0.60</td>
<td>0.80</td>
</tr>
<tr>
<td>Fibreboard, medium</td>
<td>EN 622-3</td>
<td>MBH.LA1 or 2</td>
<td>1</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MBH.HLS1 or 2</td>
<td>2</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
<td>1.10</td>
</tr>
<tr>
<td>Fibreboard, MDF</td>
<td>EN 622-5</td>
<td>MDF.LA, MDF.HLS</td>
<td>1</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MDF.HLS</td>
<td>2</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
<td>1.10</td>
</tr>
</tbody>
</table>

3.1.4 DEFORMATION MODIFICATION FACTORS ($K_{\text{def}}$) FOR SERVICE CLASSES

There is nothing here that you have not learnt so far.
Please note that, based on the ‘Application rule’ (4), the value of $k_{def}$ given in Table 3.2 should be increased by 1.0 when the timber installed is very wet (near fibre saturation point) and is likely to dry out while under load.

### 3.2 Solid timber

(1)P Solid rectangular timber should comply with EN 14081 – Parts 1 to 4 (strength classes of timber are given in EN 338) and round cross-sectional timber should comply with EN 14544.

(2) & (3): Dimensions of timber (depth for bending, width and length for tension) have an effect on the strength of timber. It must be noted that the characteristic mechanical properties values obtained and given in relevant standards (EN 338 for solid timber, EN 1194 for glulam, and EN 14374 for LVL) have been based on certain dimensions (called ‘reference dimensions’). Therefore, when member sizes differ from those used for obtaining characteristic mechanical properties, modification factors are given in this section of the code for increasing or decreasing the mechanical properties in order to take account of the variation in dimensions.

For rectangular solid timber with a characteristic density $\rho_k \leq 700 \text{ kg/m}^3$, the ‘reference depth’ is 150 mm. Therefore, for depths in bending or widths in tension of less than 150 mm, the characteristic bending and tension strengths may be increased by the following factor $k_h$:

\[
\begin{align*}
    f_{m,k} &= k_h \cdot f_{m,k} \text{ for bending, depth less than 150 mm} \\
    f_{t,0,k} &= k_h \cdot f_{t,0,k} \text{ for tension, width less than 150 mm}
\end{align*}
\]
where
\[ f_{m,k} = \text{is characteristic bending strength} \]
\[ f_{t,0,k} = \text{is characteristic tension parallel to grain strength} \]
\[ k_h = \text{minimum of } (150/h)^{0.2} \text{ or } 1.3 \text{ where } h \text{ is depth for bending case and width for tension case in mm.} \]

(4) The value of \( k_{\text{def}} \) given in Table 3.2 should be increased by 1.0 when the timber installed is very wet (near fibre saturation point) and is likely to dry out while under load.

(5) P Finger joints shall comply with EN 385.

3.3 Glued laminated timber

(1) P Glulam shall comply with EN 14080 and its strength and stiffness mechanical characteristic properties with EN 1194.

(2) & (3) For rectangular solid glulam, the ‘reference depth or width’ is 600 mm. Therefore, for depths in bending or widths in tension of less than 600 mm, the characteristic bending strength \( (f_{m,k}) \) and tension strengths \( (f_{t,0,k}) \) may be increased by the following factor \( k_h \):

\[ f_{m,k} = k_h \cdot f_{m,k} \text{ for bending, depth less than 150 mm} \]
\[ f_{t,0,k} = k_h \cdot f_{t,0,k} \text{ for tension, width less than 150 mm} \]

where
\[ f_{m,k} = \text{is characteristic bending strength} \]
\[ f_{t,0,k} = \text{is characteristic tension parallel to grain strength} \]
\[ k_h = \text{minimum of } (600/h)^{0.1} \text{ or } 1.1 \text{ where } h \text{ is depth for bending case and width for tension case in mm.} \]

(4) P: Large finger joints complying with ENV 387 shall not be used in products in service class 3 where the direction of grain changes at the joint.

(5): This ‘Application rule’ says that the effect of member size on the tensile strength perpendicular to grain shall be taken into account, but it does not say how!

3.4 Laminated Veneer Lumber (LVL)

LVL is a material which consists of veneers like plywood, but where the grain direction of veneers are the same. LVL can be manufactured for different sizes and thicknesses.
It should be noted that there are two harmonised standards for LVL; one for LVL as structural member and one for LVL as panel/boards.

(1)P: Structural LVL shall comply with EN 14374 and LVL as panels with EN 14279.

(2)P & (3): For rectangular LVL, the ‘reference depth’ is 300 mm. Therefore, for different depths in bending, the characteristic bending strength \( f_{m,k} \) shall be multiplied by the following factor \( k_h \):

\[
f_{m,k} = k_h \cdot f_{m,k} \text{ for bending, depth other than 300 mm}
\]

where

\[
f_{m,k} = \text{is characteristic bending strength}
\]

\[
k_h = \text{minimum of } (300/h)^s \text{ or } 1.2 \text{ where } h \text{ is depth in mm for bending case}
\]

and ‘s’ is the size effect exponent which can be obtained from EN 14374.

(4): For rectangular LVL, the ‘reference length’ in tension is 3000 mm. Therefore, for lengths in tension other than 3000 mm, the characteristic tension strength \( f_{t,0,k} \) shall be multiplied by the following factor \( k_l \):

\[
f_{t,0,k} = k_l \cdot f_{t,0,k} \text{ for tension, length other than 3000 mm}
\]

where

\[
f_{t,0,k} = \text{is characteristic tension parallel to grain strength}
\]

\[
k_l = \text{minimum of } (3000/l)^s/2 \text{ or } 1.1 \text{ where } l \text{ is the length in mm and ‘s’ is the size effect exponent which can be obtained from EN 14374.}
\]

(5)P: is already dealt with above.

(6)P: Large finger joints complying with ENV 387 shall not be used in products in service class 3 where direction of grain changes at the joint.

(7)P: This ‘Application rule’ says that the effect of member size on the tensile strength perpendicular to grain shall be taken into account, but it does not say how!

### 3.5 Wood-based panels

(1)P & (2): It must be noted that for the design and application of wood-based panels, designers are required to refer to EN 12871 as there is not much guidance for the structural design of panel products. All specified wood-based panels must comply with the harmonised standard EN 13986 and its supporting CEN standards. For LVL used as panels, EN 14279 must be complied with.

Please note the use of ‘Softboards, according to EN 622-4’ are not allowed unless it is designed by testing which would be costly for one-off design.
3.6 Adhesives

Different adhesive types are being researched currently. The choice of adhesive is vital as it is required to maintain structural integrity for various conditions throughout the life of the structure. However, these are allowed:

<table>
<thead>
<tr>
<th>Adhesive type</th>
<th>Compliance with</th>
<th>Environment used in (ie Service class)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td>EN301</td>
<td>All service classes</td>
</tr>
<tr>
<td>Type 2</td>
<td>EN301</td>
<td>ONLY in Service classes 1 and 2 and NOT exposed to prolonged temperatures in excess of 50°C</td>
</tr>
</tbody>
</table>

3.7 Metal fasteners

All metal fasteners shall comply with EN 14592.

All metal connectors shall comply with EN 14545.

2.11 ‘Section 4: Durability’ (page 30)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

4.1 Resistance to biological organisms

This part of design is very important for timber, wood-based panels and the connections used in the timber structures in different environments called ‘Hazard Classes’. There are usually two options which must be chosen:

– Design by natural durability (EN 350-2) for a particular hazard class (EN 335-1, EN 335-2 and EN 335-3).

– Design by preservative treatment (EN 351-1 and EN 460).

4.2 Resistance to corrosion

This part has the utmost crucial importance in the design of connections (joints) as the majority of timber structures depend on their joint design. Ensuring the durability of connections is a must, either by inherent corrosion-resistance or by protective measures against corrosion. Table 4.1 gives the MINIMUM specifications for material protection against corrosion for fasteners (related to ISO 2081):
2.12 ‘Section 5: Basis of structural analysis’
(pages 31 – 35)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

5.1 General

Any experienced good designer will automatically consider and meet all the requirements given in this section in their analyses. It is necessary to highlight the importance of using adequate analyses, models and their verifications.

5.2 Members

(1)P: Any experienced designer would consider the imperfection of members and in-homogeneities of the material. The code already contains and considers these.

(2)P & (3): The only thing that designers should take into consideration is the reduction of the cross-sectional area of members due to holes, notches, etc. Holes ≤ 6mm diameter and holes in compression members which are filled with materials with higher stiffness and strength than wood can be ignored.

(4): The easier way of dealing with this ‘Application rule’ is to consider the reduction of cross-sectional area (effective area) due to any hole in a multiple fasteners joint which is at a distance ≤ half the minimum fastener spacing.
5.3 Connections

Again, there is nothing here that a good and experienced designer will not consider in his/her design of connections/joints.

5.4 Assemblies

5.4.1 GENERAL

(1)P: All this is saying is that the analysis should use static models which consider the behaviour of structure and its supports with an acceptable level of accuracy.

(2): The analysis of trussed rafters is performed by frame models and two methods are allowed: rigorous (5.4.2) and simplified methods (5.4.3).

(3): In addition to static linear analysis, this ‘Application rule’ allows elasto-plastic analysis for plane frames (portal frame) and arches.

5.4.2 FRAME STRUCTURE

This part is known as the rigorous analysis method for trussed rafter design or framed elements. It is common practice in the UK that manufacturers use one of the models (design programmes) provided by one of the system owners of the Trussed Rafter Association (TRA) for designing the trusses. Therefore, you do not usually need sections 5.4.2 and 5.4.3 as they are already considered in the design of trusses by the system owners’ models. However, all the requirements given in sections 5.4.2 & 5.4.3 are to be considered if you write your own model or should be included in a commercial model.

Please note, the above models provided by the system owners are usually for trusses with punched metal plate connectors. For any other type of connection, the requirements given in this section must be used but it is simpler if the simplified method of analysis (5.4.3) is used.

5.4.3 SIMPLIFIED ANALYSIS OF TRUSSES WITH PUNCHED PLATE FASTENERS

This part is known as the simplified analysis method for trussed rafter design or framed elements. Again, it is common practice in the UK that manufacturers use one of the models (design programmes) provided by one of the system owners of the ‘Trussed Rafter Association (TRA) for designing the trusses. Therefore, you do not usually need this section as it is already considered in the design of trusses by the system owners’ models. However, all the requirements given in this section are to be considered if you write your own model or should be included in a commercial model.
Please note, the above models provided by the system owners are usually for trusses with punched metal plate connectors. For any other type of connections, this section would be easier to use for the design.

(1): The following conditions should be complied with:

- no re-entrant angles in the external profile
- bearing width within the length $a_1$, and the distance $a_2$ in Figure 5.2 is not greater than $a_1/3$ or 100 mm whichever is the greater
- truss height is greater than 0.15 times the span and 10 times the maximum external member depth.

(2): All the nodal points are assumed to be pinned.

(3): Bending moment in single or multi-span bay members should be calculated assuming simply-supported nodal points.

Effect of deflection at nodal points and partial fixity at the connections should be taken into account by a reduction of 10% of the moments at the inner supports of the member. The inner support moment should be used to calculate the span bending moments.
5.4.4 PLANE FRAMES AND ARCHES

(1): Any experienced designer would consider the imperfection of members and in-homogeneities of the material. Designers should take into consideration the reduction of the cross-sectional area of members due to holes, notches, etc. Holes \( \leq 6\text{mm} \) diameter and holes in compression members which are filled with materials with greater stiffness and strength than wood can be ignored. Consider the reduction of cross-sectional area (effective area) due to any hole in a multiple fasteners joint which is at a distance \( \leq \) half the minimum fastener spacing.

(2): The effect of induced deflection on internal forces and moments shall be taken into account using the elasto-plastic analysis with the following assumption:

– consider imperfect shape by assuming a deflection by applying an angle of \( \phi \) inclination to the structure or relevant parts. In addition, a sinusoidal curvature between the nodes of the structure corresponding to a maximum eccentricity of \( e \) should be assumed where:

\[
e = 0.0025l.
\]

\[
\phi = 0.005 \text{ (for height or length of the member } h \leq 5\text{m}) \text{ OR } 0.005\sqrt{5/h} \text{ (for height or length of the member } h > 5\text{m}) \text{ (please note these values are minimum. Therefore, larger values may be used). Some examples are given in Figure 5.3}
\]
2.13 ‘Section 6 – Ultimate Limit States’
(Pages 36 – 54)

The following clause numbers are kept the same as those in EN 1995-1-1 for clarity:

6.1 Design of cross-sections subjected to stress in one principal direction

6.1.1 GENERAL

There are some general points to be remembered throughout EN 1995-1-1. Beware that the principal axes are defined differently than in BS 5268. In EN 1995-1-1 the principal axis is defined y-y, whilst the weak axis is now z-z, the axes are designated in minor case. The direction of the grain (1), the length of the member is now in the x axis!

![Diagram of member axes]

Key:
(1) direction of grain

Figure 6.1 – Member Axes

For comparison the BS 5268 definition is shown below.

**BS 5268**

![Diagram of member axes]

Please also note the use of comma (,) and dot (.) in numbers in EN 1995-1-1! Decimal places are separated using a comma rather than a dot. Hence 1x10^3 would be expressed as:

1,000.00 ✓ European
1,000.00 ✗ UK
In the following section the design checks for the load-bearing ability of cross sections in one principal direction are listed. EN 1995-1-1 asks designers to check the ultimate limit states based on stresses in the section, based on the following

$$\frac{\sigma_d}{f_d} \leq 1$$

where

$$\sigma_d = S_d / \text{cross section property (such as } A, W)$$

$$S_d \quad \text{Action (factored to a design load)}$$

$$f_d = k_{\text{mod}} \times f_k / \gamma_M$$

### 6.1.2 TENSIONS PARALLEL TO GRAIN

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

This clause states that the tensile design stress parallel to the grain of the timber section must be smaller or equal to the permissible tension stress $f_{t,0,d}$

**Permissible stress, $f_{t,0,d}$**

The following subscripts and symbols are used:

$t$ Tension design strength of the timber material

$0$ Parallel to the grain of the material (zero degree from the orientation of the grain)

$d$ States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic tensile strength of the material according to the formula given below.

$$f_{t,0,d} = k_{\text{mod}} \times f_{t,0,k} / \gamma_M$$

The following reference tables in EN 1995-1-1 have to be consulted

For $k_{\text{mod}},$ Table 3.1:
### Table 3.1 – Values of $k_{mod}$

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
<th>Permanent action</th>
<th>Long-term action</th>
<th>Medium-term action</th>
<th>Short-term action</th>
<th>Instantaneous action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
<td>EN 14081-1</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>EN 14080</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>LVL</td>
<td>EN 14374, EN 14279</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Plywood</td>
<td>EN 636</td>
<td>Part 1, Part 2, Part 3</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 2, Part 3</td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 3</td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
</tr>
<tr>
<td>OSB</td>
<td>EN 300</td>
<td>OSB/2</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>OSB/3, OSB/4</td>
<td>2</td>
<td>0.30</td>
<td>0.40</td>
<td>0.55</td>
<td>0.70</td>
</tr>
<tr>
<td>Particle-board</td>
<td>EN 312</td>
<td>Part 4, Part 5</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 5</td>
<td>2</td>
<td>0.20</td>
<td>0.30</td>
<td>0.45</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 6, Part 7</td>
<td>1</td>
<td>0.40</td>
<td>0.50</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Part 7</td>
<td>2</td>
<td>0.30</td>
<td>0.40</td>
<td>0.55</td>
<td>0.70</td>
</tr>
<tr>
<td>Fibreboard, hard</td>
<td>EN 622-2</td>
<td>HB,LA, HB,HLA, 1 or 2</td>
<td>1</td>
<td>0.30</td>
<td>0.45</td>
<td>0.65</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td></td>
<td>HB,HLA1 or 2</td>
<td>2</td>
<td>0.20</td>
<td>0.30</td>
<td>0.45</td>
<td>0.60</td>
</tr>
<tr>
<td>Fibreboard, medium</td>
<td>EN 622-3</td>
<td>MBHLA1 or 2</td>
<td>1</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MBHL,HL1 or 2</td>
<td>1</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MBHL,HL2 or 2</td>
<td>2</td>
<td>0.45</td>
<td>0.60</td>
<td>0.80</td>
<td>1.00</td>
</tr>
<tr>
<td>Fibreboard, MDF</td>
<td>EN 622-5</td>
<td>MDF,LA, MDF, HLS</td>
<td>1</td>
<td>0.20</td>
<td>0.40</td>
<td>0.60</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>MDF,HLS</td>
<td>2</td>
<td>0.45</td>
<td>0.60</td>
<td>0.80</td>
<td>1.00</td>
</tr>
</tbody>
</table>
For $\gamma_M$, Table NA.3 (National Annex):

<table>
<thead>
<tr>
<th>Type of Material</th>
<th>$\gamma_m$ Partial factor for material properties and resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>For ‘Fundamental combinations’</strong></td>
<td></td>
</tr>
<tr>
<td>Solid timber, untreated</td>
<td>1.3</td>
</tr>
<tr>
<td>Solid timber, preservative treated</td>
<td>1.3</td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>1.25</td>
</tr>
<tr>
<td>LVL, plywood, OSB</td>
<td>1.2</td>
</tr>
<tr>
<td>Particleboards</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, hard</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, medium</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, MDF</td>
<td>1.3</td>
</tr>
<tr>
<td>Fibreboards, soft</td>
<td>1.3</td>
</tr>
<tr>
<td>Connections (except for punched metal plate fasteners)</td>
<td>1.3</td>
</tr>
<tr>
<td>Punched metal plate fasteners, anchorage strength</td>
<td>1.3</td>
</tr>
<tr>
<td>Punched metal plate fasteners, plate (steel) strength</td>
<td>1.15</td>
</tr>
<tr>
<td><strong>For ‘Accidental combinations’</strong></td>
<td></td>
</tr>
<tr>
<td>‘Accidental combinations’</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values $f_{t,0,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>D30</td>
</tr>
<tr>
<td>C16</td>
<td>D40</td>
</tr>
<tr>
<td>C18</td>
<td>D50</td>
</tr>
<tr>
<td>C20</td>
<td>D60</td>
</tr>
<tr>
<td>C22</td>
<td>D70</td>
</tr>
<tr>
<td>C24</td>
<td></td>
</tr>
<tr>
<td>C27</td>
<td></td>
</tr>
<tr>
<td>C30</td>
<td></td>
</tr>
<tr>
<td>C35</td>
<td></td>
</tr>
<tr>
<td>C40</td>
<td></td>
</tr>
<tr>
<td>C45</td>
<td></td>
</tr>
<tr>
<td>C50</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>18</td>
</tr>
<tr>
<td>10</td>
<td>24</td>
</tr>
<tr>
<td>11</td>
<td>30</td>
</tr>
<tr>
<td>12</td>
<td>36</td>
</tr>
<tr>
<td>13</td>
<td>42</td>
</tr>
</tbody>
</table>

**Example of design tensile strength, $f_{t,0,d}$:**

For service class 1, load-duration of medium-term, C16 solid timber and using the formula

$$f_{t,0,d} = k_{mod} \times \frac{f_{t,0,k}}{\gamma_M}$$

We can obtain $k_{mod}$ of 0.8 and $\gamma_M = 1.3$ and $f_{t,0,k}$ of 10 N/mm² from the above table (ie Table 1 of EN338), we can calculate $f_{t,0,d}$ to be 6.15 N/mm².
Design stress, $\sigma_{t,0,d}$

The following subscripts and symbols are used:

$t$ Tension property of the timber material

$0$ Parallel to the grain of the material (zero degree from the orientation of the grain)

$d$ States the design stresses must be used

The design loads and resulting design stress need to be determined to prove compliance. Characteristic values of the forces parallel to the grain, caused by the permanent and variable actions have to be summed and multiplied by the respective partial load safety factors, according to Equation 6.10 in EN 1990, giving the design tensile load $F_{t,0,d}$.

The design tensile stress parallel to the grain, $\sigma_{t,0,d}$ is then determined

$$\sigma_{t,0,d} = \frac{F_{t,0,d}}{A}$$

For more guidance on how to compute $A$, see clause 2.4.2 for design value of geometrical data.

(1): Geometrical data for cross-sections may be taken as nominal values from product standards or drawings for execution.

(NOTE: Nominal values are ‘Target’ values)

6.1.3 TENSION PERPENDICULAR TO GRAIN

(1)P: The effect of member size shall be taken into account.

This clause becomes important when designing multilayered materials and elements, curved sections and connections. See also section 6.4 and section 8.

6.1.4 COMPRESSION PARALLEL TO GRAIN

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

This clause states that the compressive design stress parallel to the grain of the timber section must be smaller or equal to the permissible tension stress $f_{c,0,d}$

Design compressive strength, $f_{c,0,d}$

The following subscripts and symbols are used:
c  Compression property of the timber material

0  Parallel to the grain of the material (zero degree from the orientation of the grain)

d  States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic compressive strength of the material according to the formula given below.

\[ f_{c,0,d} = k_{\text{mod}} \times \frac{f_{c,0,k}}{\gamma_M} \]

For \( k_{\text{mod}} \gamma_M \), and see tables above

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values \( f_{c,0,k} \) (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>16</td>
<td>17</td>
</tr>
</tbody>
</table>

**Design stress, \( \sigma_{c,0,d} \)**

The following subscripts and symbols are used:

- \( c \)  Compression property of the timber material
- \( 0 \)  Parallel to the grain of the material (zero degree from the orientation of the grain)
- \( d \)  States the design stresses must be used

The design loads and resulting design stress need to be determined to prove compliance. Characteristic values of the forces parallel to the grain, caused by the permanent and variable actions have to be summed and multiplied by the respective partial load safety factors, according to Equation 6.10 in EN 1990, giving the design tensile load \( F_{c,0,d} \).

The design tensile stress parallel to the grain, \( \sigma_{c,0,d} \) is then determined

\[ \sigma_{c,0,d} = \frac{F_{c,0,d}}{A} \]
For more guidance on how to compute $A$ see clause 2.4.2 for design value of geometrical data.

(1): Geometrical data for cross-sections may be taken as nominal values from product standards or drawings for execution.

(NOTE: Nominal values are ‘Target’ values)

6.1.5 COMPRESSION PERPENDICULAR TO GRAIN

$$\sigma_{c,90,d} \leq k_{c,90} \times f_{c,90,d}$$

This clause states that the compressive design stress perpendicular to the grain of the timber section must be smaller or equal to the design compressive strength $f_{c,90,d}$ modified by an enhancement factor $k_{c,90}$, which accounts for the crushing resistance of timber under localised stresses. In all cases, the loading scenario and construction details have to be checked for serviceability as covered in section 7 serviceability.

**Design compressive strength, $f_{c,90,d}$**

The following subscripts and symbols are used:

$c$  
Compression design stress of the timber material

$90$  
Perpendicular to the grain of the material (90 degree from the orientation of the grain)

$d$  
States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic compressive strength of the material according to the formula given below.

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

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values $f_{c,90,k}$ (N/mm$^2$)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>2.0</td>
<td>2.2</td>
</tr>
</tbody>
</table>
The modification factor, $k_{c,90}$

The modification factor $k_{c,90}$ is dependent on the location of the load and the support conditions of the member. It takes account of the load configuration, possibility of splitting and the degree of compressive deformation.

$1 \leq k_{c,90} \leq 4$

In the following configuration $k_{c,90}$ can be above 1.0 (Beware: observe limiting upper value of 4.0!)

- Case 1: Beam member resting on supports
- Case 2: Beam resting on continuous or discrete support.

**CASE 1: BEAMS RESTING ON SUPPORTS**

![Beam on supports](image)

**Factor $k_{c,90}$ for intermittent support case:**

<table>
<thead>
<tr>
<th></th>
<th>$a &gt; \frac{h}{3}$</th>
<th>$a \leq \frac{h}{3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>End support</td>
<td>1</td>
<td>$\left(2.38 - \frac{l}{250}\right) \left(1 + \frac{h}{12l}\right)$</td>
</tr>
<tr>
<td>Internal support</td>
<td>2.38 - $\frac{l}{250}$ $\left(1 + \frac{h}{6l}\right)$</td>
<td></td>
</tr>
</tbody>
</table>

**CASE 2: BEAMS RESTING ON CONTINUOUS OR DISCRETE SUPPORTS**

Discrete supports are defined in EN 1995-1-1 as supports to very wide beams, where the stress distribution in the member can be considered as widely spread.
Factor $k_{c,90}$ for continuous and discrete supports, $b \leq 2.5b$:

\[
\begin{array}{c}
\begin{array}{c}
\text{Further conditions:} \\
* \text{Compressive force over full width } b
\end{array}
\end{array}
\]

Factor $k_{c,90}$ for continuous and discrete supports, $b > 2.5b$:

\[
\begin{array}{c}
\begin{array}{c}
\text{Further conditions:} \\
* \text{Compressive force applied over full width } b \\
* \text{Contact length } l < h \text{ or } 10\text{mm whichever is greater}
\end{array}
\end{array}
\]
The effective length, $l_{ef}$

- **Continuous support conditions, $b \leq 2.5b$**

<table>
<thead>
<tr>
<th>Loading condition</th>
<th>$l_{ef}$</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Diagram (a)" /></td>
<td>$l_{ef} = l + \frac{h}{3}$</td>
</tr>
<tr>
<td><img src="image" alt="Diagram (b)" /></td>
<td>$l_{ef} = l + \frac{2h}{3}$, $h \geq 40\text{mm}$</td>
</tr>
</tbody>
</table>

Conditions:
- Loads adjacent to the end of the member
- $h \leq 2.5b$

- Distance from edge of concentrated load
  - $a \geq \frac{2}{3}h$
  - $l_{ef}$ Curtailed $\frac{a}{2}$ from any end and $\frac{h}{4}$ from any adjacent compressed area

Companion to EN 1995-1-1
- **Discrete support conditions,** \( b \leq 2.5b \)

\[
\text{Loading condition} \quad \text{\begin{array}{c}
\text{Conditions:} \\
\text{\quad Distance from edge of concentrated load} \\
\text{\quad } a \geq h \\
\text{\quad } l_i \geq 2h \text{ from any adjacent compressed area} \\
\text{\quad } \ell_{ef} = 0.5 \left( \ell + \ell_s + \frac{2h}{3} \right) \\
\text{\quad } h \geq 40\text{mm}
\end{array}}
\]

- **Continuous and discrete support conditions,** \( b > 2.5b \)
Design stress, $\sigma_{c,90,d}$

The following subscripts and symbols are used:

- $c$ Compression property of the timber material
- $90$ Perpendicular to the grain of the material (90 degree from the orientation of the grain)
- $d$ States the design stresses must be used
The design loads and resulting design stress need to be determined to prove compliance. Characteristic values of the forces parallel to the grain, caused by the permanent and variable actions have to summed and multiplied by the respective partial load safety factors, according to Equation 6.10 in EN 1990, giving the design tensile load $F_{c,90,d}$.

The design tensile stress parallel to the grain, $\sigma_{c,90,d}$ is then determined

$$\sigma_{c,0,d} = \frac{F_{c,90,d}}{A}$$

For more guidance on how to compute $A$ see clause 2.4.2 for design value of geometrical data.

(1): Geometrical data for cross-sections may be taken as nominal values from product standards or drawings for execution.

(NOTE: Nominal values are ‘Target’ values)

6.1.6 BENDING

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

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

Bending about one axis:

$$\sigma_{m,d} = \frac{M_d}{W_n}$$

The factor $k_m$ does account for linear interaction of stresses in sections subjected to bi-axial bending. When rectangular sections are subjected to bi-axial bending maximum stresses only occur in two corners of the cross section, and there only in the outermost fibre of the section. To account for the less likely occurrence of maximum stress being reached in both corners, the less critical corner stress (compared to uni-axial stress situation) can be reduced by the modification factor $k_m$.

$k_m$ accounts for two-axial bending effects, for solid, glue laminated and LVL timber:
$k_m = 0.7$ for rectangular sections

$k_m = 1.0$ for other cross sections

$k_m = 1.0$ for other wood-based structural products, for all cross sections

**Design bending strength $f_{m,d}$**

The following subscripts and symbols are used:

$m$  Moment property

d  States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic bending strength of the material according to the formula given below.

$$f_{m,d} = k_{mod} \times \frac{f_{m,k}}{\gamma_M}$$

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values $f_{m,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>14</td>
<td>16</td>
</tr>
</tbody>
</table>

Please remember to always check that the relevant stability criteria are met, see section 6.3.

### 6.1.7 SHEAR

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

This clause states that the design shear stress must be smaller or equal to the permissible design shear strength $f_{v,d}$.

Two types of shear are distinguished in EN 1995-1-1:

**Type 1**: Where one shear stress component is parallel to the grain
**Type 2:** Where both stress components are perpendicular to the grain. This is called rolling shear. In EN 1995-1-1 the rolling shear of timber and timber products can be assumed to be equal to twice the tension strength perpendicular to the grain.

![Diagram showing rolling shear](image)

**Permissible shear stress, \( f_{v,d} \)**

The following subscripts and symbols are used:

- \( v \)  Shear property of the timber material
- \( d \)  States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic tensile strength of the material according to the formula given below.

\[
f_{v,d} = k_{\text{mod}} \times \frac{f_{v,k}}{\gamma_M}
\]

Here for solid timber:

**Extract of EN338 structural timber strength classes, characteristic values \( f_{v,k} \) (N/mm²)**

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>1,7</td>
<td>1,8</td>
</tr>
</tbody>
</table>

(2): When beams are directly supported (at underside of beam) and support loads are acting at the upper side of the beam, the compression in the beam section enhances the shear ability of structural timber, this is recognised in clause 6.1.7 (2)

The contribution of a concentrated load at supports can be disregarded if:

- The concentrated load (F) acts on the top side of the beam
- The load is situated at a distance smaller than the height of the beam from the edge of the support.
For beams with a notch at the support the concentrated load can only be ignored when the notch is located as shown below. In these configurations the load must be located within a distance \( b_{ef} \) from the edge of the support.

6.1.8 TORSION

\[ \tau_{\text{tor,}d} \leq k_{\text{shape}} f_{\text{v,}d} \]

This clause states that the design torsional shear stress must be smaller or equal to the design shear strength \( f_{\text{v,}d} \), multiplied by a shape factor, which accounts for the torsional properties of two different cross sections.

**The shape factor, \( k_{\text{shape}} \)**

For circular cross section:

\[ k_{\text{shape}} = 1.2 \]

For rectangular cross section:

\[ k_{\text{shape}} = 1 + 0.15 \frac{b}{h} \]

The minimum value must be used
Design shear strength, $f_{v,d}$

The following subscripts and symbols are used:

$tor$ Torsional property of the timber section

$d$ States the design stresses must be used

$shape$ Properties are influenced by shape of section

$b$ The larger cross-sectional dimension

$b$ The smaller cross-sectional dimension

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic tensile strength of the material according to the formula given below.

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

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values $f_{v,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>1,7</td>
<td>1,8</td>
</tr>
</tbody>
</table>

Since EN 338 does not specifically state torsional properties, the design can be undertaken with the shear properties of the material, which is assumed to be a conservative design approach.

Based on research by Möhler and Hemmer (1977) the torsion stresses for circular and rectangular cross sections can be calculated by:

For circular cross section

$$\tau_{tor} = \frac{2 \cdot M_{tor}}{\pi \cdot r^3}$$

For rectangular cross section

$$\tau_{tor} = 3 \left(1 + 0.6 \cdot \frac{b}{h}\right) \cdot \frac{M_{tor}}{h \cdot b^2}$$
For combined torsional and vertical load shear *in softwoods only* (DIN 1052, 2004), the following equation can be used:

\[
\left( \frac{\tau_d}{f_{v,d}} \right)^2 + \frac{\tau_{nor,d}}{f_{v,d}} \leq 1
\]

This design clause only applies to softwoods.

### 6.2 Design of cross-sections subjected to combined stresses

#### 6.2.1 GENERAL

(1)P: The following design guidance applies to sections:

- Straight, solid timber
- Glue laminated timber
- Wood-based products

of

- Constant cross section
- Grain running parallel to the length of the member.

#### 6.2.2 COMPRESSION STRESSES AT AN ANGLE TO THE GRAIN

Interaction of compressive stresses in two or more directions shall be taken into account.

The compressive stress at an angle \( \alpha \) to the grain (figure below)

![Figure 6.7 – Compressive stresses at an angle to the grain](image-url)
should satisfy

\[ \sigma_{c,d} \leq \frac{f_{c,0,d}}{k_{c,0} \cdot f_{c,90,d}} \sin^2 \alpha + \cos^2 \alpha \]

**Design compressive strength, \( f_{c,0,d} \)**

The following subscripts and symbols are used:

- \( c \) Compression property of the timber material
- \( 0 \) Parallel to the grain of the material (zero degree from the orientation of the grain)
- \( d \) States the design stresses must be used

The design resistance of the material needs to be computed in EN 1995 by reducing the ultimate, characteristic compressive strength of the material according to the formula given below.

\[ f_{c,0,d} = k_{\text{mod}} \times \frac{f_{c,0,k}}{\gamma_M} \]

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values \( f_{c,0,k} \) (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>16</td>
<td>17</td>
</tr>
</tbody>
</table>

**Design compressive strength, \( f_{c,90,d} \)**

\[ f_{c,90,d} = k_{\text{mod}} \times \frac{f_{c,90,k}}{\gamma_M} \]

Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values \( f_{c,90,k} \) (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>2,0</td>
<td>2,2</td>
</tr>
</tbody>
</table>
The modification factor, $k_{c,90}$

Here the effect of stresses perpendicular to the grain are considered, see also section 6.1.5 for determination of $l_{ef}$

**Factor $k_{c,90}$ for continuous and discrete supports, $b \leq 2.5b$:**

$$\begin{array}{c}
h \leq 2.5b \\
\left(2.38 - \frac{l}{250}\right)\left(\frac{l_{ef}}{l}\right)^{0.5}
\end{array}$$

Further conditions:
- Compressive force over full width $b$

**Factor $k_{c,90}$ for continuous and discrete supports, $b > 2.5b$:**

$$\begin{array}{c}
h > 2.5b \\
\left(\frac{l_{ef}}{l}\right)
\end{array}$$

Further conditions:
- Compressive force applied over full width $b$
- Contact length $l < h$ or 10mm whichever is greater

6.2.3 COMBINED BENDING AND AXIAL TENSION

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{k_m \sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1$$

$k_m$ accounts for two-axial bending effects, for solid, glue laminated and LVL timber. The factor accounts for linear interaction of stresses in sections subjected to bi-axial bending. When rectangular sections are subjected to bi-axial bending, maximum stresses only occur in two corners of the cross section, and there only in the outermost fibre of the section. To account for the less likely occurrence of maximum stress being reached in both corners, the less critical corner stress (compared to uni-axial stress situation) can be reduced by the modification factor $k_m$. 
6.2.4 COMBINED BENDING AND AXIAL COMPRESSION

\[
\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

\[
\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1
\]

\(k_m\) accounts for two-axial bending effects, for solid, glue laminated and LVL timber. The factor accounts for linear interaction of stresses in sections subjected to bi-axial bending. When rectangular sections are subjected to bi-axial bending, maximum stresses only occur in two corners of the cross section, and there only in the outermost fibre of the section. To account for the less likely occurrence of maximum stress being reached in both corners, the less critical corner stress (compared to uni-axial stress situation) can be reduced by the modification factor \(k_m\).

\(k_m\) accounts for two-axial bending effects, for solid, glue laminated and LVL timber:

\(k_m = 0,7\) for rectangular sections

\(k_m = 1,0\) for other cross sections

\(k_m = 1,0\) for other wood-based structural products, for all cross sections.

Always remember to check the instability criterion, as given in 6.3.

6.3 Stability of members

6.3.1 GENERAL

Bending stresses induced by initial curvature, eccentricities or induced deflections must be taken into account. Please note the use of \textit{characteristic} material properties (for example \(f_{c,0,k}\)) in the stability design steps shown below.
The stability of columns subjected to either compression or combined compression and bending should be verified as summarised in 6.3.2. The lateral torsional stability of beams subjected to either bending or combined bending and compression should be verified in accordance with 6.3.3.

### 6.3.2 COLUMNS SUBJECTED TO EITHER COMPRESSION OR COMBINED COMPRESSION AND BENDING

The design approach for columns in this chapter of EN 1995-1-1 can be subdivided into two main cases:

(a) Columns in pure, centric compression

(b) Columns in combined compression and bending.

In both cases the design compression stress of the column is compared with the design compressive strength multiplied by the buckling coefficient $k_c$ and the bending coefficient $k_m$, for case (b).

So for Case (a): Column in pure, centric compression, the following condition has to be met:

\[
\frac{\sigma_{c,0,d}}{k_c f_{c,0,d}} \leq 1
\]

The following have to be computed in the order given below:

(1): Relative slenderness $\lambda_{rel}$ of the columns needs to be determined for both main axes, y and z

\[
\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}
\]

\[
\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}
\]

For this, the designer needs to determine and compute $l_{ef}$, $i$ and $\lambda = \frac{l_{ef}}{i}$.
### Effective length, $l_{ef}$

Follows structural design theory

<table>
<thead>
<tr>
<th>Configuration</th>
<th>$l_{ef}$</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported: Constant moment</td>
<td>$l_{ef} = 1.0l$</td>
<td></td>
</tr>
<tr>
<td>Simply supported: Uniformly distributed load</td>
<td>$l_{ef} = 0.9l$</td>
<td></td>
</tr>
<tr>
<td>Simply supported: Concentrated force at the middle of the span</td>
<td>$l_{ef} = 0.8l$</td>
<td></td>
</tr>
<tr>
<td>Cantilever: Uniformly distributed load</td>
<td>$l_{ef} = 0.5l$</td>
<td></td>
</tr>
<tr>
<td>Cantilever: Concentrated force at free end</td>
<td>$l_{ef} = 0.8l$</td>
<td></td>
</tr>
</tbody>
</table>

$$i_y = \sqrt{\frac{I_y}{A}}$$

$$i_z = \sqrt{\frac{I_z}{A}}$$
Extract of EN338 structural timber strength classes, characteristic values $f_{c,0,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>16</td>
<td>17</td>
</tr>
</tbody>
</table>

Extract of EN338 structural timber strength classes, characteristic values $E_{0.05}$ (kN/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>4.7</td>
<td>5.4</td>
</tr>
</tbody>
</table>

Buckling coefficient $k_c$

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}$$

with

$$k_y = 0.5 \left(1 + \beta_c (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2 \right)$$

$$k_z = 0.5 \left(1 + \beta_c (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2 \right)$$

$eta_c$ for

**Solid timber** | **Glue laminated and LVL**
---|---
0.2 | 0.1

**Observe straightness limits**
The deviation from straightness is measured midway between support. It should be measured in elements where lateral instability can occur, hence for:

- Columns
- Beams
- Members in frames.
The following limitations are imposed by EN 1995-1-1

Glue laminated timber and LVL

Glue laminated timber and LVL

Solid timber*

Solid timber*

*The limitations on bow in most strength grading rules are inadequate for the selection of material for these members and particular attention should therefore be paid to their straightness.

6.3.3 BEAMS SUBJECTED TO EITHER BENDING OR COMBINED BENDING AND COMPRESSION

Lateral torsional stability of beams shall be verified when:

- Case 1: Only a moment $M_y$ (about the strong axis) exists:
- Case 2: Where a combination of moment $M_y$ and $N_c$ exists

For both cases the relative slenderness of the beam needs to be determined:

$$\lambda_{rel,m}$$

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

$f_{m,k}$

Extract of EN338 structural timber strength classes, characteristic values $f_{m,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14 C16 C18 C20 C22 C24 C27 C30 C35 C40 C45 C50</td>
<td>D30 D40 D50 D60 D70</td>
</tr>
<tr>
<td>14 16 18 20 22 24 27 30 35 40 45 50 30 40 50 60 70</td>
<td></td>
</tr>
</tbody>
</table>

$\sigma_{m,\text{crit}}$

For softwood with a solid rectangular cross-section:

$$\sigma_{m,\text{crit}} = \frac{0.78b^2}{hl_{ef}}E_{0.05}$$
For all others:

\[ \sigma_{w,cr} = \frac{M_{y,cr}}{W_y} = \frac{\pi E_{0.05} I_z G_{0.05} I_{tor}}{I_y W_y} \]

\( E_{0.05} \)

Extract of EN338 structural timber strength classes, characteristic values \( E_{0.05} \) (kN/mm\(^2\))

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>4.7</td>
<td>5.4</td>
</tr>
</tbody>
</table>

\( G_{0.05} \)

Extract of EN338 structural timber strength classes, characteristic values \( G_{0.05} \) (kN/mm\(^2\))

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>0.29</td>
<td>0.34</td>
</tr>
</tbody>
</table>

The above values for \( G_{0.05} \) are obtained from \( E_{0.05}/16 \).

\( I_{ef} \)

\[ l_{ef} = \frac{l}{l_{ef}} \]

**Table 6.1 – Effective length as a ratio of the span**

<table>
<thead>
<tr>
<th>Beam type</th>
<th>Loading type</th>
<th>( \ell_{ef}/\ell^{*} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported</td>
<td>Constant moment</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Uniformly distributed load</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Concentrated force at the middle of the span</td>
<td>0.8</td>
</tr>
<tr>
<td>Cantilever</td>
<td>Uniformly distributed load</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Concentrated force at the free end</td>
<td>0.8</td>
</tr>
</tbody>
</table>

* The ratio between the effective length \( \ell_{ef} \) and the span \( \ell \) is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, \( \ell_{ef} \) should be increased by \( 2h \) and may be decreased by \( 0.5h \) for a load at the tension edge of the beam.
$I_{tor}$ Torsional properties of cross section (structural design theory)

For rectangular sections (structural design theory):

$$I_{tor} = \frac{bh^3}{3} \left[ 1 - 0.63 \frac{t}{b} + 0.052 \left( \frac{h}{b} \right)^2 \right] \quad ! \ h < b$$

For circular sections:

$$I_{tor} = \frac{1}{2} \pi r^4$$

**CASE 1: WHERE THERE IS ONLY A MOMENT $M_y$**

All stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_{crit} f_{m,d}$$

$k_{crit}$

For beams with an initial lateral deviation from straightness within limits defined in the National Annex (see below) the following can be used, depending on:

- $\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}}$ (see above)

<table>
<thead>
<tr>
<th>$\lambda_{rel,m}$</th>
<th>$k_{crit}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda_{rel,m} \leq 0.75$</td>
<td>1</td>
</tr>
<tr>
<td>$0.75 &lt; \lambda_{rel,m} \leq 1.4$</td>
<td>$1.56 - 0.75 \lambda_{rel,m}$</td>
</tr>
<tr>
<td>$1.4 &lt; \lambda_{rel,m}$</td>
<td>$\frac{1}{\lambda_{rel,m}^2}$</td>
</tr>
</tbody>
</table>

**Note:** $k_{crit}$ can be taken as 1,0 for a beam where lateral displacement of its compressive edge is prevented throughout its lengths and torsional rotation is prevented at supports. For example in roofs where beams rest within fork supports.
Observe straightness limits
The deviation from straightness is measured midway between support. It should be measured in elements where lateral instability can occur, hence for:

- Columns
- Beams
- Members in frames

The following limitations are imposed by EN 1995-1-1:

<table>
<thead>
<tr>
<th>Glue laminated timber and LVL</th>
<th>Solid timber*</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{l}{500} )</td>
<td>( \frac{l}{300} )</td>
</tr>
</tbody>
</table>

*The limitations on bow in most strength grading rules are inadequate for the selection of material for these members and particular attention should therefore be paid to their straightness

CASE 2:

Where a combination of moment about the strong axis \( M_y \) and a compressive force exists.

The stresses in the section should satisfy the following expression:

\[
\left( \frac{\sigma_m}{f_{m,d}} \right)^2 + \frac{\sigma_c}{f_{c,0,d}} \leq 1
\]

\[
k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}}
\]

\[
k_z = 0.5 \left( 1 + \beta_c (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2 \right)
\]

\[
\lambda_{rel,z} = \frac{\lambda_z}{\pi} \sqrt{\frac{f_{c,0,k}}{E_{0.05}}}
\]
Here for solid timber:

Extract of EN338 structural timber strength classes, characteristic values $f_{c,0,k}$ (N/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14 16</td>
<td>C16 17</td>
</tr>
</tbody>
</table>

$E_{0,05}$

Extract of EN338 structural timber strength classes, characteristic values $E_{0,05}$ (kN/mm²)

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14 4.7</td>
<td>C16 5.4</td>
</tr>
</tbody>
</table>

**Straightness factor, $\beta_c$**

**Solid timber**

0.2

**Glue laminated and LVL**

0.1

**Observe straightness limits**

The deviation from straightness is measured midway between support. It should be measured in elements where lateral instability can occur, hence for:

- Columns
- Beams
- Members in frames

The following limitations are imposed by EN 1995-1-1:

Glue laminated timber and LVL

$$\frac{1}{500}$$

Solid timber*

$$\frac{1}{500}$$

*The limitations on bow in most strength grading rules are inadequate for the selection of material for these members and particular attention should therefore be paid to their straightness
6.4 Design of cross-sections in members with varying cross-sections or curved shape

6.4.1 GENERAL

Timber designers should take the effects of combined axial and bending force into account. Stresses at a cross-section from an axial force may be calculated as usual from:

\[ \sigma_N = \frac{N}{A} \quad \text{N axial force} \]
\[ \text{A area of cross-section:} \]

6.4.2 SINGLE TAPERED BEAMS

Since single tapered beams have one edge cut at an angle to the grain of the wood, additional stresses are present within the member along this tapered edge. These stresses are additional to the normal stresses (along the grain of the timber), occur perpendicular to the grain and are also complemented by shear stresses. Therefore the bending stresses occurring in tapered beams are not linear as in beams of constant height. Whether the superimposition of all the stress components along the tapered edge of the beam is safe needs to be verified.

The design bending stresses \( \sigma_{m,\alpha,d} \) and \( \sigma_{m,0,d} \) may be taken as

\[ \sigma_{m,\alpha,d} = \sigma_{m,0,d} = \frac{6M_d}{bh^2} \]

At the outermost fibre of the tapered edge, the stresses should satisfy the following expression

\[ \sigma_{m,\alpha,d} \leq k_{m,\alpha} \sigma_{m,d} \]
Adjustment factor $k_{m,\alpha}$

The adjustment factor is dependent on whether the stress parallel to the tapered edge is in tension or in compression. The following expressions can be used to compute the adjustment factor:

<table>
<thead>
<tr>
<th>Tension</th>
<th>Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>$1 + \frac{f_{m,d}}{0.75f_{v,d}} \tan \alpha \left( \frac{f_{m,d}}{f_{c,90,d}} \tan^2 \alpha \right)^2$</td>
<td>$1 + \frac{f_{m,d}}{1.5f_{v,d}} \tan \alpha \left( \frac{f_{m,d}}{f_{c,90,d}} \tan^2 \alpha \right)^2$</td>
</tr>
</tbody>
</table>

### 6.4.3 DOUBLE TAPERED, CURVED AND PITCHED CAMBERED BEAMS

This section covers three main types of structures:

- Double tapered beam with lower straight edge
- Curved beam
- Pitched cambered beam with curved lower edge.

The design clauses given in this section apply **ONLY** to glued laminated timber and LVL.

The design clauses apply to the apex zone of the beams, as shown below. Please note that in curved and pitched beams the apex zone extends over the curved part of the beam.

The following expression shall be satisfied

$$\sigma_{m,d} \leq k_r f_{m,d}$$
**Manufacturing adjustment factor \( k_r \)**

This factor accounts for the strength reduction due to bending of the laminates during manufacture.

<table>
<thead>
<tr>
<th>( \frac{r_m}{t} \geq 240 )</th>
<th>( \frac{r_m}{t} &lt; 240 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( 0.76 + 0.001 \frac{r_m}{t} )</td>
</tr>
</tbody>
</table>

\( r_m \) inner radius of beam

\( t \) lamination thickness

**Apex bending stress \( \sigma_{m,d} \)**

\[
\sigma_{m,d} = k_i \frac{6 M_{ap,d}}{b h_{ap}^2}, \text{ with}
\]

\[
k_i = k_1 + k_2 \left( \frac{h_{ap}}{r} \right) + k_3 \left( \frac{h_{ap}}{r} \right)^2 + k_4 \left( \frac{h_{ap}}{r} \right)^3
\]

The following expressions compute the factors:

\[
k_1 = 1 + 1.4 \tan \alpha_{ap} + 5.4 \tan^2 \alpha_{ap}
\]

\[
k_2 = 0.35 - 8 \tan \alpha_{ap}
\]

\[
k_3 = 0.6 + 8.3 \tan \alpha_{ap} - 7.8 \tan^2 \alpha_{ap}
\]

\[
k_4 = 6 \tan^2 \alpha_{ap}
\]

\[
r = r_m + 0.5 h_{ap}
\]

\( M_{ap,d} \)

Is the design moment at the apex.
**Tensile stress perpendicular to grain** $\sigma_{t, 90, d}$

In the apex zone, the greatest tensile stress perpendicular to grain should satisfy:

$$\sigma_{t, 90, d} \leq k_{\text{dis}} k_{\text{vol}} f_{1, 90, d}$$

**Stress distribution factor** $k_{\text{dis}}$

The factor takes into account the effect of the stress distribution in the apex zone.

<table>
<thead>
<tr>
<th>Stress Distribution Factor</th>
<th>$k_{\text{dis}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For double tapered and curved beams</td>
<td>1.4</td>
</tr>
<tr>
<td>For pitched cambered beams</td>
<td>1.7</td>
</tr>
</tbody>
</table>

**Volume factor** $k_{\text{vol}}$

<table>
<thead>
<tr>
<th>Volume Factor</th>
<th>$k_{\text{vol}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For solid timber</td>
<td>1.0</td>
</tr>
<tr>
<td>For glued laminated timber and LVL*</td>
<td>$\frac{V_0}{V}$, where $V_0$ is the reference volume of 0.01m$^3$ and $V$ is stressed volume of apex zone in m$^3$ but $V \leq \frac{2V_b}{3}$, where $V_b$ is the total volume of the beam</td>
</tr>
</tbody>
</table>

*all veneers parallel to the beam axis

**Combined tensions perpendicular to grain and shear**: $\sigma_{t, 90, d}$ and $\tau_d$

The following expression must be satisfied:

$$\frac{\tau_d}{f_{\tau, d}} + \frac{\sigma_{t, 90, d}}{k_{\text{dis}} k_{\text{vol}} f_{1, 90, d}} \leq 1$$

**Stress distribution factor** $k_{\text{dis}}$

The factor takes into account the effect of the stress distribution in the apex zone.
Volume factor $k_{vol}$

<table>
<thead>
<tr>
<th></th>
<th>$k_{vol}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For solid timber</td>
<td>1,0</td>
</tr>
<tr>
<td>For glued laminated timber</td>
<td>$\left(\frac{V_0}{V}\right)^{0.2}$</td>
</tr>
<tr>
<td>and LVL*</td>
<td></td>
</tr>
</tbody>
</table>

*all veneers parallel to the beam axis

Design tensile stress perpendicular to grain $\sigma_{t,90,d}$

\[
\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{bh_{ap}^2}
\]

or

\[
\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{bh_{ap}^2} - 0,6 \frac{P_d}{b}
\]

$M_{ap,d}$ design moment at apex of beam

$b$ width of beam

$h_{ap}$ height of beam at apex

$P_d$ uniformly distributed load acting on the top of the beam in apex area

\[
k_p = k_5 + k_6 \left(\frac{h_{ap}}{r}\right) + k_7 \left(\frac{h_{ap}}{r}\right)^2
\]

$k_5 = 0,2 \tan \alpha_{ap}$
6.5 Notched members

6.5.1 GENERAL

The effect of stress concentration must be taken into account.

It can only be disregarded if:

- Tension or compression is parallel to the grain
- Bending with tensile stresses at the notch if the taper is not steeper than \( 1: i = 1:10 \)

\[
k_\theta = 0.25 - 1.5 \tan \alpha_{ap} + 2.6 \tan^2 \alpha_{ap}
\]

\[
k_\gamma = 2.1 \tan \alpha_{ap} - 4 \tan^2 \alpha_{ap}
\]

Use of Structural Eurocodes – EN 1995 (Design of Timber Structures) BD2405

\[
k_\alpha = 2 \tan \alpha_{ap} - 4 \tan^2 \alpha_{ap}
\]
6.5.2 BEAMS WITH A NOTCH AT SUPPORT

Two cases of end-notched beams are considered in EN 1995-1-1

![Diagram](image)

**Figure 6.11 – End-notched beams**

Design rules apply for:

- Rectangular cross sections
- Grain runs essentially parallel to grain.

Shear stresses at supports must be calculated using the reduced depth $h_{ef}$

End notched beams must be checked to verify that:

$$\tau_d = \frac{1.5V}{bh_{ef}} \leq k_v f_{v,d}$$

$k_v$

<table>
<thead>
<tr>
<th>MIN</th>
<th>1.0</th>
<th>-</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$\frac{k_v \left(1 + \frac{1.1h^{1.5}}{\sqrt{h}}\right)}{\sqrt{h} \left(\alpha^2(1-\alpha) + 0.8 \frac{x}{h} \frac{1}{\alpha^2 - \alpha^2}\right)}$</td>
<td></td>
</tr>
</tbody>
</table>

*See above for variables for below.*
is the notch inclination (see Figure 6.11a)

\( b \) is the beam depth in mm

\( x \) is the distance from line of action of the support reaction to the corner of the notch

\[ \alpha = \frac{h_{ef}}{h} \]

\( k_n \)

For LVL 4,5

For solid timber 5

For glued laminated timber 6,5

### 6.6 System strength

1. This section allows the designer to take account of boundary conditions in the design of single members. Components which are laterally connected by a continuous load distribution system, such as roof trusses which are assembled in conjunction with tiling and battens, can be designed with strength properties enhanced by a system strength factor.

2. The system strength factor, \( k_{sys} \) is assumed to be 1.1.

3. The strength verification of the load distribution system should be carried out under short-term loading.

In all cases, the load distribution members must be:

- Continuous over at least two spans
- Joints within the load distribution members are staggered.

For roof trusses when the maximum centre-to-centre distance is 1.2 m, it can be assumed that the tiling battens, purlins or panels can transfer loads to the neighbouring trusses, provided the above conditions are met.

4. For laminated timber decks the value of the system strength factor varies and is given in the figure below, depending on how the member is laminated:

- Nailed or screwed laminations
- Laminations pre-stressed or glued together.
Section 7 ‘Serviceability limit states’
(pages 55 – 58)

Serviceability limit states are defined to enable designers to avoid unduly high deflections or vibrations, which would compromise the usability of the building, its appearance and the comfort of its inhabitants. In contrast to the ultimate limit states, design failures in the serviceability limit states do not endanger the life of building inhabitants or cause excessive damage to the property in most cases. Therefore, the serviceability design uses characteristic loads and mean material characteristics without additional material reduction factors.

7.1 Joint slip

(1): The slip modulus $K_{sec}$ (known as $K_s$ in EN 26891) for per shear plane per fastener should be calculated in accordance with Table 7.1 over page.
Table 7.1 – Values of $K_{ser}$ for fasteners and connectors in N/mm in timber-to-timber and wood-based panel-to-timber connections

<table>
<thead>
<tr>
<th>Fastener type</th>
<th>$K_{ser}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowels</td>
<td>$\rho_m^{1,5}d/23$</td>
</tr>
<tr>
<td>Bolts with or without clearance*</td>
<td>$\rho_m^{1,5}d^3/30$</td>
</tr>
<tr>
<td>Screws</td>
<td>$\rho_m^{1,5}d^3/80$</td>
</tr>
<tr>
<td>Nails (without pre-drilling)</td>
<td>$\rho_m^{1,5}d^3/30$</td>
</tr>
<tr>
<td>Staples</td>
<td>$\rho_m^{1,5}d^3/80$</td>
</tr>
<tr>
<td>Split-ring connectors type A according to EN 912</td>
<td>$\rho_m d/2$</td>
</tr>
<tr>
<td>Shear-plate connectors type B according to EN 912</td>
<td>$\rho_m d/2$</td>
</tr>
<tr>
<td>Toothed-plate connectors:</td>
<td></td>
</tr>
<tr>
<td>– Connectors types C1 to C9 according to EN 912</td>
<td>$1,5\rho_m d/4$</td>
</tr>
<tr>
<td>– Connectors type C10 and C11 according to EN 912</td>
<td>$\rho_m d/2$</td>
</tr>
</tbody>
</table>

* The clearance should be added separately to the deformation.

Where:

– Mean density, $\rho_m$, is in kg/m$^3$.

– $d$ (diameter) and $d_c$ (diameter of connector) are in mm.

(2) If a joint has two members of timber with different densities, then the mean density $\rho_m$ should be taken as:

$$\rho_m = \sqrt{\rho_{m,1} \cdot \rho_{m,2}}$$  \hspace{1cm} (equation 7.1 of EN 1995-1-1)

(3) For steel to timber or concrete to timber joints, $K_{ser}$ should be calculated using $\rho_m$ of timber and then multiplied by 2.

### 7.2 Limiting values for deflections of beams

(1) Components of deflection resulting from a combination of actions are shown in Figure 7.1 below:

![Figure 7.1 – Components of deflection](image_url)
For simplicity, some of the guidance given in Chapter 2 is repeated here.

Instantaneous deformation ($w_{\text{inst}}$) should be calculated using $E_{\text{mean}}, G_{\text{mean}}$ & $K_{\text{ser}}$ for a combination of loads (EN 1990, clause 6.5.3(2)a).

Final deformation ($w_{\text{fin}}$) should be calculated using quasi-permanent combination of loads [EN 1990, clause 6.5.3(2)c)]. When a structure has components and elements with the same creep behaviour, $w_{\text{fin}}$ should be:

$$w_{\text{fin}} = w_{\text{fin},G} + w_{\text{fin},Q1} + w_{\text{fin},Qi}$$

Where,

$$w_{\text{fin},G} = w_{\text{inst},G} (1+k_{\text{def}}) \text{ for permanent loads } G$$

$$w_{\text{fin},Q1} = w_{\text{inst},Q1} (1+\psi_{2,1} k_{\text{def}}) \text{ for the main imposed load } Q_1$$

$$w_{\text{fin},Qi} = w_{\text{inst},Qi} (\psi_{0,i}+\psi_{2,i} k_{\text{def}}) \text{ for the other imposed loads } Q_i \text{ (i>1)}$$

*It is important that $\psi_2$ factors are omitted from formulae 6.16a and 6.16b of EN 1990 when using the above formulae.*

$w_{\text{inst},G}, w_{\text{inst},Q1}$ and $w_{\text{inst},Qi}$ are instantaneous deformations for actions $G$, $Q1$ and $Qi$.

$\psi_{2,1}$ and $\psi_{2,i}$ are factors for the quasi-permanent value of imposed loads.

$\psi_{0,1}$ is the factor for the combination value of imposed loads.

$k_{\text{def}}$ for timber and wood-based materials, is the ‘deformation factor’ better known as the ‘creep factor’ given in Table 3.2 of the code which is a factor for the evaluation of creep deformation taking into account the relevant service class.

$k_{\text{def}}$ for connections, is the ‘deformation factor’ better known as the ‘creep factor’ given in 2.3.2.2(3) and 2.3.2.2(4) of the code which is a factor for the evaluation of creep deformation of connections taking into account the relevant service class.

$k_{\text{def}}$ should be doubled when the connection has timber members with the same creep behaviour. $k_{\text{def}} = 2 \sqrt{k_{\text{def},1} k_{\text{def},2}}$ when the connection has timber members with different creep behaviour where $k_{\text{def},1}$ and $k_{\text{def},2}$ are factors for the two members.

Final deformation should be calculated using final mean values of the appropriate moduli given below ($E_{\text{mean,fin}}, G_{\text{mean,fin}}$ and $K_{\text{ser,fin}}$) when the structure has components or elements with different materials (ie different creep behaviour):

$$E_{\text{mean,fin}} = E_{\text{mean}}/(1+k_{\text{def}})$$
\[ G_{\text{mean,fin}} = \frac{G_{\text{mean}}}{1 + k_{\text{def}}(1)} \]

\[ K_{\text{ser,fin}} = \frac{K_{\text{ser}}}{1 + k_{\text{def}}(1)} \]

**E_{\text{mean}}, G_{\text{mean}} and K_{\text{ser}} should be used for vibrations**

(2) Four main deflections are distinguished in Figure 7.1:

- Precamber
- Deflection from permanent loads
- Deflection from variable loads
- Deflection from creep.

These are then summarised in an overall deflection:

\[ w_{\text{net,fin}} = w_{\text{inst}} + w_{\text{creep}} - w_{c} = w_{\text{fin}} - w_{c} \]

As stated in BS EN 1990: 2002, the serviceability criteria should be specified for each project and agreed with the client. The recommended range of permissible deflections for beams with span \( l \) is given in Table 7.2 below:

<table>
<thead>
<tr>
<th>Type of member</th>
<th>Simply-supported single span ( l )</th>
<th>Cantilever span ( l )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof or floor members with a plasterboard or plasterboard ceiling</td>
<td>( 1/250 )</td>
<td>( 1/125 )</td>
</tr>
<tr>
<td>Roof or floor members without a plasterboard or plasterboard ceiling</td>
<td>( 1/150 )</td>
<td>( 1/75 )</td>
</tr>
</tbody>
</table>

Table 7.2: Examples of permissible deflection values for beams

### 7.3 Vibrations

#### 7.3.1 GENERAL

All (1)P & (2) are saying is that we need to check the deflection and vibration so that the structure functions properly and users are not uncomfortable when using the structure.

(3) It is important that a modal damping \( \xi = 0.02 \) is used for UK floors.
7.3.2 VIBRATION FROM MACHINERY

(1) P: This says that we need to use the unfavourable combinations of permanent and variable loads that can be expected from the machinery used on the floor.

(2): If the vibration is continuous, then Figure 5a in Appendix A of ISO 2631-2 with a multiplying factor of 1.0 should be used.

7.3.3 RESIDENTIAL FLOORS

(1) For floors with a fundamental frequency less than 8 Hz ($f_1 \leq 8$ Hz) special investigation should be carried out that is not covered by this code.

(2) We need to satisfy the following requirements for residential floors with a fundamental frequency more than 8 Hz ($f_1 \gt 8$ Hz):

\[
\frac{w}{F} \leq a \text{ mm per kN} \quad \text{(equation 7.3 of EN 1995-1-1)}
\]

and

\[
v \leq b \left( f_1 \xi - 1 \right) \text{ m/(Ns}^2) \quad \text{(equation 7.4 of EN 1995-1-1)}
\]

Where:

- $F$ is applied force at any point on the floor in kN
- $w$ is maximum instantaneous vertical deflection caused by a vertical concentrated static load force $F$ applied at any point on the floor, taking account of load distribution.
- $a$ is deflection of floor under a 1kN point load

The limits of $a$ are:

\[
a = 1.8 \text{ mm for span } l \leq 4000 \text{ mm}
\]

\[
a \leq 16500/l^{1.1} \text{ mm for span } l > 4000 \text{ mm}
\]

- $v$ is the unit impulse velocity response, i.e. the maximum initial value of the vertical floor vibration velocity (in m/s) caused by an ideal unit impulse (1 Ns) applied at the point of the floor giving maximum response. Components above 40 Hz may be disregarded
- $b$ is a constant for the control of unit impulse velocity response in m/Ns$^2$

The limits of $b$ are:

\[
b \geq 150 - [30(a - 0.5)/0.5] \text{ for } a \leq 1 \text{ mm}
\]

\[
b \geq 120 - 40(a - 1) \text{ for } a > 1 \text{ mm}
\]

- $\xi$ is the modal damping ratio = 0.02
Please note: Figure 7.2 of EN 1995-1-1 is replaced by the above expressions for $\xi$ of 0.02. It is also true that $v$ the unit impulse velocity response will not normally govern the size of floor joists in residential timber floors.

The above recommended limits on $a$ may be compared with a corresponding floor deflection calculated as:

$$a = 1000 \, k_{\text{dist}} \cdot l_{eq}^3 \cdot k_{\text{shear}} / [48 \, (EI)_{\text{joist}}] \text{ in mm}$$

where:

$$(EI)_{\text{joist}} = \text{bending stiffness of a joist in Nmm}^2 \text{ (calculated using } E_{\text{mean}} \text{)}$$

$k_{\text{dist}} = \text{proportion of point load acting on a single joist}$

$= \text{maximum of 0.3 OR } k_{\text{strut}} \{ 0.38 - [0.08 \ln (14 \, (EI)_b)] / s^4 \}$

$k_{\text{strut}} = 0.97 \text{ for single or multiple lines of strutting, installed in accordance with …NCCI document… otherwise } k_{\text{strut}} = 1.0$

$s = \text{joist spacing in mm}$

$(EI)_b = \text{floor flexural rigidity of the floor decking perpendicular to the joists in Nmm}^2 / \text{m, using } E_{\text{mean}} \text{ for } E. \text{ Discontinuities at the edge of floor panels or the ends of floor boards may be ignored}$

$(EI)_b = \text{may be increased by adding the flexural rigidity of plasterboard ceilings fastened directly to the soffit of the floor joists, assuming } E_{\text{plasterboard}} = 2000 \, \text{N/mm}^2$\n
$(EI)_b = \text{may be increased for open web joists with a continuous transverse bracing member fastened to all the joists within 0.1 } l \text{ of mid-span, by adding the bending stiffness of the transverse member in Nmm2 divided by the span } l \text{ in metres}$

$l_{eq} = \text{equivalent floor span in mm for simply-supported single span joists}$

$= 0.9 \, l \text{ for the end spans of continuous joists}$

$= 0.85 \, l \text{ for the internal spans of continuous joists}$

$k_{\text{shear}} = \text{amplification factor to account for shear deflections}$

$= 1.05 \text{ for simply-supported glued thin-webbed joists}$

$= 1.1 \text{ for continuous solid timber joists}$

$= 1.15 \text{ for simply-supported glued thin-webbed joists}$

$= 1.3 \text{ for continuous glued thin-webbed joists}$
(3) The above calculations are made under the assumption that the floor is unloaded ie only the self-weight of the floor and other permanent loads exist.

(4) For rectangular floors \((l \times \text{width } b)\) simply-supported along all four edges and with timber joists spanning \(l\), the fundamental frequency should be:

\[
\nu_1 = \frac{\pi}{2 \ell^2} \sqrt{\frac{(EI)_l}{m}}
\]  
(7.5)

where:

- \(m\) is the mass per unit area in kg/m²;
- \(\ell\) is the floor span, in m;
- \((EI)_l\) is the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction, in Nm²/m.

Please note the mass of the floor should be the permanent actions only without including partition loads or any variable actions.

In calculating the equivalent plate bending stiffness \((EI)_l\) no allowance should be made for composite action unless the floor is designed in accordance with 9.1.2 and with adhesives meeting the requirements of 3.6.

(5) For a rectangular floor \((l \times \text{width } b)\) simply-supported along all four edges, the value of \(v\) may be taken as:

\[
v = \frac{4(0.4 + 0.6 n_{40})}{mb\ell + 200}
\]  
(7.6)

where:

- \(v\) is the unit impulse velocity response, in m/(Ns²);
- \(n_{40}\) is the number of first-order modes with natural frequencies up to 40 Hz;
- \(b\) is the floor width, in m;
- \(m\) is the mass, in kg/m²;
- \(\ell\) is the floor span, in m.

The value of \(n_{40}\) may be calculated from:

\[
n_{40} = \left\{ \left( \frac{40}{\nu_1} \right)^2 - 1 \right\} \left\{ \frac{b}{\ell} \right\} \left( \frac{(EI)_l}{(EI)_b} \right)^{0.25}
\]  
(7.7)

where \((EI)_b\) is the equivalent plate bending stiffness, in Nm²/m, of the floor about an axis parallel to the beams, where \((EI)_b < (EI)_l\).

Please note: \(b\) is the width of the floor in the equations 7.6 and 7.7 and is not the \(b\) given in equation 7.4 earlier in this section.
‘Section 8’ Connections with metal fasteners (pages 59 – 88)
Companion to EN 1995-1-1

Screwed connections
Laterally loaded screws
Axially loaded screws
Combined laterally-axially

Multiple fastener connections

Multiple shear plane connections

Multiple shear plane connections

Alternating connection forces

Metal dowel-type fasteners
General failure modes

Timber-Timber and Panel-Timber

Steel-Timber

Screwed connections

Punched metal and plate fasteners

Geometry

Strength

Plate anchorage

Connection strength

Plate capacity

Split ring and shear plate

Toothed plate connectors

Multiple shear plane connections

General failure modes

Timber-Timber and Panel-Timber

Steel-Timber

Alternating connection forces

Metal dowel-type fasteners

Fastener requirements + Durability, table 4.1 section 2
8.1 General

8.1.1 FASTENER REQUIREMENTS

Unless design rules are given in the section below, the load-carrying capacity of fasteners shall be determined from tests according to:

- EN 1075
- EN 1380
- EN 1381
- EN 28970

{Characteristic load-carrying capacity in tension}

The design rules cover the design and construction of load-bearing mechanically fastened joints in structural timber, between softwood and heading and wood-based panel products complying with EN 1995-1-1.

It is of paramount importance to ensure that connections are durable and resist corrosion. EN 1995-1-1 addresses this issue in Section 4 Durability. Under subheading 4.2 ‘Resistance to corrosion’, guidance for fasteners and other structural connections is given. EN 1995-1-1 stipulates that metal fasteners and other structural connections must be inherently corrosion-resistant or must be protected against corrosion. The minimum corrosion protection is dependent on the service classes and examples of suitable minimum specifications are given below (in EN 1995-1-1: Table 4.1, page 30)

<table>
<thead>
<tr>
<th>Fastener</th>
<th>Service Class¹</th>
<th>Service Class²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nails and screws with ( d \leq 4 ) mm</td>
<td>None</td>
<td>Fe/Zn 12c³</td>
</tr>
<tr>
<td>Bolts, dowels, nails and screws with ( d &gt; 4 ) mm</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Staples</td>
<td>Fe/Zn 12c³</td>
<td>Fe/Zn 12c³</td>
</tr>
<tr>
<td>Punched metal plate fasteners and steel plates up to 3 mm thickness</td>
<td>Fe/Zn 12c³</td>
<td>Fe/Zn 12c³</td>
</tr>
<tr>
<td>Steel plates from 3 mm up to 5 mm in thickness</td>
<td>None</td>
<td>Fe/Zn 12c³</td>
</tr>
<tr>
<td>Steel plates over 5 mm thickness</td>
<td>None</td>
<td>None</td>
</tr>
</tbody>
</table>

¹ If hot dip zinc coating is used, Fe/Zn 12c should be replaced by Z275 and Fe/Zn 25c by Z350 in accordance with EN 10147

² For especially corrosive conditions consideration should be given to heavier hot dip coatings or stainless steel.

8.1.2 MULTIPLE FASTENER CONNECTIONS

EN 1995-1-1 gives five principal, overriding general guidelines and design principles for fasteners in timber structures.
1. To obtain design strength and stiffness of a connection, fasteners must be inserted and designed into a connection using the appropriate spacing, edge and end distances.

2. The summation of fasteners, of the same type and dimensions, in a multiple fastener connection is always less than the summation of the individual fasteners’ capacities.

3. When different types of fasteners are used in a connection or the stiffness of the connection is different, the compatibility of fasteners and connections must be verified.

4. A row of fasteners parallel to the grain has a characteristic load-carrying capacity of

\[ F_{v,ef,Rk} = n_{ef} F_{v,Rk} \]  

(8.1)

- \( F_{v,ef,Rk} \): effective load-carrying capacity of one row of fasteners parallel to the grain
- \( n_{ef} \): effective number of fasteners in line

The effective number of fasteners has to be determined depending on the connector used:

**For nails:**

\[ n_{ef} = n^{xe} \]

where:

- \( n_{ef} \) is the effective number of nails in the row;
- \( n \) is the number of nails in a row;
- \( k_{ef} \) is given in Table 8.1.

<table>
<thead>
<tr>
<th>Spacing(^a)</th>
<th>( k_{ef} )</th>
<th>Not predrilled</th>
<th>Predrilled</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_t &gt; 14d )</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>( a_t = 10d )</td>
<td>0.85</td>
<td>0.85</td>
<td></td>
</tr>
<tr>
<td>( a_t = 7d )</td>
<td>0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>( a_t = 4d )</td>
<td>-</td>
<td>0.5</td>
<td></td>
</tr>
</tbody>
</table>

\(^a\) For intermediate spacings, linear interpolation of \( k_{ef} \) is permitted.

**Figure 8.6 — Nails in a row parallel to grain staggered perpendicular to grain by \( d \)**
For bolts:

\[ n_{ef} = \min \left\{ \frac{n}{n_{a1}}, \frac{1}{\sqrt[13]{d}} \right\} \]  
\[ (8.34) \]

where:

- \( a_1 \) is the spacing between bolts in the grain direction.
- \( d \) is the bolt diameter.
- \( n \) is the number of bolts in the row.

For loads perpendicular to grain, the effective number of fasteners should be taken as

\[ n_{ef} = n \]  
\[ (8.35) \]

For angles \( 0^\circ < \alpha < 90^\circ \) between load and grain direction, \( n_{ef} \) may be determined by linear interpolation between expressions (8.34) and (8.35).

\( F_{v1,Rk} \) characteristic load carrying capacity of each fastener (parallel to the grain)

5. For a force acting at an angle to the direction of the row, it should be verified that the force component parallel to the row is less than or equal to the load-carrying capacity calculated according to expression (8.1).

Note: Here is a link to section 8.1.4 Forces at an angle to the grain, see below:

In forces at an angle to the grain there is the risk of splitting, caused by the tension force component of the angled load. EN 1995-1-1 includes a design check for this situation.

\( F_{v1,Ed} \leq F_{v0,Rd} \)  
\[ (8.2) \]

with

\[ F_{v1,Ed} = \max \left\{ F_{v,Ed,1}, F_{v,Ed,2} \right\} \]  
\[ (8.3) \]

where:

- \( F_{v0,Rd} \) is the design splitting capacity, calculated from the characteristic splitting capacity \( F_{v0,Rd} \) according to 2.4.3;
- \( F_{v,Ed,1}, F_{v,Ed,2} \) are the design shear forces on either side of the connection. (see Figure 8.1).
You also will need the following sections in EN 1995-1-1.

2.4.3 Design resistances

The design value $R_d$ of a resistance (load-carrying capacity) shall be calculated as:

$$ R_d = k_{mod} \frac{R_b}{\gamma_M} \quad (2.17) $$

where:

- $R_b$ is the characteristic value of load-carrying capacity;
- $\gamma_M$ is the partial factor for a material property;
- $k_{mod}$ is a modification factor taking into account the effect of the duration of load and moisture content.
### Table 3.1 – Values of $k_{mod}$

<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
<th>Load-duration class</th>
<th>Permanent action</th>
<th>Long term action</th>
<th>Medium term action</th>
<th>Short term action</th>
<th>Instantaneous action</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid timber</td>
<td>EN 14031-1</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>EN 14090</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.80</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>0.50</td>
<td>0.55</td>
<td>0.65</td>
<td>0.70</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>LVL</td>
<td>EN 14374, EN 14279</td>
<td>1</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.80</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.90</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
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For softwood connections, determine splitting potential as follows

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<th>Type of Material</th>
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<td>Solid timber, preservative treated</td>
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<td>Glued laminated timber</td>
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<td>LVL, plywood, OSB</td>
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<tr>
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<td>1.3</td>
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<td>Fibreboards, hard</td>
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<td>Fibreboards, medium</td>
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<td>Punched metal plate fasteners, anchorage strength</td>
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For softwood connections, determine splitting potential as follows

![Figure 8.1 – Inclined force transmitted by a connection](image-url)

Figure 8.1 – Inclined force transmitted by a connection
8.1.3 MULTIPLE SHEAR PLANE CONNECTIONS

In multiple shear plane connections the resistance of shear plane should be determined by assuming that each shear plane is part of a series of three-member connections.

For the designer to be able to combine the resistance from individual shear planes in the multiple shear plane connection, the failure modes must be compatible.

\[
F_{\text{work}} = 14hw \left( \frac{h}{h - \frac{h}{h}} \right)^{0.65}
\]

where:

\[
W = \begin{cases} 
\max \left( \frac{w_{pl}}{100} \right) & \text{for punched metal plate fasteners} \\
1 & \text{for all other fasteners}
\end{cases}
\]

and:

- \( F_{\text{work}} \) is the characteristic splitting capacity, in N;
- \( w \) is a modification factor;
- \( h_0 \) is the loaded edge distance to the centre of the most distant fastener or to the edge of the punched metal plate fastener, in mm;
- \( h \) is the timber member height, in mm;
- \( b \) is the member thickness, in mm;
- \( w_{pl} \) is the width of the punched metal plate fastener parallel to the grain, in mm.

8.1.3 MULTIPLE SHEAR PLANE CONNECTIONS

In multiple shear plane connections the resistance of shear plane should be determined by assuming that each shear plane is part of a series of three-member connections.

For the designer to be able to combine the resistance from individual shear planes in the multiple shear plane connection, the failure modes must be compatible.

So do **NOT** combine these failure modes

- a
- b
- g
- h
with the remaining failure modes in **timber-panel** connections

![Diagram of timber-panel connections](image)

(1) Single shear
(2) Double shear

Key:
- (1) Single shear
- (2) Double shear

and do **NOT** combine these following failure modes

![Diagram of failure modes](image)

with the remaining failure modes in **steel-to-timber** connections

![Diagram of steel-to-timber connections](image)
8.1.4 CONNECTION FORCES AT AN ANGLE TO THE GRAIN

In connections, forces can act at an angle to the grain of the solid timber pieces connected:

![Diagram of force transmission](image)

Figure 8.1 – Inclined force transmitted by a connection

When a force acts at an angle to the grain, as shown above, there is the possibility of splitting in the timber:

\[(2)\] To take account of the possibility of splitting caused by the tension force component, \(F_{Ed}\) at an \(\alpha\), perpendicular to the grain, the following shall be satisfied:

\[F_{v,Ed} \leq F_{90,Ed}\]  \hspace{1cm} \text{[8.2]}

with

\[F_{v,Ed} = \max \left\{ F_{v,Ed1}, F_{v,Ed2} \right\}\]  \hspace{1cm} \text{[8.3]}

where:

- \(F_{90,Ed}\) is the design splitting capacity, calculated from the characteristic splitting capacity \(f_{90,ct}\) according to 2.4.3;
- \(F_{v,Ed1}, F_{v,Ed2}\) are the design shear forces on either side of the connection. (see Figure 8.1).
Determine $F_{90,Rk}$ for softwood connections

$$F_{90,Rk} = 14 \cdot b \cdot \sqrt{\frac{h}{1 - \frac{h}{b}}}$$  \hspace{1cm} (8.4)

where:

$$w = \begin{cases} 
\max \left( \frac{w_{pl}}{100} \right)^{0.35} & \text{for punched metal plate fasteners} \\
1 & \text{for all other fasteners} 
\end{cases}$$  \hspace{1cm} (8.5)

and:

- $F_{90,Rk}$ is the characteristic splitting capacity, in N;
- $w$ is a modification factor;
- $h_l$ is the loaded edge distance to the centre of the most distant fastener or to the edge of the punched metal plate fastener, in mm;
- $h$ is the timber member height, in mm;
- $b$ is the member thickness, in mm;
- $w_{pl}$ is the width of the punched metal plate fastener parallel to the grain, in mm.

You also will need the following sections in EN 1995-1-1 to determine the design resistance of the member:

### 2.4.3 Design resistances

1(P) The design value $R_d$ of a resistance (load-carrying capacity) shall be calculated as:

$$R_d = k_{\text{mod}} \frac{R_\gamma}{\gamma_M}$$  \hspace{1cm} (2.17)

where:

- $R_\gamma$ is the characteristic value of load-carrying capacity;
- $\gamma_M$ is the partial factor for a material property;
- $k_{\text{mod}}$ is a modification factor taking into account the effect of the duration of load and moisture content.

Observe the different $\gamma_M$ partial factors to be used depending on the type of connection designed.
<table>
<thead>
<tr>
<th>Material</th>
<th>Standard</th>
<th>Service class</th>
<th>Permanent action</th>
<th>Long term action</th>
<th>Medium term action</th>
<th>Short term action</th>
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8.1.5 ALTERNATING CONNECTION FORCES

If a connection is subject to internal alternating forces due to long- or medium-term actions, the characteristic load-carrying capacity of the connection shall be reduced.

The effect of long or medium-term actions alternating between

- a tensile design force $F_{t,Ed}$ and
- a compressive design force $F_{c,Ed}$

should be taken into account by designing the connection for the following loading scenario:

$$ (F_{t,Ed} + 0.5F_{c,Ed}) \text{ and } (F_{c,Ed} + 0.5F_{t,Ed}) $$

### Type of Material

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<tr>
<th>Type of Material</th>
<th>$Y_M$</th>
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Companion to EN 1995-1-1
8.2 Lateral load-carrying capacity of metal dowel-type fasteners

For the design of a connection, the following characteristic load-carrying capacities of the metal dowel-type fasteners must be used:

- Yield strength
- Embedment strength
- Withdrawal strength

8.2.2 TIMBER-TO-TIMBER AND PANEL-TO-TIMBER CONNECTIONS

Single shear  Double shear

Depending on the relative stiffness of the fastener, timber failure modes can vary. The possible failure modes are given for single and double shear and the \( F_{v,Rk} \) characteristic load-carrying capacity of each fastener to the grain is to be determined depending on its failure mode in the connection.

Single shear and double shear are distinguished. The lowest load-carrying capacity governs the design of the connection. The design equations consist of two parts: the first part (derived from Johansen yield theory) and the second part which is called the ‘rope effect’. Accounting for the rope effect enhances the load-carrying capacity of the fastener and must be limited to a percentage of the Johansen part! Hence the rope effect cannot be included for dowels and can always be included for screws!

Determine \( F_{v,Rk} \), depending on the minimum value failure mode as shown overleaf.
### Single shear

<table>
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<th>Section</th>
<th>Formula</th>
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</table>

\[
\frac{f_{h,1,k} t_1 d}{1 + \beta} \left[ \beta + 2\beta^2 \left( 1 + \frac{t_2}{t_1} \right) \left( \frac{t_2}{t_1} \right)^2 - \beta \left( 1 + \frac{t_2}{t_1} \right) \right] + \frac{F_{M,\text{sl}}}{4} + 1.05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[ 2\beta^2 (1 + \beta) \frac{4\beta(1 + 2\beta) M_{y,\text{sl}}^{\text{inc}}}{f_{h,1,k} d} \right] - \beta + \frac{F_{M,\text{sl}}}{4}
\]

\[
1.05 \frac{f_{h,1,k} t_1 d}{1 + 2\beta} \left[ 2\beta^2 (1 + \beta) + \frac{4\beta(1 + 2\beta) M_{y,\text{sl}}^{\text{inc}}}{f_{h,1,k} d} \right] - \beta + \frac{F_{M,\text{sl}}}{4} + 1.15 \frac{2\beta}{1 + \beta} \sqrt{2 M_{y,\text{sl}} f_{h,1,k} d} + \frac{F_{M,\text{sl}}}{4}
\]

### Double shear

<table>
<thead>
<tr>
<th>Section</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{h,1,k} t_1 d$</td>
<td>$0.5 f_{h,1,k} t_2 d$</td>
</tr>
<tr>
<td>$f_{h,1,k} t_1 d$</td>
<td>$f_{h,1,k} t_2 d$</td>
</tr>
</tbody>
</table>

\[
0.5 f_{h,1,k} t_1 d \left[ 1.05 \frac{f_{h,1,k} t_1 d}{2 + \beta} \left[ 2\beta^2 (1 + \beta) + \frac{4\beta(2 + \beta) M_{y,\text{sl}}^{\text{inc}}}{f_{h,1,k} d} \right] - \beta + \frac{F_{M,\text{sl}}}{4} \right] + 1.15 \frac{2\beta}{1 + \beta} \sqrt{2 M_{y,\text{sl}} f_{h,1,k} d} + \frac{F_{M,\text{sl}}}{4}
\]
These design equations are best checked for the governing failure mode in a connection using a calculation routine, easily programmed in a Microsoft Excel sheet or similar software packages. The following variables are to be inserted at the beginning of the routine by the designer for each connection to be designed:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_b,1,k )</td>
<td>Characteristic embedment strength in timber</td>
<td>EN 383 and EN 14358</td>
</tr>
<tr>
<td>( f_b,2,k )</td>
<td>Characteristic embedment strength in timber</td>
<td>EN 383 and EN 14358</td>
</tr>
<tr>
<td>( t_1 )</td>
<td>Timber or board thickness or penetration depth of fastener</td>
<td></td>
</tr>
<tr>
<td>( t_2 )</td>
<td>Timber or board thickness or penetration depth of fastener</td>
<td></td>
</tr>
<tr>
<td>( M_y,\text{Rk} )</td>
<td>Characteristic fastener yield moment</td>
<td>EN 409 and EN 14358</td>
</tr>
<tr>
<td>( F_{ax,\text{Rk}} )</td>
<td>Characteristic axial withdrawal capacity of fastener</td>
<td></td>
</tr>
<tr>
<td>( d )</td>
<td>Fastener diameter</td>
<td></td>
</tr>
</tbody>
</table>

The values for the above variables are dependent on the fastener type used in a connection. Below the determination/values of these parameters is shown in detail under the various fastener sections. If there are no design guidelines the values are to be determined by tests and the relevant test standard is given in column three in the table above.

### 8.2.3 STEEL-TO-TIMBER CONNECTIONS

The failure modes in steel-to-timber connections are slightly different and therefore the critical design formulas are adapted. The characteristic load-carrying capacity of steel-to-timber connections depends on the thickness of the steel plates. The following definitions are needed for connection design in EN 1995-1-1:

Thin steel plates: \( \leq 0.5d \)

Thick steel plates: \( \geq d \) (tolerance of hole diameter being \(<0.1d\)) and \( d \) is the fastener diameter.

Characteristic load-carrying capacity of connections with steel plate thicknesses between thin and thick plates should be calculated by linear interpolations between the limiting thick and thin values.

The strength of the steel plate must always be checked.

The characteristic load-carrying capacity \( F_{v,\text{Rk}} \) per shear plane and fastener for:

- nails
- bolts
- dowels
- screws

is the minimum from the expressions overleaf.
### Single shear

<table>
<thead>
<tr>
<th>Thin steel plate (≤0.5d)</th>
<th>Thick steel plate (≥d)</th>
<th>Check on diameter tolerance!</th>
</tr>
</thead>
<tbody>
<tr>
<td>MIN!</td>
<td>MIN!</td>
<td></td>
</tr>
</tbody>
</table>

\[
0.4 f_{hk} t_1 d \
1.15 \sqrt{\frac{2 M_{y,ki}}{f_{hk} d t_1^2}} + \frac{F_{m,ki}}{4} \
2,3 \sqrt{M_{y,ki} f_{hk} d + \frac{F_{m,ki}}{4}}
\]

### Double shear: Steel plate central member

<table>
<thead>
<tr>
<th>Thin steel plate (≤0.5d)</th>
<th>Thick steel plate (≥d)</th>
<th>Check on diameter tolerance!</th>
</tr>
</thead>
<tbody>
<tr>
<td>MIN!</td>
<td>MIN!</td>
<td></td>
</tr>
</tbody>
</table>

\[
f_{h,k1} t_1 d \
f_{h,k1} t_1 d \left[ 2 + \frac{4 M_{y,ki}}{f_{h,k1} d t_1^2} - 1 \right] + \frac{F_{m,ki}}{4} \
2,3 \sqrt{M_{y,ki} f_{h,k1} d + \frac{F_{m,ki}}{4}}
\]

### Double shear: Steel plate outer members

<table>
<thead>
<tr>
<th>Thin steel plate (≤0.5d)</th>
<th>Thick steel plate (≥d)</th>
<th>Check on diameter tolerance!</th>
</tr>
</thead>
<tbody>
<tr>
<td>MIN!</td>
<td>MIN!</td>
<td></td>
</tr>
</tbody>
</table>

\[
0.5 f_{h,2k} t_2 d \
1.15 \sqrt{2 M_{y,ki} f_{h,2k} d + \frac{F_{m,2k}}{4}} \
0.5 f_{h,2k} t_2 d \
2,3 \sqrt{M_{y,ki} f_{h,2k} d + \frac{F_{m,2k}}{4}}
\]
The lowest load-carrying capacity governs the design of the connection. The limitation of the rope effect equally applies, so as before, the design equations consist of two parts: the first part (derived from Johansen yield theory) and the second part which is called the ‘rope effect’. Accounting for the rope effect enhances the load-carrying capacity of the fastener and must be limited to a percentage of the Johansen part.

Hence the rope effect cannot be included for dowels and can always be included for screws.

(5)P: It shall be taken into account that the load-carrying capacity of steel-to-timber connections with a loaded end may be reduced by failure along the perimeter of the fastener group. Annex A gives guidance for this case but please note that this should be ignored for nails and screws in accordance with the UK National Annex.

8.3 Nailed connections

8.3.1 LATERALLY LOADED NAILS

Definitions in nailed connection design:

**Single shear connection**

\[ t_1 \] Headside thickness in a single shear connection

\[ t_2 \] Point side penetration
Double shear connection

EN 1995-1-1 advises to pre-drill timber when

- characteristic density of timber is $> 500 \frac{kg}{m^3}$
- diameter of nail is $> 8 mm$

Guidance on determining the diameter $d$ of a nail

- Squared nails
- Grooved nails
- Standard nails

EN 14592 not yet available.

Characteristic properties of connections with wire nails (minimum tensile strength $600 \frac{N}{mm^2}$)

- Characteristic value for the yield moment of the nail, $M_{y,Rk}$

$$M_{y,Rk} = \begin{cases} \frac{f_u}{600}180d^{2.6} & \text{for round nails} \\ \frac{f_u}{600}270d^{2.5} & \text{for square nails} \end{cases} \quad (8.14)$$

where:
- $M_{y,Rk}$ is the characteristic value for the yield moment, in Nmm;
- $d$ is the nail diameter as defined in EN 14592, in mm;
- $f_u$ is the tensile strength of the wire, in N/mm².

- Characteristic embedment strength of the timber, $f_{b,k}$

The design formulae to determine the embedment strength of the timber depend on three main variables:
- The nail diameter: \( d \leq 8 \text{ mm}, d > 8 \text{ mm}, \text{ bead} \geq 2d \)

- The type of materials to be connected: timber/LVL to timber/LVL, panel to timber (also the type of panel product!)

- Insertion technique: with or without pre-drilling

\( d \leq 8 \text{ mm} \) – **timber/LVL to timber/LVL**

- without predrilled holes

\[ f_{u,k} = 0.082 \rho_k d^{0.3} \text{ N/mm}^2 \]  
\[ (8.15) \]

- with predrilled holes

\[ f_{u,k} = 0.082 (1 - 0.01d) \rho_k \text{ N/mm}^2 \]  
\[ (8.16) \]

where:

\( \rho_k \) is the characteristic timber density, in kg/m\(^3\);

\( d \) is the nail diameter, in mm.

\( d > 8 \text{ mm} \) – **timber/LVL to timber/LVL**

Perpendicular to grain:

\[ f_{h,0,k} = 0.082 (1 - 0.01d) \rho_k \]  
\[ (8.32) \]

at an angle \( \alpha \) to grain

\[ f_{h,0,k} = \frac{f_{h,0,k}}{k_{90} \sin^2 \alpha + \cos^2 \alpha} \]  
\[ (8.31) \]

where:

\[ k_{90} = \begin{cases} 
1.35 + 0.015 d & \text{for softwoods} \\
1.30 + 0.015 d & \text{for LVL} \\
0.90 + 0.015 d & \text{for hardwoods} 
\end{cases} \]  
\[ (8.33) \]

and:

\( f_{h,0,k} \) is the characteristic embedment strength parallel to grain, in N/mm\(^2\);

\( \rho_k \) is the characteristic timber density, in kg/m\(^3\);

\( \alpha \) is the angle of the load to the grain;

\( d \) is the bolt diameter, in mm.
Nail head diameter ≥ 2d – panel to timber

- for plywood:
  \[ f_{kh} = 0.14 \rho_k d^{0.3} \]  \( (8.20) \)
  where:
  \( f_{kh} \) is the characteristic embedment strength, in N/mm²;
  \( \rho_k \) is the characteristic plywood density in kg/m³;
  \( d \) is the nail diameter, in mm;

- for hardboard in accordance with EN 622-2:
  \[ f_{kh} = 30 d^{-0.7} t^{0.5} \]  \( (8.21) \)
  where:
  \( f_{kh} \) is the characteristic embedment strength, in N/mm²;
  \( d \) is the nail diameter, in mm;
  \( t \) is the panel thickness, in mm.

- for particleboard and OSB:
  \[ f_{kh} = 65 d^{-0.7} t^{0.1} \]  \( (8.22) \)
  where:
  \( f_{kh} \) is the characteristic embedment strength, in N/mm²;
  \( d \) is the nail diameter, in mm;
  \( t \) is the panel thickness, in mm.

**Double shear connection with overlapping nails**

Nails are allowed to overlap in the central member provided that

\[ t - t_2 > 4d \]

![Figure 8.5 – Overlapping nails](image-url)
Effective number of fasteners in a nailed connection

If a row of nails is inserted into a connection in a staggered manner by at least 1d no effective number has to be calculated for the connection.

Otherwise for one row of n nails (parallel to the grain) the load-carrying capacity should be calculated using the effective number of fasteners rather than the actual number.

\[ n_{ef} = n^{ke} \]  \hspace{1cm} (8.17)

where:
- \( n_{ef} \) is the effective number of nails in the row;
- \( n \) is the number of nails in a row;
- \( k_{ef} \) is given in Table 8.1.

<table>
<thead>
<tr>
<th>Spacing ( s )</th>
<th>Not predrilled</th>
<th>Predrilled</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s \leq 14d )</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>( s = 10d )</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>( s = 7d )</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>( s = 4d )</td>
<td>-</td>
<td>0.5</td>
</tr>
</tbody>
</table>

\* For intermediate spacings, linear interpolation of \( k_{ef} \) is permitted.

Figure 8.6 – Nails in a row parallel to grain staggered perpendicular to grain by \( d \)

General guidelines and detailing

- There should always be at least two nails in a connection
- Structural detailing and control of a nailed connection:
  1. Unless otherwise specified, nails should be driven in at right angles to the grain and to such depth that the surfaces of the nail heads are flush with the timber surface.
  2. Unless otherwise specified, slant nailing should be carried out in accordance with Figure 8.8(b).
  3. The diameter of pre-drilled holes should not exceed 0.8d, where \( d \) is the nail diameter.
8.3.1.2 Nailed timber-to-timber connections
Pointside penetration length ($t_2$)

- For smooth nails should be at least $8d$
- All other nails (defined in EN 14592) at least $6d$

Design guidelines for the use of smooth and other nails in timber connections

EN 1995-1-1 considers the use of smooth and other nails in detail. Different connection design philosophies are allowed and can be used to justify a design.

- Smooth nails inserted into the end grain of the timber should not be considered capable of transmitting lateral forces (in the direction or perpendicular to the direction of the nail).
- In secondary structures (ie non-loadbearing structures such as fascia boards) smooth nails may be used. The load-carrying capacity of the nail should then be taken as $\frac{1}{3}$ of the values for nails, inserted at right angles to the grain.
- Nails other than smooth nails (see EN 14592) may be used in all structures. The design values should be taken as $\frac{1}{3}$ of the values for smooth nails of equivalent diameter at right angles to the grain when:
  - nails are only laterally loaded
  - there are at least three nails per connection
  - pointside penetration is at least $10d$
  - connection is not exposed to service class 3 conditions
  - prescribed spacings and edge distances are satisfied.
Minimum spacings and edge distances

Table 8.2 – Minimum spacings and edge and end distances for nails

<table>
<thead>
<tr>
<th>Spacing or distance (see Figure 8.7)</th>
<th>Angle $\alpha$</th>
<th>Minimum spacing or end edge distance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>without predrilled holes</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with predrilled holes</td>
</tr>
<tr>
<td></td>
<td>$\rho_0 &lt; 420 \text{ kg/m}^3$</td>
<td>$20 \text{ kg/m}^3 \leq \rho_0 &lt; 500 \text{ kg/m}^3$</td>
</tr>
<tr>
<td>Spacing $a_1$ (parallel to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$d &lt; 5 \text{ mm}: (5 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td></td>
<td>$\geq 5 \text{ mm}: (5 + \cos \alpha) d$</td>
<td></td>
</tr>
<tr>
<td>Spacing $a_2$ (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$5d$</td>
</tr>
<tr>
<td>Distance $a_{3,2}$ (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$(10 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td>Distance $a_{3,c}$ (unloaded end)</td>
<td>$0^\circ \leq \alpha \leq 270^\circ$</td>
<td>$10d$</td>
</tr>
<tr>
<td>Distance $a_{4,2}$ (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>$d &lt; 5 \text{ mm}: (5 + 2 \sin \alpha) d$</td>
</tr>
<tr>
<td></td>
<td>$\geq 5 \text{ mm}: (5 + 5 \sin \alpha) d$</td>
<td>$d \geq 5 \text{ mm}: (7 + 5 \sin \alpha) d$</td>
</tr>
<tr>
<td>Distance $a_{4,e}$ (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$5d$</td>
</tr>
</tbody>
</table>

- $a_1$ is the spacing of nails within one row parallel to grain;
- $a_2$ is the spacing of rows of nails perpendicular to grain;
- $a_{3,2}$ is the distance between nail and unloaded end;
- $a_{3,c}$ is the distance between nail and loaded end;
- $a_{4,2}$ is the distance between nail and unloaded edge;
- $a_{4,e}$ is the distance between nail and loaded edge;
- $\alpha$ is the angle between the force and the grain direction.
Pre-drilling of timber before insertion of nail

EN 1995-1-1 suggests the pre-drilling of timber depending on

- Size of timber member
- Species of timber

Size of timber member

\[
t = \max \left( \frac{7d}{13d - 30}, \frac{\rho_k}{400d} \right)
\]

where:
- \( t \) is the minimum thickness of timber member to avoid pre-drilling, in mm;
- \( \rho_k \) is the characteristic timber density in kg/m³;
- \( d \) is the nail diameter, in mm.

Species of timber member

Certain timber species are especially sensitive to splitting upon insertion of nails, for example
• Fir (Abies alba)
• Douglas fir (Pseudotsuga menziesii)
• Spruce (Picea abies)

EN 1995-1-1 recommends pre-drilling for Fir and Douglas fir.

Pre-drilling is recommended for timber members of the these species smaller than

\[ t = \max \left( \frac{14d}{(13d - 30) \rho_k}, \frac{7d}{(13d - 30) \rho_k} \right) \]

If edge distances are

\[ a_1 \geq 10d \quad \text{for } \rho_k \leq 420 \text{ kg/m}^3 \]
\[ a_1 \geq 14d \quad \text{for } 420 \text{ kg/m}^3 \leq \rho_k \leq 600 \text{ kg/m}^3. \]

then minimum timber thickness to avoid pre-drilling can be determined using

\[ t = \max \left( \frac{7d}{(13d - 30) \rho_k}, \frac{14d}{(13d - 30) \rho_k} \right) \]

### 8.3.1.4 Nailed steel-to-timber connections

**Minimum edge and end distances**

- The minimum edge and end distances below apply.
- Minimum nail spacings \( a_1 \) must be multiplied by a factor 0.7
### 8.3.2 AXIALLY-LOADED NAILS

**General guidelines**

- Smooth nails must not be used to resist permanent or long-term axial loading.
- For threaded nails, only the threaded part should be considered capable of transmitting axial load.
- Nails in end grain are not deemed to allow transfer of axial load.

**Characteristic withdrawal capacity of nails** $F_{ax,Rk}$

The following loading conditions are covered:

---

**Table 8.2 – Minimum spacings and edge and end distances for nails**

<table>
<thead>
<tr>
<th>Spacing or distance (see Figure 8.7)</th>
<th>Angle $\alpha$</th>
<th>Minimum spacing or end/edge distance</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>without predrilled holes</td>
</tr>
<tr>
<td></td>
<td>$\alpha$</td>
<td>$F_{k} \leq 420 \text{ kgf/m}^2$</td>
</tr>
<tr>
<td></td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$5d$</td>
</tr>
<tr>
<td></td>
<td>$90^\circ \leq \alpha \leq 270^\circ$</td>
<td>$(10 + 5 \cos \alpha) d$</td>
</tr>
<tr>
<td></td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$d &lt; 5 \text{ mm}$</td>
</tr>
<tr>
<td></td>
<td>$d \geq 5 \text{ mm}$</td>
<td>$(5 + 5 \sin \alpha) d$</td>
</tr>
</tbody>
</table>
The characteristic withdrawal capacity $F_{ax,\text{fek}}$ should be taken as the smaller values from the following expressions:

- For nails other than smooth nails, as defined in EN 14592:

\[
F_{ax,\text{rk}} = \begin{cases} 
 f_{m,k} d t_{pm} & \text{(a)} \\
 f_{\text{head}} d_n^2 & \text{(b)} 
\end{cases}
\]  
\(8.23\)

- For smooth nails:

\[
F_{ax,\text{rk}} = \begin{cases} 
 f_{m,k} d t_{pm} & \text{(a)} \\
 f_{m,k} d t + f_{\text{head}} d_n^2 & \text{(b)} 
\end{cases}
\]  
\(8.24\)

where:
- $f_{m,k}$ is the characteristic pointside withdrawal strength;
- $f_{\text{head}}$ is the characteristic headside pull-through strength;
- $d$ is the nail diameter according to 8.3.1.1;
- $t_{pm}$ is the pointside penetration length or the length of the threaded part in the pointside member;
- $t$ is the thickness of the headside member;
- $d_n$ is the nail head diameter.
Characteristic pointside withdrawal strength $f_{ax,k}$ and characteristic headside pull-through strength $f_{head,k}$

These values can be determined in two ways, either

- By tests in accordance with EN 1382, EN 1383 and EN 14258
- For smooth nails with pointside penetration of at least $12d$

$$f_{ax,k} = 20 \times 10^{-6} \rho_k^2$$
$$f_{head,k} = 70 \times 10^{-3} \rho_k^2$$

where:

$\rho_k$ is the characteristic timber density in kg/m³.

The following recommendations should be observed:

**Nails with different pointside penetration depths**

- For smooth nails: pointside penetration $t_{pen} \geq 8d$
- For nails with a pointside penetration $t_{pen} < 12d$, the withdrawal capacity should be multiplied by $\frac{t_{pen}}{4d - 2}$
- For threaded nails: pointside penetration: $t_{pen} \geq 6d$
- For nails with $t_{pen} < 8d$ the withdrawal capacity should be multiplied by $\frac{t_{pen}}{2d - 3}$

**Structural timber installed wet**

- Timber installed near fibre saturation point (moisture content at which wood cells are completely saturated), which is likely to dry out under load

  - $f_{ax,k} \times \frac{2}{3}$
  - $f_{head,k} \times \frac{2}{3}$

**Spacings, end and edge distances**

- same requirements for laterally and axially-loaded nails

**Slant nailing**

- Distance to the loaded edge at least $10d$
- At least to slant nails per connections
8.3.3 **COMBINED LATERALLY AND AXIALLY-LOADED NAILS**

Connections subjected to combinations of axial load \( (F_{ax,Ed}) \) and lateral load \( (F_{v,Ed}) \) the following should be satisfied:

- for smooth nails:
  \[
  \frac{F_{ax,Ed}}{F_{ax,Ed}} \frac{F_{v,Ed}}{F_{v,Ed}} \leq 1
  \]

- for nails other than smooth nails, as defined in EN 14592:
  \[
  \left( \frac{F_{ax,Ed}}{F_{ax,Ed}} \right)^2 \left( \frac{F_{v,Ed}}{F_{v,Ed}} \right)^2 \leq 1
  \]

\( F_{ax,Ed} \) and \( F_{v,Ed} \) are the design load-carrying capacities of the connection loaded with axial load or lateral load respectively.

8.4 **Stapled connections**

EN 1995-1 covers the design with staples and in clauses (1) to (8) the following staples and conditions are covered.

**Shape and number of staples in connections**

- EN 1995-1-1 covers design with staples which are round, nearly round or rectangular in shape with bevelled or symmetrical pointed legs. These staples can be designed as nails and the previous section 8.3 does apply except for clauses 8.3.1.1(5) and (6) and 8.3.1.2(7) – Clause (1)

<table>
<thead>
<tr>
<th>Bevelled pointed legs</th>
<th>Symmetrical pointed legs</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1" alt="Bevelled pointed legs" /></td>
<td><img src="image2" alt="Symmetrical pointed legs" /></td>
</tr>
</tbody>
</table>

There should always be at least two staples in a connection – Clause (4)

**Minimum dimensions of staples**

- The diameter \( d \) of the staple is determined by taking the square root of the product of both dimensions.
The width $b$ of the staple crown should be at least $6d$, and the pointside penetration length $t_2$ should be at least $14d$. (Clause (3))
Material properties and load-carrying capacities – requirements and assumptions:

- Staples should be produced from wire with a minimum tensile strength of 800 N/mm²

- The characteristic yield moment $M_{y,Rk}$ that should be used per leg is determined using $d$:

  $$M_{y,Rk} = 240 \cdot d^{2.6}$$  \hspace{1cm} (8.29)

  where:
  
  - $M_{y,Rk}$ is the characteristic yield moment, in Nmm;
  - $d$ is the staple leg diameter, in mm.

- The lateral design load-carrying capacity per staple per shear plane $F_{v,Rk}$ should be considered as; either

  - equivalent to that of two nails with the staple diameter, IF the angle between the crown and the direction of the grain of the timber under the crown is greater than 30°

  - 0.7 of that of two nails, IF the angle between the crown and the direction of the grain under the crown is equal to or less than 30°
Effective number of fasteners $n_{ef}$

For a row of $n$ staples parallel to the grain, the load-carrying capacity in that direction should be calculated using the effective number of fasteners $n_{ef}$ (according to 8.3.1.1(8), below)

$\begin{equation}
  n_{ef} = n^{k_{ef}}
\end{equation}
$

where:

- $n_{ef}$ is the effective number of nails in the row;
- $n$ is the number of nails in a row;
- $k_{ef}$ is given in Table 8.1.

### Table 8.1 – Values of $k_{ef}$

<table>
<thead>
<tr>
<th>Spacing $^a$</th>
<th>Not predrilled</th>
<th>Predrilled</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha &gt; 14,^\circ$</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>$\alpha = 10,^\circ$</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>$\alpha = 7,^\circ$</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>$\alpha = 4,^\circ$</td>
<td>-</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*For intermediate spacings, linear interpolation of $k_{ef}$ is permitted.

![Diagram](image)

Key:
1. Nail
2. Grain direction

Figure 8.6 – Nails in a row parallel to grain staggered perpendicular to grain by $\alpha$

Reference to 8.1.2 (4)

$F_{\text{req},k} = n_{ef}F_{\text{v,eff},k}$

where:

- $F_{\text{v,eff},k}$ is the effective characteristic load-carrying capacity of one row of fasteners parallel to the grain;
- $n_{ef}$ is the effective number of fasteners in line parallel to the grain;
- $F_{\text{v,eff},k}$ is the characteristic load-carrying capacity of each fastener parallel to the grain.

NOTE: Values of $n_{ef}$ for rows parallel to grain are given in 8.3.1.1(6) and 6.5.1.1(5).
Spacings, edge and end distances

Minimum staple spacings, edge and end distances are given in Table 8.3, and illustrated in Figure 8.10 where $\Theta$ is the angle between the staple crown and the grain direction.

![Diagram](image)

Table 8.3 – Minimum spacings and edge and end distances for staples

<table>
<thead>
<tr>
<th>Spacing and edge/end distances (see Figure 8.7)</th>
<th>Angle $0^\circ \leq \alpha \leq 360^\circ$</th>
<th>Minimum spacing or edge/end distance $d$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_1$ (parallel to grain)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>for $\theta \geq 30^\circ$</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$(10 + 5 \cos \alpha) \cdot d$</td>
</tr>
<tr>
<td>for $\theta &lt; 30^\circ$</td>
<td></td>
<td>$(15 + 5 \cos \alpha) \cdot d$</td>
</tr>
<tr>
<td>$\alpha_2$ (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$15 \cdot d$</td>
</tr>
<tr>
<td>$\alpha_{3,1}$ (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$(15 + 5 \cos \alpha) \cdot d$</td>
</tr>
<tr>
<td>$\alpha_{3,e}$ (unloaded end)</td>
<td>$90^\circ \leq \alpha \leq 270^\circ$</td>
<td>$15 \cdot d$</td>
</tr>
<tr>
<td>$\alpha_{4,1}$ (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>$(15 + 5 \sin \alpha) \cdot d$</td>
</tr>
<tr>
<td>$\alpha_{4,e}$ (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$10 \cdot d$</td>
</tr>
</tbody>
</table>

8.5 Bolted connections

8.5.1 LATERALLY LOADED BOLTS

8.5.1.1 General and bolted timber-to-timber connections

**Characteristic yield moment of bolt $M_{y,Rk}$ – clause (1)**

(1) For bolts the following characteristic value for the yield moment should be used:

$$M_{y,Rk} = 0.3 \cdot f_{u,k} \cdot d^{2.6}$$  \hspace{1cm} (8.30)

where:

- $M_{y,Rk}$ is the characteristic value for the yield moment, in Nmm;
- $f_{u,k}$ is the characteristic tensile strength, in N/mm$^2$;
- $d$ is the bolt diameter, in mm.
Embedment strength values $f_{hak}$ for bolts $d \leq 30\text{mm}$ in timber and LVL, inserted at angle to the grain

$$f_{hak} = \frac{f_{hak}}{k_{so} \sin^2 \alpha - \cos^2 \alpha}$$  \hspace{1cm} (8.31)

$$f_{h0k} = 0.082(1 - 0.01d) \rho_k$$  \hspace{1cm} (8.32)

where:

$$k_{so} = \begin{cases} 1.35 + 0.015d & \text{for softwoods} \\ 1.30 + 0.015d & \text{for LVL} \\ 0.90 + 0.015d & \text{for hardwoods} \end{cases}$$  \hspace{1cm} (8.33)

and:

- $f_{h0k}$ is the characteristic embedment strength parallel to grain, in $\text{N/mm}^2$;
- $\rho_k$ is the characteristic timber density, in $\text{kg/m}^3$;
- $\alpha$ is the angle of the load to the grain;
- $d$ is the bolt diameter, in mm.

Minimum spacings, edge and end distances

![Diagram showing spacings and end and edge distances]

- $-90^\circ \leq \alpha \leq 90^\circ$
- $90^\circ \leq \alpha \leq 270^\circ$
- $0^\circ \leq \alpha \leq 180^\circ$
- $180^\circ \leq \alpha \leq 360^\circ$

Key:
1. Loaded end
2. Unloaded end
3. Loaded edge
4. Unloaded edge
1. Fastener
2. Grain direction

Figure 8.7 – Spacings and end and edge distances
(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge
Table 8.4 – Minimum values of spacing and edge and end distances for bolts

<table>
<thead>
<tr>
<th>Spacing and end/edge distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum spacing or distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_1 ) (parallel to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (4 + \cos \alpha) d )</td>
</tr>
<tr>
<td>( \alpha_2 ) (perpendicular to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 4 d )</td>
</tr>
<tr>
<td>( \alpha_{31} ) (loaded end)</td>
<td>( -90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( \max(7 d, 80 \text{ mm}) )</td>
</tr>
<tr>
<td>( \alpha_{3u} ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( \max(1 + 6 \sin \alpha, 4d) )</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( 4 d )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha \leq 270^\circ )</td>
<td>( \max(1 + 6 \sin \alpha, 4d) )</td>
</tr>
<tr>
<td>( \alpha_{41} ) (loaded edge)</td>
<td>( 0^\circ \leq \alpha \leq 180^\circ )</td>
<td>( \max(2 + 2 \sin \alpha, 3d) )</td>
</tr>
<tr>
<td>( \alpha_{4u} ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 3 d )</td>
</tr>
</tbody>
</table>

Effective number of bolts to be used for calculating the effective load-carrying capacity of the connection parallel to the grain, \( n_{\text{ef}} \)

\[ F_{v,\text{ef},Rk} = n_{\text{ef}}F_{v,Rk} \]  \hspace{1cm} (8.1)

where:
- \( F_{v,\text{ef},Rk} \) is the effective characteristic load-carrying capacity of one row of fasteners parallel to the grain;
- \( n_{\text{ef}} \) is the effective number of fasteners in line parallel to the grain;
- \( F_{v,Rk} \) is the characteristic load-carrying capacity of each fastener parallel to the grain.

NOTE: Values of \( n_{\text{ef}} \) for rows parallel to grain are given in 8.3.1.1(8) and 8.5.1.1(5).

\( n_{\text{ef}} \)

\[ n_{\text{ef}} = \min \left( n, \frac{n}{\sqrt{\frac{\alpha_1}{13d}}} \right) \] \hspace{1cm} (8.34)

where:
- \( \alpha_1 \) is the spacing between bolts in the grain direction;
- \( d \) is the bolt diameter;
- \( n \) is the number of bolts in the row.

For loads perpendicular to grain, the effective number of fasteners should be taken as

\[ n_{\text{ef}} = n \] \hspace{1cm} (8.35)

For angles \( 0^\circ < \alpha < 90^\circ \) between load and grain direction, \( n_{\text{ef}} \) may be determined by linear interpolation between expressions (8.34) and (8.35).

Minimum washer dimensions

(1) Bolt holes in timber should have a diameter not more than 1 mm larger than the bolt. Bolt holes in steel plates should have a diameter not more than 2 mm or 0.1d (whichever is the greater) larger than the bolt diameter \( d \).
(2) Washers with a side length or a diameter of at least $3d$ and a thickness of at least $0.3d$ should be used under the head and nut. Washers should have a full bearing area.

(3) Bolts and lag screws should be tightened so that the members fit closely, and they should be re-tightened if necessary when the timber has reached equilibrium moisture content to ensure that the load-carrying capacity and stiffness of the structure is maintained.

(4) The minimum diameter requirements given in Table 10.1 apply to bolts used with timber connectors, where:

$d_c$ is the connector diameter, in mm

$d$ is the bolt diameter, in mm

$d_1$ is the diameter of centre hole of connector

---

8.5.1.2 Bolted panel-to-timber connections

Embedment strengths $f_{b,k}$ at all angles

<table>
<thead>
<tr>
<th>Material</th>
<th>$f_{b,0,k}$</th>
<th>Definitions</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood</td>
<td>$f_{b,0,k} = 0.11 (1 - 0.01d) \rho_k$</td>
<td>$\rho_k$ is the characteristic plywood density, in kg/m$^3$; $d$ is the bolt diameter, in mm.</td>
<td>(8.36)</td>
</tr>
<tr>
<td>Particleboard/OSB</td>
<td>$f_{b,0,k} = 50 d^{0.8} r^2$</td>
<td>$d$ is the bolt diameter, in mm; $r$ is the panel thickness, in mm.</td>
<td>(8.37)</td>
</tr>
</tbody>
</table>

8.5.1.3 Bolted steel-to-timber connections

See section 8.3.2

8.5.2 AXIALLY-LOADED BOLTS

Taken from EN 1995-1-1

(1) The axial load-bearing capacity and withdrawal capacity of a bolt should be taken as the lower value of:

– the bolt tensile capacity

– the load-bearing capacity of either the washer or (for steel-to-timber connections) the steel plate.
(2) The bearing capacity of a washer should be calculated assuming a characteristic compressive strength on the contact area of $3,0f_{c,90,k}$.

(3) The bearing capacity per bolt of a steel plate should not exceed that of a circular washer with a diameter which is the minimum of:

- $12t$, where $t$ is the plate thickness
- $4d$, where $d$ is the bolt diameter.

8.6 Dowelled connections

**Minimum diameter of dowels**

$6mm < d < 30mm$

**Design rules**

Follow procedures for laterally-loaded bolted connections, section 8.5.1, but use minimum spacings, edge and end distances given below.

**Minimum spacings, edge and end distances**

![Diagram of dowelled connections with annotations for spacings and angles](image)

- $90^\circ \leq \alpha \leq 270^\circ$
- $0^\circ \leq \alpha \leq 180^\circ$
- $180^\circ \leq \alpha \leq 360^\circ$

Key:

1. Fastener
2. Grain direction

Figure 8.7 – Spacings and end and edge distances

(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge
Dowel hole tolerances (Section 10.4.4)

The minimum dowel diameter should be 6 mm. The tolerances on the dowel diameter should be −0/+0.1 mm. Pre-bored holes in the timber members should have a diameter not greater than that of the dowel.

### 8.7 Screwed connections

#### 8.7.1 LATERALLY-LOADED SCREWS

**Design dimensions**

Two types of screws are considered in EN 1995-1-1- clauses (1) and (2)

- threaded screw

- smooth shank screw
  - The threaded part of the screw shall be taken into account as an effective diameter $d_{eff}$ to determining the load-carrying capacity
  - For smooth shank screws (outer thread diameter is equal to shank diameter) all design rules in 8.2 apply.

$$d_{eff} \text{ taken as the smooth shank diameter}$$

- Other screws: Load-carrying capacity should be calculated using $d_{eff} = 1.1 \times$ the thread root diameter

- Smooth shank screws with $d > 6\text{mm}$ ➤ see section 8.5.1

- Smooth shank screws with $d \leq 6\text{mm}$ ➤ see section 8.3.1

<table>
<thead>
<tr>
<th>Spacing and edge/end distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum spacing or edge/end distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_1$ (parallel to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$(3 + 2 \cdot \cos \alpha) \cdot d$</td>
</tr>
<tr>
<td>$\alpha_2$ (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha \leq 360^\circ$</td>
<td>$3 \cdot d$</td>
</tr>
<tr>
<td>$\alpha_3$, (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$\max (7 \cdot d, 80 \text{mm})$</td>
</tr>
<tr>
<td>$\alpha_3$, (unloaded end)</td>
<td>$90^\circ \leq \alpha &lt; 150^\circ$</td>
<td>$\max (\alpha_{\text{th}} \cdot \sin \alpha) \cdot d, 3d$</td>
</tr>
<tr>
<td></td>
<td>$150^\circ \leq \alpha &lt; 210^\circ$</td>
<td>$3 \cdot d$</td>
</tr>
<tr>
<td></td>
<td>$210^\circ \leq \alpha &lt; 270^\circ$</td>
<td>$\max (\alpha_{\text{th}} \cdot \sin \alpha) \cdot d, 3d$</td>
</tr>
<tr>
<td>$\alpha_4$, (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>$\max (2 + 2 \cdot \sin \alpha) \cdot d, 3d$</td>
</tr>
<tr>
<td>$\alpha_4$, (unloaded edge)</td>
<td>$180^\circ \leq \alpha &lt; 360^\circ$</td>
<td>$3 \cdot d$</td>
</tr>
</tbody>
</table>
Requirements for structural detailing and control (EN 1995-1-1 Section 10.4.5)

(1) For screws in softwoods with a smooth shank diameter \( d \leq 6\text{mm} \) pre-drilling is not required. For all screws in hardwoods and for screws in softwoods with a diameter \( d > 6\text{mm} \), pre-drilling is required, with the following requirements:

- The lead hole for the shank should have the same diameter as the shank and the same depth as the length of the shank.
- The lead hole for the threaded portion should have a diameter of approximately 70% of the shank diameter.

(2) For timber densities greater than 500 kg/m³, the pre-drilling diameter should be determined by tests.

8.7.2 AXIALLY-LOADED SCREWS

Failure modes – clause (1)

Failure modes should be verified when assessing the load-carrying capacity of the connections with axially-loaded screws. The following need to be recorded:

- the withdrawal capacity of the threaded part of the screw
- for screws used in combination with steel plates, the tear-off capacity of the screw head should be greater than the tensile strength of the screw
- the pull-through strength of the screw head
- the tension strength of the screw
- for screws used in conjunction with steel plates, failure along the circumference of a group of screws (block shear or plug shear)

Minimum spacing, edge and end distances – clause (2)

<table>
<thead>
<tr>
<th>Screws driven</th>
<th>Minimum spacing</th>
<th>Minimum edge distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>At right angle to the grain</td>
<td>( 4d )</td>
<td>( 4d )</td>
</tr>
<tr>
<td>In end grain</td>
<td>( 4d )</td>
<td>( 2.5d )</td>
</tr>
</tbody>
</table>

Pointside penetration length

\[ t_2 = 6d \]
Characteristic withdrawal capacity $F_{ax,α,Rk}$ – clause (4)

$$F_{ax,α,Rk} = n_d (\pi d l_d)^{0.5} f_{ax,k}$$  \hspace{1cm} (8.38)

where:

- $F_{ax,α,Rk}$ is the characteristic withdrawal capacity of the connection at an angle $α$ to the grain;
- $n_d$ is the effective number of screws;
- $d$ is the outer diameter measured on the threaded part;
- $l_d$ is the pointside penetration length of the threaded part minus one screw diameter;
- $f_{ax,k}$ is the characteristic withdrawal strength at an angle $α$ to the grain.

where

$$f_{ax,k} = \frac{f_{ax,1}}{\sin^2 α + 1.5 \cos^2 α}$$  \hspace{1cm} (8.39)

$$f_{ax,1} = 3.6 \times 10^3 p_k^{1/6}$$  \hspace{1cm} (8.40)

where:

- $f_{ax,k}$ is the characteristic withdrawal strength at an angle $α$ to the grain;
- $f_{ax,1}$ is the characteristic withdrawal strength perpendicular to the grain;
- $p_k$ is the characteristic density, in kg/m$^3$.

Please note the above formula is in the process of being amended by TC250/SC5:

$$F_{ax,α, Rk} = n_{ef} \left( \frac{8d}{l_{ef}} \right)^{0.2} f_{ax, α, k} dl_{ef}$$

where the characteristic withdrawal strength perpendicular to grain $f_{ax,k}$ in N/mm$^2$, should be determined either by testing in accordance with EN 1382 and EN 14358 or calculated from the following expression:

$$f_{ax,k} = 0.037 p_k$$

Pull-through capacity of screw head – clause (6)

To be determined by tests to EN 1383.

Effective number of screws – clause (7)

Effective screws loaded parallel to the shank.

$$n_{el} = n^{0.9}$$  \hspace{1cm} (8.41)

where:

- $n_{el}$ is the effective number of screws;
- $n$ is the number of screws acting together in a connection.
8.7.3 COMBINED LATERALLY AND AXIALLY-LOADED SCREWS

Screwed connections subjected to a combination of axial and lateral load

- for nails other than smooth nails, as defined in EN 14592:

\[
\left( \frac{F_{x,Ed}}{F_{y,Ed}} \right)^2 + \left( \frac{F_{y,Ed}}{F_{x,Ed}} \right)^2 \leq 1
\]  

(8.28)

where:

- \(F_{x,Ed}\) and \(F_{y,Ed}\) are the design load-carrying capacities of the connection loaded with axial load or lateral load respectively.

8.8 Connections made with punched metal and plate fasteners

8.8.1 GENERAL

Connections with punched metal plate fasteners shall comprise plates of the same size, orientation on each side of the timber member – Clause (1)

All rules below apply to punched metal plate fasteners with two orthogonal directions – Clause (2)

8.8.2 PLATE GEOMETRY

Symbols used

- \(x\)-direction main direction of plate;
- \(y\)-direction perpendicular to the main plate direction;
- \(\alpha\) angle between the \(x\)-direction and the force (tension: \(0^\circ \leq \gamma < 90^\circ\), compression: \(90^\circ \leq \gamma < 180^\circ\));
- \(\beta\) angle between the grain-direction and the force;
- \(\gamma\) angle between the \(x\)-direction and the connection line;
- \(A_d\) area of the total contact surface between the plate and the timber, reduced by 5 mm from the edges of the timber and by a distance in the grain direction from the end of timber equal to 6 times the fastener’s nominal thickness;
- \(i\) dimension of the plate measured along the connection line.
8.8.3 PLATE STRENGTH PROPERTIES

The following properties need to be determined in accordance with EN 14545 from tests carried out in accordance with EN 1705.

- $f_{k,0,0}$ the anchorage capacity per unit area for $\alpha = 0^\circ$ and $\beta = 0^\circ$;
- $f_{k,0,90}$ the anchorage capacity per unit area for $\alpha = 90^\circ$ and $\beta = 90^\circ$;
- $f_{s,0}$ the tension capacity per unit width of plate for $\alpha = 0^\circ$;
- $f_{s,90}$ the compression capacity per unit width of plate for $\alpha = 90^\circ$;
- $f_{s,0}$ the shear capacity per unit width of plate in the x-direction;
- $f_{s,90}$ the tension capacity per unit width of plate in the y-direction;
- $k_1, k_2, \alpha_0$ constants.

**Design capacities**

Design tension, compression and shear capacity of the plate is calculated using $k_{\text{mod}} = 1.0$

\[ X_d = k_{\text{mod}} \frac{X_k}{\gamma_M} \quad (2.9) \]

where:

- $X_k$ is the characteristic value of a strength property,
- $\gamma_M$ is the partial factor for a material property,
- $k_{\text{mod}}$ is a modification factor taking into account the effect of the duration of load and moisture content.

**NOTE 1:** Values of $k_{\text{mod}}$ are given in 3.1.3.

**NOTE 2:** The recommended partial factors for material properties ($\gamma_M$) are given in Table 2.3. Information on the National choice may be found in the National annex.
8.8.4 PLATE ANCHORAGE STRENGTHS

Characteristic anchorage strength $f_{a, \alpha, \beta, k}$ per plate

This can either be determined from tests or calculated from

$$f_{a, \alpha, \beta, k} = \max \left( f_{a,0,0,k} - \left( f_{a,0,0,k} - f_{a,0,0,0,k} \right) \frac{\beta}{45^\circ} \right)$$

for $\beta \leq 45^\circ$, or

$$f_{a,0,0,k} - \left( f_{a,0,0,k} - f_{a,0,0,0,k} \right) \sin(\max(\alpha, \beta))$$

for $45^\circ < \beta \leq 90^\circ$ \hspace{1cm} (8.42)

$$f_{a,\alpha,\beta,0} = f_{a,0,0,k} - \left( f_{a,0,0,k} - f_{a,0,0,0,k} \right) \sin(\max(\alpha, \beta))$$

(2) The characteristic anchorage strength per plate parallel to grain should be taken as:

$$f_{a,\alpha,0,k} = \begin{cases} f_{a,0,0,k} + k_1 \alpha & \text{when } \alpha \leq \alpha_0 \\ f_{a,0,0,k} + k_1 \alpha_0 + k_2 (\alpha - \alpha_0) & \text{when } \alpha_0 < \alpha \leq 90^\circ \end{cases}$$

(8.44)

The constants $k_1$, $k_2$ and $\alpha_0$ should be determined from anchorage tests in accordance with EN 1075 and derived in accordance with the procedure given in EN 14545 for the actual plate type.

8.8.5 CONNECTION STRENGTH VERIFICATION

8.8.5.1 Plate anchorage capacity

Design anchorage stress $\tau_{Ed}$ or $\tau_{M,d}$

$\tau_{Ed}$ is the stress imposed by a force $F_{Ed}$

$\tau_{M,d}$ is the stress imposed from a moment $M_{Ed}$
Accounting for contact pressure between timber members

Contact pressure between timber members can be taken into account to reduce $F_{Ed}$. The gap between members shall not be greater than 1.5 mm and a maximum value of 3 mm.

(2) As an alternative to expression (8.47), $W_p$ may be conservatively approximated from:

$$W_p = \frac{A_{ef} \cdot d}{4} \tag{8.48}$$

with:

$$d = \sqrt{\frac{A_{ef}}{h_{ef}}} + h_{ef}^2 \tag{8.49}$$

where:

$h_{ef}$ is the maximum height of the effective anchorage area perpendicular to the longest side.

Accounting for contact pressure between timber members

Contact pressure between timber members can be taken into account to reduce $F_{Ed}$. The gap between members shall not be greater than 1.5 mm and a maximum value of 3 mm.

(4) Contact pressure between the timber members in chord splices in compression may be taken into account by designing the single plate for a design force, $F_{A Ed}$, and a design moment $M_{A Ed}$, according to the following expressions:

$$F_{A Ed} = \sqrt{\frac{F_{Ed}}{2}} \left( \cos \beta + \frac{3M_{Ed}}{2h} \right)^2 + (f_{Ed} \sin \beta)^2 \tag{8.50}$$

$$M_{A Ed} = \frac{M_{Ed}}{2} \tag{8.51}$$

where:

$F_{Ed}$ is the design axial force of the chord acting on a single plate (compression or zero);

$M_{Ed}$ is the design moment of the chord acting on a single plate;

$h$ is the height of the chord.
Interaction

(5) The following expression should be satisfied:

\[ \left( \frac{f_{\text{F,Ed}}}{f_{\text{s,Ed}}} \right)^2 + \left( \frac{f_{\text{M,Ed}}}{f_{\text{k,Ed}}} \right)^2 \leq 1 \]  

(3.52)

8.8.5.2 Plate capacity

Determine forces in two main directions

\[ F_{x,\text{Ed}} = F_{\text{Ed}} \cos \alpha \pm 2F_{\text{M,Ed}} \sin \gamma \]  

(8.53)

\[ F_{y,\text{Ed}} = F_{\text{Ed}} \sin \alpha \pm 2F_{\text{M,Ed}} \cos \gamma \]  

(8.54)

where:

- \( F_{\text{Ed}} \) is the design force in a single plate (i.e., half of the total force in the timber member)
- \( F_{\text{M,Ed}} \) is the design force from the moment on a single plate \( F_{\text{M,Ed}} = 2M_{\text{Ed}}/l \)

Satisfy

\[ \left( \frac{F_{x,\text{Ed}}}{F_{x,\text{RL}}} \right)^2 + \left( \frac{F_{y,\text{Ed}}}{F_{y,\text{RL}}} \right)^2 \leq 1 \]  

(8.55)

where:

- \( F_{\text{x,Ed}} \) and \( F_{\text{y,Ed}} \) are the design forces acting in the x and y direction,
- \( F_{\text{x,RL}} \) and \( F_{\text{y,RL}} \) are the corresponding design values of the plate capacity. They are determined from the maximum of the characteristic capacities at sections parallel or perpendicular to the main axes, based upon the following expressions for the characteristic plate capacities in these directions

\[ F_{x,\text{RL}} = \max \left( \frac{f_{x,0,k}}{f_{x,0,k}} \cos \gamma \right) \]  

(8.56)

\[ F_{y,\text{RL}} = \max \left( \frac{f_{y,0,k}}{f_{y,0,k}} \sin \gamma \right) \]  

(8.57)

with

\[ f_{\text{N},0,k} = \begin{cases} f_{\text{x},k} & \text{for } F_{\text{x},\text{Ed}} > 0 \\ f_{\text{y},k} & \text{for } F_{\text{x},\text{Ed}} \leq 0 \end{cases} \]  

(8.58)

\[ f_{\text{M},0,k} = \begin{cases} f_{\text{x},0,k} & \text{for } F_{\text{x},\text{Ed}} > 0 \\ f_{\text{y},0,k} & \text{for } F_{\text{x},\text{Ed}} \leq 0 \end{cases} \]  

(8.59)

\[ k = \begin{cases} 1 + \frac{1}{2} \sin(2\gamma) & \text{for } F_{\text{x},\text{Ed}} > 0 \\ 1 & \text{for } F_{\text{x},\text{Ed}} \leq 0 \end{cases} \]  

(8.60)

where \( \gamma_0 \) and \( k \) are constants determined from shear tests in accordance with EN 1075 and derived in accordance with the procedure given in EN 14545 for the actual plate type.

(3) If the plate covers more than two connection lines on the member then the forces in each straight part of the connection line should be determined such that equilibrium is fulfilled and that expression (8.55) is satisfied in each straight part of the connection line. All critical sections should be taken into account.
8.9 Split ring and shear plate connectors

Figure 8.12 – Dimensions for connections with split ring and shear plate connectors

Dimension of connection

(2) The minimum thickness of the outer timber members should be $2.25h_c$, and of the inner timber member should be $3.75h_c$, where $h_c$ is the embedment depth, see Figure 8.12.

$t_1 \geq 2.25h_c$

$t_2 \geq 3.75h_c$

Characteristic load-carrying capacity parallel to grain $F_{v,0,Rk}$ per connector per shear plane

For connections with one connector per shear plane loaded in an unloaded end situation ($150^\circ \leq \alpha \leq 210^\circ$), the condition (a) in expression (8.61) should be disregarded.

\[
F_{v,0,Rk} = \min \left\{ \frac{k_1}{k_2} \frac{k_3}{k_4} \frac{k_5}{k_6} \right\} \quad (a) \]

\[
F_{v,0,Rk} = \frac{k_1}{k_2} \frac{k_3}{k_4} \frac{k_5}{k_6} \quad (b)
\]

where:

- $F_{v,0,Rk}$ is the characteristic load-carrying capacity parallel to the grain, in N;
- $d_c$ is the connector diameter, in mm;
- $h_c$ is the embedment depth, in mm;
- $k_i$ are modification factors, with $i = 1$ to 4, defined below.

\[
k_1 = \min \left\{ \frac{1}{k_3} \frac{1}{3k_4} \frac{1}{5k_6} \right\} \quad (8.62)
\]
(4) The factor $k_2$ applies to a loaded end ($-30^\circ \leq \alpha \leq 30^\circ$) and should be taken as:

$$k_2 = \min \left\{ \frac{d_{\alpha}}{2d_c} \right\} \quad (8.63)$$

where:

$$k_\alpha = \begin{cases} 1.25 & \text{for connections with one connector per shear plane} \\ 1.0 & \text{for connections with more than one connector per shear plane} \end{cases} \quad (8.64)$$

$\alpha_\alpha$ is given in Table 8.7.

For other values of $\alpha$, $k_2 = 1.0$.

Table 8.7 – Minimum spacings and edge and end distances for ring and shear plate connectors.

<table>
<thead>
<tr>
<th>Spacing and edge/end distances (see Figure 8.7)</th>
<th>Angle to grain</th>
<th>Minimum spacings and edge/end distances</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_1$ (parallel to grain)</td>
<td>$0^\circ \leq \alpha &lt; 360^\circ$</td>
<td>$(1.2 + 0.8</td>
</tr>
<tr>
<td>$\alpha_2$ (perpendicular to grain)</td>
<td>$0^\circ \leq \alpha &lt; 360^\circ$</td>
<td>$1.2 \cdot d_c$</td>
</tr>
<tr>
<td>$\alpha_3$ (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$1.5 \cdot d_c$</td>
</tr>
<tr>
<td>$\alpha_4$ (unloaded end)</td>
<td>$90^\circ \leq \alpha &lt; 150^\circ$</td>
<td>$(0.4 + 1.6</td>
</tr>
<tr>
<td></td>
<td>$150^\circ \leq \alpha &lt; 210^\circ$</td>
<td>$1.2 \cdot d_c$</td>
</tr>
<tr>
<td></td>
<td>$210^\circ \leq \alpha &lt; 270^\circ$</td>
<td>$(0.4 + 1.6</td>
</tr>
<tr>
<td>$\alpha_4$ (loaded edge)</td>
<td>$0^\circ \leq \alpha \leq 180^\circ$</td>
<td>$(0.6 + 0.2</td>
</tr>
<tr>
<td>$\alpha_5$ (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$0.6 \cdot d_c$</td>
</tr>
</tbody>
</table>

(5) The factor $k_3$ should be taken as:

$$k_3 = \min \left\{ \frac{1.75}{\rho_k} \right\} \quad (8.65)$$

where $\rho_k$ is the characteristic density of the timber, in kg/m³.

(6) The factor $k_4$, which depends on the materials connected, should be taken as:

$$k_4 = \begin{cases} 1.0 & \text{for timber-to-timber connections} \\ 1.1 & \text{for steel-to-timber connections} \end{cases} \quad (8.66)$$

**For forces at angle $\alpha$ to the grain, $F_{\alpha,Rk}$ per connector, per shear plane**

$$F_{\alpha,Rk} = \frac{F_{\alpha,Rk_{\parallel}}}{k_{\alpha} \sin^2 \alpha + \cos^2 \alpha} \quad (8.67)$$

with:

$$k_{\alpha} = 1.3 + 0.001d_c \quad (8.68)$$

where:

- $F_{\alpha,Rk_{\parallel}}$ is the characteristic load-carrying capacity of the connector for a force parallel to grain according to expression (8.61).
- $d_c$ is the connector diameter, in mm.
Minimum spacings, edge and end distances

Table 8.7 – Minimum spacings and edge and end distances for ring and shear plate connectors.

<table>
<thead>
<tr>
<th>Spacing and edge/end distances (see Figure 8.7)</th>
<th>Angle to grain</th>
<th>Minimum spacings and edge/end distances</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_1$ (parallel to grain)</td>
<td>$0 \leq \alpha \leq 360^\circ$</td>
<td>$(1.2 + 0.8</td>
</tr>
<tr>
<td>$d_2$ (perpendicular to grain)</td>
<td>$0 \leq \alpha \leq 360^\circ$</td>
<td>$1.2 d_c$</td>
</tr>
<tr>
<td>$d_{3l}$ (loaded end)</td>
<td>$-90^\circ \leq \alpha \leq 90^\circ$</td>
<td>$1.5 d_c$</td>
</tr>
<tr>
<td>$d_{3u}$ (unloaded end)</td>
<td>$90^\circ \leq \alpha &lt; 150^\circ$</td>
<td>$(0.4 + 1.6</td>
</tr>
<tr>
<td></td>
<td>$150^\circ \leq \alpha &lt; 210^\circ$</td>
<td>$1.2 d_c$</td>
</tr>
<tr>
<td></td>
<td>$210^\circ \leq \alpha \leq 270^\circ$</td>
<td>$(0.4 + 1.6</td>
</tr>
<tr>
<td>$d_{4l}$ (loaded edge)</td>
<td>$0 \leq \alpha \leq 180^\circ$</td>
<td>$(0.6 + 0.2</td>
</tr>
<tr>
<td>$d_{4u}$ (unloaded edge)</td>
<td>$180^\circ \leq \alpha \leq 360^\circ$</td>
<td>$0.6 d_c$</td>
</tr>
</tbody>
</table>

Figure 8.7 – Spacings and end and edge distances
(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge
Staggered connectors

When connectors are staggered the minimum spacings parallel and perpendicular to grain must be observed and comply with the following expression:

\[(k_{a1})^2 + (k_{a2})^2 \geq 1\]

with \[0 \leq k_{a1} \leq 1\]
\[0 \leq k_{a2} \leq 1\]  \hspace{1cm} (8.66)

where:

\(k_{a2}\) is a reduction factor for the minimum distance \(a_2\) perpendicular to the grain.

Possibility to reduce spacing parallel to grain

(13) Connectors should be considered as positioned parallel to the grain where \(k_{a2} a_2 < 0,5 k_{a1} a_1\).

(11) The spacing parallel to grain, \(k_{a1} a_1\), may further be reduced by multiplication by a factor \(k_{c,red}\), with \(0,5 \leq k_{c,red} \leq 1,0\), provided that the load-carrying capacity is multiplied by a factor:

\[k_{c,red} = 0,2 + 0,8k_{c,red}\]  \hspace{1cm} (6.70)

Effective number of connectors

\[n_{e1} = 2 + \left(1 - \frac{n}{20}\right)(n - 2)\]  \hspace{1cm} (6.71)

where:

\(n_{e1}\) is the effective number of connectors;
\(n\) is the number of connectors in a line parallel to grain.
8.10 Toothed plated connectors

(1) The characteristic load-carrying capacity of connections made using toothed-plate connectors should be taken as the summation of the characteristic load-carrying capacity of the connectors themselves and the connecting bolts according to 8.5.

![Figure 8.12 – Dimensions for connections with split ring and shear plate connectors](image)

**Dimension of connection**

(2) The minimum thickness of the outer timber members should be $2.25h_c$, and of the inner timber member should be $3.75h_c$, where $h_c$ is the embedment depth, see Figure 8.12.

\[ t_1 \geq 2.25h_c \]
\[ t_2 \geq 3.75h_c \]

**Characteristic load-carrying capacity parallel to grain $F_{v,Rk}$ per tooth-plate connector per shear plane**

(2) The characteristic load-carrying capacity $F_{v,Rk}$ per toothed-plate connector for connectors of type C according to EN 912 (single-sided: type C2, C4, C7, C9, C11; double sided: type C1, C3, C5, C6, C8, C10) and EN 14545 should be taken as:

\[
F_{v,Rk} = \begin{cases} 
18 \ k_i k_i k_s \ d_i^{1.5} & \text{for single-sided types} \\
25 \ k_i k_i k_s \ d_i^{1.5} & \text{for double-sided types}
\end{cases}
\]

(8.72)

where:

- $F_{v,Rk}$ is the characteristic load-carrying capacity per toothed-plate connector, in N.
- $k_i$ are modification factors, with $i = 1$ to 3, defined below.
- $d_i$ is:
  - the toothed-plate connector diameter for types C1, C2, C5, C7, C10 and C11, in mm;
  - the toothed-plate connector side length for types C5, C8 and C9, in mm;
  - the square root of the product of both side lengths for types C3 and C4, in mm.
(4) The factor $k_1$ should be taken as:

$$k_1 = \min \left\{ \frac{1}{3l_t}, \frac{l_o}{5l_t} \right\}$$  \hspace{1cm} (8.73)

where:

- $l_t$ is the side member thickness;
- $l_o$ is the middle member thickness;
- $l_e$ is the tooth penetration depth, in mm.

(5) The factor $k_2$ should be taken as:

- For types C1 to C9:

$$k_2 = \min \left\{ \frac{1}{a_{31}}, \frac{1.5}{1.5} \right\}$$  \hspace{1cm} (8.74)

with

$$a_{31} = \max \left\{ \frac{1.1 d_t}{7d}, \frac{1.1 d_t}{80 \text{ mm}} \right\}$$  \hspace{1cm} (8.75)

where:

- $d_t$ is the bolt diameter, in mm;
- $d_o$ is explained in (2) above.

- For types C10 and C11:

$$k_2 = \min \left\{ \frac{1}{a_{31}}, \frac{2.0}{2.0} \right\}$$  \hspace{1cm} (8.76)

with

$$a_{31} = \max \left\{ \frac{1.5 d_t}{7d}, \frac{1.5 d_t}{80 \text{ mm}} \right\}$$  \hspace{1cm} (8.77)

where:

- $d$ is the bolt diameter in mm;
- $d_o$ is explained in (2) above.

(6) The factor $k_3$ should be taken as:

$$k_3 = \min \left\{ \frac{15}{\rho,} \right\}$$  \hspace{1cm} (8.78)

where $\rho$ is the characteristic density of the timber, in kg/m$^3$. 
Minimum spacings, edge and end distances

(7) For toothed-plate connector types C1 to C9, minimum spacings and edge and end distances should be taken from Table 8.8, with the symbols illustrated in Figure 8.7.

<table>
<thead>
<tr>
<th>Spacings and edge/end distances (see Figure 8.7)</th>
<th>Angle to grain</th>
<th>Minimum spacings and edge/end distances</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_1 ) (parallel to grain)</td>
<td>( 0 \leq \alpha \leq 360^\circ )</td>
<td>( (1.2 + 0.3 \cos \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_2 ) (perpendicular to grain)</td>
<td>( 0 \leq \alpha \leq 360^\circ )</td>
<td>( 1.2 \phi )</td>
</tr>
<tr>
<td>( \alpha_{3,1} ) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( 2.0 \phi )</td>
</tr>
<tr>
<td>( \alpha_{3,2} ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( (0.9 + 0.6 \sin \alpha) \phi )</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( 1.2 \phi )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (0.9 + 0.6 \sin \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_{4,1} ) (loaded edge)</td>
<td>( 0 \leq \alpha \leq 180^\circ )</td>
<td>( (0.6 + 0.2 \sin \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_{4,2} ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 0.6 \phi )</td>
</tr>
</tbody>
</table>

(8) For toothed-plate connector types C10 and C11, minimum spacing and edge and end distances should be taken from Table 8.9, with the symbols illustrated in Figure 8.7.

<table>
<thead>
<tr>
<th>Spacings and edge/end distances (see Figure 8.7)</th>
<th>Angle to grain</th>
<th>Minimum spacings and edge/end distances</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_1 ) (parallel to grain)</td>
<td>( 0 \leq \alpha \leq 360^\circ )</td>
<td>( (1.2 + 0.8 \cos \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_2 ) (perpendicular to grain)</td>
<td>( 0 \leq \alpha \leq 360^\circ )</td>
<td>( 1.2 \phi )</td>
</tr>
<tr>
<td>( \alpha_{3,1} ) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( 2.0 \phi )</td>
</tr>
<tr>
<td>( \alpha_{3,2} ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( (0.4 + 1.5 \sin \alpha) \phi )</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( 1.2 \phi )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (0.4 + 1.5 \sin \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_{4,1} ) (loaded edge)</td>
<td>( 0 \leq \alpha \leq 180^\circ )</td>
<td>( (0.6 + 0.2 \sin \alpha) \phi )</td>
</tr>
<tr>
<td>( \alpha_{4,2} ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 0.6 \phi )</td>
</tr>
</tbody>
</table>
Staggered connectors

When connectors are staggered, the minimum spacings parallel and perpendicular to grain must be observed and comply with the following expression:

\[(k_{a1})^2 + (k_{a2})^2 \geq 1\]  \[\text{with } \begin{cases} 0 \leq k_{a1} \leq 1 \\ 0 \leq k_{a2} \leq 1 \end{cases}\]  \hspace{1cm} (8.69)

where:

- \(k_{a1}\) is a reduction factor for the minimum distance \(a_1\) parallel to the grain;
- \(k_{a2}\) is a reduction factor for the minimum distance \(a_2\) perpendicular to the grain.

Figure 8.7 – Spacings and end and edge distances
(a) Spacing parallel to grain in a row and perpendicular to grain between rows, (b) Edge

Figure 8.13 – Reduced distances for connectors
When bolts are used with toothed-plate connectors

Please observe also (EN 1995-1-1: Section 10.4.3):

(1) Bolt holes in timber should have a diameter not more than 1 mm larger than the bolt. Bolt holes in steel plates should have a diameter not more than 2 mm or 0,1d (whichever is the greater) larger than the bolt diameter d.

(2) Washers with a side length or a diameter of at least 3d and a thickness of at least 0,3d should be used under the head and nut. Washers should have a full bearing area.

(3) Bolts and lag screws should be tightened so that the members fit closely, and they should be re-tightened if necessary when the timber has reached equilibrium moisture content to ensure that the load-carrying capacity and stiffness of the structure is maintained.

(4) The minimum diameter requirements given in Table 10.1 apply to bolts used with timber connectors, where:

\[
d_c \quad \text{is the connector diameter, in mm;}
\]
\[
d \quad \text{is the bolt diameter, in mm}
\]
\[
d_c \quad \text{is the diameter of centre hole of connector.}
\]

<table>
<thead>
<tr>
<th>Type of connector EN 912</th>
<th>(d_c)</th>
<th>(d) minimum</th>
<th>(d) maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 – A6</td>
<td>≤ 130</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>A1, A4, A6</td>
<td>&gt; 130</td>
<td>0,1 (d_c)</td>
<td>24</td>
</tr>
<tr>
<td>B</td>
<td>(d_c)</td>
<td>(d_c)</td>
<td></td>
</tr>
</tbody>
</table>

’sSection 9: Components and assemblies’
(Pages 89 – 104)

9.1 Components

9.1.1 GLUED THIN-WEBBED BEAMS

It is important to familiar yourself with Figure 9.1 regarding where the stresses are referred to specially at section 1-1 where rolling shear takes place. There are two types of beams shown – ‘I’ and ‘Box’ beams – but other types also exist.
It is vital that the manufacturing of these beams is carried out properly with adequate process and materials. There is a European Technical Approval Guidance ETAG … which should be complied with. However, this section sets certain design analyses and requirements which are self-explanatory. There are 1 Principle and 6 Application Rules in this section.

The main Principle Rule is number 6 which recommends that glued splices should have sufficient strength. The Application rules are all for checking various design criteria:

(1) Equations 9.1 to 9.4 should be used for checking stresses in the flanges when the beam is in bending:

\[
\begin{align*}
\sigma_{f,m,ext} & \leq f_{md} \\
\sigma_{t,ext} & \leq f_{td} \\
\sigma_{c,ext} & \leq k_c f_{c,d} \\
\sigma_{t,ext} & \leq f_{t,d}
\end{align*}
\]

where:
- \(\sigma_{f,m,ext}\) is the extreme fibre flange design compressive stress;
- \(\sigma_{t,ext}\) is the extreme fibre flange design tensile stress;
- \(\sigma_{c,ext}\) is the mean flange design compressive stress;
- \(\sigma_{t,ext}\) is the mean flange design tensile stress;
- \(k_c\) is a factor which takes into account lateral instability.

(2) The factor \(k_c\) can be found from 6.3.2 using:

\[
k_c = \sqrt{\frac{2}{\lambda}}
\]

where:
- \(\lambda\) is the distance between the sections where lateral deflection of the compressive flange is prevented;
- \(b\) is given in Figure 9.1.
(3) The tension and compression stresses in the web caused by bending should be:

\[
\sigma_{w,t,d} \leq f_{w,t,d} \quad \text{(9.6)}
\]

\[
\sigma_{w,c,d} \leq f_{w,c,d} \quad \text{(9.7)}
\]

where:

- \(\sigma_{w,t,d}\) and \(\sigma_{w,c,d}\) are the design compressive and tensile stresses in the webs;
- \(f_{w,t,d}\) and \(f_{w,c,d}\) are the design compressive and tensile bending strengths of the webs.

(4) The above design compression and tension stresses should be taken as the design bending strength.

(5) It would be useful to have the glued splices tested for strength.

(6) Bucking of web must be checked using the following:

\[
h_w \leq 70b_w \quad \text{(9.8)}
\]

and

\[
F_{w,Ed} \leq \begin{cases} 
  h_w h_w \left(1 + \frac{0.5(l_{t,c} + l_{t,c})}{h_w}\right) f_{c,0,d} & \text{for } h_w \leq 35b_w \\
  35 l_w^2 \left(1 + \frac{0.5(l_{t,c} + l_{t,c})}{h_w}\right) f_{c,0,d} & \text{for } 35b_w \leq h_w \leq 70b_w 
\end{cases} \quad \text{(9.9)}
\]

where:

- \(F_{w,Ed}\) is the design shear force acting on each web;
- \(h_w\) is the clear distance between flanges;
- \(h_{t,c}\) is the compressive flange depth;
- \(h_{t,c}\) is the tensile flange depth;
- \(b_w\) is the width of each web;
- \(f_{c,0,d}\) is the design panel shear strength.

(7) Stresses at the glue lines between the flanges and web(s) must be determined so that rolling shear can be checked using the following expressions. It is important to note that rolling shear of certain panel products (such as Finnish plywood) used as web members may occur at the first glue line of the board and not necessarily at the glue line between the flanges and webs. This usually depends on the face veneer grain direction to the span which in Finnish plywood’s case is perpendicular to the grain. Therefore, it should be noted that the rolling shear may occur in the first glue line of the panels if their face veneer grain direction is perpendicular to the span.
The ‘glued thin-flanged beams’ are usually referred to as ‘stressed-skins’ in the UK implying that most stresses take place at the skin of the structure, such as an aeroplane’s wing. The stressed skin structures are not commonly used in UK. However, their design must be carried out properly, especially when gluing is involved which plays a significant role in the structural performance of such elements. Familiarisation with Figure 9.2 is recommended:

Gluing on site is not recommended and the manufacturing and production of such structures must be carried out under proper control. ETAG 011 provides guidance for all the requirements for such structures/components.

(1) Figure 9.2 assumes that there is a linear variation of stresses across the depth of the component (i.e. like a ‘Box’ beam).

(2) This is a Principle rule which must be adhered to: when verifying strength, the non-uniform stress distribution in the skin due to shear lag and buckling must be considered.

(3) The structure should be considered as a series of ‘I’ or ‘U’ beams as shown in Figure 9.2 by finding out what the effective width \((b_{ef})\) of the top and bottom skins (flanges) are using the following equations:

\[
\tau_{\text{max,}\text{d}} \leq \begin{cases} 
   f_{c,90,\text{d}} & \text{for } h_t \leq 4 b_{\text{ef}} \\
   f_{c,90,\text{d}} \left( \frac{4b_{\text{ef}}}{h_t} \right)^{0.8} & \text{for } h_t > 4 b_{\text{ef}}
\end{cases}
\]  

(9.10)

where:

- \(\tau_{\text{max,}\text{d}}\) is the design shear stress at the sections 1-1, assuming a uniform stress distribution;
- \(f_{c,90,\text{d}}\) is the design planar (rolling) shear strength of the web;
- \(h_t\) is either \(b_{\text{ef}}\) or \(h_w\);
- \(b_{\text{ef}} = \frac{h_w}{2}\) for boxed beams
- \(b_{\text{ef}} = b_{w}/2\) for I-beams

\[9.11\]
Table 9.1 gives the values of shear lag and plate buckling for different types of skins (flanges) when the span of the beam is $l$:

<table>
<thead>
<tr>
<th>Flange material</th>
<th>Shear lag</th>
<th>Plate buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plywood, with grain direction in the outer plies:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Parallel to the webs</td>
<td>$0.1l$</td>
<td>$20h_l$</td>
</tr>
<tr>
<td>- Perpendicular to the webs</td>
<td>$0.1l$</td>
<td>$25h_l$</td>
</tr>
<tr>
<td>Oriented strand board</td>
<td>$0.15l$</td>
<td>$25h_l$</td>
</tr>
<tr>
<td>Particleboard or fibreboard with random fibre orientation</td>
<td>$0.2l$</td>
<td>$30h_l$</td>
</tr>
</tbody>
</table>

(5) All this Application Rule says is that $b_f \leq 2b_{cf}$ to prevent the plate buckling of the skins (flanges).

(6) The shear stresses at the glue lines between the skin and the web (section 1-1 of Figure 9.1) must be checked for rolling shear using the following equations:

$$
\tau_{\text{max},d} \leq \begin{cases} 
    f_{\tau,90,d} & \text{for } b_w \leq 8h_f \\
    f_{\tau,90,d} \left( \frac{8h_f}{b_w} \right)^{0.8} & \text{for } b_w > 8h_f 
\end{cases} \quad (9.14)
$$

where:

- $\tau_{\text{max},d}$ is the design shear stress at the sections 1-1, assuming a uniform stress distribution;
- $f_{\tau,90,d}$ is the design planar (rolling) shear strength of the flange.

For ‘U’ shaped components, $8b_f$ must be replaced by $4b_f$ in equation 9.14.

(7) Compressive and tensile stresses of the skins (flanges) must be checked by the following expressions using the effective width of the skins (flanges):
(8) It would be useful to have the glued splices tested for strength to comply with this Principle Rule.

(9) The tension and compression stresses in the webs caused by bending should be:

\[
\begin{align*}
\sigma_{t,\text{d}} & \leq f_{t,\text{d}} \quad (9.15) \\
\sigma_{c,\text{d}} & \leq f_{c,\text{d}} \quad (9.16)
\end{align*}
\]

where:

\(\sigma_{t,\text{d}}\) is the mean flange design compressive stress; \\
\(\sigma_{c,\text{d}}\) is the mean flange design tensile stress; \\
\(f_{t,\text{d}}\) is the flange design compressive strength; \\
\(f_{c,\text{d}}\) is the flange design tensile strength.

9.1.3 MECHANICALLY JOINTED BEAMS

(1) P and (2) All these clauses say is that consider slip any joint and assume a linear relation between slip and force causing the slip.

(3) If the spacing of the fasteners varies in the longitudinal direction according to the shear force between \(s_{\text{min}}\) and \(s_{\text{max}}\) (4 \(s_{\text{min}}\)), an effective \(s_{\text{ef}}\) may be used as follows:

\[
\begin{align*}
s_{\text{ef}} &= 0.75 s_{\text{min}} + 0.25 s_{\text{max}} \quad (9.17)
\end{align*}
\]

The method of calculating the load-carrying capacity of mechanically jointed beams is given in Annex B which is informative and recommended by the UK National Annex.

9.1.4 MECHANICALLY JOINTED AND GLUED COLUMNS

(1) P All this clause says is that the deformations due to slip in any type of joint in any parts of columns and beams must be considered.

The method of calculating the load-carrying capacity of I and box columns, spaced columns and lattice columns is given in Annex C which is informative and recommended by the UK National Annex.
9.2 Assemblies

9.2.1 TRUSSES

You do not really need this section because most of the design work usually is done by the system owners in UK. The system owners are the organisations that provide design software for truss rafter manufacturers in return for manufacturers buying the punched metal plates from them. They are usually a member of Trussed Rafter Associations (TRA) too. However, not all trussed rafter manufacturers or designers are a member of TRA. Therefore, for clarity this section is explained:

(1) For trusses loaded at the nodes, the following expressions should be used for combined bending and compression:

\[
\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 0.9
\]

\[
\left( \frac{\sigma_{c,0,d}}{f_{c,0,d}} \right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 0.9
\]

\( k_m \) accounts for two-axial bending effects, for solid, glue laminated and LVL timber. The factor accounts for linear interaction of stresses in sections subjected to bi-axial bending. When rectangular sections are subjected to bi-axial bending maximum stresses only occur in two corners of the cross section, and there only in the outermost fibre of the section. To account for the less likely occurrence of maximum stress being reached in both corners, the less critical corner stress (compared to uni-axial stress situation) can be reduced by the modification factor \( k_m \).

\( k_m \) accounts for two-axial bending effects, for solid, glue laminated and LVL timber:

\( k_m = 0.7 \) for rectangular sections (and \( \frac{h}{b} \leq 4 \) German guidance)

\( k_m = 1.0 \) for other cross-sections

(2) & (3) For compression members, the effective length should be taken as the distance between two points of contraflexure. However, in fully triangulated trusses, the effective length of compression members should be taken as the bay length (Figure 9.3) if:

- members are only one bay long, without rigid end connections
- members are continuous over two or more bays and are not loaded laterally.
(4) When using simplified analysis given in 5.4.3, the following effective column length may be assumed:

- continuous members without significant end moments and when bending stress of lateral load is \( \geq 0.4 \) compressive stress:
  - in an outer bay; \( 0.8 \times \text{bay length} \)
  - in an inner bay; \( 0.6 \times \text{bay length} \)
  - at a node; \( 0.6 \times \text{largest adjacent bay length} \)

- continuous members with significant end moments and when bending stress of lateral load is \( \geq 0.4 \) compressive stress:
  - at the beam end with moment; \( 0.0 \) (ie no column effect)
  - in the penultimate bay; \( 1.0 \times \text{bay length} \)
  - remaining bays and nodes; \( 0.8 \times \text{largest adjacent bay length} \)

- for all other cases; \( 1.0 \times \text{bay length} \)

For the strength verification of members in compression and connections, the calculated axial forces should be increased by 10%.

(5) When simplified analysis is used for trusses loaded at nodes, the ratio of tensile and compressive stresses as well as the connection capacity \( \leq 70\% \).

(6) P & (7)P These are Principle rules which are common sense i.e. we need to check out the plane stability of trusses and the connections must take forces during the transportation and erection.

(8) All joints should be capable of transferring a force \( F_{rd} \) acting in any direction within the plane of the truss where \( F_{rd} = 1.0 + 0.1L \), where \( L \) is the overall length of the truss in m and \( F_{rd} \) has short term duration acting in service class 2 and is in kN.
9.2.2 TRUSSES WITH PUNCHED METAL PLATE FASTENERS.

(1) The trusses must comply with the requirements of EN 14250.

(2) The requirements of 5.4.1 (a) Analysis should be using static models which consider the behaviour of structure and its supports with an acceptable level of accuracy. (b) The analysis of trussed rafters is performed by frame models and two methods are allowed: Rigorous (5.4.2) and simplified methods (5.4.3). (c) In addition to static linear analysis, this 'Application rule' allows elasto-plastic analysis for plane frames (portal frame) and arches.

The requirements of 9.2.1 above should also apply.

(3) When a concentrated perpendicular load of < 1.5 kN is applied to members of trusses and when $\sigma_{c,d} < 0.4 f_{c,d}$ and $\sigma_{t,d} < 0.4 f_{t,d}$ then the requirements of 6.2.3 and 6.2.4 may be replaced by:

$$\sigma_{u,d} \leq 0.75f_{u,d} \tag{9.10}$$

(4) Punched metal plates should overlap at least 40 mm or 1/3 of timber member height whichever is greater.

(5) Punched metal plate splices should cover at least 2/3 of the height of member.

9.2.3 ROOF AND FLOOR DIAPHRAGMS

9.2.3.1 General

Please note, it is not that often that we design floors and roofs as diaphragms. However, if needed for proof of stability then the following can be used. Please also note that EN 12872 provides all the recommendations for installation and performance requirements of floors walls and roofs.

Please note, it is always a good practice to stagger the sheets used for the diaphragm.
This section applies to floors and roofs acting as a diaphragm with their diaphragm being a type of wood-based panels fastened to the timber supports by mechanical fasteners. The load-carrying capacity of fasteners at the edges of sheets can be increased by 20% over the values given in Section 8.

### 9.2.3.2 Simplified analysis of roof and floor diaphragms

1. For diaphragms with a uniformly distributed load (Figure 9.4) this simplified analysis can be used provided that:
   - The span \( 2b < l < 6b \) where \( b \) is diaphragm width (Figure 9.4)
   - The design condition is failure at fasteners (not in the panel)
   - The panels are fixed in accordance with: nail spacings around the board/panel edges should be \( \leq 150 \text{mm} \). For within the boards/panels they should be \( \leq 300 \text{mm} \). Please note square-edge boards (Figure 10.1) must be supported at all their edges but long edges of T&G boards can be unsupported provided they cross the supporting joists or rafters and their short edges are supported by joists or rafters. Nails should comply with EN 14592.

There are many recommendations for using panel products as diaphragm/flooring given in EN 12871 which should be followed for best practice.
(2) & (3) Edge beams should be designed for maximum bending moment in
the diaphragm and shear forces should be assumed to act uniformly over
the width of diaphragm.

(4) It is always a good practice to stagger sheets (figure 9.4)

9.2.4 WALL DIAPHRAGMS

9.2.4.1 General
There are many issues in the design of wall panels that need to be considered.
However, not all UK design practice criteria are included in this Eurocode. The
readers are recommended to refer to NCCI for further guidance for additional
design criteria such as the effect of openings, plasterboard contribution,
shielding effect of brick cladding, contribution of racking resistance of brick
cladding to the overall racking resistance, etc.

(1)P: Both horizontal and vertical loads should be resisted by the wall panels.

(2)P: Wall panels should be adequately restrained against sliding and
overturning.

(3)P: This principle allows wall panels to resist in plane racking forces by the
sheathing, diagonal bracing or moment connections.

(4)P: The racking resistance of a wall panel can be obtained by using EN 594
test method or by calculations.

(5)P: The design of wall panel shall take account of both the material and
geometric make-up of the wall.

(6)P: Serviceability limits for wall panels must be met.

(7): There are two alternative methods of analysis given, namely Method A
(9.2.4.2) and Method B (9.2.4.3). **UK should use Method B.**

9.2.4.2 Simplified analysis of wall diaphragms – Method A
This method should be ignored.

9.2.4.3 Simplified analysis of wall diaphragms – Method B
9.2.4.3.1 Construction of walls and panels to meet the requirements of the
simplified analysis
(1): This Application Rule is just the description of the wall shown in Figure 9.7
with panels made of timber studs and panel sheathing.
(2) A panel is considered to contribute towards the racking resistance if the panel has a width of at least \( h/4 \) where \( h \) is the height of panel.

Nail spacings around the board/panel edges should be \( \leq 150\text{mm} \) and \( \leq 300\text{mm} \) within the boards/panels.

Screw spacings around the board/panel edges should be \( \leq 200\text{mm} \) and \( \leq 300\text{mm} \) within the boards/panels.

Again, best practice application guidance is given in EN 12871 which is recommended.

(3) Where an opening is formed within the panel, the widths of each panel at either side of the opening should be considered as a separate panel.

(4) Where panels are combined to form a wall:

- The tops of individual panels should be linked (ie a binder).
- Vertical connection between two panels should have a design strength of at least 2.5 kN/m.
• The wall should resist overturning and sliding forces by either anchorage or dead loads applied to the wall.

9.2.4.3.2 Design procedure

(1) All this paragraph says is that the racking strength \( F_{v,Rd} \) against a racking force of \( F_{v,Ed} \) (acting at top of a cantilevered wall that is secured against uplift and sliding by anchorage or vertical loads) must be calculated.

(2) The racking resistance of a wall consisting of several wall panels is the sum of racking resistance of wall panels:

\[
F_{v,Rd} = \sum F_{v,i,Rd}
\]  

where:
\( F_{v,i,Rd} \) is the design racking strength of a wall in accordance with (3) below.

(3) & (4): The design racking strength of a wall ‘i’ is:

\[
F_{v,i,Rd} = \frac{F_{tx,Rd} \cdot h}{z_b} \cdot k_i \cdot k_{a} \cdot k_{b}
\]  

where:
\( F_{tx,Rd} \) is the lateral design capacity of an individual fastener;
\( b_i \) is the wall length, in m;
\( z_b \) is the basic fastener spacing, see (4) below;
\( k_0 \) is the dimension factor for the panel, see (4) below;
\( k_a \) is the uniformly distributed load factor for wall ‘i’, see (4) below;
\( k_i \) is the fastener spacing factor, see (4) below;
\( k_b \) is the sheathing material factor, see (4) below.

\[
z_b = \frac{9700 \cdot d}{\lambda_1}
\]  

where:
\( d \) is the fastener diameter, in mm;
\( \lambda_1 \) is the characteristic density of the timber frame;

\[
k_i = \begin{cases} 
\left( \frac{b_i}{h} \right)^{0.4} & \text{for } \frac{b_i}{h} \leq 1.0 \\
\left( \frac{4.8}{h} \right)^{0.4} & \text{for } \frac{b_i}{h} > 1.0 \text{ and } h_i \leq 4.8 \text{ m} \\
\left( \frac{4.8}{h} \right)^{0.4} & \text{for } \frac{b_i}{h} > 1.0 \text{ and } h_i > 4.8 \text{ m}
\end{cases}
\]

where \( h \) is the height of the wall, in m;

\[
k_{ia} = 1 + (0.083 \cdot q_i - 0.0008 \cdot q_i^2) \left( \frac{2.4}{h} \right)^{0.4}
\]

where \( q_i \) is the equivalent uniformly distributed vertical load acting on the wall, in kN/m, with \( q_i \geq 0 \), see (5) below.
Use of Structural Eurocodes – EN 1995 (Design of Timber Structures) BD2405

\[ k_i = \frac{1}{0.86 \frac{z}{z_p} + 0.57} \]  \hspace{1cm} (9.29)

where \( z \) is the spacing of the fasteners around the perimeter of the sheets;

\[ k_n = \begin{cases} 10 & \text{for sheathing on one side} \quad (a) \\ \frac{F_{\text{v,Ed,max}}}{F_{\text{v,Ed,max}}} + 0.5 \frac{F_{\text{u,Ed,min}}}{F_{\text{u,Ed,min}}} & \text{for sheathing on both sides} \quad (b) \end{cases} \]

where:

- \( F_{\text{v,Ed,max}} \) is the design racking strength of the stronger sheathing,
- \( F_{\text{v,Ed,min}} \) is the design racking strength of the weaker sheathing.

(5) The equivalent vertical load, \( q_i \), used to calculate \( k_u \) should be determined using only permanent actions and any net effects of wind together with the equivalent actions arising from concentrated forces, including anchorage forces, acting on the panel. For the purposes of calculating \( k_u \), concentrated vertical forces should be converted into an equivalent uniformly distributed load on the assumption that the wall is a rigid body e.g. for the load \( F_{\text{w,Ed},i} \) acting on the wall as shown in Figure 9.8

\[ q_i = \frac{2}{b_i^2} \frac{a F_{\text{w,Ed},i}}{2} \]  \hspace{1cm} (9.31)

where:

- \( a \) is the horizontal distance from the force \( F \) to the leeward corner of the wall;
- \( b \) is the length of the wall.

![Figure 9.8 – Determination of equivalent vertical action \( q_i \) and reaction forces from vertical and horizontal actions](image-url)
(6) The external forces $F_{i,s,Ed}$ and $F_{i,c,Ed}$ (see Figure 9.9) from the horizontal action $F_{i,Ed}$ on wall $i$ should be determined from

$$F_{i,s,Ed} = F_{i,c,Ed} = \frac{F_{i,Ed}}{\delta_i} h$$

(9.32)

where $h$ is the height of the wall.

These external forces can be transmitted to either the adjacent panel through the vertical panel-to-panel connection or to the construction above or below the wall. When tensile forces are transmitted to the construction below, the panel should be anchored with stiff fasteners. Compression forces in the vertical members should be checked for buckling in accordance with 6.3.2. Where the ends of vertical members bear on horizontal framing members, the compression perpendicular to the grain stresses in the horizontal members should be assessed according to 6.1.5.

(7) The buckling of the sheets under the action of shear force $F_{r,Ed}$ may be disregarded provided

$$\frac{h_{\text{pat}}}{t} \leq 100$$

(9.33)

where:

$b_{\text{pat}}$ is the clear distance between vertical members of the timber frame;

$t$ is the thickness of the sheathing.

9.2.5 BRACING

9.2.5.1 General

(1) P & (2) P: All these Principle Rules are common sense for making sure instability or excessive deflections will be prevented by bracing.

(3) P: Bracing will be determined for most unfavourable combination of imperfections and induced deflections.

9.2.5.2 Single members in compression

(1) For single elements in compression, requiring lateral support at intervals $a$ (see Figure 9.9), the initial deviations from straightness between supports should be within $a/500$ for glued laminated or LVL members, and $a/300$ for other members.

(2) Each intermediate support should have a minimum spring stiffness $C$

$$C = k_s \frac{N_a}{a}$$

(9.34)

where:

$k_s$ is a modification factor;

$N_a$ is the mean design compressive force in the element;

$a$ is the bay length (see Figure 9.9).
NOTE: For $k_1$, see note in 9.2.5.3(1)

(3) The design stabilizing force $F_d$ at each support should be taken as:

$$F_d = \begin{cases} \frac{N_d}{k_{c1}} & \text{for solid timber} \\ \frac{N_d}{k_{c2}} & \text{for glued laminated timber and LVL} \end{cases}$$

(9.35)

where $k_{c1}$ and $k_{c2}$ are modification factors.

NOTE: For $k_{c1}$ and $k_{c2}$, see note in 9.2.5.3(1)

![Diagram](image)

Figure 9.9 – Examples of single members in compression braced by lateral supports.

(4) The design stabilizing force $F_d$ for the compressive edge of a rectangular beam should be determined in accordance with 9.2.5.2(3) where:

$$N_d = (1-k_{mt})\frac{M_d}{h}$$

(9.36)

The value of $k_{mt}$ should be determined from 6.3.3(4) for the unbraced beam, and $M_d$ is the maximum design moment acting on the beam of depth $h$.

9.2.5.3 Bracing of beams or truss systems

(1) For a series of $n$ parallel members which require lateral supports at intermediate nodes A, B, etc. (see Figure 9.10) a bracing system should be provided, which, in addition to the effects of external horizontal load (e.g. wind), should be capable of resisting an internal stability load per unit length $q$, as follows:

$$q_d = k_{d} \frac{N_d}{k_{c}}$$

(9.37)

where:

$$k_d = \min \left\{ \frac{1}{k_{c1}}, \frac{1}{k_{c2}} \right\}$$

(9.38)

$N_d$ is the mean design compressive force in the member,
\( \ell \) is the overall span of the stabilizing system, in m;

\( k_{t,1} \) is a modification factor.

Table 9.2

<table>
<thead>
<tr>
<th>Modification factor ( K )</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>( K_s )</td>
<td>4</td>
</tr>
<tr>
<td>( K_{f,i} )</td>
<td>60</td>
</tr>
<tr>
<td>( K_{f,2} )</td>
<td>100</td>
</tr>
<tr>
<td>( K_{f,3} )</td>
<td>50 (members spaced ≤ 600 mm) \n</td>
</tr>
</tbody>
</table>

(2) The horizontal deflection of the bracing system due to force \( q_d \) and any other external load (e.g. wind), should not exceed \( \ell/500 \).
‘Section 10: Structural detailing and Control’

10.1 GENERAL

(1)P This section is generally for workmanship, best practice and certain detailed information that supports all previous sections.

10.2 MATERIALS

(1) For solid timber imperfections such as bow measured at mid-length should be \( \leq \frac{L}{300} \) specially for columns and beams.

For glulam and IVL imperfections such as bow measured at mid-length should be \( \leq \frac{L}{500} \)

(2) It is best practice not to expose or use timber in climatic conditions that are more severe than that expected in the finished structure.

(3) It is best practice before use to dry timber as close as possible to the m.c. which the structure will attain during the service.

10.3 GLUED JOINTS

(1) It is safer and best practice to use glues which have quality control certification to ensure the reliability and quality of joints which are crucial for safety of structures.

(2) Glue manufacturers’ recommendations must be followed.

(3) For glues which require a setting-time, the structure/element/component must not be loaded before the specified setting-time is reached. Avoid using gluing on site.

10.4 CONNECTIONS WITH FASTENERS

10.4.1 General

(1)P: It is important that deflects such as wane, splits, knots, etc. are limited (ideally not permitted) at joints which will reduce the load-carrying capacity of the joint.
10.4.2 Nails

(1) Nails should ideally be driven with their heads flush with the surface of timber.

(2) Skew nailing should ideally be as shown below:

![Diagram showing skew nailing](image)

(3) Diameters of pre-drilled holes for nails should be \( \leq 0.8d \) where \( d \) is the diameter of nail.

10.4.3 Bolts and washers

(1) Tolerance of bolt holes in timber should not be > 1mm of the bolt diameter.

Tolerance of bolt holes in steel plates should not be > 2mm or 0.1\( d \) (whichever is greater) of the bolt diameter ‘\( d \)’.

(2) Side of square washers and diameter of round washers (underneath nuts and bolts) should be at least 3\( d \) and have a thickness of at least 0.3\( d \).

(3) Bolts and screws should be tight and re-tightened again if necessary when the timber has reached its equilibrium moisture content, especially when shrinkage of timber has taken place.

(4) Bolts used with connectors should have the following diameters:

<table>
<thead>
<tr>
<th>Type of connector EN 912</th>
<th>( d_c )</th>
<th>( d_{\text{minimum}} )</th>
<th>( d_{\text{maximum}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1 – A6</td>
<td>( \leq 130 )</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>A1, A4, A6</td>
<td>&gt; 130</td>
<td>0.1 ( d_c )</td>
<td>24</td>
</tr>
<tr>
<td>B</td>
<td>( d_c – 1 )</td>
<td>( d_1 )</td>
<td></td>
</tr>
</tbody>
</table>

\( d_c \) = connector diameter in mm

\( d \) = bolt diameter in mm

\( d_1 \) = hole diameter in mm
10.4.4 Dowels
(1) Minimum dowel diameter should be 6mm with \(-0/+0.1\)mm tolerance. Pre-drilled holes in timber members must not be more than the dowel diameter. It is always good practice to use dowel materials with higher machined properties than timber members.

10.4.5 Screws
(1) Pre-drilling is not required for screws in softwoods with a smooth shank diameter \(\leq 6\)mm.

Pre-drilling is required for screws in hardwoods and in softwoods with diameter > 6mm with the following requirements:

- The lead hole (member that the screw is being entered) diameter should be = diameter of the shank and the same length as the shank.

- The lead hole for the threaded portion should have a diameter of approx. 70\% of the shank diameter.

(2) For densities > 600 kg/m\(^3\), pre-drilling diameter should be determined by tests.

10.5 ASSEMBLY
(1) It is good practice not to overstress members and connections if they don’t fit together easily in assembly or erection. Do not use components which exhibit splits, warping etc.

10.6 TRANSPORTATION AND ERECTION
(1) It is good practice to store, transport and erect component/elements/structures in an adequate manner so that unnecessary and unaccounted for loads are not imposed on them, especially vibration forces.

Lifting procedures and lifting points should be designed so that no distortion and damage can occur during the lifting.

10.7 CONTROL
(1) These requirements are again good practice issues for which any good design or consulting engineering company should have a quality control procedure in place. This is a very relevant issue in EN 1990.
10.8 SPECIAL RULES FOR DIAPHRAGM STRUCTURES

10.8.1 Floor and roof diaphragms
(1) Nail spacings around the board/panel edges should be ≤ 150mm. For within the boards/panels they should be ≤ 300mm. Please note, square-edged boards (Figure 10.1) must be supported at all their edges but long edges of T&G boards can be unsupported provided they cross the supporting joists or rafters and their short edges are supported by joists or rafters. Nails should comply with EN 14592.

![Figure 10.1 – Example of connection of panels not supported by a joist or a rafter](image)

There are many recommendations for using panel products as diaphragm/flooring given in EN 12871 which should be followed for best practice.

10.8.2 Wall diaphragms
(1): Nail spacings around the board/panel edges should be ≤ 150mm and ≤ 300mm within the boards/panels.

Screw spacings around the board/panel edges should be ≤ 200mm and ≤ 300mm within the boards/panels.

![Figure 10.2 – Panel fixings](image)

Again, best practice application guidance is given in EN 12871 which is recommended.
10.9 SPECIAL RULES FOR TRUSSES WITH PUNCHED METAL PLATE FASTENERS

10.9.1 Fabrication
Please refer to EN 14250 for fabrication. Usually any good and reputable fabricator would have quality control in place complying with EN 14250.

10.9.2 Erection of trusses
(1) Again this is good practice to ensure that the trusses are erected straight and vertically before the bracing or facing are fixed to them.

(2) Trusses used should comply with EN 14250. **You should ignore the rest of the clause because it is not good practice or acceptable that members are allowed to distort between fabrication and erection.**

(3) This Application rule is another one for which national choice is allowed. Maximum bow $a_{bow}$ in any truss member after erection should be avoided or prevented from increasing after the facing is fixed. However, the permitted bow $a_{bow, perm.}$ should be 10 mm.

(4) This Application rule is another one for which national choice is allowed. Major deviation from vertical $a_{dev}$ after erection should be:

$$a_{dev, perm.} = \text{minimum of } 10 + 5(H-1) \text{ OR } 25 \text{ mm}$$

where $H =$ height of truss in metres.
ANNEX A (Informative)

Block shear and plug shear failure at multiple dowel-type steel-to-timber connections

Please note that this annex is only relevant to dowel-type fasteners. It does NOT apply to nails or screws. However, for clarity the following are explained for using dowel-type fasteners other than nails and screws:

For steel-to-timber connections comprising multiple dowel-type fasteners which are subjected to a force component parallel to grain near the end of the timber member, block shear failure and plug shear failure can occur (Figures A1 and A2 respectively):

The characteristic load-carrying capacity of the fracture along the perimeter of the fastener area can be determined using the expression below:

\[
F_{bs,Rk} = \max \left\{ 1.5 A_{\text{net,t}} f_{t,0,k}, 0.7 f_{v,k} \right\}
\]
\[ A_{\text{net,t}} \]

\[ A_{\text{net,t}} = L_{\text{net,t}} t_1 \]

\[ A_{\text{net,v}} \] the failure area, dependent on failure modes

➤ For failure modes:

![Diagram of failure modes](image)

\[ A_{\text{net,v}} = L_{\text{net,v}} t_1 \]

➤ For failure modes:

![Diagram of failure modes](image)

\[ A_{\text{net,v}} = \frac{L_{\text{net,v}}}{2} (L_{\text{net,t}} + 2t_{ef}) \]

\[ L_{\text{net}} \] Determining the failure area

Total net length of the shear fracture area

\[ L_{\text{net,v}} = \sum_i l_{v,i} \]

Net width of the cross-section perpendicular to grain

\[ L_{\text{net,t}} = \sum_i l_{t,i} \]
Effective depth, depending on failure mode and steel plate thickness

**Thin steel plate** (thickness ≤ 0.5\(d\) (\(d\) is the fastener diameter))

\[ t_{ef} = 0.4t_1 \]

\[ t_{ef} = 1.4 \sqrt{\frac{M}{f}} \]

\(M_{y,Rk}\) is the characteristic yield moment of the fastener. If not available for the fastener, it should be determined according to EN 409 and EN 14358.

**Thick steel plates** (thickness ≥ \(d\) (\(d\) is the fastener diameter))
Timber properties needed for calculations

\( f_{t,0,k} \) characteristic tensile strength of timber member

Extract of EN338 structural timber strength classes, characteristic values \( f_{t,0,k} \) (N/mm\(^2\))

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>8</td>
<td>10</td>
</tr>
</tbody>
</table>

\( f_{v,k} \) characteristic shear strength of timber member

Extract of EN338 structural timber strength classes, characteristic values \( f_{v,k} \) (N/mm\(^2\))

<table>
<thead>
<tr>
<th>Softwood and poplar species</th>
<th>Hardwood species</th>
</tr>
</thead>
<tbody>
<tr>
<td>C14</td>
<td>C16</td>
</tr>
<tr>
<td>1,7</td>
<td>1,8</td>
</tr>
</tbody>
</table>

\( f_{h,k} \) characteristic embedding strength of timber member

See section 8
ANNEX B (Informative)
Mechanically jointed beams

The UK national application document allows this informative Annex to EN 1995-1-1 to be used in the UK.

B.1 Simplified analysis

B.1.1 CROSS SECTION

The following cross sections are covered in this annex:

![Diagram of cross sections](image-url)
B.1.2 ASSUMPTIONS

Annex B and the design method described therein is based on the following assumptions:

- The design method is based on the theory of linear elasticity
- The beams are simply supported beams with a span \( l \)

For design use:

\[
\begin{array}{c}
\text{For design use:} \\
\begin{array}{c}
\text{l} \\
\hline
\text{l}
\end{array}
\end{array}
\]

- For continuous beams the design expressions may be used with \( l = 0.8l \)

For design use:

\[
\begin{array}{c}
\begin{array}{c}
\text{For design use:} \\
\begin{array}{c}
0.8 \times l_1 \\
\hline
0.8 \times l_2
\end{array}
\end{array}
\end{array}
\]

- For cantilevered beams use double length

For design use:

\[
\begin{array}{c}
\begin{array}{c}
\text{For design use:} \\
\begin{array}{c}
l_1 \\
\hline
2 \times l_2
\end{array}
\end{array}
\end{array}
\]

- The individual parts (wood, wood-based panels) are either full length or made with glued end joints
- The individual parts are connected to each other by mechanical fasteners with a slip modulus \( K \)
- The spacing between the fasteners is constant or varies uniformly to the shear between \( s_{\text{min}} \) and \( s_{\text{max}} \). Please note that \( s_{\text{max}} \leq 4 \times s_{\text{min}} \)
- The load is acting in the z-direction giving a moment \( M = M(\alpha) \) (varying sinusoidally or parabolically (does this mean loading either uniformly distributed or …) and a shear force \( V = V(\alpha) \)

B.1.3 SPACINGS

When a flange consists of two parts jointed to a web (like in an I beam) or where a web consists of parts (like in a box beam) the spacing \( s_1 \) is determined by the sum of the fasteners per unit length in the two jointing planes, below an example is shown – jointing planes (1) and (2)
B.1.4 DEFLECTIONS RESULTING FROM BENDING MOMENTS

Deflections must be calculated by using an effective bending stiffness \((EI)_{eff}\) determined in accordance with B.2 and B.3

B.2 Effective bonding stiffness

(1) The effective bending stiffness should be taken as:

\[
(EI)_{eff} = \sum_{i=1}^{3} (E_i l_i + \gamma_i E_i A_i \alpha_i^2)
\]

using mean values of \(E\) and where:

\[
A_i = h_i \lambda_i
\]

\[
l_i = \frac{h_i \lambda_i^2}{12}
\]

\[
\gamma_2 = 1
\]

\[
\gamma_i = \left[1 + \pi^2 E_i A_i \alpha_i^2 \frac{1}{K_i^2}\right]^{-1} \quad \text{for}\ i = 1 \text{ and } i = 3
\]

\[
\alpha_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2) - \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^{3} \gamma_i E_i A_i}
\]

where the symbols are defined in Figure B.1:

\(K_i = K_{si,1}\) for the serviceability limit state calculations;

\(K_i = K_{u,1}\) for the ultimate limit state calculations.

For T-sections \(h_3 = 0\)
B.3 The normal stresses

(1) The normal stresses should be taken as:

\[ \sigma_i = \frac{\gamma_i E_t a_i M}{(E I)_{cf}} \quad (B.7) \]

\[ \sigma_{m,i} = \frac{0.5 E_i h_i M}{(E I)_{cf}} \quad (B.8) \]
B.4 Shear stresses

The maximum shear stresses occur where the normal stresses are zero. The maximum shear stresses in the web member (part 2 in Figure B.1) should be taken as:

$$\tau_{2_{\text{max}}} = \frac{\gamma_3 E_3 A_3 \alpha_3 + 0.5 E_2 b_2 h_2^2}{b_2 (EI)_{ef}}$$  \hspace{1cm} (B.9)

B.5 Fastener load

The load per fastener should be taken as

$$F_i = \frac{\gamma_i E_i A_i s_i}{(EI)_{ef}} V$$  \hspace{1cm} (B.10)

where:

- $i = 1$ and 3, respectively;
- $s_i = s_i(x)$ is the spacing of the fasteners as defined in B.1.3(1).
ANNEX C (Informative)

C.1 Built-up columns

This annex is relevant to columns

- which are simply supported (with length \( l \))
- where the individual parts are full length
- where the applied load is an axial force (\( F_c \)) acting at the geometric centre of gravity
- in cases where small moments (for example from self weight) are applied in addition to the axial load. Section 6.3.2(3) applies

C.1.2 LOAD-CARRYING CAPACITY

For column deflection in the y-direction, the load carrying capacity should be taken as the sum of the load-carrying capacities of the individual members.

For column deflection in the z-direction (see figures below), it should be verified that

\[ \sigma_{c,0,d} \leq k_c f_{c,0,d} \]

where:

\[ \sigma_{c,0,d} = \frac{F_{c,d}}{A_{tot}} \]

where:

- \( A_{tot} \) is the total cross-sectional area;
- \( k_c \) is determined in accordance with 6.3.2 but with an effective slenderness ratio \( \lambda_{ef} \) determined in accordance with sections C.2 - C.4.
Figure C.1 – Spaced columns

Figure C.2 – Shear force distribution and loads on gussets or packs
C.2 Mechanically jointed columns

The effective slenderness ratio should be taken as

\[ \lambda_{ct} = \sqrt{\frac{A_{ct}}{l_{ct}}} \]  \hspace{1cm} (C.3)

with

\[ l_{ct} = \frac{(EI)_{ct}}{E_{mean}} \]  \hspace{1cm} (C.4)

where \((EI)_{ct}\) is determined in accordance with Annex B (informative).

B.2 Effective bending stiffness

(1) The effective bending stiffness should be taken as:

\[ (EI)_{ct} = \sum_{i=1}^{3} (E_i I_i + \gamma_i E_i A_i a_i^2) \]  \hspace{1cm} (B.1)

using mean values of \(E\) and where:

\[ A_i = b_i h_i \]  \hspace{1cm} (B.2)

\[ I_i = \frac{b_i h_i^3}{12} \]  \hspace{1cm} (B.3)

\[ \gamma_2 = 1 \]  \hspace{1cm} (B.4)

\[ \gamma_i = \left[ 1 + \pi^2 E_i A_i s_i (K_i I_i^2) \right]^{-1} \]  \hspace{1cm} for \( i = 1 \) and \( i = 3 \) \hspace{1cm} (B.5)

\[ a_i = \frac{\gamma_2 E_i A_i (h_1 + h_2) + \gamma_3 E_3 A_3 (h_2 + h_3)}{2 \sum_{i=1}^{3} \gamma_i E_i A_i} \]  \hspace{1cm} (B.6)

where the symbols are defined in Figure B.1:

\( K_i = K_{w,i} \) for the serviceability limit state calculations;

\( K_i = K_{u,i} \) for the ultimate limit state calculations.

For T-sections \( h_3 = 0 \)

C.2.2 LOAD ON FASTENERS

The load on a fastener should be determined in accordance with Annex B.

(1) The load on a fastener should be taken as:

\[ F_i = \frac{\gamma_i E_i A_i a_i s_i V}{(EI)_{ct}} \]  \hspace{1cm} (B.10)

where:

\( i = 1 \) and \( 3 \), respectively;

\( s_i = s_i(x) \) is the spacing of the fasteners as defined in B.1.3(1).
C.3 Spaced columns with packs or gussets

The following assumptions apply:

- The cross section is composed of two, three or four identical shafts
- The cross sections are symmetrical about both axes
- The number of unrestrained bays is at least three (shafts are at least connected at the ends and at the third points)
- For columns with packs $a \leq 3b$, for columns with gussets $a \leq 6b$
- The joints, packs and gussets are designed in accordance with C.2.2
- The pack length $l_2$ satisfies $\frac{l_2}{a} \geq 1.5$
- The gusset satisfies the condition $\frac{l_2}{d} \geq 2$
- There are at least four nails, or two bolts with connectors in each shear plane. For nailed joints there are at least four nails in a row at each end in the length of the column
- The column is subjected to concentric axial loads

**Columns with two shafts**

\[ A_{tot} = 2A \]
\[ I_{\text{tot}} = \frac{b\left[(3h + 2a)^3 - (h + 2a)^3 + h^3\right]}{12} \]

Columns with three shafts

\[ A_{\text{tot}} = 3A \]

\[ I_{\text{tot}} = \frac{b\left[(3h + 2a)^3 - (h + 2a)^3 + h^3\right]}{12} \]
C.3.2 AXIAL LOAD-CARRYING CAPACITY

(1) For a column deflection in the y-direction, the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual members.

For a column deflection in z-direction C.1.2 applies

\[ \sigma_{c,0,d} \leq k_c \bar{f}_{c,0,d} \]

(2) For column deflection in the z-direction C.1.2 applies with

\[ \lambda_{cf} = \sqrt{\lambda^2 + \eta \frac{n}{2} \lambda_1^2} \quad (C.10) \]

where:
- \( \lambda \) is the slenderness ratio for a solid column with the same length, the same area \( A_{ks} \) and the same second moment of area \( I_{km} \), i.e.,
  \[ \lambda = \sqrt{\frac{A_{ks}}{I_{km}}} \quad (C.11) \]
- \( \lambda_1 \) is the slenderness ratio for the shafts and has to be set into expression (C.10) with a minimum value of at least 30, i.e.
  \[ \lambda_1 = \sqrt{\frac{\ell_1}{h}} \quad (C.12) \]
- \( n \) is the number of shafts;
- \( \eta \) is a factor given in Table C.1.
### Table C.1 – The factor $\eta$

<table>
<thead>
<tr>
<th></th>
<th>Packs</th>
<th></th>
<th></th>
<th>Gussets</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Glued</td>
<td>Nailed</td>
<td>Bolted*</td>
<td>Glued</td>
<td>Nailed</td>
<td></td>
</tr>
<tr>
<td>Permanent/long-term loading</td>
<td>1</td>
<td>4</td>
<td>3,5</td>
<td>3</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>Medium/short-term loading</td>
<td>1</td>
<td>3</td>
<td>2,5</td>
<td>2</td>
<td>4,5</td>
<td></td>
</tr>
</tbody>
</table>

* with connectors

#### C.3.3 Load on fasteners, gussets or packs

1. The load on the fasteners and the gussets or packs are as shown in Figure C.2 with $V_d$ according to section C.2.2.

2. The shear forces on the gussets or packs, see Figure C.2, should be calculated from:

$$f_d = \frac{V_d}{a_1}$$  \hspace{1cm} (C.13)

![Figure C.2 – Shear force distribution and loads on gussets or packs](image)

#### C.4 Lattice columns with glued or nailed joints

##### C.4.1 Assumptions

1. Lattice columns with N- or V-lattice configurations and with glued or nailed joints are considered in this section, see Figure C.3.

2. The following assumptions apply:
   - the structure is symmetrical about the $y$- and $z$-axes of the cross-section. The lattice on the two sides may be staggered by a length of $r_i/2$, where $r_i$ is the distance between the nodes;
   - there are at least three bays;
   - in nailed structures there are at least four nails per shear plane in each diagonal at each nodal point.
- each end is braced;
- the slenderness ratio of the individual flange corresponding to the node length \( l \) is not greater than 60;
- no local buckling occurs in the flanges corresponding to the column length \( l \);
- the number of nails in the verticals (of an N-truss) is greater than \( n \sin \theta \) where \( n \) is the number of nails in the diagonals and \( \theta \) is the inclination of the diagonals.

### C.4.2 Load-carrying capacity

(1) For column deflection in the y-direction (see Figure C.2), the load-carrying capacity should be taken as the sum of the load-carrying capacities of the individual flanges.

(2) For column deflection in the z-direction C.1.2 applies with

\[
\lambda_{ef} = \max \left( \frac{\lambda_{tot} \sqrt{1 + \mu}}{1.05 \lambda_{tot}} \right)
\]

where:

\[
\lambda_{tot} \quad \text{is the slenderness ratio for a solid column with the same length, the same area and the same second moment of area, i.e.}
\]

\[
\lambda_{tot} = \frac{2l}{h}
\]

\( \mu \) takes the values given in (3) to (6) below.

(3) For a glued V-truss:

\[
\mu = 4 \frac{e^2 A_t}{\ell} \left( \frac{h}{\ell} \right)^2
\]

where (see Figure C.3):

- \( e \) is the eccentricity of the joints;
- \( A_t \) is the area of the flange;
- \( l_t \) is the second moment of area of the flange;
- \( \ell \) is the span;
- \( h \) is the distance of the flanges.
Figure C.3 – Lattice columns: (a) V-truss, (b) N-truss

(4) For a glued N-truss:

$$
\mu = \frac{2A_h}{L_t \left( \frac{h}{\ell} \right)^2}
$$  \hspace{1cm} (C.17)

(5) For a nailed V-truss:

$$
\mu = 25 \frac{h E_p\text{mean} A_f}{\ell^2 n K_u \sin 2\theta}
$$  \hspace{1cm} (C.18)

where:

- \(n\) is the number of nails in a diagonal. If a diagonal consists of two or more pieces, \(n\) is the sum of the nails (not the number of nails per shear plane),

Key:

1. number of nails: \(n\)
2. number of nails: \(m\)
3. number of nails: \(2n \sin \theta\)
4. number of nails: \(m\)
\( E_{\text{mean}} \) is the mean value of modulus of elasticity; 
\( K_u \) is the slip modulus of one nail in the ultimate limit state.

(5) For a nailed N-truss:

\[
\mu = 50 \frac{h E_{\text{mean}} A_t}{\ell^2 n K_u \sin 2\theta}
\]

\[(C.16)\]

where:
- \( n \) is the number of nails in a diagonal. If a diagonal consists of two or more pieces, \( n \) is the sum of the nails (not the number of nails per shear plane);
- \( K_u \) is the slip modulus of one nail for the ultimate limit states.

C.4.3 Shear forces

(1) C.2.2 applies.
ANNEX D (Informative) P

Bibliography

- In this Annex a reference for NCCI will be entered when available.
- The UK National Annex is already included in this companion document but it should be listed here for reference.
- BS EN 1990: ‘002 Eurocode – Basis of structural design
- BS EN 1991 (all parts), Eurocode 1 – Actions on structures.
CHAPTER 3
Design guide

3.1 Design flow chart and quick clause finder

The design flow chart and the quick clause finder are to help designers to get a quick overview of the various interrelated design clauses in EN 1995-1-1. It is to help overcome the unfamiliarity of layout and content arrangement in EN 1995-1-1. The entry screen of the software tool is shown below.

![Entry screen for the quick clause finder and the design flow chart](image)

The design flow chart illustrates how EN 1995-1-1 is to be used by the designer from initial concept design through to the detailed design of members and assemblies.
The quick clause finder allows the user to type a keyword and gives quick and comprehensive access to all the design clauses related to the keyword search. This ensures all relevant design cases are being displayed and can be considered by the designer.

If a designer, for example, is interested in designing a beam under combined bending and compression loading, the quick clause finder will display the various design clauses related to the search term ‘beam’. In a further drop-down menu the designer can narrow the search and further specify the element type, material and which limit state is to be considered.
Quick clause finder: Drop-down menu for choosing the structural element

Drop-down menu for choosing the material of the structural element type to be considered
Below the results for the search ‘beam’ in solid timber and for the ultimate limit state the list of design clauses related to the search term are also overviewed. This gives a comprehensive list of the various design rules to be considered for this type of structural element and also links in with secondary design steps, such as joints. The clause is displayed in full text with an additional feature to view all related clauses, also in full text (circle line in second figure).
Use of Structural Eurocodes – EN 1995 (Design of Timber Structures) BD2405

EuroCode 5 - Design of Timber Structures
EN 1995-1-1:2004

Quick Clause Finder

Design Details:
- Element Category: Simple Beam
- Element Type: Solid Timber
- Limit State: Ultimate

Relevant Clauses:
- 6.1.6 Bending
- 6.1.7 Shear
- 6.1.8 Torsion
- 6.3.1 Stability of members - General
- 6.3.3 Beams subjected to either bending or combined bending and torsion
- 6.5.1 Notched members - General
- 6.5.2 Beams with a notch at the support
- 9.1.3 Mechanically jointed beams
6.3.3 Beams subjected to either bending or combined bending and compression

(1) Lateral torsional stability shall be verified both in the case where only a moment M_{11} acts about the strong axis y and where a combination of moment M_{11} and compressive force N_{1} exists.

(2) The relative slenderness for bending should be taken as:

$$\lambda_{cr} = \frac{c_{cr}}{\sqrt{\frac{N_{cr}}{N_{y}}}}$$

where c_{cr} is the critical bending stress calculated according to the classical theory of stability, using 5 percentile stiffness values.

The critical bending stress should be taken as:

$$\sigma_{cr} = \frac{M_{cr}}{W_{cr}}$$

where:
- $S_{cr}$ is the 5th percentile value of modulus of rupture parallel to grain;
- $S_{cr}$ is the 5th percentile value of shear modulus parallel to grain;
- $I_{cr}$ is the second moment of area about the weak axis z;
- $I_{cr}$ is the torsional moment of inertia;

Quick clause finder results for 'beam'
REFERENCES


g) P Hoffmeyer. Wood as a building material. Centrum Hout, The Netherlands, Timber Engineering STEP 1 Basis of design, material properties, structural components and joints, 1995, Section A4.


