The New Provisions for the Seismic Design of Timber Buildings in Europe

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Highlights

- A review of the different previous versions of Chapter 8 of Eurocode 8 is presented.
- New definition of structural types is presented with graphical description.
- Capacity design rules, ductility provisions and over-strength factors are presented for the different structural types.
- Other changes including modified definitions, material properties and safety verifications equations are presented.
- Some provisions regarding the application of non-linear static analysis of timber structures is introduced.

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Abstract

This paper presents the results of the ongoing work on the revision of the provisions for the seismic design of timber buildings in Europe included within Chapter 8 of Eurocode 8. The most recent research results and technical developments regarding both wood-based materials and structural systems have been implemented into the proposed new version together with the application of the capacity design to each structural system. The main objectives are to update the few and incomplete provisions included in the current version to the current state-of-the-art and to correct some misleading rules. This manuscript represents the authors' point of view on the basis of a scientific research background and the design common practice regarding different key aspects in the seismic design of timber structures.

keywords: Eurocodes, seismic design, capacity design, behaviour factors, over-strength factors

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1 Introduction

Timber structural systems have increasingly become a viable alternative to other traditional structural materials like concrete, steel and masonry, mainly because of their excellent properties related to sustainability, energy efficiency, speed of construction and high seismic capacity. According to [1] the market share of wood-based residential buildings goes from less than 1% in Spain to 12% in Germany, 15% in Austria, 18% in Switzerland and Belgium, 21% in UK and 30% in Ireland, in 2006. A similar percentage (6.4%) has been estimated in Italy in 2014 [2] with an increasing expected growth in the next years. With specific attention to the mechanical behaviour of timber structural systems, several shaking table tests and extensive numerical simulations have been carried out in the last years within international research programmes, showing their excellent structural performances in case of seismic events. A tangible outcome of the obtained results in the research field is given by the increasing number of medium-rise buildings constructed in earthquake-prone areas with different level of seismicity in the last 10-15 years (Figure 1).

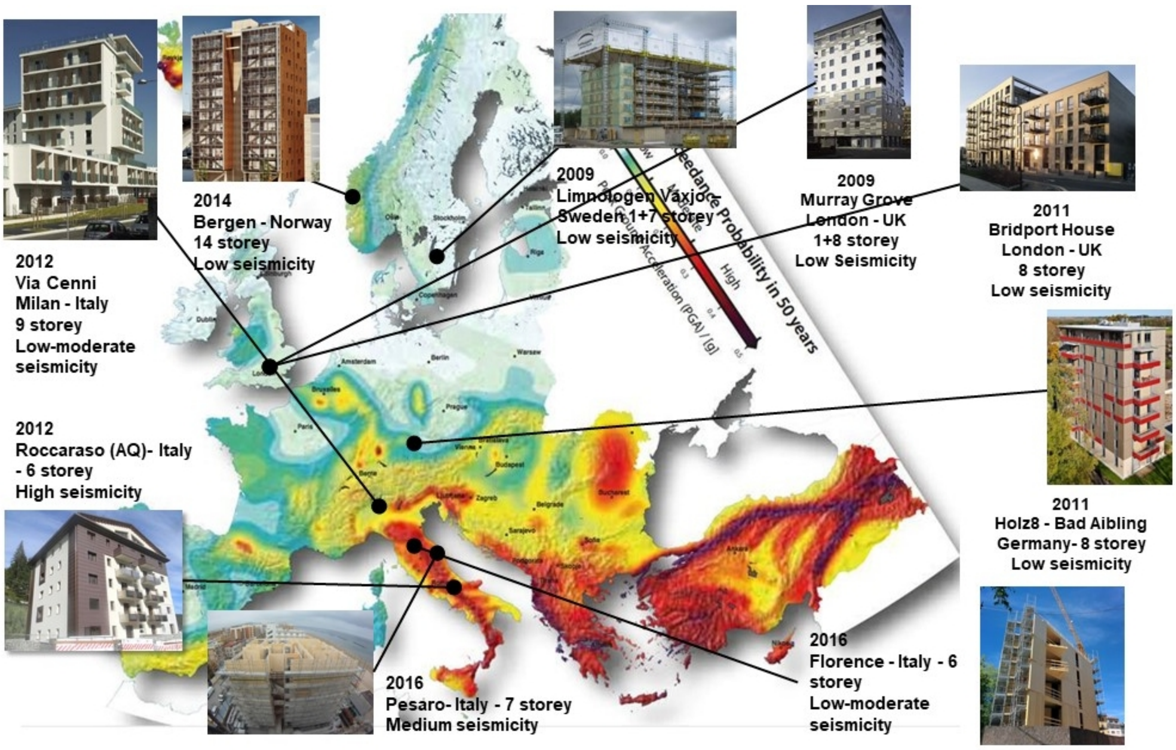


Figure 1: Medium -rise timber buildings built in recent years in European areas with different levels of seismic hazard (European Seismic Hazard map from the SHARE web site http://www.share-eu.org).

The revision process of the structural Eurocodes and therefore of Eurocode 8 [3] began in 2015 with the formal establishment of CEN (European Committee of Standardization) Project Teams tasked to prepare new drafts of the different sections, and the final updated version is expected to be released around 2020.

Among the different materials, the Chapter related to the seismic design of timber buildings is probably the one which needs major changes, being the current version rather old and short and considering that the construction practice for timber buildings evolved in the last years much more rapidly and radically than for other materials, especially concerning earthquake design.

This paper presents a proposal of modification of the current provisions; the proposal has been partly presented in [4] and it is still under discussion within the CEN/TC250/SC8 committee 'Design for Earthquake Actions', sub-group WG3 ‘Timber’ and for this reason it should considered as a draft version, since many changes may occur before its final published version. This manuscript represents the authors’ point of view on the basis of a scientific research background and the design common practice, and it shall be not assumed as the final Standard version.

2 Brief history of the timber Chapter in Eurocode 8

The provisions for the seismic design of timber buildings are included within the Chapter 8 of Eurocode 8. Three different versions of this Chapter have been released, starting from the first, 1988, up to the current, 2004, version as discussed in the next sub-sections. Figure 2 shows a timeline of the different issues.

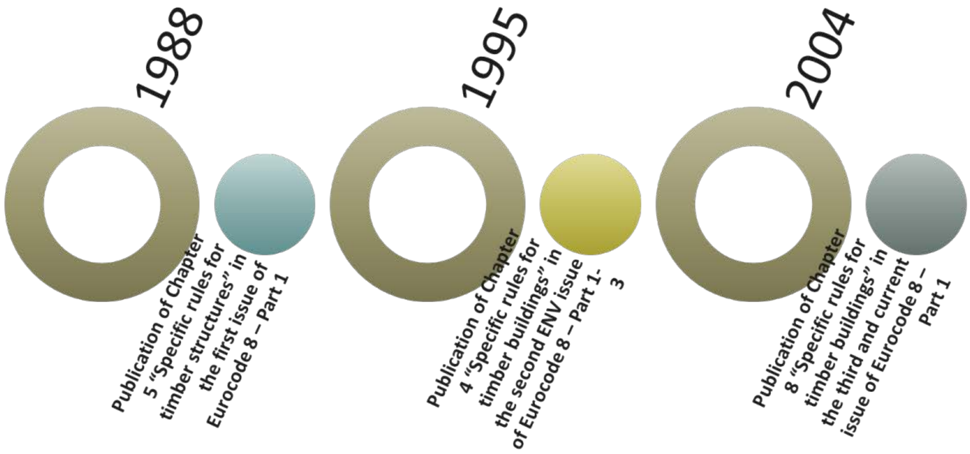


Figure 2: Timeline of the different issues of the chapter for the seismic design of timber buildings of Eurocode 8.

2.1 The first 1988 edition

The first edition of the Chapter related to the seismic design of timber buildings, included in the first issue of Eurocode 8 in 1988 [5], was composed by only four pages, and it was based on the Background Document presented by Ceccotti and Larsen [6]. Since this first release, the Chapter already contained the general framework of the current version and was divided into different parts: (i) *General criteria*, where the general principles of the seismic design of timber structures were given; (ii) *Materials*, which made reference to the relevant parts of Eurocode 5 [7] and where a first ductility classification was provided for joints with mechanical fasteners; (iii) *Structural types and Ductility Classes*, where three Ductility Classes (respectively Non-dissipative, Low-dissipative and Medium-dissipative structures) and some structural types were defined; (iv) *Behaviour factors and damping ratio*, where a conservative value of the behaviour factor q=1 was proposed for the three Ductility Classes and for all structural types (however, in the Background Document [6], a first proposal of behaviour factor greater than one was given, with q values ranging from 1 to 2.5); (v) *Safety verifications*, *limitations*, *detailing* where values of the partial safety factors for material properties and of the strength modification factor k_{mod} were proposed, together with some specific rules for joints and diaphragms.

2.2 The 1995 ENV version

A comprehensive revision and a substantial improvement of the 1988 edition was provided with the second release of the chapter for timber buildings, included in the ENV (European Prestandard) version of Eurocode 8 published in 1995 [8], and based on the rules and provisions presented at the 26th CIB Meeting held in Athens, Georgia in 1993 [9]. The main modifications included: (i) the introduction of new paragraphs (*Safety verifications*, *Detailing Rules and Control of design and construction*); (ii) the improvement of the existing paragraphs (the “General criteria” paragraph was detailed with definitions and design concepts to be adopted in the design, the “Material” paragraph was detailed with new provisions about properties of wood-based panels and of dissipative connections, the “Structural types” section was largely improved and modified); (iii) the increased number of Ductility Classes (from 3 to 4, basically introducing a new High Ductility Class) and structural types for each class also with the aid of graphical sketches; and (iv) the modification of the values of the behaviour factors to be used in the design (now ranging from 1 to 3 depending on the Ductility Class).

Moreover, the ductility classification for dissipative zones was modified with respect to the 1988 edition introducing a new rule, still included in the current version, stating that “*In order to ensure that the given values of the behaviour factor may be used, the dissipative zones shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio of 4 for ductility class M structures and at a static ductility ratio of 6 for ductility class H structures, without more than a 20% reduction of their resistance*”. Prescriptive ductility rules for the dissipative zones were introduced, based on the fastener diameter and the thickness of the connected timber or wood-based members and the values of the partial safety factors for material properties to be adopted for the design according to the dissipative and non-dissipative behaviour were modified with respect to the 1988 edition.

For the verifications according to the dissipative structural behaviour, the value for fundamental load combinations (i.e. γ_M =1.3) was proposed, whilst for the verifications according to non-dissipative behaviour, the value for accidental load combinations (i.e. γ_M =1.0) was suggested.

2.3 The current 2004 edition

The 1995 ENV edition of Eurocode 8 was completely redrafted between 1999 and 2003 and published in the current EN version in 2004 [3]. However, unlike the previous editions, no scientific background was provided for the proposed changes. The modifications included: (i) the reduction and modification of structural types; (ii) the introduction of some structural assemblies for building roofs like trusses with nailed, doweled or bolted joints; (iii) the reduction of Ductility Classes from 4 to 3, in accordance with other material chapters; (iv) the modification for the different structural types of the values of the behaviour factor q which were largely increased with respect to the 1995 ENV edition, ranging from 1.5 to 5; (v) the deletion of the graphical sketches used to describe the different structural types; and (vi) the modification of the partial safety factors γ_M for fundamental and accidental load combinations for the ultimate limit state verifications in case of dissipative and non-dissipative structural behaviour, which were inverted with respect to the ENV version.

2.4 Critical review of the current 2004 edition

In the force based design approach of Eurocode 8 [3], the energy dissipation capacity of the whole structure is implicitly considered by dividing the seismic forces obtained from a linear (static or dynamic) analysis by the behaviour q-factor associated to the relevant ductility classification. This approach can be applied only if the following conditions are satisfied:

1. The structural systems are clearly described without any possible misinterpretation.
2. The dissipative zones (ductile) and the non-dissipative (brittle) parts are unequivocally identified for each structural system.
3. The over-strength factors to be used for the design of the brittle components are provided.

Conversely, by analysing in detail the content of the current version of Chapter 8 of Eurocode 8, it could be observed that:

1. As mentioned above, the structural systems are not clearly described, the short definition of some of them may be misleading without an explanatory drawing, some systems are repeated

twice or refers only to structural components and not to lateral load resisting systems of buildings. And, above all, some structural systems such as the CLT and the Log House systems, which are nowadays widely used in the construction practice are not even mentioned.

2. The capacity design rules for each structural system are not completely defined since only few prescriptive rules are given regarding joints with dowel type fasteners.
3. The over-strength factors are not provided. A value of 1.3 is given only regarding the verification of shear stress in carpentry joints.

Therefore, to align the content of the chapter related to timber buildings to the provisions given for the other materials, a fundamental revision is needed, considering that the current few rules are left to the interpretation of the structural designer.

3 The new proposal of Chapter 8 of Eurocode 8

While trying to keep the same order of headings and topics of the former versions also to keep consistency with the other materials chapters within Eurocode 8, the proposed modifications to the current version are substantial. Figure 3 shows the table of contents of the new Chapter: with respect to the current version, section 8.4 “Capacity design rules” and Annex D (informative) “Non-linear static (pushover) analysis of timber structures” are completely new.

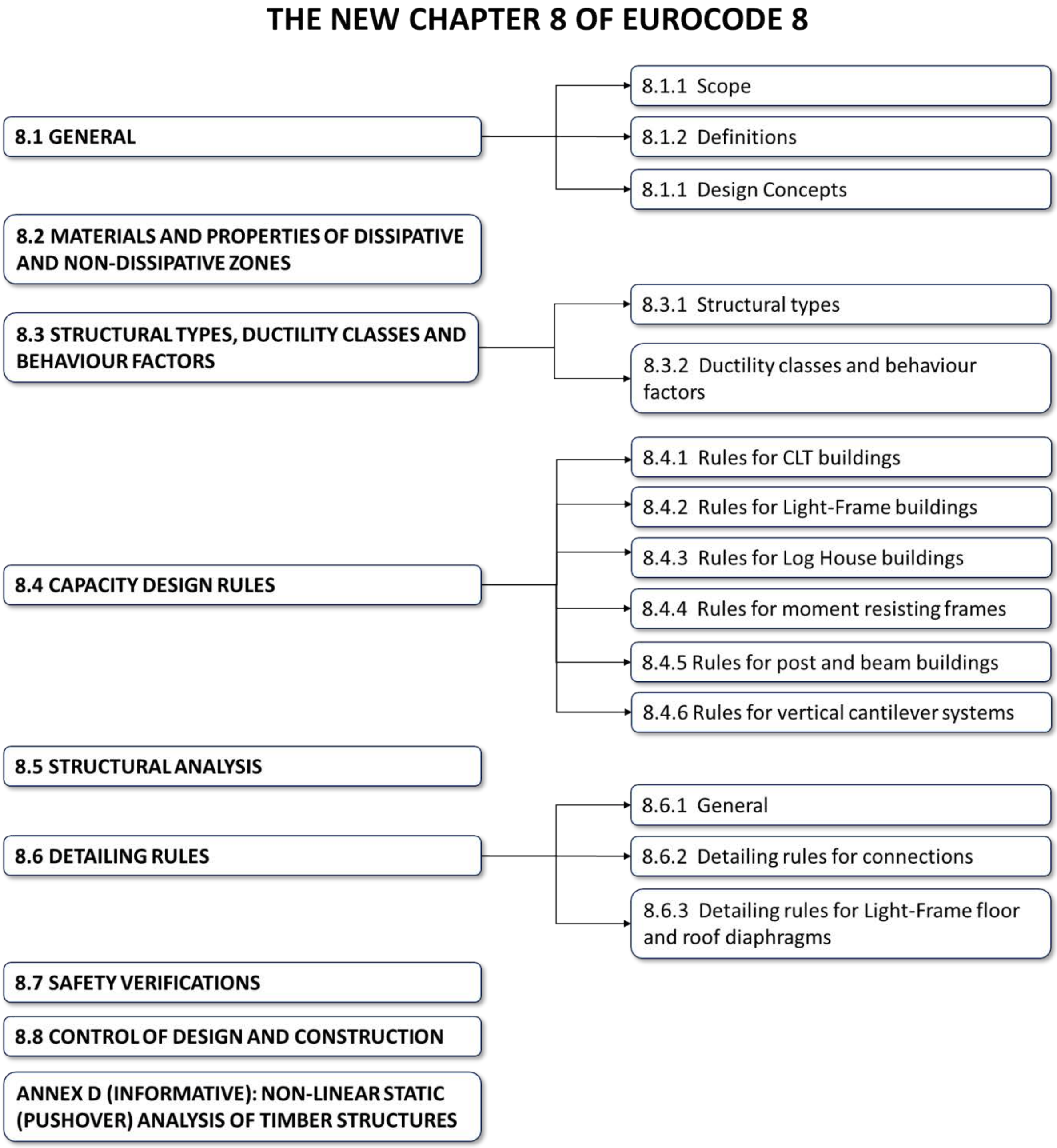


Figure 3: Table of contents.

The main changes are however included in the code text and are briefly summarized in this paper.

3.1 Definitions and design concepts

Some definitions were slightly changed with respect to the current version. Regarding the definition of static ductility, a reference to the definition given in EN 12512 [10] was added, while for carpentry joints a further clarification was given, reporting that "*loads are transferred through to the connected elements by means of compression areas*".

According to the current definition of static ductility given in Chapter 8 of Eurocode 8, i.e. the "*ratio between the ultimate deformation and the deformation at the end of elastic behaviour, calculated according to EN 12512, evaluated in quasi-static cyclic tests*". By comparing six different methods used in the calculations of the yield point and ductility ratio in various types of connections and wall assemblies, Munoz et al. [11] demonstrated that differences up to 100% can be found in the calculations of the ductility ratio. While there is an international agreement about the definition of the ultimate displacement (defined as the displacement corresponding to 80% of the maximum load in the descending portion of the 1st cycle backbone curve in a cyclic test), different methods are proposed for the evaluation of the yield displacement of mechanical joints in timber structures and of the loading protocol for cyclic testing. This may have a great influence in the determination of the ductility provisions given in Eurocode 8 for ductility class medium (DCM) and high (DCH) for different structural systems. However, the current provisions of EN 12512 are under review and is expected that new definitions of yield point and ductility ratio will be given in a future edition of this Standard.

Differently from the current generic distinction between dissipative and low dissipative structural behaviour, the classification of timber buildings according to the design concept is modified specifying that "*Earthquake-resistant timber buildings shall be designed in accordance with one of the following concepts:*

a) High- or Medium-dissipative structural behaviour;

b) Low-dissipative structural behaviour."

For the design of structures classified as low-dissipative, no account is taken of any hysteretic energy dissipation and the behaviour factor cannot be taken as being greater than the value of 1.5, considered to account for overstrengths. For High- or Medium-dissipative structures the behaviour factor is taken as being greater, accounting for the hysteretic energy dissipation that mainly occurs in specifically designed zones, called dissipative zones or critical regions.

Later it is also specified that "*Other structural types, classified in ductility class M (medium, DCM) or H (high, DCH) may be designed with concept b) provided that the corresponding provisions given in the reference parts of this section for the general rules at building level are satisfied.*"

The possibility of designing every structural type for DCL is given in the relevant chapters of all other materials in Eurocode 8. Regarding the general rules at building level, further specifications are given later within the Capacity Design Rules section.

For the dissipative zones, the current definition specifies that the dissipative zones shall be located in joints and connections, whereas the timber members themselves shall be regarded as behaving elastically. A further clarification is given, more specifically it is stated that "*The energy dissipation is provided by plasticization of metal fasteners combined with embedment of timber at the interface with the fasteners, and for some systems also by friction.*"

A further provision is given later specifying that: "*As an alternative, dissipative zones could be located outside of joints and connections in purposely developed energy dissipators (e.g. lead extruded or hydraulic dampers, dog-bone steel plates, etc.). In this case, both the timber members and the joints and connections shall be regarded as behaving elastically. These connections, the other joints and connections between timber members and all the timber members shall be designed as non-dissipative members according to the capacity-based design rules. The appropriate behaviour factor q should not be determined according to Table 8.2 but reference should be made to the relevant part of EN1998*

3.2 Materials and properties of dissipative and non-dissipative zones

Wood-based materials such as OSB panels, Gypsum Fibre boards and CLT panels, which were not included in the current version, have been added. Regarding the structural panels used as structural components or sheathing material for shear walls and diaphragms, the proposal is in the following:

a) particleboard-sheathing (according to EN 312) has a density of at least 650 kg/m^3;

b) plywood-sheathing (according to EN 636) is at least 9 mm thick and has at least 5 layers;

c) particleboard- and fibreboard (according to EN 622)-sheathing are at least 12 mm thick;

d) Oriented Strand Board sheathing (OSB) type 3 or 4 according to EN 300 and has a minimum thickness of 12 mm;

e) Gypsum Fibre boards (GF) sheathing according to EN 15283-2 has a minimum thickness of 12 mm;

(5) CLT panels produced according to EN 16351 have a minimum thickness of 60mm for shear walls and 18 mm for floor and roof diaphragms.

A large number of experimental results about the good dissipation properties of Light-Frame shear walls sheathed with OSB panels are reported in [12, 13, 14].

Light-Frame buildings sheathed with Gypsum Fibre boards (GF) sheathing and stapled connections are becoming more and more used in the current construction practice. Moreover, recent research conducted at the University of Trento, Italy [14] and within the SERIES Project [15, 16] have proved the suitability of Gypsum Fibre Panels (GF) connected to the timber framing with staples as a sheathing material for shear walls in Light-Frame construction. The limitation of 18 mm for CLT floor panels is given according to the current specifications included in the European Standard for CLT EN 16351 [17], which states that CLT may be made of timber layers having thicknesses between 6 mm and 60 mm. The limitation to 60 mm of panel thickness for CLT walls is given according to current production of most European producers.

As for steel material to be used for connections the following provisions are given, already partly included in the current version of Chapter 8:

a) steel plate elements shall fulfil the relevant requirements in EN 1993;

b) steel fasteners shall fulfil the relevant requirements in EN 409;

c) the ductility properties of the dissipative connections in Ductility Class M or H structures (see (8.3)) shall be tested for compliance with 8.3.2(3)P by cyclic tests on the relevant combination of the connected parts and fastener;

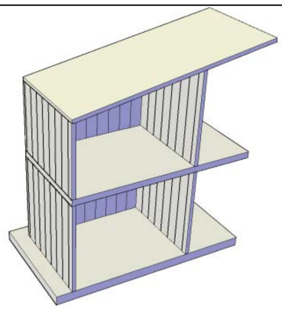
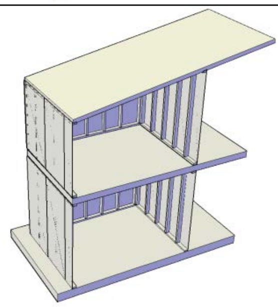
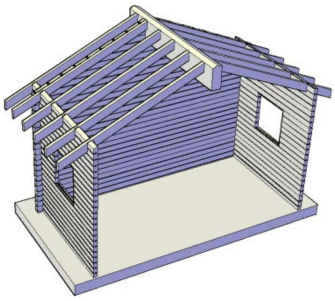
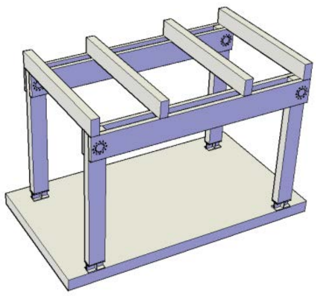
(d) the low-cycle fatigue capacity of fasteners used in the dissipative zones shall satisfy the requirements reported in the Annex F of EN 14592.

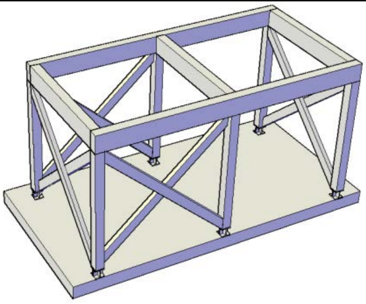
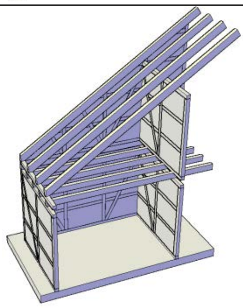
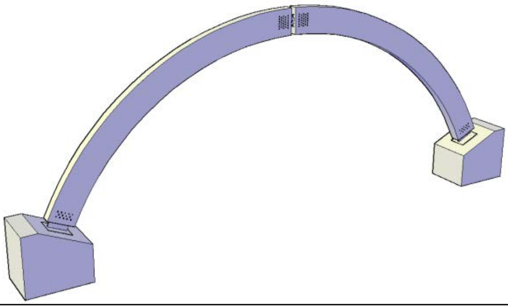
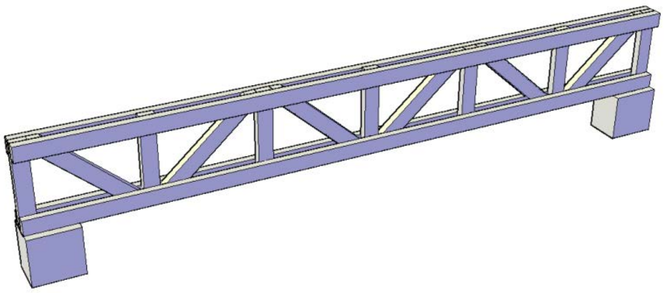
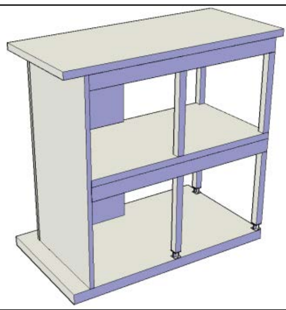
Point (d) has been introduced in order to take into account the low-cycle fatigue capacity of fasteners.

3.3 Structural types, ductility types and behaviour factors

This part has been completely redrafted with respect to the current version. First, a clear definition of the different structural types is given, explained also by means of schematic figures. According to the proposal, nine different structural types are identified and briefly described in Table 1.

Table 1: Structural types for timber buildings and schematic graphical description.

1	Cross laminated timber (CLT) buildings.	
2	Light -frame (LF) buildings.	
3	Log House buildings.	
4	Moment resisting frames.	

5	Post and beam timber buildings with vertical bracings made of timber trusses.	
6	Timber framed walls with carpentry connections and masonry infill.	
7	Large span arches with two or three hinged joints.	
8	Large span trussed frames with nailed, screwed, doweled and bolted joints.	
9	Vertical cantilever systems made with structurally continuous Glulam or CLT wall elements.	

New structural systems for timber buildings, already widely used in seismic regions such as the Cross Laminated Timber (CLT) system and the Log House system, were introduced. With respect to the current version, all the structural types referring to structural assemblies for building roofs like trusses with nailed, doweled or bolted joints or with connectors were removed. The reason for this change was that the timber trusses were introduced in the 2004 edition probably overlooking the meaning of timber trusses given in the previous 1995 ENV edition where this system referred to vertical bracing systems used in buildings (even large span glulam roofs, where the timber elements are directly connected to the foundation and resist vertical and horizontal loads). As this chapter refers to lateral load resisting systems in timber building, there is no reason to make reference to structural assemblies used for roofs. The structural type referenced in 2004 edition as “Hyperstatic portal frames” is here referenced with the most common definition of “Moment resisting frames” and two values of the behaviour factor q are given for DCM and DCH. Also the vertical cantilever system is a new structural type not referenced in the 2004 edition which is nevertheless widely used in seismic regions. The graphic description was re-introduced like in the 1995 ENV edition.

The proposed value of the behaviour q-factor given for each structural type and for the corresponding ductility class (Medium or High) are given in Table 2. For structures designed in accordance with the concept of low-dissipative structural behaviour (DCL), the behaviour q-factor should be taken not greater than 1.5.

Table 2: Structural types and upper limit values of the behaviour q-factors for buildings regular in elevation

Structural type		**DCM**	**DCH**
1	CLT buildings	2.0	3.0
2	Light-Frame buildings	2.5	4.0
3	Log House buildings	2.0	-
4	Moment resisting frames	2.5	4.0
5	Post and beam timber buildings	2.0	-
6	Mixed structures made of timber framing and masonry infill resisting to the horizontal forces	2.0	-
7	Large span arches with two or three hinged joints	-	-
8	Large span trusses with nailed, screwed, doweled and bolted joints	-	-
9	Vertical cantilever systems made with glulam or CLT wall elements	2.0	-

New values for the behaviour q-factors were introduced, specifying two different values, if applicable, for DCM and DCH ductility classes. The values given for CLT structures are based on experimental [20] research results and numerical investigations [22,23,24] conducted within the Sofie Project for buildings designed according to the capacity design rules given in the relevant section (see § 3.4).

For Light-Frame structures two different values of the behaviour factor q are given for DCM and DCH. The highest q values of 5.0 given in the 2004 edition, and the corresponding higher values of the R-factor, equal to $R_d x R_0$=5.1, given in the National Building Code of Canada [22] and R=6.5 used in ASCE-7 [23] in the US confirmed as part of the FEMA P-695 [24] study, are not confirmed by other international codes (e.g. New Zealand [25]) and by all the numerical investigations conducted so far (see [26] as a reference). Therefore, a more conservative value of 4.0 is proposed according to experimental [14, 27, 28, 29] and numerical studies [30] carried out in the last years. For the seismic design according to DCM a value of 2.5, given in [31], is proposed in order to include Light-Frame buildings sheathed with gypsum fibre boards and stapled connections. Unlike the 2004 edition, and according to the provisions given in the previous 1995 ENV edition, no distinction is made between glued and nailed diaphragms. For Log-House buildings, reference have been made to [32].

Other provisions are related to (i) the design of building with different Lateral Load Resisting Systems (LLRS) working at the same level, (ii) the continuity of shear walls and (iii) the design of structural systems and elements not included in the list of structural types given in the new proposal.

As for (i), the new provision is the following: "*In principle, all seismic forces in one direction shall be resisted by one system type. If different lateral load resisting systems are used in the same direction, even if made of other materials, the lower value of the behaviour q-factor of the two systems shall be used. In order to use a higher value for the behaviour q-factor (not higher than the maximum value of the two systems), non-linear static (push-over) or non-linear dynamic (time-history) analyses shall be carried out to design the system. In this last case, the deformation compatibility between the different lateral load resisting systems needs to be verified*". Studies are currently ongoing about a proposal of analytical formulation for the calculation of the behaviour factor of mixed CLT/Light-Frame buildings [33].

Regarding the continuity of shear walls, the following provision is given: "*Shear walls shall be structurally continuous from the foundation or base of the timber part of the building to a certain floor,* *namely they cannot be interrupted below a certain floor in elevation in order to avoid the occurrence of soft storey mechanisms (see* Figure 4*). Partition walls and structural walls which are not intended to be part of the seismic resistant system (secondary seismic walls according to 4.2.2 of EN 1998-1), shall be detailed so as not to take part in the seismic lateral load resisting system.”*

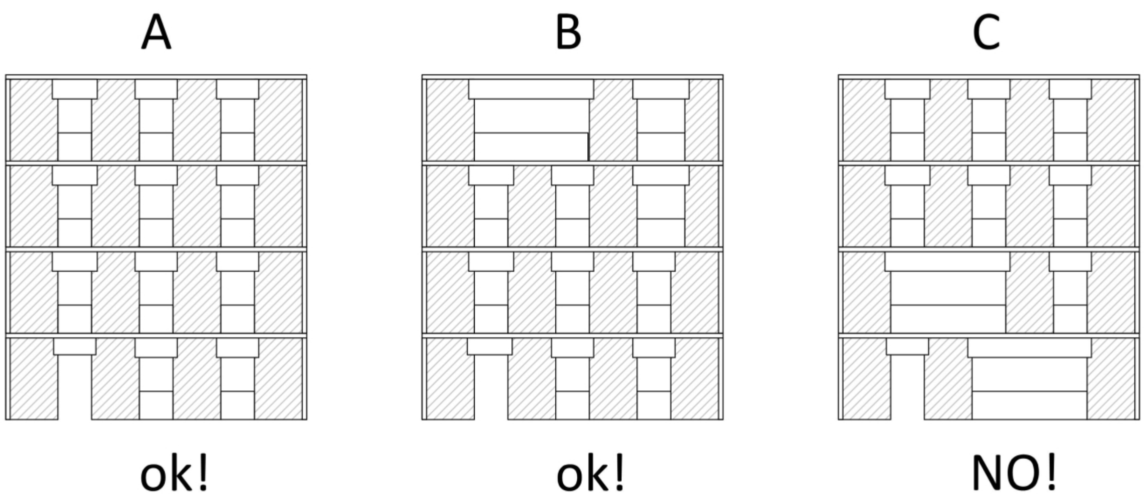


Figure 4: A: Building with all shear walls structurally continuous from the foundation to the roof. B: Building with part of the shear walls structurally continuous from the foundation to the roof and part interrupted at the top storey. C: Building with part of the shear walls interrupted below the second and third storey (possible soft storey mechanism at the first or second storey).

The continuity of shear walls along the building height is an important issue regarding the seismic design. Note that the continuity is referred only to shear walls and not to walls supporting only vertical loads and should start from the foundation or the “base of the timber part”, signifying that a multi-storey timber building can be built over one or more concrete storeys, of course provided that the timber walls are supported by corresponding masonry walls or reinforced concrete frames. Shear walls continuity can be interrupted at a “certain floor”, signifying that some shear wall can be interrupted in the last storeys like for example in case B of Figure 4, provided that of course the remaining shear walls at the same storey are able to withstand the seismic storey shear.

With regard to the possibility of occurrence of soft-storey mechanisms it is specified that “*In the seismic design, the resistance of shear walls should be proportional to the storey seismic shear in order to ensure a simultaneous plasticization of as many storeys as possible, avoid soft storey mechanisms, and increase the ductility and energy dissipation of the structure.”*

Regarding new structural types not yet included in the current list of "known" building systems, they are not excluded, provided that the ductility properties of dissipative zone are demonstrated. The corresponding provision specifies that "*Different structural elements and systems not listed above may be used provided that the properties of dissipative zones are determined by tests either on single joints, on whole structures or on parts thereof in accordance with EN 12512 and with Annex D of EN 1990. The appropriate behaviour factor q should be determined based on non-linear dynamic numerical simulations of the structure by implementing the non-linear cyclic behaviour of the dissipative zones obtained from the experimental tests*."

The ductility properties of the dissipative zones should be fulfilled for each structural type in order to ensure that the above given values of the behaviour factor may be used. Three alternative possibilities are given:

1. *Ensuring that "the dissipative zones, specified in the capacity design rules for each structural type, shall be able to deform plastically for at least three fully reversed cycles at a static ductility ratio reported in* Table 3, *without more than a 20% reduction of their resistance between the first and third cycle backbone curve. For the same structural type these provisions shall be satisfied by only one type of dissipative sub-assembly/element provided that the Capacity Design Rules as defined in the relevant sections of each structural type are satisfied."*

Table 3: Required static ductility values of dissipative zones tested according to EN12512 without more than a 20% reduction of their resistance between the first and third cycles backbone curve for all structural types depending on the Ductility Class.

Structural type	**Dissipative sub-assembly/element/connector**	**Type of ductility**	**DCM**	**DCH**
CLT buildings	Shear wall	Displacement ductility	3.0	4.0
CLT buildings	Hold-downs, angle brackets, screws	Displacement ductility	3.0	4.0
Light-Frame buildings	Shear wall	Displacement ductility	3.0	5.0
Light-Frame buildings	Fastener (nail/screw/staple)	Displacement ductility	5.0	7.0
Log House buildings	Shear wall	Displacement ductility	2.0	-

Moment resisting frames	Portal Frame	Displacement ductility	2.5	4.0
Moment resisting frames	Beam-column joint	Rotational ductility	6.0	10.0
Post and beam timber buildings	Braced Frame	Displacement ductility	2.0	-
Timber framed walls with masonry infills	Shear wall	Displacement ductility	2.0	-
Vertical cantilever systems made with glulam or CLT wall elements	Shear wall	Displacement ductility	2.5	-

The values proposed in Table 3 are based on researches conducted so far (see [27, 28, 29, 30, 34] for Light-Frame), however more research is needed in order to check their validity. As an alternative, the above given provisions may be regarded as satisfied in the dissipative zones of all structural types classified in ductility class H if the following provisions are met:

a) in doweled, bolted and nailed timber-to-timber and steel-to-timber joints, the minimum thickness of the timber connected members is 10d and the fastener-diameter d does not exceed 12 mm;

b) in shear walls and diaphragms of Light-Frame construction, the sheathing material is wood-based with a minimum thickness of 4d, where the nail diameter d does not exceed 3,1 mm.

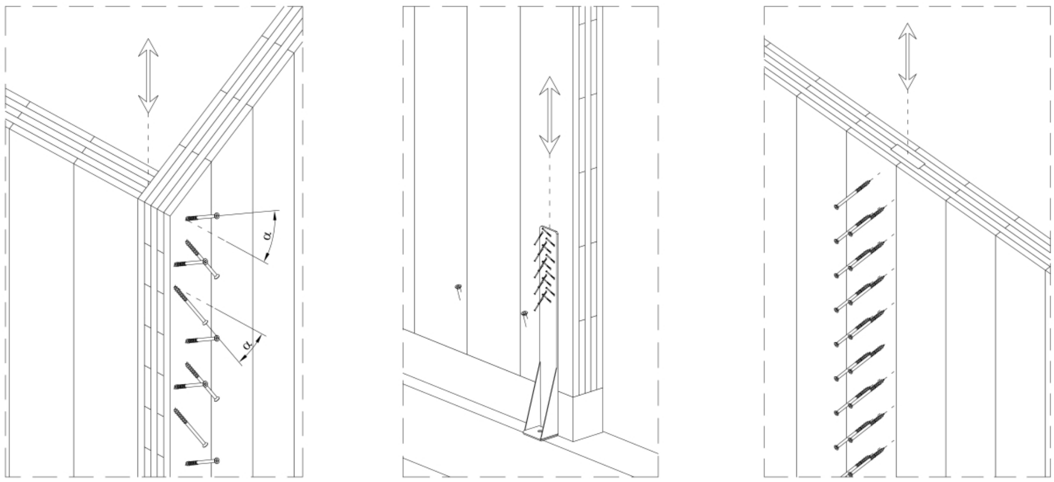
If the above requirements are not met, but the minimum member thickness of 8d and 3d for case a) and case b), respectively, is assured, the dissipative zones of all structural types can be regarded as ductility class M.

3. As an alternative to #2 the provisions of #1 are satisfied if the following conditions are met:
   - *for the dissipative zones of all ductility class M structural types, of the ductility class H CLT system with segmented wall and for the sheathing-to-framing connection, when a ductile failure mechanism characterized by the formation of at least one plastic hinge in the mechanical fasteners is attained for the seismic design load condition;*

- *for the nailed and screwed connections between the sheathing material and timber frame used in class H in Light-Frame buildings, when a ductile failure mechanism characterized by the formation of at least one plastic hinge in the nail (or screw) is attained for the seismic design load condition;*
- *for the dissipative zones of all ductility class H structural types, when a ductile failure mechanism characterized by the formation of two plastic hinges in the mechanical fasteners is attained for the seismic design load condition.*

Referring to 8.2.2 of EN 1995-1-1 for timber-to-timber and panel-to-timber connections, failure modes a, b and c for fasteners in single shear, and g and h for fasteners in double shear characterized by only embedding of timber and no fastener plasticization shall be avoided. Referring to 8.2.3 of EN 1995-1-1 for steel-to-timber connections, failure modes a, c for fasteners in single shear, and f, j and l for fasteners in double shear characterized by only embedding of timber and no fastener plasticization shall be avoided. Special care should be taken in avoiding brittle failures characterized by splitting, shear plug, tear out and tensile fracture of wood in the connection regions. In the case of connections with multiple fasteners in dissipative zones, adequate reinforcement should be added to avoid the aforementioned brittle failure mechanisms.

Another provision is given for dowel-type fasteners transferring most of the load via axial resistance, which cannot be considered as dissipative. Referring to Figure 5, A and B cannot be considered as dissipative connections, while C can be considered as dissipative.



A B C

Figure 5: A and B: connections inserted inclined with respect to the direction of the shear force, transferring most of the load via axial resistance, which cannot be considered as dissipative. C: connections inserted perpendicular with respect to the direction of the shear force, transferring most of the load via shear resistance, which can be considered as dissipative

3.4 Capacity design rules

As mentioned above, in order to apply the force-based procedure of Eurocode, capacity design rules are needed for each structural type and material in order to achieve the desired level of ductility and energy dissipation capacity for the whole building and therefore to apply the given values of the behaviour q-factor for the different Ductility Classes.

Therefore, for each structural type, capacity design rules are provided both at building level and at connection level in order to ensure that the energy dissipation will occur in the ductile components. Regarding the latter, in order to ensure a ductile failure mode characterized by yielding of fasteners in steel-to-timber or timber-to-timber connections, it is specified that any anticipated brittle failure like tensile and pull-through failure of anchor bolts or screws, steel plate tensile and shear failure in the weaker section of hold-down and angle brackets connections or any other brittle failures such as splitting, shear plug, tear-out and tensile fracture of wood in the connection regions should be always avoided.

a) b) c)

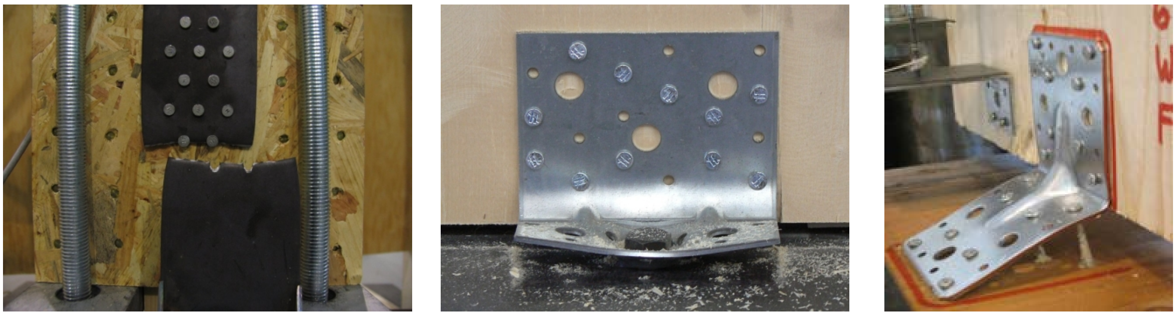


Figure 6: Brittle failure mechanisms in angle brackets and hold-down connections due to the steel plate failure in the weaker section of hold-down connections (a), due to the pull-through of the head of the anchor bolt through the steel plate in steel bracket (b) and due to the sudden withdrawal of nails in the inter-story wall-to floor angle brackets connection (c).

Table 4 shows the Capacity design rules at building level for each structural system defined in the new proposal for the two Ductility Classes.

Table 4: Capacity design rules for DCM and DCH for the different structural types.

Structural Type	**Ductility Class Medium (DCM)**		**Ductility Class High (DCH)**	
	Components to be overdesigned	**Dissipative components/mechanisms**	**Elements to be overdesigned**	**Dissipative components/mechanisms**
CLT (Cross Laminated Timber)	– all CLT wall and floor panels – connections between adjacent floor panels – connections between floors and underneath walls – connections between perpendicular walls	– Shear-restrain connections at wall base – Uplift-restrain connections at wall ends	– all CLT wall and floor panels – connections between adjacent floor panels – connections between floors and underneath walls – connections between perpendicular walls	– Shear-restrain connections at wall base – Uplift-restrain connections at wall ends – vertical step joints between wall panels in segmented shear walls
LF (Light-Frame)	– nailed sheathing-to-framing connections in floors – connections between floors and underneath walls – connections between perpendicular walls	– nailed, stapled or screwed sheathing-to-framing connections – Shear-restrain connections at wall base – Uplift-restrain connections at wall ends	– nailed sheathing-to-framing connections in floors – connections between floors and underneath walls – connections between perpendicular walls	– nailed, stapled or screwed sheathing-to-framing connections

Structural Type	Ductility Class Medium (DCM)		Ductility Class High (DCH)	
	Components to be overdesigned	**Dissipative components/mechanisms**	**Elements to be overdesigned**	**Dissipative components/mechanisms**
	– sheathing panels and framing members		– sheathing panels and framing members – Shear-restrain connections at wall base – Uplift-restrain connections at wall ends	
Log House buildings	– shear verification of carpentry joints – timber logs – Shear-restrain connections at wall base – Uplift-restrain connections at wall ends	– friction between logs	-	-
Moment-resisting frames	– all timber components	– all dowel-type mechanical fasteners	– all timber components	– high-ductility joints, i.e. special systems which incorporate beam-column joints
Post&beam timber buildings	– all timber components	– all dowel-type mechanical fasteners	-	-
Vertical cantilever system	– wall panels	– fasteners at base connections		

The new proposal of capacity design rules defined for each structural type is that *the design strength of the brittle parts $F_{Rd,b}$ should be greater than or equal to the design strength of the ductile parts $F_{Rd,d}$ multiplied by an overstrength factor γ_{Rd} and divided by a reduction factor for strength degradation β_{sd} due to cyclic loading according to the following equation:*

$$\frac{\gamma_{Rd}}{\beta_{sd}} \cdot F_{Rd,d} \leq F_{Rd,b} \quad (1)$$

where the values of γ_{Rd} are provided in Table 5, and the value of β_{sd} is equal to 0.8.

Table 5: Values of the overstrength factors γ_{Rd}

Structural type	**Overstrength factor γ_{Rd}**
CLT buildings, Light-Frame buildings, Log House buildings, High ductility moment resisting frames with expanded tube fasteners, Mixed structures made of timber framing and masonry infill resisting to the horizontal forces	1.3
Moment resisting frames (except for high ductility moment resisting frames with tube fasteners and Densified Veneer Wood), Post and beam timber buildings, Vertical cantilever systems made with glulam or CLT wall elements	1.6

3.5 Safety verifications

As reported also in [4], the strength values of timber shall be determined taking into account the k_{mod}-values for instantaneous loading and the partial factors for material properties γ_M for accidental load combinations.

For ultimate limit state verifications of structures designed in accordance with the concept of dissipative structural behaviour (Ductility classes M or H), the strength degradation of the dissipative zones shall be taken into account by multiplying the characteristic strength in static conditions by the reduction factor β_{sd}. The design strength shall then be calculated as:

$$F_{Rd,d} = k_{mod} \cdot \beta_{sd} \cdot \frac{F_{Rk,d}}{\gamma_M} \qquad (2)$$

The strength degradation of the non-dissipative zones may not be taken into account. The design strength should be calculated as:

$$F_{Rd,b} = k_{mod} \cdot \frac{F_{Rk,b}}{\gamma_M} \qquad (3)$$

This formulation for the safety verifications is quite different from the one present in the current 2004 version where the partial safety factor γ_M for fundamental load combinations is proposed for ultimate limit state verifications of structures designed in accordance with the concept of low-dissipative structural behaviour and no reduction factor β_{sd} for strength degradation is given.

3.6 Non-linear static (pushover) analysis of timber structures

Some general provisions are given in a new Annex for the application of non-linear static (pushover) analysis to timber buildings. With this regard, some references on the application of the N2 method for timber structures may be found in [35]. Timber components and mechanical connections or devices characterized by a brittle failure shall be modelled as elastic elements adopting the mean values of mechanical properties. Reference to the experimental data provided by the producers on the dissipative mechanical connections and mechanical devices shall be made. In order to model the mechanical behaviour of mechanical connections reference shall be made to the mean backbone curve obtained from the experimental test carried out according to EN 12512 [10].

The seismic verification shall be performed in terms of actions for brittle/non-dissipative elements and in terms of displacements (or rotations) for ductile/dissipative elements.

4 Future improvements

The research projects carried out so far and referenced above brought a large amount of experimental data and useful information which has been used to develop the proposal presented herein. At the same time, due also to the development of powerful software packages for structural analysis, new numerical models for the linear and non-linear analysis of timber structures have been developed and used for research purposes especially in the evaluation of the seismic performance of medium to high-rise timber buildings [19, 20, 36, 37, 38].

The new frontier is now represented by the “tall wood buildings” with a number of storeys ranging from 10 to 30 [39]. A 10-storey building has been recently built in Australia and a 14-storey building is already under construction in Norway, even if in a non-seismic area; an 18-storey hybrid concrete-mass timber building has been built in Vancouver, Canada in 2016 and there are projects for the construction of buildings up to 30 storeys in Canada [40] and USA.

Therefore, considering these new trends for the next few years, a future generation of EC8 for timber structures should address the following issues, not included in the revision presented in this paper:

- More detailed provisions about non-linear static and dynamic analysis methods should be provided in order to foster their use in seismic design. However, the non-linear behaviour of timber structural systems is essentially based on the non-linear properties of connections. Furthermore, structural designers do not have usually easy access to experimental data (which should refer to the same connection with the same type, number and diameter of fasteners used in the actual design). Therefore, in order to improve the ease of use of these methods, the products certification (ETA, CE marking based on product standards) for connections and fasteners should contain also details about the non-linear properties of such elements.
- Some guidance should also be given for the retrofit of existing timber [41] and non-timber (e.g. masonry, [42]) buildings using wood-based products.
- Recommendations for the estimation of the connection ductility in the dissipative regions should also be provided, together with detailing rules such as the use of specific reinforcement to avoid brittle failure modes such as shear plug, splitting, etc.
- Guidelines for the design of tall (10 storeys and more) timber buildings should also be provided so as to account for the specific behaviour of timber (e.g. the influence of the higher vibration modes in the seismic design due to the low modulus of elasticity of timber). With the aim of investigating the seismic performance of tall timber buildings, new types of connections and/or new design approaches should be provided. For instance, the hold-down connectors commonly available for the construction of timber buildings have a maximum characteristic strength of 100 kN. However, it is not unusual to calculate uplift forces up to 500-700 kN even in low seismicity areas for medium-rise buildings (6-7 storeys). Therefore, in case these uplift forces are resisted only by hold-down connectors, this may lead to an excessively large number of connectors to be placed at the same position, with risk of brittle failure (e.g. splitting) within the connected timber parts. So there is a demand for stronger connection systems for medium to high-rise buildings in seismic areas or

alternative design methods which yields smaller seismic forces in the connections. This is the reason why new approaches for the seismic design of such tall buildings, including alternative design procedures with innovative low-damage structural systems such as pre-stressed re-centring walls [43]the use of new types of dissipative steel connections, innovative energy dissipators [44] and tuned mass dampers [45, 46] deformable floor diaphragms or multi-storey segmental rocking walls should be further investigated [39] advanced materials such as superelastic shape memory alloys [4746] or even the use of passive base isolation systems for timber buildings [48].

5 Conclusions

The ongoing work on the revision of the Chapter 8 for the seismic design of timber buildings of Eurocode 8 was presented. The new proposal, which is markedly different from the previous and current short, concise and outdated version, is based on the following main modifications: (i) changes in the general definitions and design concepts, (ii) update of the list of wood based and other materials and properties of dissipative and non-dissipative zones, (iii) update of the list of structural types with consideration of new structural widely used types not included in the current version, (iv) modification of the description of the existing structural types with the aid of graphical descriptions, (v) modification of the values of the behaviour factors for the different Ductility Classes, (vi) introduction of capacity design rules for each structural type and of the over-strength factors to be used in the design of the brittle components, (vii) modification of the current equations for the safety verifications and (viii) some new provisions for the application of the non-linear static (pushover) analysis.

More research is of course needed about the applicability of the new provisions on multi-storey buildings also considering other structural systems and especially for medium to high-rise buildings in medium to high seismicity areas, where the common commercially available connection devices seem inapplicable and the seismic design requires a different philosophy or different types of connection devices.

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