# The Italian instructions for the design, execution and control of timber constructions (CNR-DT 206 R1/2018)

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ABSTRACT

In 2007 the Italian National Research Council published the technical document CNR-DT 206/2007 "Instructions for the design, execution and control of timber structures", with the purpose to provide a technical support to the operators of the sector, in line with the most advanced knowledge at that time. Since then, the world of timber engineering has largely used such document, even though the instructions are not mandatory standard rules, so that they became the most common tool in Italy for the structural use of timber, opening the markets and favouring competition and new applications. In latest years new studies, researches and innovative proposals have promoted the development and the growth of timber constructions also in the civil residential field. The framework of standard rules for constructions and products, both Europeans and Italians, has evolved too. For these reasons, CNR has considered as opportune to proceed to the editing of a new version, updated and widened, namely with the acronym DT 206-R1. The document comes from the spontaneous cooperation of an open group of specialists and operators of the sector, based on a wide discussion on the common scientific and technical expertise and knowledge. The current version has already taken into account the results of the public inquiry phase (concluded by now).

The paper presents the main contents of the document, evidencing the innovations.

## 1. Introduction

Timber structures are high-efficient structural systems that, thanks to their prerogatives of easiness and quickness of construction, transportation, material sustainability, energy efficiency and good seismic response have become a valid alternative with respect to traditional structures.

Since the past, solid wood elements have been used as construction material all over the world, especially in those areas with abundance of forests and woodlands such as in North America, North Europe, or Japan.

The introduction of new engineered timber products, such as glue-laminated timber, Cross-Laminated Timber (*CLT*) and Laminated Veneer Lumber (*LVL*), caused a significant spread of timber-based structures in the last two–three decades also in Europe [1], not only in North Europe, which is a zone particularly devoted to the use of timber, but also in the Mediterranean area countries, like in Italy, Spain and Portugal. A massive use of timber buildings in the European market constructions has ensued, especially for *CLT*, light and heavy timber frame and blockhaus buildings, allowing a rapid technological progress [2–4].

Such progress has not been accompanied by an immediate updating of the European Standards, either at the international or national levels. This has often led to an unaware employ of timber by engineers, who

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could just rely on technical specifications, manuals and data sheets provided by manufacturers and not on codified rules.

As a consequence, the need for updating the existing standards for design and constructions becomes unavoidable in Europe. Eurocode 5 (EC5) [5] and Eurocode 8 (EC8) [6] are the two main documents, valid for all European countries, for the design of timber constructions. EC5 provides the design and safety check rules for solid and glulam timbers members and subassemblies (trusses, braced systems, diaphragms, etc.) as well as for connections. EC8 provides the rules for the design of seismic-resistant structures made with whatever material, including also timber.

To fill the existing gap between technological and research advancements and the codes for construction in timber, a reviewing phase of both Eurocodes is started and is still ongoing. Revision of Eurocode 5 started in 2012. It collects the background of information deriving from researchers, producers and innovation field. Detailed descriptions of the revisions are summarized in Dietsch and Winter 2012 [7] and Kleinhenz et al., 2016 [8]: they illustrates improvements relative to the existing rules, such as methods for calculating vibrations of floors, more detailed information on members stability, additional requirements for design perpendicular to grain compression and new provisions for structural elements, like *CLT* panels, timber-to-concrete composites structures, holes in beams.

Also Chapter 8 of Eurocode 8, related to seismic design of timber structures, is under review, whose main scope consists of aligning the design criteria for wood-based buildings to that of other building types (i.e., reinforced concrete or steel buildings), exploiting the knowhow and the advancements derived from the scientific community [9]. To date, the drafts of the consolidate versions of EC5 and EC8 have been are available in [10] and [11], respectively.

A recent research conducted by the Italian industrial association 'Federlegno Arredo' estimates that Italy occupies a relevant position concerning the use of timber in structural applications. In 2018 the percentage of timber buildings stock has been of 7.1% (3147 buildings) with respect to the overall building market in Italy, where the 79% is represented by residential buildings, while the remaining 21% by non-residential ones. In addition, the production of timber elements (i.e., glulam beams, arches, columns, panels, etc.) used for realizing large-span roofs, bridges, footbridges, diaphragms, industrial buildings or similar structures, must be also considered. In 2019, Italy is classified as fourth in Europe with regards to the production of residential and not residential timber buildings, after Germany, Sweden and United Kingdom [12].

With the purpose of providing a useful tool able to address engineers towards a rational design of timber structures and in line with the technological advancements, the Technical Document CNR-DT 206-R1/2018 "Instructions for the design, execution and control of timber structures" has been issued in Italy. It has been elaborated by a working group of Italian researchers and released in October 2018 by the Italian Council of Research. The document, issued for the first time in 2007, was considered in the last years the main Italian technical support for practitioners and researchers operating in the field of timber structures.

This paper discusses the contents of the Italian CNR Technical Document, emphasizing the novelty introduced with respect to EC5 and EC8. The document, which includes the most important experimental and theoretical advancements derived from the research, has been conceived in harmonization with the current and the future provisions that will be incorporated in the Eurocodes.

In Table 1 the List of Contents of the Technical Document divided into sections is reported. The sections revised or expanded, those affected by small changes and those introduced ex-novo with respect to the current version of Eurocodes are indicated in the same Table. In particular, the comparison is done with EC5, except for Section 10 that must be compared with EC8.

The new timber products, such as *CLT* panels, Solid Wood Panels (*SWP*) and glued solid panels have been introduced in Section 4 of the Document. They are not contemplated yet in the current version of the EC5. Section 5 introduces a subdivision between the adhesives for elements glued in the factory and those for elements glued on-site.

**Table 1**
List of Contents of the Technical Document CNR DT206 R1/2018 and comparison with Eurocodes.

| Chapter | Small changes | Revised and expanded | New section |
|---|---|---|---|
| 1. Introduction | | | |
| 2. Field of application | | | |
| 3. Reference standards | | | |
| 4. Materials and products | | ✔ | |
| 5. Complementary material | | ✔ | |
| 6. Elements, typologies and structural systems | | ✔ | ✔ |
| 7. Design criteria and rules | | ✔ | |
| 8. Connections | | ✔ | ✔ |
| 9. Specific rules for typologies and structural systems | | ✔ | ✔ |
| 10. Seismic design | | ✔ | ✔ |
| 11. Durability design | ✔ | | |
| 12. Structural robustness | | | ✔ |
| 13. Fire behaviour | | | |
| 14. Execution rules | ✔ | | |
| 15. Controls | ✔ | | |
| 16. Existing structures | | | ✔ |
| 17. Appendices | | ✔ | ✔ |

The main body of the CNR-DT 206 R1/2018 Technical Document has been reorganized with respect to the Eurocodes. A hierarchical order among *elements* (beams, columns and panels), *typologies* (diaphragms, shear walls, trusses arches and frames) and *structural systems* (timber buildings) has been established and replicated in all the chapters when necessary, such as in the Sections 6 and 9. In Section 6 they are described by a morphological point of view, in particular the definition and analyses of the technological aspects of structural elements, structural types and structural systems most recurrent in timber constructions are provided. In Section 9 the rules of elements, typologies and structural systems with regards to both gravity and seismic loads is illustrated. It is worth noticing that the introduction of rules concerning the multi-story timber buildings represents the main innovative feature of the CNR Document within the European regulatory framework.

Particular attention has been focused on the environmental exposition classes for timber structures in Section 7: according to EC5 three 'service classes' are to be considered, while CNR also extends to those situations characterized by different environmental conditions, namely 'non-standard classes'.

Based on research developments, some amendments have interested the safety checks of members at Ultimate Limit States (*ULS*) with respect to EC5. In particular, orthogonal to grain check has been revised introducing specific formulations for the evaluation of the 'effective length' involved by compression stresses, as a function of different loading conditions. The 'effective length' for evaluating the critical bending moment of beams have been modified according to the flexural buckling theory. The shape factors used for torsion check have been revised and the formulation for combined shear-torsion check has been introduced.

With regards to both mechanical and carpentry connections (Section 8), a specific section is related to *ULS* checks and dimensional limits for the most common types of carpentry joints, while in EC5 no indications are provided yet for such type of connections.

The significant diffusion of timber buildings in earthquake-prone countries, like in Italy and Portugal, identifies the current need to define specific seismic design rules. Section 10 refers to the capacity design rules necessary for achieving a dissipative behaviour under earthquakes [13–17]. The Section is in part based on the new provisions included in the revision of the chapter for the seismic design of timber buildings still under discussion within the Standard Committee TC 250/

SC8/WG3 [9]. Two ductility classes (high and medium), instead of three indicated in the Eurocode 8, are allowable for timber buildings and for each of them the behaviour factors values are specified as a function of the building typology. Moreover, the hierarchy of strength rules necessary to ensure a dissipative behaviour in the connection zones of the constructions, for both high and medium ductility class, are defined.

With regards to the aspects related to the durability of timber, fire behaviour, maintenance and control reference is made to EC5. Two new Sections concerning the structural robustness and the existing timber structures have been included.

To provide an ease-of-use by engineers and researchers, many Appendices are reported in the document. They deal with timber grades (App. A), partial safety factors of material (App. B), coefficient for calculation of deformations (App. C), reference standards (App. D), design of composites beams with flexible connections (App. E) and example of calculations (App. F).

In the following sections of the paper all the amendments and novelties introduced in the documents are accurately described, highlighting the most relevant aspects, which have motivated the reviewing process.

## 2. Materials and products (*Section 4*)

### *2.1. Material properties (Section 4.2)*

This topic plays a crucial role with regards to the durability of timber structures, then it deserves particular attention. Firstly, it is remarked that timber elements must have a moisture content as much as possible equal to the environmental conditions of the construction in service. Moreover, it is highlighted that the dimensional variations due to the moisture gradient cannot be neglected in the design phase. To this aim, for the first time, the CNR Document as respect to EC5, introduces a formulation to calculate the linear dimensional variations of the elements as a function of moisture content (eq. 4.3, CNR-DT 206 R1/2018) and swelling and shrinking coefficient of the wood, as it follows [18]:

$$l_f = l_i[1 + k(u_f - u_i)] \tag{1}$$

where $l_i$ and $l_f$ are the dimensions of the element related to the initial and final moisture contents, respectively; $k$ is the swelling and shrinking coefficient of the wood in the considered anatomic direction, which assumes different values for solid [19] and for glue-laminated timber [20] (Table 2); $u_i$ and $u_f$ are the initial and final moisture contents (%), respectively. It is assumed that the Fiber Saturation Point (*FSP*) of the cell walls is achieved when the timber moisture content is equal to 30% independently from timber species and that for values less than 30% no dimensional variations are allowable.

With respect to EC5, the CNR Document introduces the issue of the thermal variations. In particular, the Section 4.2.3 of the Document point out that such variations are usually negligible for a single structural element. Conversely, in case of hybrid elements, made with different materials, the interaction among them must be accurately checked with reference to both dimensional hygrometric variation of timber (or derived products) and dimensional variations due to thermal effect of material different from timber.

The issue of the viscosity, not discussed in EC5, is also introduced in the CNR Document. It is clarified that timber has a rheological behaviour characterized by viscous deformations (also namely creep or fluage) when subjected to long-term loads. In fact, in addition to the instantaneous elastic deformations, both permanent and medium-term loads produce increments of deformations of the timber elements over the time. Mechanical connections as well show viscous deformations higher (about twice) than those exhibited by the timber elements.

**Table 2**
Values of coefficient ($k$) of linear swelling and shrinking (Table 4-2, CNR-DT 206 R1/2018).

| Timber type | Longitudinal | Radial | Tangential |
|---|---|---|---|
| Softwood, oak, chestnut, aspen | 0.0001 | 0.0012 | 00,024 |
| Cerris | 0.0001 | 0.0020 | 0.0040 |
| Glue-lam (EN14080) | 0.0001 | 0.0025 | 0.0025 |

Furthermore, it is underlined that in the case of timber elements coupled with materials affected by different viscous behaviour (for instance concrete or steel), such difference must be taken into account at Serviceability (*SLS*) and Ultimate (*ULS)* Limit State checks.

### *2.2. Products (Section 4.3)*

The range of structural products suitable for engineering applications is provided, distinguishing between one-dimensional elements (beams) and two-dimensional elements (panels), as represented in Table 3.

With regards to one-dimensional elements, CNR Document (as well as EC5) contemplates:

- solid wood elements with rectangular cross-sections (in compliance with UNI-EN 14081 [21]).
- glue-laminated timber elements (in compliance with UNI-EN14080 [22])

As respect to EC5, it introduces also the following elements:

- solid wood elements glued through finger joints (that cannot be used in service class 3);
- Laminated Veneer Lumber (*LVL)* beams, realized by gluing each other timber sheets with thickness from 3 to 6 mm; the grains of the sheets are oriented parallel to the axis of the element and cross-sections of the beam must have at least five-layers.

Due to its diffusion in Italy, the CNR Document includes also the so called “Fiume” and “Trieste” beams (in compliance with UNI 11035-3 [22]): the first ones are beams with irregular-shaped cross-sections (rounded corners) constant over the longitudinal axis of the element, while the second ones have irregular and variable cross-section over the longitudinal axis.

As far as timber-based panels, all the products contained in the EC5 are included in the CNR Document, but in addition Solid Wood Panels (*SWP*) and Cross-Laminated Timber (*CLT*) panels are considered.

*SWP* are made with timber boards glued on the edges and, if multi-layered, glued also on the upper and bottom surfaces. Maximum thickness of the panels is 80 mm and characteristics values of strength and stiffness are indicated in the standard UNI EN 12369-3 [23].

*CLT* panels consists of cross-layered timber boards rotated with a right angle to each other. Each layer, glued on the upper and bottom surfaces with those adjacent, is made with solid wood boards with thickness between 10 and 45 mm and base greater than 10 cm. The boards are joined each other by means of finger joints over the longitudinal direction. The minimum number of layer of the panels is equal to three.

## 3. Complementary materials (section 5)

In Section 5.1 it is specified that adhesives, who assure the integrity of the bonding in the assigned service class and all along the service life of the construction, are those regulated by the standards UNI-EN 301 [24] (for phenolic and aminoplastic adhesives) and UNI-EN 15425 [25] (for polyuretanic monocomponent adhesives). In particular, adhesives are subdivided in two categories: *i*) Adhesives for elements glued in the factory (Section 5.1.1, CNR-DT 206 R1/2018) and *ii*) Adhesives for bonding in situ (Section 5.1.2, CNR-DT 206 R1/2018). For the first category, adhesives classified according to UNI-EN 301 or UNI-EN 15425 can be used. Different adhesives not classified according to

**Table 3**
Timber-based products.

| One-dimensional elements (beams) | Two-dimensional elements (panels) | |
|---|---|---|
| Solid wood (with and without finger jointed) | Solid wood panels (SWP) | Oriented Strand Board (OSB) |
|  |  |  |
| *www.archiexpo.it* | *www.dataholz.eu/* | *www.legnolab.it/* |
| Glued Laminated Timber | Cross-Laminated Timber (CLT) | Particle board |
|  |  |  |
| *www.kindlholz.at/* | *www.legnoedilizia.it/* | *www.europlac.com/* |
| Laminated Veneer Lumber (LVL) | Laminated Veneer Lumber (LVL) | Fibreboard (hard or medium density) |
|  |  |  |
| www.*cwc.ca* | *www.woodproducts.fi* | *www.theprojectestimate.com* |

these standards can be used if they provide performances no worse than those of a phenolic or aminoplastic adhesive approved according to the UNI-EN 301 standard, also in relation to creep (to be verified through suitable comparative tests or literature evidence).

The Document recommend to not realize glued connections in situ. However, in relation to the importance of structural bonding and the delicacy of the procedures to be followed, it is essential that the application of the adhesives in situ is performed by specialized personnel. As no specific standard is currently available for adhesive products to be used in situ, it is appropriate, when they are not classified according to UNI-EN 301 [24], to ask to the producer to characterize the adhesives through tests performed according to suitable test protocols (which also includes an accelerated aging test). This is important to demonstrate that the shear strength is not less than the timber one under the same conditions as for the test protocol.

Furthermore, when the bonding strength is a requirement for design at Ultimate Limit State, the following recommendations must be observed:

- To follow the instructions of the adhesive producer regarding mixing, environmental conditions for application and curing, moisture content of the elements and all the factors relevant to the correct use of the adhesive. This information, if not present in the technical data sheet, should be explicitly requested to the producer;
- To not alter the rheology of the adhesives in situ, for example by adding other materials (so called inert), unless this practice is expressly permitted in the technical data sheets of the products, with indications regarding the types and quantities;
- To not alter the mixing ratio between the two components of an adhesive; a variation of this ratio, in fact, does not lead to a higher reaction speed, but only to a decay of the mechanical characteristics of the final product.

For adhesives that require a conditioning period (after initial curing) to achieve the full strength, the joint should not be loaded before the necessary time prescribed by the producer.

## 4. Elements, typologies and structural systems (*section 6*)

### *4.1. Elements (Section 6.1)*

The following structural elements are defined:

- Beams and columns: one-dimensional elements made with solid wood or other timber-based product used both as vertical (or sub-vertical) elements predominantly subjected to compression and as horizontal (or sub-horizontal) elements predominantly subjected to bending. Such members, with constant or slightly variable cross-sections, consist of a single elements or more than one connected each other along the longitudinal direction (Fig. 1a).
- Special-shaped beams: single or double tapered beams; curved beams with or without straight end segments; notched beams; perforated-web beams. The latter are not included in the current version of the EC5 (Fig. 1b).
- Composite beams and columns: one-dimensional elements with cross-section composed also by timber in combination with different materials (such as steel or concrete), mechanically or glued connected among them. In particular timber-to-timber and timber-to-concrete composites beams joined with mechanical connections, glued thin-webbed beams, glued thin-flanged beams and composite columns with gussets or packs (Fig. 1c) are treated.
- Panels: two-dimensional elements, having thickness smaller than length and depth, capable to sustain both in-plane and/or out-of-plane loads. Accordingly, panels can be used either as floors or as walls (Fig. 1d).

### *4.2. Structural types (Section 6.2)*

The following structural types, obtained by assembling the above-defined structural elements, are defined:

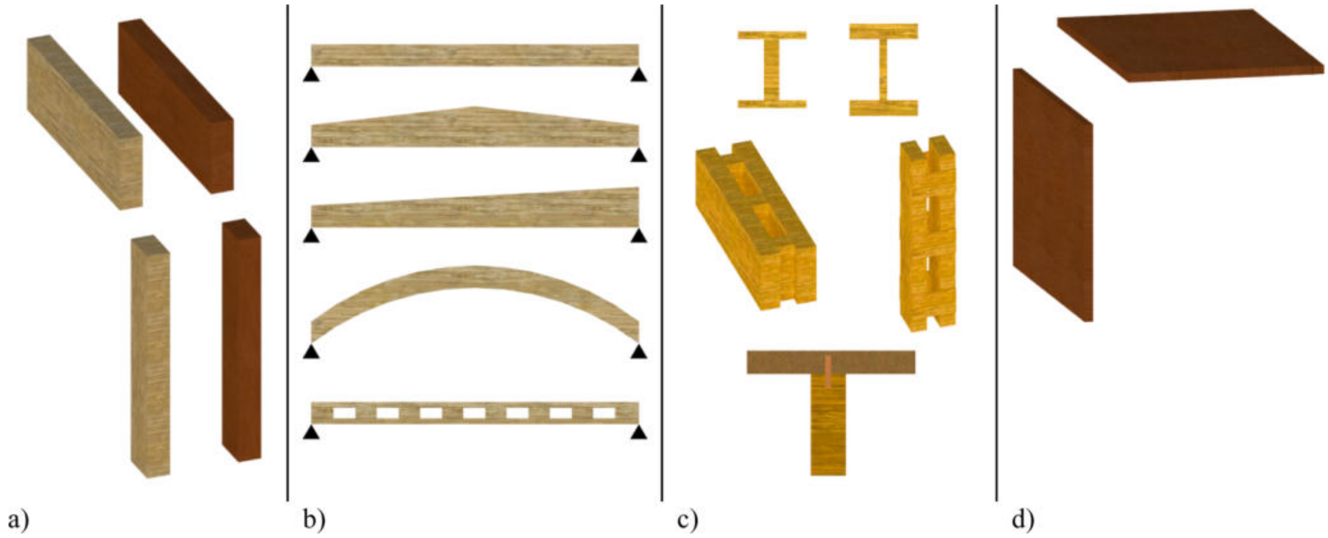



**Fig. 1.** Structural elements: a) Beams and columns; b) Special-shaped beams; c) Composite beams and columns; d) Panels.

- Floor slabs: horizontal load-bearing structures with flexural behaviour able to transfer the vertical loads to other horizontal elements (beams) or directly to the vertical elements (columns). They can undergo one or two-directional bending depending on the static schemes. They can be made with beams, in a single or multi-layered layout, supporting timber or timber-based boards or panels or reinforced concrete slabs or combination of them (Fig. 2a).
- Walls: two-dimensional structures placed in the vertical plane, generally realized with solid timber panels (such as *CLT*) or by assembling one-dimensional vertical and horizontal elements with timber or timber-based thin panels. They can resist both vertical and/or horizontal loads (Fig. 2b).
- Lattice girders: one-dimensional elements, made with solid timber or timber-based products, joined together by *metallic*, carpentry or glued connections. They mainly undergo axial forces in compression or in tension. (Fig. 2c).
- Arches: one-dimensional curved *structures* (statically determinate or indeterminate), whose stability is guaranteed by the horizontal thrust at the base constraint, which produces compressive and flexural forces inside the arch itself (Fig. 2d).
- Framed structures: they consist of beams and columns arranged in orthogonal grids, connected by rigid or semi-rigid joints or pinned with braces. They undergo prevalent shear and bending forces in the first case, axial forces if they are braced (Fig. 2e).

### *4.3. Structural systems (Section 6.3)*

- Structural systems for buildings are grouped in the following three categories:
- Shear-wall structures (light-stiffened frames or CLT panels);
- Heavy timber framed structure;
- Blockhaus.

Light-stiffened frame structures are made with timber framed panels, composed by small-section members, spanning between 40 and 70 cm [26]. Wood-based panels are connected to the timber frame by means of nails, small diameter screws or staples to framed panels, giving rise to a shear-wall, which can resist both vertical and horizontal loads.

The floor slabs are generally realized with small beams, having the same spacing as the vertical walls studs and a slab made with wood-based panels, generally connected to the beams through nails or screws. The wall to floor connections, as well as the connection to foundations are realized through metallic steel-to-timber connections resisting against uplift and shear sliding.

Light-stiffened timber frames can be realized in two structural types: Platform frame (Fig. 3a) and Balloon frame (Fig. 3b). The first one is characterized by timber studs, which are interrupted at each story, while the latter has continuous timber studs along the height of the building.

Cross-Laminated Timber structures are made with *CLT* panels, used for both vertical shear-walls and floor slabs [3,27,28]. Shear-walls resist both vertical and horizontal loads and are used for external and internal

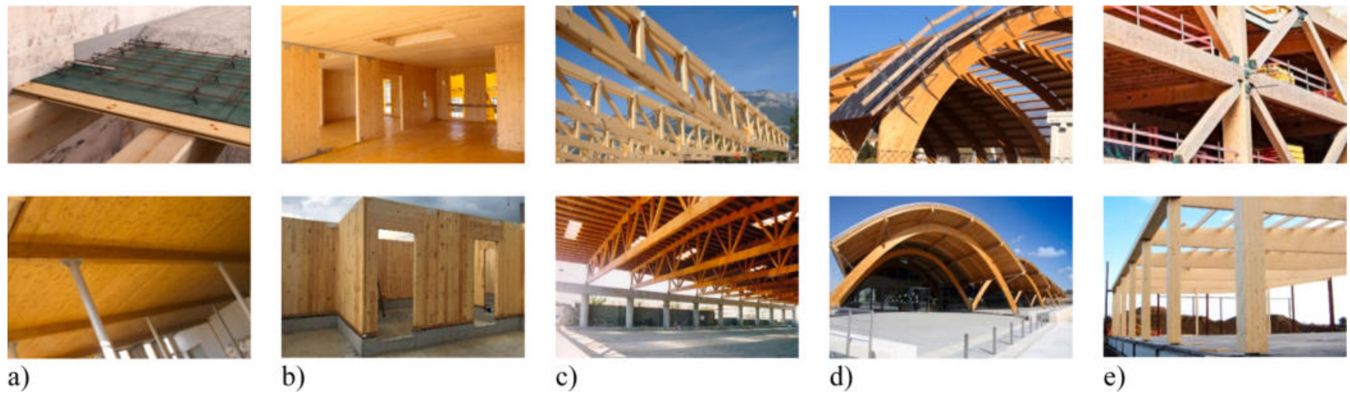
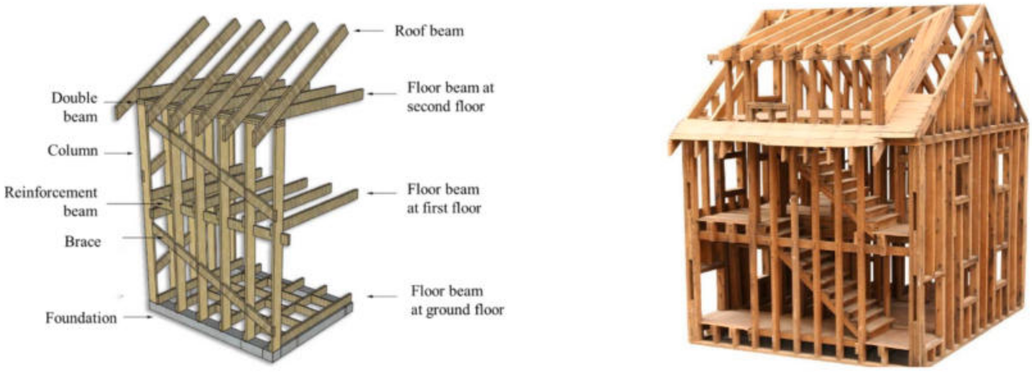



**Fig. 2.** Structural types: a) Floor slabs (www.archiexpo.it; www.galloppinilegnami.it); b) Walls (www.teknoring.com; www.zoppelletto.net); c) Lattice girders (www.canducci.net; www.vgastudio.it); d) Arches (www.venetatetti.com; www.casapratica.org); e) Framed structures (www.arataecobuilding.eu; www.archiexpo.it).

a)




b)

**Fig. 3.** Shear wall structure example. Light stiffened frames: a) Platform frame (www.pbctoday.co.uk; www.ableskills.co.uk); b) balloon frame (www.novocom.top).

walls of the building. Walls may be monolithic, i.e. made with a single panel including openings for doors and windows, with height equal to the inter-storey height of the building and width depending on the maximum producible and movable dimensions. As an alternative, segmented panels of limited sizes (width ranging between 2.0 and 3.0 m), connected together by means of vertical mechanical devices, which guarantee the continuity of the wall, can be used.

The horizontal panels, having width ranging between 1.20 and 3.0 m, can be directly supported by the vertical walls or also by primary beams. They are connected one each other through mechanical joints and realized to guarantee the necessary distribution of loads and required in-plane strength and stiffness, providing a rigid diaphragm behaviour. Reinforced concrete slabs can be also cast at the extrados, realizing a composite timber-concrete structure.

Multi-storey building can be obtained by overlapping more levels of vertical walls and floor slabs, realizing a multicellular box-like system. The panel to panel and panel to foundation connections are realized by means of metallic devices (generally hold-downs and angle brackets) screwed or bolted to timber panels (Fig. 4) able to resist uplift and shear sliding. Compression forces are assumed to be transferred through contact.

In case of two or maximum three-story buildings the walls can be also continuous along the whole height.

Heavy timber frame structures are made by beams and columns with rigid joints, or beams and columns with semi-rigid joints or pinned joints coupled with bracing systems [13]. The latter can be realized through 2D Moment Resisting Frames (*MRF*), concentric (*CBF*) or eccentric (*EBF*) braced frames made with timber or steel, or reinforced concrete core (Fig. 5). *MRF*s are able to resist both vertical and horizontal loads, while in braced frames the vertical loads are resisted by the pinned beam and column structural system and the horizontal loads by the braced system.

Heavy timber structures can be made with solid or glue-laminated timber elements, with square or rectangular solid cross-sections, or with box cross-sections. Columns can also be composed by two vertical elements adjacent each other or connected by battens disposed at constant spaces or by lattice members. Connections between the elements can be realized by means of metallic devices.

The Blockhaus structural system (also named as log-house) consists

**Fig. 4.** Shear-wall structure example: CLT structures (www.unikore.it).

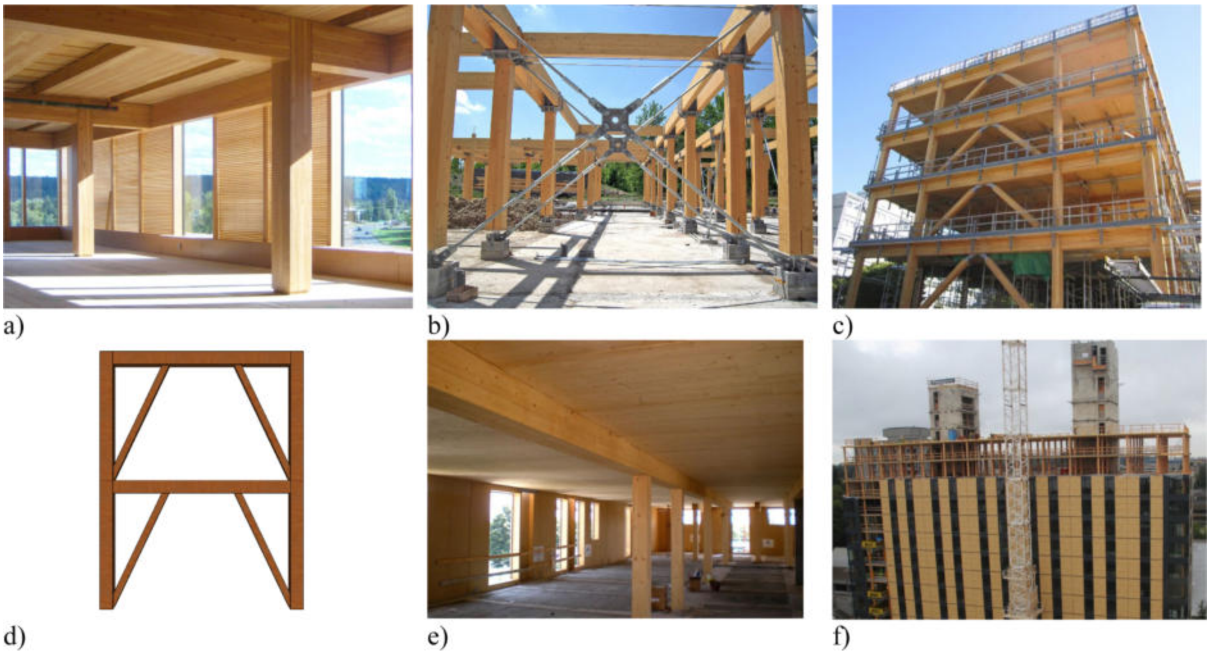


Fig. 5. Heavy timber frame example: a) *MRF* (www.archdaily.com.br); b) *CBF-X* (www.teknoring.com), c) *CBF-V* (www.studiokamyk.com.pl); d) *EBF*; e) *MRF* with *CLT* floors (www.prefabbricatisulweb.it); f) Timber frame with reinforced concrete core (www.firegroundleadership.com).

of walls made with overlapped horizontal timber logs (Fig. 6a) [29,30]. Timber logs can be solid wood or glue-lam elements having circular or rectangular-shaped cross-sections. To enhance the beam-to-beam connections, the members are notched in the longitudinal direction at both the upper and lower sides; in some cases also metallic fasteners are used to increase in-plane stability and stiffness of the walls. Connections between orthogonal log-walls is provided by carpentry joints (Fig. 6b). Floors are made by solid or glue-timber beams, which support timber-based panels (mainly *OSB* panels or plywood, but also *CLT* or timber-to-concrete composite floors), screwed to the beams to ensure in-plane rigid diaphragms. Connection with the foundation are realized by means of metallic fasteners (bolt, screws or nails), which transfer bending moment and shear.

## 5. Design rules and criteria (*section 7*)

### 5.1. Service classes (Section 7.2)

For the three service classes, defined according to weather temperature and humidity, as well as to the moisture content of wood, the corresponding design "environment" has been supplied [18], as reported in Table 4.

Moreover, hygroscopic curves are also given (Fig. 7) for determining the service class in case of non-standard thermo-hygrometric

**Table 4**
Service classes (Table 7-2, CNR-DT 206 R1/2018).

| Service Class | Environment description |
|---|---|
| *1* | Indoor environment, heated during the winter.<br>Mean temperature equal to 20 °C and humidity higher than 65% for few weeks per year.<br>Mean equilibrium humidity of wood, for the great part of softwood, less than 12%. |
| *2* | Indoor environment, also not heated during the winter; outdoor environment, but not directly exposed to weather.<br>Mean temperature equal to 20 °C and humidity higher than 85% for few weeks per year.<br>Mean equilibrium humidity of wood, for the great part of softwood, less than 20%. |
| *3* | Environment where structures are directly exposed to weather, or frequently undergo moistening, or they are even immersed.<br>Mean equilibrium humidity of wood, generally greater than 20% or this value is overpasses for long period per years. |

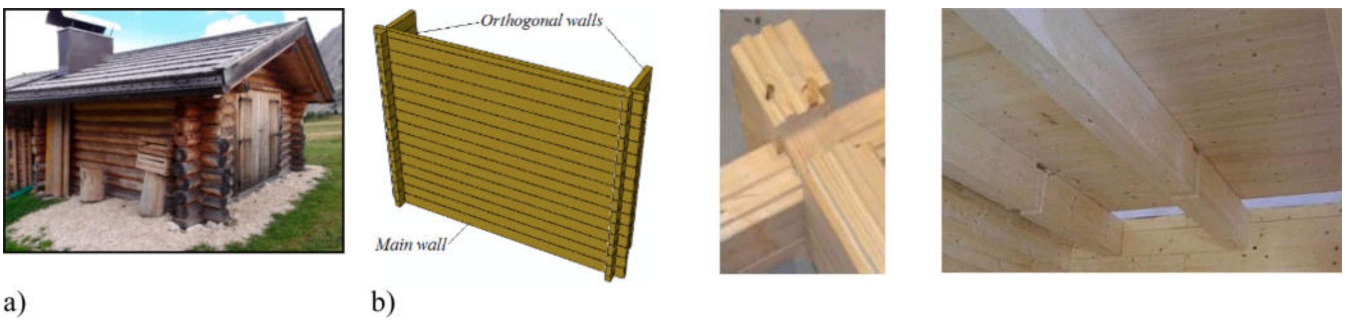



Fig. 6. Blockhaus example: a) traditional rural constructions; b) typical shear-wall, detail of connection between orthogonal walls and floor [30].

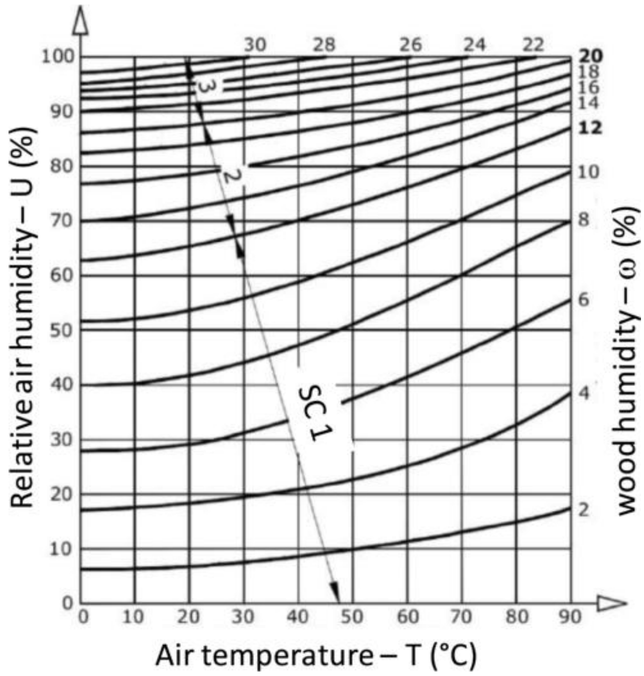



**Fig. 7.** Hygroscopic curves for the determination of the service class (Fig. 7-1, CNR-DT 206 R1/2018).

environmental conditions [31].

### 5.2. *Serviceability limit states (Section 7.5)*

#### 5.2.1. *Instantaneous and final deformation (Section 7.5.1)*

A simple calculation method for deformation is introduced. The total deformation of timber elements can be calculated as it follows (eq. 7.2, CNR-DT 206 R1/2018):

$$w_{fin} = w_{inst} + w_{creep} \tag{2}$$

where $w_{inst}$ is the initial instantaneous deformation, calculated with reference to the characteristic combination of loads, while $w_{creep}$ is the deformation due to creep phenomena which is calculated as it follows (eq. 7.3, CNR-DT 206 R1/2018):

$$w_{creep} = w_i^{\cdot} k_{def} \tag{3}$$

where $w_i$ is the initial instantaneous deformation calculated with reference to the quasi-permanent load combination, while $k_{def}$ is the coefficient that characterize the creep and depends on the service class.

#### 5.2.2. *Vibrations (Section 7.5.4)*

For the vibration check of the floors, reference is made to EC5. In the case of floors with fundamental frequency $\leq 8$ *Hz*, it is also recommended to limit the maximum vertical acceleration due to a dynamic load representative of the walking phenomenon on the floor, in order to avoid possible resonance.

Moreover, it is suggested to calculate the mass of floors in quasi-permanent load condition.

For more detailed analyses reference is made to the international standard ISO 10137 [32].

### 5.3. *Ultimate limit states for structural elements (Section 7.6)*

#### 5.3.1. *Compression perpendicular to grain (Section 7.6.1.1.4)*

The perpendicular to grain compression stress ($\sigma_{c,90,d}$) is determined accounting for an effective area (Fig. 8), which can be greater than the actual area interested by the loads.

In particular, the effective area is evaluated with reference to the effective length $l_{ef}$ determined as it follows, where $l$ is the actual length and $l_{sc}$ is the length of the unloaded zone parallel to grain (Fig. 9), expressed in *mm*:

$l \geq 400$ *mm*, $l_{ef} = l$;

$l$ less than 400 *mm*, $l_{ef} \geq l$, to be evaluated as it follows:

*i)* if on both sides of loaded zone the unloaded zone is extended for a length parallel to grain at least equal to $1/6h$, then $l_{ef}$ is calculated as it follows (eq. 7.9, CNR-DT 206 R1/2018):

$$l_{ef} = min(l + 1/3h;\ 2l;\ 400mm) \tag{4}$$

*ii)* if on a single side of the loaded zone the unloaded zone is extended for a length parallel to grain at least equal to $1/6h$, then $l_{ef}$ is calculated as (eq. 7.10, CNR-DT 206 R1/2018):

$$l_{ef} = min(l + 1/6h;\ 1.5l;\ 400mm) \tag{5}$$

*iii)* if the unloaded length parallel to grain ($l_{sc}$) is less than $1/6h$ the effective length can be calculated as it follows (eq. 7.11, CNR-DT 206 R1/2018):

$$l_{ef} = min(l + 2l_{sc};\ 2l;\ 400mm) \tag{6}$$

*iv)* if on a single side of the loaded zone the unloaded zone is extended for a length parallel to grain at greater or at least equal to $1/6h$,

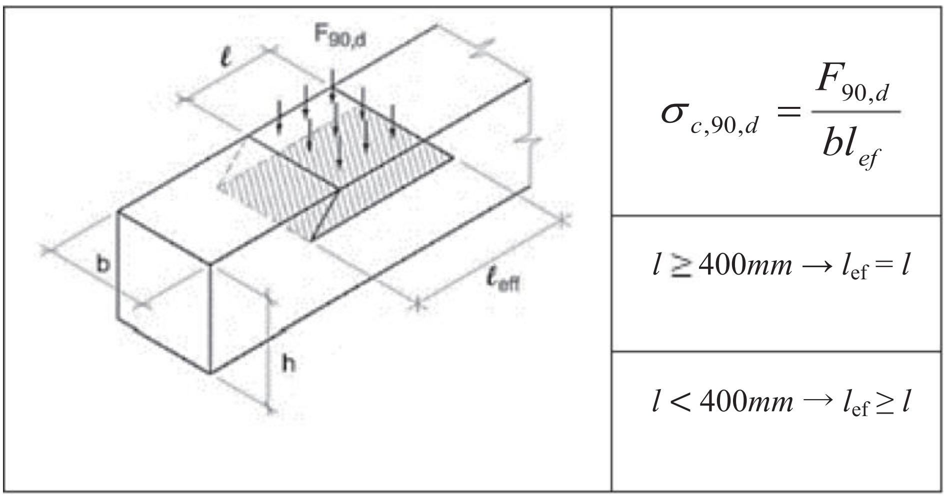



**Fig. 8.** Determination of the effective length.

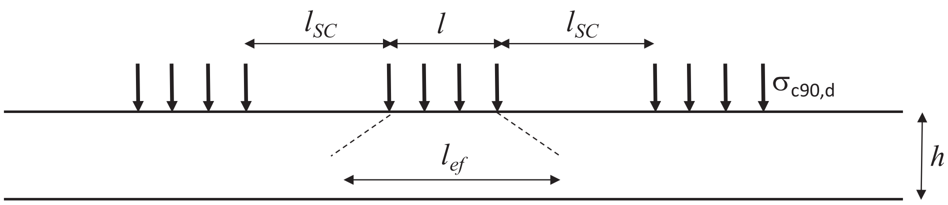



**Fig. 9.** Reference parameters for evaluating the effective length ($l_{ef}$).

then the $l_{ef}$ is calculated as (eq. 7.11, CNR-DT 206 R1/2018):

$$l_{ef} = min(l + l_{sc};\ 1.5l;\ 400mm) \tag{7}$$

*5.3.2. Lateral torsional stability (Section 7.6.1.2.1)*

The values of β coefficients for the determination of the effective length $l_{eff}$ used for the calculation of the critical moment $M_{cr}$ have been slightly modified with respect to EC5 (Tab. 5) and are in accordance with those reported in the German standard DIN 1052 [33]. They vary according to the type of restraint and load condition of the structural element, as summarized in Table 5.

*5.3.3. Webbed-holes beams (Section 7.6.2.3)*

Webbed-holes beams are introduced for the first time. In particular, holes are defined as openings in beams with dimension $d >$ min (50 *mm*; 0.3 *h*), with *d* and *h* expressed in *mm* (Fig. 10). Each hole should be centred with respect to the longitudinal axis of the beam as far as possible, and, in any case, the minimum distances and dimensions shown in Fig. 10 should be respected.

For perpendicular to grain tensile stresses, or for beams and structural elements in service class 3, appropriate transversal reinforcement around the holes (such as glued metallic bars, fully thread screws, external plywood panels or other technologies of proven validity) should be adopted in addition to the requirements given in Fig. 10.

In case of hole with dimension $d <$ min(50 *mm*; 0.3 *h*) safety checks should be referred to the reduced cross-section due to the hole.

In presence of through-holes with $d > 50$ mm the following check should be satisfied (eq. 7.48, CNR-DT 206 R1/2018):

$$\frac{F_{t90,d}}{0.5 l_{t,90,d}\, b f_{t,90,d}} \leqslant 1 \tag{8}$$

where $F_{t,90,d}$ is the perpendicular to grains tensile force, *b* is the section width, $f_{t,90,d}$ is the perpendicular to grain tensile strength and $l_{t,90,d}$ is the length of the beam subjected to the perpendicular to grain tensile stresses. Such length can be evaluated according to the geometry of the hole, as it follows:

$$l_{t,90} = \begin{cases} (0.35h_d + 0.5h) & \text{for circular holes} \\ 0.5(h_d + h) & \text{for rectangular holes} \end{cases} \tag{9}$$

where $h_d$ and $h$ are defined in Fig. 10.

The perpendicular to grain tensile force $F_{t,90,d}$ is to be evaluated, as it follows (eq. 7.49, CNR-DT 206 R1/2018):

$$F_{t,90,d} = F_{t,V,d} + F_{t,M,d} \tag{10}$$

where $F_{t,V,d}$ is the contribution due to the design shear evaluated along the axis of the hole, which should take into account any localized loads

**Table 5**
Coefficient **β** for the calculation of $l_{eff}$.

| Beam type | Loading type | β |
|---|---|---|
| **Simple supported** | Constant moment | 1.00 |
| | Uniformly distributed load | 0.88 |
| | Concentrated force at the mid span | 0.74 |
| **Cantilever** | Uniformly distributed load | 0.490.78 |
| | Concentrated force at the free end | |

and $F_{t,M,d}$ is the contribution due to the design bending moment evaluated along the hole axis. These forces can be evaluated, as it follows:

$$F_{t,V,d} = \frac{V_d}{4}\frac{h_d}{h}\left[3 - \frac{h_d^2}{h^2}\right] \tag{11}$$

$$F_{t,M,d} = 0.008\frac{M_d}{h_r} \tag{12}$$

where $V_d$ and $M_d$ are the design shear and design bending moment at the edge of the hole, respectively, while $h_r$ is evaluated as it follows:

$$h_r = \begin{cases} \min(h_{r0} + 0.15h_d;\ h_{ru} + 0.15h_d) & \text{for circular holes} \\ \min 0.5(h_{r0};\ h_{ru}) & \text{for rectangular holes} \end{cases} \tag{13}$$

with $0.7h_d$ instead of $h_d$ in case of circular holes; where $h_{ro}$ and $h_{ru}$ are defined in Fig. 10.

## 6. Connections (*Section 8*)

### *6.1. Carpentry joints (Section 8.2)*

*6.1.1. General features*

The Section 8.2 dedicated to the carpentry joints represents a novelty with respect to EC5 (which does not include such types of connections). It describes the most common types of joints, giving formulas to check the compressive and shear stresses and dimensional limits.

Carpentry joints deserve particular attention. Several types of carpentry joints are present in ancient buildings, where their assessment is often required [34–36]. Due to the high costs of skilled labour, they were abandoned many years ago in favour of mechanical connections. Despite this, carpentry joints can be still today realized, if provided with mechanical devices able to prevent the joint opening in case of cyclic loads.

The mechanical behaviour of the carpentry joints is characterized by the stress transmission through the contact surfaces, also depending on the friction between them. The structural nodes are made through notches and indentations for which it is necessary to impose dimensional limits in order to avoid excessive weakening of the elements.

The following types of carpentry joints are identified:

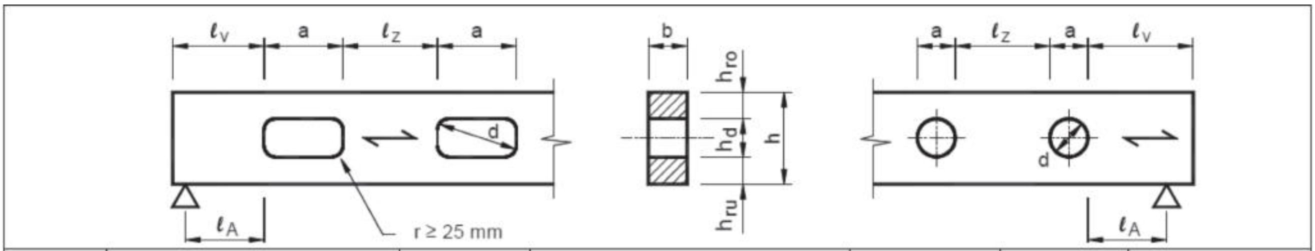
- Simple birdmouth joint (Fig. 11a);
- Double birdmouth joint (Fig. 11b);
- Simple back birdmouth joint (Fig. 11c);
- Scarf joint (Fig. 11d).

The geometrical dimensional limits of each joint useful for design are also defined in Fig. 11. Moreover, the types of strength checks necessary for each type of joint are specified and those for the cases of simple birdmouth joint and scarf joint are illustrated.

*6.1.2. Simple birdmouth joint*

With reference to Fig. 12, under the hypothesis that the strut is subjected to pure compression only and that the friction between the surfaces is neglected, the following strength checks must be carried out:

- Shear on the heel surface (eq. 8.3a, CNR-DT 206 R1/2018):




| | | | | | | |
|---|---|---|---|---|---|---|
| $l_v \geq h$ | $l_z \geq \max(1{,}5h;\ 300\ mm)$ | $l_A \geq 0{,}5h$ | $h_{ro} \geq 0{,}35h$ $h_{ru} \geq 0{,}35h$ | $a \leq 0{,}4h$ | $h_d \leq 0{,}3h$ | $r \geq 25\ mm$ |

**Fig. 10.** Spacing and distances for webbed-holes beams (Figs. 7-9, CNR DT 206 R1/2018).

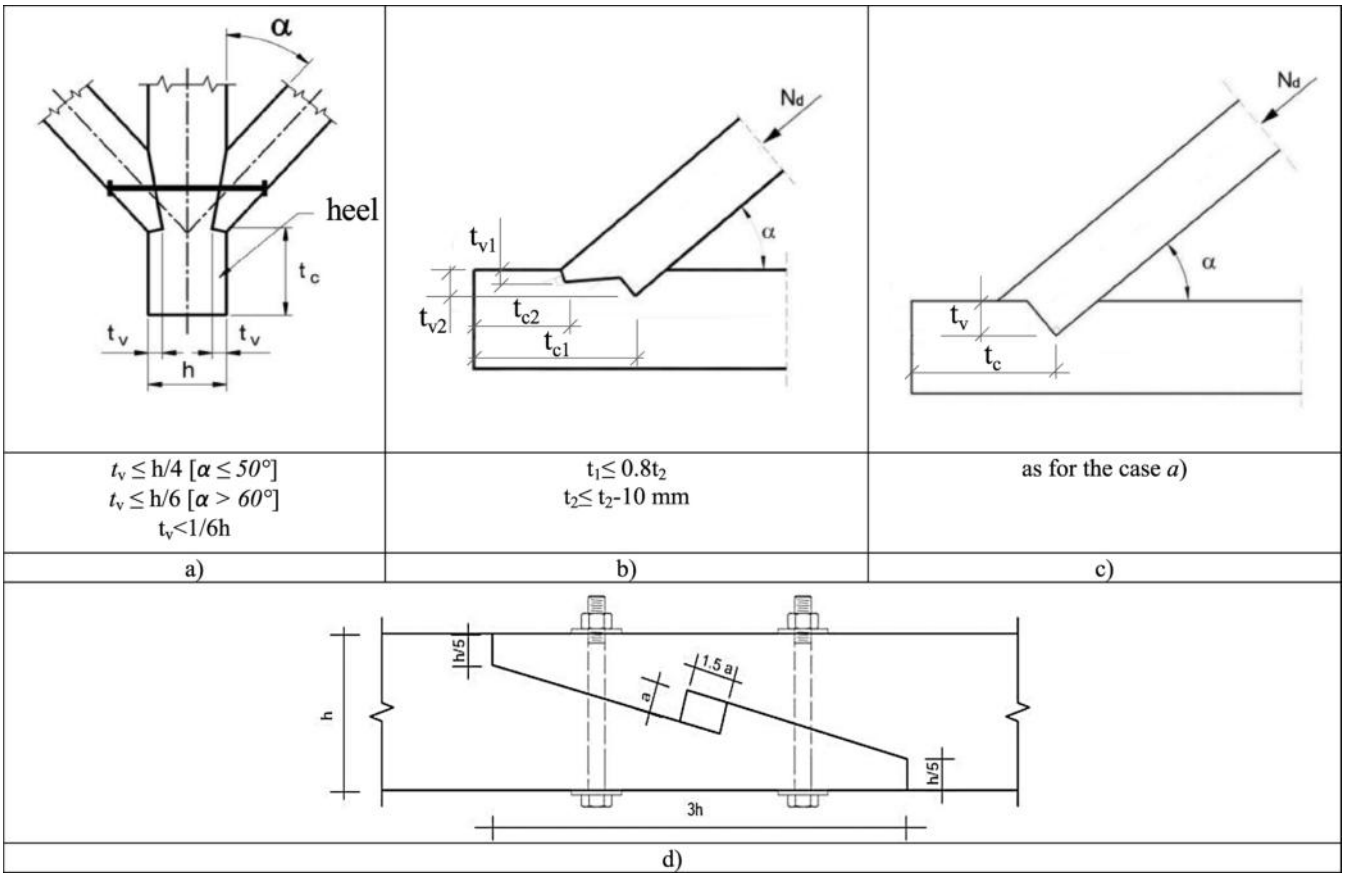



**Fig. 11.** Carpentry joints and dimensional limits: a) Simple birdmouth (Fig. 8-1, CNR-DT 206 R1/2018); b) double birdmouth (Fig. 8-2, CNR-DT 206 R1/2018); c) Simple back birdmouth (Fig. 8-4, CNR-DT 206 R1/2018), d) scarf joint (Fig. 8-5, CNR-DT 206 R1/2018).

$$\tau_d = \frac{F_{1d}}{bt_c} \leqslant f_{v,d} \tag{14}$$

- Inclined to grain compression on tooth surface (eq. 8.3b, CNR-DT 206 R1/2018):

$$\sigma_{c,d} = \frac{F_{1d}}{b[t_v(\cos(\alpha/2))]} \leqslant f_{c,\alpha/2,d} \tag{15}$$

- Inclined to grain compression for the posterior tooth (eq. 8.3b, CNR-DT 206 R1/2018):

$$\sigma_{c,d} = \frac{F_{2d}}{bd} \leqslant f_{c,90-\beta,d} \tag{16}$$

where $F_{1d}$ and $F_{2d}$ are the two components of the axial force of the strut $N_d$ (Fig. 12), the first inclined by $\alpha/2$ with respect to the horizontal axis and the second inclined by β with respect to the vertical axis, $b$ is the width of the cross section of the tie beam; $d$ is the length of the posterior compressed area, which can be assumed equal to $0.25t_v/\sin\beta$ (where $t_v$ is the birdmouth thickness), $t_c$ is the heel length, $f_{vd}$ is the design shear strength of timber, $f_{c,\alpha/2,d}$ is the design compression strength of timber inclined by $\alpha/2$ with respect to the horizontal axis, and $f_{c,90\text{-}\beta,d}$ is the design compression strength of timber inclined by 90-β with respect to the vertical axis.

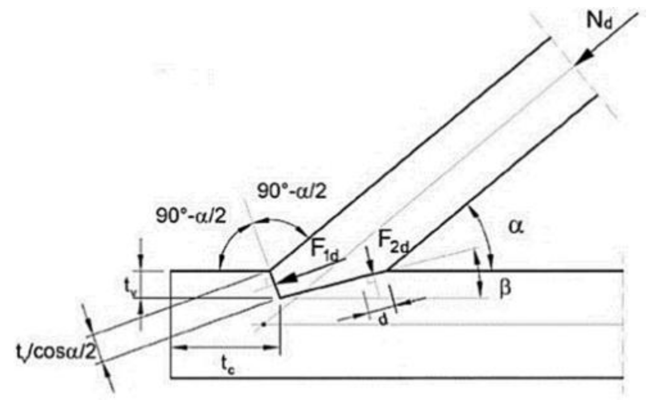



**Fig. 12.** Internal forces on simple birdmouth joint and geometrical features (Fig. 8-3, CNR-DT 206 R1/2018).

### 6.1.3. *Double birdmouth joint*

The double birdmouth configuration involves an increase of both the shear and the compression resistance, with an increase of the heel length and the ratio between the bearing surface and the depth of the notch. It is therefore necessary to perform the inclined to grain compression check for the surface of each birdmouth and the shear check for the two surfaces of the heel, behind the first and second birdmouths.

### 6.1.4. *Simple back birdmouth joint*

The dimension of the notch allows to increase the length of the heel with respect to the case of simple birdmouth joint, resulting in a greater shear-resistant surface. The checks to be carried out are the following ones:

- Inclined compression $\alpha$ respect to the grain for the birdmouth surface;
- Shear of the heel.

### 6.1.5. *Scarf joint*

The configuration of scarf joint with timber key and tightening bolts is considered (Fig. 13). The strength verifications to be carried out are the following ones:

– Shear strength of the birdmouth (eq. 8.4, CNR-DT 206 R1/2018):

$$\tau_d = \frac{N_d}{bl_{ef}} \leqslant f_{v,d} \tag{17}$$

where $\tau_d$ is the design shear stress, $f_{v,d}$ is the design shear strength, $N_d$ is the design axial tensile force, $l_{ef}$ is the effective length of the birdmouth surface and $b$ is the cross-section width (Fig. 13).

– Combined axial tensile force-bending strength of the upper surface of the joint (eq. 8.5, CNR-DT 206 R1/2018):

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leqslant 1 \tag{18}$$

where $\sigma_{t,0,d} = N_d / (b\, h_e)$ is the design tensile stress, $\sigma_{m,z,d} = 6\, M_d / (b\, h_e^2)$ is the design value of the maximum normal stress due to bending, $f_{t,0,d}$ is the design tensile strength, $f_{m,z,d}$ is the design bending strength, $h_e$ is the effective height of the cross-section (Fig. 13), $b$ is the cross-section width, $M_d$ is the bending moment equal to ($N_d\, e$), $e$ is the eccentricity of the axial force with respect to the verification cross-section.

– Inclined to grain compressive strength of the birdmouth in contact with the timber key (eq. 8.6a, CNR-DT 206 R1/2018):

$$\sigma_{c,\alpha,d} = \frac{N_{2,d}}{ba} \leqslant \frac{f_{c,\alpha,d}}{\frac{f_{c,0,d}}{f_{c,90,d}} sin^2\alpha + cos^2\alpha} \tag{19}$$

where $\sigma_{c,\alpha,d}$ is the design value of the compression stress inclined with respect to the grain, $b$ is the the cross-section width, $a$ is the cross-section length, $f_{c,0,d}$ is the parallel to the grain design compression strength, $f_{c,90,d}$ is the perpendicular to grain design compression strength.

– Perpendicular to grain compression strength of the timber key (eq. 8.6b, CNR-DT 206 R1/2018):

$$\sigma_{c,90,d} = \frac{N_{2,d}}{ba} \leqslant f_{c,90,d} \tag{20}$$

where $a$ is the height of the timber key.

e) Tensile strength of the bolts (eq. 8.7, CNR-DT 206 R1/2018):

$$\sigma_{c,90,d} = \frac{N_{1,d}}{dL} \leqslant F_{ax,Rd} \tag{21}$$

where $F_{ax,Rd}$ is the design axial force of the bolt, $d$ is the width of the timber key and $L$ is the distance between bolts.

## 6.2. *Connections with metal fasteners (Section 8.3)*

The related Section 8.3 has been reorganized in order to be more clear and comprehensive. Cylindrical shank fasteners, as nails, dowels, bolts, screws are firstly described [37], together with the principal mechanical characteristics, to address also the practitioners who face for the first time the design of timber structures.

The capacity of multiple fastener connections and multiple shear plane connections is described in its generality. Timber-to-timber, wood based panel-to-timber and steel-to-timber connections resistances are based on Johansen's theory. The different failure modes are illustrated in Refs. [7,38].

Then, the equations for the evaluation of the capacity of the different types of connections with metal fasteners are given according to the in force version of EC5. Main issues are first described in a general view and then declined for each different type. In particular formulas have been modified as dimensionless, wherever possible.

In case of bolted connections the equation for the evaluation of the characteristic value of the yielding moment $M_{y,k}$ is rewritten in such a way that the mechanical meaning is clearer with respect to the corresponding formula of EC5, giving anyhow the same results, valid for bolts with diameter up to 30 *mm* (eq. 8.32, CNR-DT 206 R1/2018):

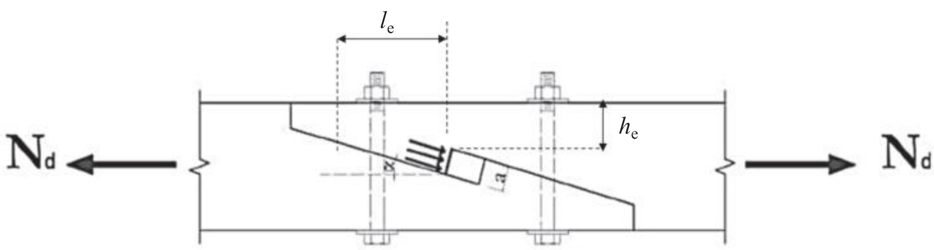



**Fig. 13.** Verification surfaces for scarf joint (Fig. 8-8, CNR-DT 206 R1/2018).

$$M_{yk} = \zeta f_{uk} W_{pl,b} \quad (22)$$

where $f_{u,k}$ is the characteristic value of the ultimate tensile strength; $\zeta = (d_0/d)^{0.4}$ is the reduction factor of the plastic moment that accounts for the ultimate behavior of the connector; $d_0$ is a reference conventional diameter equal to 4.35 *mm* and $d$ is the bolt diameter in *mm*; $W_{pl,b} = d^3/6$ is the plastic modulus of the bolt.

New types of joints as the glued-in rod joints are described and equations are given for the capacity evaluation in case of bars embedded in the grain direction or inclined [26].

The slip moduli ($k_{ser}$) for cylindrical shank connectors included in the CNR Document are those provided by the current version of EC5, thus they have not been reported in the present paper.

Particular attention is devoted to the minimum spacing and end edge distances for axially loaded screws details, including in the CNR Document all the limitations provided by EC5. An example of a steel to timber bolted connection in the Appendix F illustrates the application of the code equations.

## 7. Specific rules for structural types and systems (*Section 9*)

### *7.1. Structural types (Section 9.1)*

#### *7.1.1. Trusses (Section 9.1.1)*

Trusses should be generally assumed as systems of beams, taking into account the joints deformability and possible eccentricity of the connections. For the resistance verification, when the maximum transverse dimension of each element is less than 1/10 of the maximum height of the truss, a structural model with hinged joints can be assumed for evaluating the axial forces in the elements. For the evaluation of the bending stress, in case of a continuous element on multiple spans, the structural member can be modelled as a continuous beam supported at the joints of the reticular structure. It is possible to take into account the joint displacement by a 10% reduction or increment of the maximum values of bending moments respectively at the joints and within the bay.

#### *7.1.2. Arches (Section 9.1.2)*

The in-plane stability of arches should be verified through a second order analysis. Approximate methods of verification can be adopted, by referring to an equivalent compressed element having a predefined buckling length. In particular, for arches with two or three hinges, having height to width ratio between 0.15 and 0.5 and subjected to uniformly distributed load, it is possible to adopt a buckling length equal to 1.25 times the half length of the arch. Arch structures (statically determined or undetermined) should always be designed also for out-of-plane stability, so that they should be suitably braced. For arches and all thrusting structures, the external restraints should be able to support the thrusts necessary for the structural equilibrium, without appreciable deformation, or it is necessary to provide specific ad hoc designed elements for facing the thrusts.

#### *7.1.3. Framed structures (Section 9.1.3)*

The stability of each structural member should be verified taking into account the joints deformability and the presence of bracing systems, considering the restraint and stress conditions. In general, for the global instability verification it is necessary to consider the geometrical and structural imperfections and the *P*-delta effects due to the vertical loads, using the load-class corresponding to the specific actions. The stability of the frames can be verified with a second order analysis. When the lateral stability is guaranteed by a bracing systems (in case of moment resisting joints), the columns buckling length, in the absence of a rigorous analysis, can be assumed to be equal to the column height (inter-storey height). In case of pinned joints, the global stability can be verified approximately by checking that the structure is able to resist simultaneously the most unfavourable loads combination, without wind, together with conventional horizontal action, to be considered within the medium duration load class, equal to 1/80 of the vertical loads corresponding to the loads combination used, to be considered as a fundamental combination at the ultimate limit states. Moreover, the instantaneous maximum horizontal displacement, at the service limit state, should be less than 1/500 of the total height of the frame.

#### *7.1.4. Bracing systems (Section 9.1.4)*

The structures must be adequately braced to resist the horizontal forces. The bracing system, both horizontal (floor and roofs) and vertical (walls), is assumed to be realized through either diaphragms made with timber panels or wood-based material, fixed to timber frames by mechanical connections, or single one-dimensional diagonal members, or truss systems, made with timber or wood-based material or other (e.g., steel).

Design criteria reported in the CNR Document are in agreement with the indications given by EC5. Thus, specific rules for stability checks for single members in compression supported by intermediate elastic constrains are reported (Section 9.1.4.3.1, CNR-DT 206 R1/2018), as well as those for bracing of beam or truss systems (Section 9.1.4.3.3, CNR-DT 206 R1/2018).

### *7.2. Structural systems (Section 9.2)*

#### *7.2.1. Light timber frame buildings (Section 9.2.1.1)*

Light-stiffened panels used for buildings have height equal to that of inter-storey. The connection between the orthogonal walls is made by mechanical fasteners (screws or nails) connecting the two vertical elements at the end edges of the panels. Wall to foundation connections should be made by means of mechanical connections (hold-downs, angle brackets, anchor bolts, nails or screws), capable of transferring bending moments and shear forces. The first ones are located at the ends of the panels and at the openings, while the second ones are uniformly arranged over the horizontal edge of the wall.

The floors are generally made up by a simple or double layout of beams, stiffened by upper and/or lower timber-based panels connected to the below structure by means of screws, nails, or other devices over the perimeter of each panel. On the floors perimeter it is necessary to realize a continuous tensile-resistant ring beam either for efficiently connecting the vertical walls each other or to prevent the overturning of the walls due to horizontal actions, or to transfer the in-plane horizontal actions to walls.

When *CLT* panels are used, they should be connected through horizontal joints realized by mechanical fasteners (screws or nails). In case of timber-concrete composite floors, the concrete slab should be connected to the timber elements by means of shear connector devices with sufficient strength and stiffness.

#### *7.2.2. CLT buildings (Section 9.2.1.2)*

*CLT* panels have height equal to that of inter-story. They are made with a single wall or with segmented walls (i.e., composed by panels coupled each other). In the latter case, each panel should have a length greater than *0.25 h* (*h* is the inter-story height of the building) and it is connected to the adjacent ones through mechanical fasteners (nails, screws or steel plates) placed in the vertical joints. Wall to foundation connections should be made by means of mechanical connections (hold-downs, angle brackets, anchor bolts, nails or screws), capable of transferring bending moments and shear forces. Flexural-resistant connections are located at the ends of the panels and of the openings, while the shear resistant connections are uniformly positioned over the horizontal length of the wall. Connections between vertical walls (upper and lower) are similar to the walls to foundations connections. On the external sides of the walls, metallic plates nailed or screwed to the panels are suitable. The connections between the orthogonal walls are made with mechanical elements (usually only inclined screws or nails or angle brackets connected with screws or nails).

Floors are usually made by *CLT* panels joined one each other by horizontal mechanical connections (screws or nails). They provide in-plane rigid diaphragms. Different types of floors can be also used, provided that they ensure sufficient in-plane stiffness. In any case, the floor structures is directly supported by the walls, to which efficient connections through metallic elements should be realized, assuring the stability against the wall overturning and the transmission of the in-plane shear forces.

*7.2.3. Heavy timber frame buildings (Section 9.2.1.3)*

In case of Moment Resisting Frames, beam-to-column joints connections (rigid or semi-rigid) are realized through metal fasteners and must be able to resist bending moment, while the column to foundation connections can be also pinned.

For braced frames, either beam- to-column and braces connections or connections to the foundations are pinned. Generally, they are realized with cylindrical shank metal fasteners.

Different types of floors can be also used (like parallel or orthogonal beams stiffened by panels, *CLT* panels, timber-to-concrete composite structures), provided that they have sufficient in-plane strength and stiffness and that they are opportunely connected to the frames for ensuring the transfer of the seismic actions.

*7.2.4. Blockhaus buildings (Section 9.2.1.4)*

Solid or glue-laminated timber logs with rectangular or circular cross-section used to cast the walls should have upper and lower notches to facilitate the overlap and to improve the out-of-plane stability of the walls. The wall to foundation connections consist of mechanical connections (hold-downs, angle brackets, anchor bolts, nails or screws) capable of transferring flexural and shear forces. The flexural-resistant connections are positioned at the ends of the panels and at the openings, while the shear-resistant connections are uniformly located along the wall length.

It is necessary to verify that under seismic actions the overlapped logs do not up-lift and that the vertical load is sufficient to ensure a fully compressed cross-section of the walls. Otherwise, tensile-resistant metal fasteners between the beams (steel ties, screws or other) should be provided.

Different types of floors can be also used (like parallel or orthogonal beams stiffened by panels, CLT panels, timber-to-concrete composite structures), provided that they have sufficient in-plane strength and stiffness and assure the stability against the wall overturning and the transfer of the in plane shear.

## 8. Design for earthquake resistance (*Section 10*)

### *8.1. Behaviour of the structural systems (Section 10.2)*

This Section refers to timber buildings, whose structural systems are those described in the Section 4.3 of the paper (i.e., Section 6.3 of the CNR Document).

In light timber frame buildings the seismic actions are resisted by the overall structure. The strength and stiffness of the shear walls are directly related to the geometry and to the number and diameter of the sheathing to frame connections, generally made with nails, screws or staples, and to the type and number of both wall to wall and wall to foundation connections used.

In heavy timber frame buildings, the horizontal actions are resisted by timber frames with rigid or semi-rigid joints, or by braced systems (i.e., timber or steel braces, infilled timber panels) with pinned connections. The seismic behaviour is similar to that of reinforced concrete buildings, or of steel buildings. Beam to column joints (in case of *MRF*) and braced systems are fundamental for the seismic resistance of the construction. Also the floors play the fundamental role of sharing seismic actions among the lateral resisting vertical frames, so that they should be verified against in-plane actions.

In *CLT* buildings, both internal and external walls contribute to the seismic resistance. Shear walls, made with *CLT* panels, should resist both gravity and horizontal loads. Being *CLT* panels designed with over-strength with respect to the amount of both gravity and horizontal actions, the seismic behaviour is governed by the wall to wall and wall to foundation connections. In case of segmented walls joined by vertical connections, the walls have a lower in-plane stiffness with respect to the monolithic walls, consequently it is necessary to increase the number of tensile resistant connections at the base of the walls. Less frequently, walls may be made of a single element having the same height of the building without interruption at each storey (balloon construction). In this case both tensile and shear-resistant connections are placed only at the foundation and should therefore be carefully designed with regards the horizontal loads.

Floors, generally made by *CLT* panels, undergo low in-plane stresses when the vertical walls are uniformly distributed in plane; if this is not the case, similar criteria as for heavy timber frames should be applied.

In blockhaus buildings, generally with one or two storey, the walls represent the horizontal and gravity load-resisting systems. Typically such walls are squat, thus the behaviour is dominated by shear and the strength is governed by friction between the logs (and then by the vertical load acting on the wall). The global flexural behaviour is faced by the connections against up-lift.

The wood-based sheathing materials, for light and heavy timber frames, should comply with the following requirements, given with respect to the specific relevant standard:

- Particleboard-sheathing (according to UNI EN 312 [39]) with a minimum density of 650 kg/m3;
- Plywood-sheathing (according to EN 636 [40]), at least 9 mm thick and with at least 5 layers;
- Fibreboard-sheathing (according to EN 622 [41]) with a minimum thickness of 12 *mm*;
- Oriented Strand Board (OSB) sheathing (according to EN 300 [42]) with a minimum thickness of 12 *mm*;
- Gypsum Fibre board (GF) (according to EN 15283-2 [43]) with a minimum thickness of 12 *mm*.

### *8.2. Dissipative structures, behaviour factors, ductility classes and capacity design (Section 10.3)*

Capacity design approach for timber buildings is implemented in the CNR Document. Earthquake-resistant timber structures can be designed considering two different behaviours:

a) Non-dissipative behaviour;
b) Dissipative behaviour.

In concept *a*), it is necessary to verify that all the structural elements and the relative connection zones do not exceed the elastic limit. In concept *b*), it is necessary to provide dissipative zones in the structure, capable of resisting seismic actions with sufficient deformation capacity in plastic field (ductility). Typically, seismic energy dissipation occurs in the connection zones, while timber elements remain elastic. Plastic dissipation is given by a combination of yielding of the metal fasteners and timber embedding. Instead, both glued connections, axially loaded metal fasteners and carpentry joints cannot be considered as dissipative.

In the case of not dissipative structures a behaviour factor at least equal to 1.50 can be applied to reduce the seismic actions.

**Table 6**
Behaviour factors ($q_o$) for timber buildings (Table 10-1, CNR-DT 206 R1/2018).

| *Structural systems* | *CDA* | *CDB* |
|---|---|---|
| Light-frame structures | 4.0 | 2.5 |
| Heavy moment resisting frames | 4.0 | 2.5 |
| Heavy braced frames | – | 2.0 |
| CLT buildings | 3.0 | 2.0 |
| Blockhaus buildings | – | 2.0 |

In the case of dissipative structures two different ductility classes are identified: High Ductility Class (namely *CDA* in the Italian standard, corresponding to *DCH* in EC8); Medium Ductility Class (namely *CDB* in the Italian standard, corresponding to *DCM* in EC8). Behaviour factors for both classes *CDA* and *CDB* are larger than 1.50. They are listed in Table 6 for the different seismic-resistant structural systems. In case of irregularity in elevation, the values of behaviour factors reported in Table 6 should be reduced by 20%.

The structure can be considered in *CDA* if the following two conditions are respected:

i) connections exhibit ductile failure mechanisms characterized by the activation of two plastic hinges in the metal fasteners (according to the Johansen's theory);
ii) brittle failure mechanisms do not occurs.

In the case of timber to timber and timber to steel connections with dowel-type metal fasteners, the condition *i*) can be considered satisfied if the thickness of the members is larger than 10*d* (where *d* is the diameter of the fastener) and the diameter *d* is lower than 12 *mm* or if, with respect to the specific structural system, the following conditions are satisfied:

– for light timber frame buildings: the sheating is composed by plywood or *OSB* panels having a minimum thickness equal to 4*d* (where d is the diameter of the fastener) and greater than 3.1 *mm*;
– for light-timber frame buildings: the failure mechanism of the connections, realized through ring nails or with screws, between the sheating panels and the timber frame is ductile with activation of a single plastic hinge in the metal fastener (differently from what provided in the case *i*)).
– for *CLT* buildings: walls with length to height ratio greater than 1.0 are divided in sub-panels with length varying between 0.25 *h* and *h* (where *h* is the inter-storey height), connected along the vertical joints by means of metal fasteners (screws of nails) having a ductile failure mechanisms characterized by the activation of a single plastic hinge in the metal fastener. (differently from what provided in the case *i*)).

The structure is considered of class *CDB* if in the dissipative zones occurs that:

iii) the connections exhibit ductile failure mechanisms characterized by the activation at least of one plastic hinges in the metal fasteners;
iv) brittle failure mechanisms do not occurs.

In the case of timber to timber and timber to steel connections with dowel-type metal fasteners, the condition *i*) can be considered satisfied if the thickness of the members is lower than 8*d* (where *d* is the diameter of the fastener) and the diameter *d* is lower than 12 *mm* or if, with respect to the specific structural system, the following conditions are satisfied:

– for light timber frame buildings: the sheating layer of the resistant shear walls is composed by type of panels described in the section 8.1 (i.e., Section 10.2 of the CNR-DT 2016 R1/2018) having a minimum thickness equal to 3*d* (where d is the diameter of the fastener) and greater than 3.1 *mm*;
– for light timber frame buildings: the failure mechanism of the connections, realized through nails, screws or staples, between the sheating panels and the timber frame is ductile and characterized by the activation of a single plastic hinge in the steel metal fastener (differently from what provided in the case (*i*)).

Furthermore, the application of the capacity design criteria, assuming that the design strength of the brittle elements should be always larger than that one of the ductile elements, increased by an overstrength factor, is required.

For *CDA* and *CDB*, overstrength factors should be equal to 1.60 and 1.40 for heavy timber *MRF* and vertical cantilever walls respectively, to 1.30 and 1.10 for other structural systems, respectively.

Floors should be designed for the design seismic load increased at least by 30%. Moreover, the connections to the vertical elements should be also designed with an overstrength factor as respect to the slab in-plane strength equal to 1.30 in *CDA* and 1.10 in *CDB*.

In the case of blockhaus buildings, it is required that the stabilizing moment induced by gravity loads (calculated in seismic condition) is greater than the overturning one due to seismic actions increased by an overstrength factor equal to 1.50.

The structural dissipative behaviour (especially for buildings made with shear walls) can be also achieved adopting alternative connection systems in which the energy dissipation is entrusted to specific devices connected to timber elements, while timber to timber connections are designed with sufficient overstrength (i.e., using an overstrength factor equal to 1.50 at least). As main advantage, these connections are able to reduce (or eliminate) the structural damage to the timber elements after an earthquake.

### 8.3. *Structural analysis (Section 10.4)*

Structural analysis of timber buildings can be conducted by means of linear or nonlinear methods, taking into account the stiffness of the connection systems. In case of linear analyses, such stiffness can be calculated through the slip modulus $k_{ser}$.

For panel-based structures (i.e., light timber frame buildings, *CLT* buildings or similar systems) the deformability of the panel-to-panel or panel-to-foundation connections must be considered in the structural analyses. Besides, both orthogonal to grain deformability of timber in compression and that of steel-to-timber connections in tension should be included.

Depending on the arrangement of the seismic-resistant elements, both plane or three-dimensional structural models can be used for the structural analyses. For heavy timber frame buildings, with rigid or semi-rigid moment resisting joints or bracings, frame models are suitable.

For shear wall buildings, either *a*) two-dimensional finite elements models or *b*) equivalent frame models can be adopted. In case *a*), panel-to-panel connections must be schematized considering separately the contributions of the tensile-resistant elements (i.e., hold-downs), shear-resistant elements (i.e., angle-brackets), timber-to-timber and timber-to-concrete contact. In case *b*), connections can be modelled through simplified local flexural and/or shear springs connected to the frame elements and provided of equivalent stiffness corresponding to the actual one. These latter indications remain valid also in the case of heavy or braced timber frame.

In case of nonlinear static analyses conducted on structural models defined according to the case *a*), the tension, compression and shear behaviour of the plastic hinges must be defined. For structural models corresponding to case *b*), instead, combined axial force-bending moment and shear plastic hinges must be defined. Moreover, in both case *a*) and *b*), timber elements are considered with a linear elastic behaviour.

To date, nonlinear dynamic analyses are not suggested for practical purposes. This is because they require detailed modelling of the post-elastic cyclic behaviour of the connections, as well as a correct definition of the seismic input (i.e., accelerograms).

In the case of three-dimensional structural models, floors can be schematized as in-plane rigid diaphragms, provided that at the end of the analyses such an assumption is proved. Moreover, it should be checked that they have a sufficient overstrength, as described in the Section 8.2 (i.e., Section 10.3 in the CNR-DT 206 R1/2018). If such check is not satisfied, the actual stiffness of floors should be considered in the three-dimensional structural model.

### *8.4. Ultimate Limit State (ULS) verifications (Section 10.5)*

For Ultimate Limit State verification reference is made to current standards for timber constructions both national and Eurocodes, while capacity design criteria are applied according to Section 8.2.

For dissipative zones, a strength reduction factor equal to 0.80 due to cyclic degradation should be considered. For non-dissipative components of dissipative structures or for non-dissipative structures, elastic limit strength of structural members and connections is considered, no degradation should be taken into account.

### *8.5. Damage Limit State (DLS) verification (Section 10.6)*

CNR Document contains also limitation of the inter-storey drift at the Damage Limit State (*DLS)* for the structural systems, as indicated in the EC8 (Section 4.4.3.2).

## 9. Structural robustness (*Section 12*)

With respect to EC5, the CNR Document includes practical indications devoted to the structural robustness. The robustness is defined as the capacity of the structure to resist exceptional actions not foreseen in the design phase, avoiding structural damages disproportionate as respect to the extent of the causes. To withstand unexpected exceptional actions, some design strategies are suggested, such as: to design against conventional exceptional actions; to prevent effects induced by exceptional actions; to reduce possible exceptional actions; to adopt structural solutions that withstand the exceptional actions or that are able to sustain local damage; to design redundant structures provided with ductility; to adopt control systems of the actions and their effects. Furthermore, it is clarified that such structural strategies can be implemented by adopting adequate constructional measures, that reduce the sensitivity of the timber structure to exceptional actions, both conventional (such as earthquake, or fire) and not conventional (such as unexpected weather events, or high moisture contents in the material, biotic attacks). Possible measures can be the implementation of structural systems low sensitive to partial collapses, connections low sensitive to fire, protection of the structural elements against the humidity, adoption of ductile connection systems, prevention of orthogonal to grain forces on timber elements.

## 10. Existing structures (*Section 16*)

The CNR Document allows for strength checks of existing timber structures, provided that the mechanical properties of timber elements are determined and the degradation state quantified, taking into account the possible reduction of the mechanical properties.

The satisfaction of all the requirements provided by the CNR Document are sufficient for the safety of the structure. On the other side, in many cases, especially for buildings with significant historical and cultural interest, the classification methods used for modern timber elements or the protocol of checks could provide too conservative results, which can induce to very invasive consolidation intervention. Therefore, for structure in good mechanical conditions, criteria of improvement of the behaviour as an alternative to the upgrading of the structure can be adopted for the design of retrofitting.

The following reference standards to be used for assessing the conservation state and classification of the structural elements are suggested: i.e. UNI 11118 (2004) [44], UNI 11119 (2004) [45] and UNI 11130 (2004) [46]. With regards the criteria for the execution of loading tests it refers to the standard UNI EN 380 (1994) [47], while for the design of the interventions to UNI 11138 [48], CNR-DT 201/2005 [49] and CNR-DT 212/2013 [50].

## 11. Appendices (*Section 17*)

### *11.1. Premise*

Hereafter Appendix C (i.e., Coefficient for calculation of deformations) and Appendix E (Design of composites beams with flexible connections), being similar to EC5, are not discussed.

### *11.2. Appendices A-timber grades (Section 17.1)*

EC5 does not report timber strength grades explicitly, but it refers to the specific standards (EN 338 [51]; EN 14080 [52]). Contrary, the CNR Document reports the tables of the timber strength grades for solid timber (EN 338), both hardwood and softwood species, as well as for glulam timber (EN 14080), both homogeneous and combined. Strength grades tables specifically related to Italian autochthonous species are also given, according to UNI 11035-2 standards [53].

### *11.3. Appendices B-Coefficients for the determination of the design strength (Section 17.2)*

The partial safety factors $\gamma_M$ and the coefficients $k_{mod}$ are provided for timber and timber-based products for structural applications, according to EC5 [5], Italian National Annex of EC5 [54] and Italian Standards for Constructions D.M. 17-01-2018 [55]. The values of $\gamma_M$ for the above reference standards are summarized in Table 7. It is to be noticed that the D.M. 17-01-2018 indicates two different coefficients (i.e., column *A* and *B* in Table 7): in column *A* $\gamma_M$ coefficients are currently used for structural checks, while in column *B* values are suggested in case of structural elements made with materials produced under a continuous control (whose strength is affected by a coefficient of variation lower than 15%).

Values of the coefficient $k_{mod}$ (such as the modification factor taking into account the effect of the duration of load and the moisture content) are those provided by EC5.

### *11.4. Appendices F-Examples of calculations (Section 17.5)*

The design of some frequent cases in engineering practice are proposed in the CNR Document and numerically developed. The considered examples are shown in Fig. 14.

## 12. Conclusions

The paper presents the novelties introduced by the Italian Technical Document CNR DT 206 R1/2018 concerning "The instructions for the design, execution and control of timber construction". The CNR Document collects the most advanced knowledge on the behaviour and design of the new generation of timber structures, it coming from a spontaneous cooperation of an open group of academic specialists and

**Table 7**
Values of $\gamma_M$ according to standards.

| | Eurocode 5 | National Annex to Eurocode 5 | Italian D.M. 17-01-2018 | |
|---|---|---|---|---|
| | | | Column A | Column B |
| 1. Fundamental combinations | | | 1.50 | 1.45 |
| Solid wood | 1.30 | 1.50 | 1.45 | 1.35 |
| Glulam timber | 1.25 | 1.45 | 1.45 | 1.35 |
| Fibre-based panels | 1.30 | 1.30 | 1.50 | 1.40 |
| LVL, CLT, OSB | 1.20 | 1.20 | 1.40 | 1.30 |
| Connections | 1.30 | 1.30 | 1.50 | 1.40 |
| 2. Exceptional combinations | 1.00 | 1.00 | 1.00 | 1.00 |

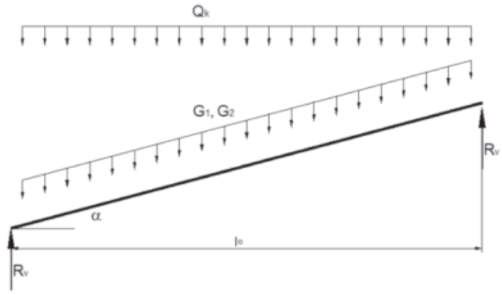
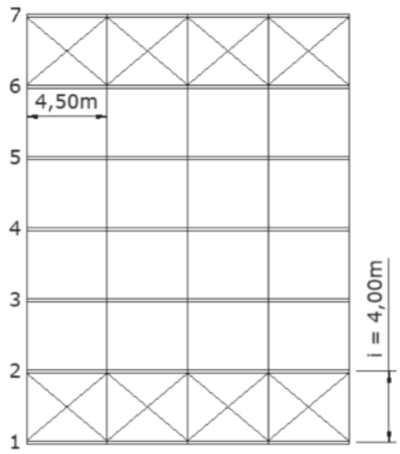
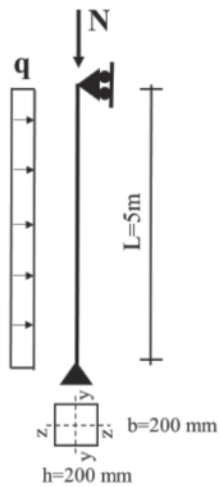
| | |
|---|---|
| Inclined roof beam |   |
| Braced roof |   |
| Column buckling |   |

**Fig. 14.** Examples of calculation.

operators of the sector.

In the perspective of the European framework, the CNR Document is harmonized with the current provisions included in the EC5 and EC8, taking also into account some improvements that are worth to be included in the revised and updated versions. Then, all the differences and the novelties introduced by the CNR Document with respect to Eurocodes have been highlighted and discussed.

As a consequence of the technological progress, the range of the structural products is rationalized and expanded including the most diffused typologies of one and two-dimensional elements used in timber constructions practice. Particular attention has been devoted to the determination of the service classes: the hygroscopic curves in case of non-standard thermo-hygrometric environmental conditions are introduced. Moreover, the problems of thermal variations and creep phenomenon are faced providing more details for evaluating the dimensional variations of the material.

The 'architecture' of the CNR Document is different with respect to Eurocode 5 and 8. The whole body of the document is based on a hierarchical order among *structural elements*, *typologies* and *systems,* replicated with the same order of importance in all the chapters. For each of them a technological and morphological description is provided in the document (Section 6), while the structural configuration to resist gravity

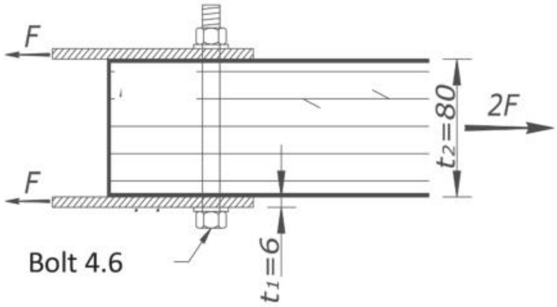
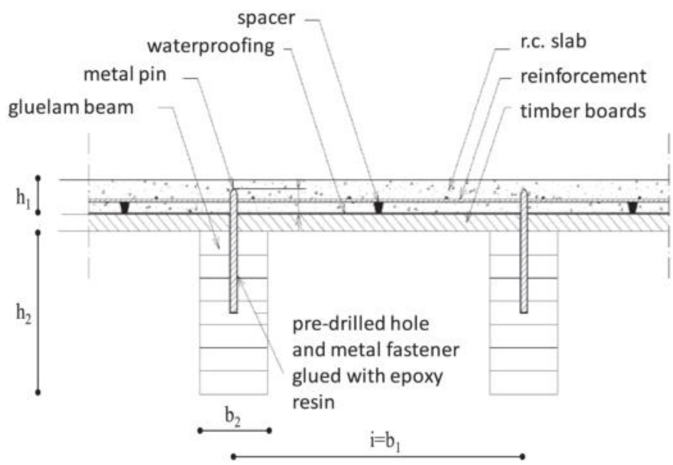
| | |
|---|---|
| Steel-timber bolted connection |  |
| Composite timber to concrete floor |  |

**Fig. 14.** (*continued*).

loads and seismic actions and the related design rules are described in the subsequent Sections (Sections 9 and 10).

The strength and stability checks of the elements at Ultimate Limit States (Section 7) have not been substantially modified, except for the orthogonal-to-grain compression and shear checks and for the lateral torsional buckling check (i.e. review of the effective length); besides, the calculation of flexural deformations of the members at Service Limit States are slightly rewritten, although the basic concepts remain the same.

The seismic design of timber buildings represented the key topic of the revision process of the CNR Document. By analogy with the other structural systems (i.e., reinforced concrete and steel buildings), the 'capacity design' approach has been also introduced for the design of seismic-resistant structures. Then, the dissipative behaviour of the connection zones, behaviour factors, ductility classes and hierarchy of strength to be used in linear static analyses are described in detail.

Strength checks of carpentry joints (Section 8) represented another novelty with respect to EC 5. The most diffused joint types present in historical constructions have been included with the relative strength checks. Conversely, few modifications are applied to the cases of steel-to-timber mechanical connections.

With the purpose of providing an easy-to-use tool for practical applications, some Appendices have been also included in the Document: Appendix A, concerning timber grades, includes Tables with strength profiles for both solid and glue-laminated timber; Appendix B indicates partial safety factors provided by Eurocodes and Italian Standards; Appendix F introduces example of calculation relative to the most common structural systems (roof beams, timber-to-concrete composites floors, mechanical connection, members stability, etc.).

Despite the good achieved results, the activity of the committee is still in progress with the aim of a continuous update and improvement of the Technical Document according with the advancement of knowledge.

## Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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