THE TIMBER CHAPTER OF THE NEW EUROCODE 8 – PART 1-2: NEW FEATURES AND FUTURE IMPROVEMENTS

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ABSTRACT: The revision of the new Timber Chapter (i.e. Chapter 13 and Annex L) of Eurocode 8 (prEN1998-1-2) began almost 10 years ago, resulting in significant changes compared to the current version. Expanding from just 4 pages in EN1998-1 to almost 50 pages, the new chapter introduces detailed provisions for engineered wood products, dissipative zones, and both new and existing structural systems. It emphasizes capacity-based design at multiple levels (i.e. connections, walls, and entire buildings) along with updated overstrength factors and behavior factors for medium (ductility class 2) and highly dissipative (ductility class 3) structures. Provisions for the use of bonded-in rods as well as a new Annex L for non-linear static analyses of timber buildings have also been included. This paper summarizes the most significant changes introduced in the Timber chapter of the new Eurocode 8, and highlights the potential of this regulation to provide the designer with up-to-date information on the seismic design of timber buildings.

KEYWORDS: Eurocode 8, Seismic analysis, Capacity-based design, Overstrength factors

1 – INTRODUCTION

The updating process of the Eurocodes began in 2010 with the definition of the mandate from the European Commission. In 2015, Project Teams were selected and prepared their initial draft modifications by 2021. These drafts were then managed by various subgroups within CEN/TC250 and underwent public inquiry through national standardization bodies (NSBs) such as DIN (Germany), OSB (Austria), UNI (Italy), and so on. Feedback from these inquiries was incorporated into the revisions, after which some of the updated Eurocodes proceeded (or will proceed) to a Formal Vote by the respective NSBs. The final revised Eurocodes are scheduled for publication in 2027, with all previous generations of Eurocodes set to be officially withdrawn by 2028.

The current version of Eurocode 8 [1], published 20 years ago, offers limited guidance on the seismic design of timber buildings and has not been updated to reflect significant advances in both technology and research. Over the past decades, the increasing interest in timber structures has driven the development of high-performance, taller timber buildings, even in seismic-prone regions. These advances have led to the adoption of new construction technologies (i.e. timber-composite solutions), engineered wood products (i.e. cross laminated timber) and connection systems which enhance structural efficiency and enable taller buildings to be constructed. To ensure a proper level of knowledge and structural reliability, many researchers have investigated the mechanical performances of novel components and the seismic behaviour of single structural elements and entire buildings involving full-scale shaking table tests as well as cyclic tests on subassemblies and components [2]. The new generation of Eurocode 8, is composed of two parts:

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FprEN 1998-1-1:2024: General rules and seismic action [3], and FprEN 1998-1-2:2024: Rules for new buildings [4].

The new Timber Chapter (i.e. Chapter 13) of Eurocode 8 [1] will consist of 41 pages and introduce significant updates across the following topics: (i) new engineered wood and non-wood products for seismic design, (ii) a newly developed section regarding structural types, (iii) general and specific provisions for dissipative structural systems, (iv) updated default values of behaviour factors for medium dissipative (ductility class DC 2) and high dissipative (ductility class DC 3) structures, (v) capacity-based design rules at connection as well as at wall-building level, (vi) minimum required ductility values of dissipative zones, and finally (vii) a new procedure for determining the load-deformation curves of dissipative timber components, either through experimental cyclic testing or analytical methods (i.e. the new Annex L).

2 – GENERAL DESIGN PRINCIPLES

The basic principle for the design of earthquake-resistant timber buildings according to the force-based approach consists in classifying the structures as low-dissipative (DC1) or dissipative (DC2 or DC3). In buildings designed for DC2 or DC3, dissipation zones should be located either in joints and connections or in dedicated energy dissipation systems, such as slip-friction connections, high-force-to-volume damping devices, self-centring mechanisms, and low-damage systems. Energy dissipation within connections and joints should occur through the flexural yielding of laterally loaded metal fasteners or bonded-in rods. Conversely, axially loaded fasteners, axially loaded bonded-in rods, and timber/wood-based elements, which do not effectively dissipate energy, should be considered as non-dissipative components. When systems suitably developed for the energy dissipation are adopted, both timber members and connections should be elastically designed.

2.1 DESIGN STRENGTH

The design strength of dissipative zones for DC2 and DC3 design at Significant Damage (SD) Limit State can be calculated according to (1):

$$F_{\mathrm{Rd,d}} = k_{\mathrm{deg}}\, k_{\mathrm{mod}} \frac{F_{\mathrm{Rk,d}}}{\gamma_{\mathrm{M}}} \text{ with } k_{\mathrm{deg}} < 1 \tag{1}$$

where $F_{\mathrm{Rd,d}}$ is the design value of the strength of the dissipative zone; k_{deg} is the strength reduction factor due to degradation under cyclic loading; k_{mod} is the modification factor for load duration and moisture content; $F_{\mathrm{Rk,d}}$ is the characteristic value of the strength of the dissipative zone; and γ_{M} is the partial factor for a material property for accidental design situations.

The design strength of the non-dissipative components of DC2 and DC3 design and of all members of DC1 design should be calculated according to (2):

$$F_{\mathrm{Rd,nd}} = k_{\mathrm{mod}} \frac{F_{\mathrm{Rk,nd}}}{\gamma_{\mathrm{M}}} \tag{2}$$

where $F_{\mathrm{Rd,nd}}$ is the design value of the strength of the non-dissipative component; $F_{\mathrm{Rk,nd}}$ is the characteristic value of the strength of the non-dissipative component; and γ_M is the partial factor for a material property for persistent and transient design situations.

For DC2 and DC3 design, the seismic verification is then carried out according to (3):

$$E_{\mathrm{d}} \leq F_{\mathrm{Rd,d}} \tag{3}$$

by ensuring that the design strength of the dissipative component (e.g. the design moment resistance of the joint) $F_{\mathrm{Rd,d}}$ is greater than or equal to the effect of the design seismic action in the component, E_{d}, calculated using the reduced spectrum. The same verification is carried out also for all components in DC1 design, where however $F_{\mathrm{Rd,d}}$ is replaced by $F_{\mathrm{Rd,nd}}$ in (3).

Capacity design rules common to all dissipative structural types

In order to ensure yielding of the dissipative zones, all non-dissipative members and connections in DC2 or DC3 structures should be designed by satisfying (4):

$$\frac{\gamma_{\mathrm{Rd}}}{k_{\mathrm{deg}}} F_{\mathrm{Rd,d}} \leq F_{\mathrm{Rd,nd}} \tag{4}$$

where γ_{Rd} is the overstrength factor listed in Table 1.

Table 1: Values of the overstrength factors γ_{Rd} to be used in capacity design.

Non-dissipative failure mode	**Overstrength factor γ_{Rd}**
Failure modes of timber	1,6
Failure of metal plates in steel-to-timber or steel-to-foundation connections	1,6
Failure of anchor bolts connecting metal plates to the foundation or of anchor bolts connecting two separate metal plates	1,6
Failure of axially loaded timber-to-timber or timber-to-steel dowel-type connections	1,6
Failure of laterally loaded timber-to-timber or timber-to-steel dowel-type connections	1,3
Stabilising moment due to gravity loads in log shearwalls	1,3

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The ductile failure modes occurring in the dowel-type fasteners of the dissipative zones, namely those characterized by flexural yielding of metal fasteners, should satisfy (5):

$$\gamma_{Rd} F_{v,Rk,d} \leq F_{v,Rk,nd} \tag{5}$$

where $F_{v,Rk,d}$ is the characteristic strength of the selected ductile failure mode providing energy dissipation, $F_{v,Rk,nd}$ is the characteristic strength of the less ductile failure modes, namely those characterized by only timber crushing in compression at the interface with the dowel, and $\gamma_{Rd,d}$ is a factor equal to 1,2.

2.2 FORCE-BASED SEISMIC DESIGN METHOD

If the force-based approach is employed in seismic design, the behaviour factor q is used to calculate the seismic action E_d.

The behaviour factor q is given by the product of the three components q_S, q_D and q_R. For buindigs designed in DC1, the behaviour factor components q_D and q_R should be taken equal to 1,0. The behaviour factor component q_S should be taken equal to 1,5. For buildings designed in DC2 and DC3, the behaviour factor components q_D and q_R may be taken greater than 1,0 depending on the ductility class and structural type (as defined in the following Section 4.2).

It is noteworthy to mention that the DC1 ductility class can be adopted only when the maximum seismic action index S_δ is lower than the values reported in Table 3 for each structural type.

3 – MATERIALS

3.1 MECHANICAL PROPERTIES OF DISSIPATIVE ZONES

The mechanical properties that characterize the required performance of dissipative zones are the impairment factor, φ_{imp} (a parameter used to assess the low-cycle fatigue strength of dissipative zones), the strength reduction factor, k_{deg} and the displacement ductility μ. The impairment factor, φ_{imp} shall be determined through cyclic tests according to the EN 12512 [5] and prCEN/TS 1998-1-101and is defined according to (6):

$$\varphi_{imp}(\delta)=\frac{\Delta F_{1\text{-}3}(\delta)}{F_1(\delta)} \tag{6}$$

where $\Delta F_{1\text{-}3}(\delta)$ represents the reduction of resistance of the tested component from the first to the third cycle at equal target deformation δ and F_1 is the peak strength at the first cycle.

The strength reduction factor, k_{deg} is calculated according to (7):

$$k_{deg}=\frac{F_1(\delta_u)}{F_N} \tag{7}$$

where $F_1(\delta_u)$ is the load at the first cycle at the ultimate deformation δ_u from a cyclic test and F_N is the resistance obtained from a monotonic test. φ_{imp} and k_{deg} should be respectively not greater than 0,3 and not smaller than 0,8.

The displacement ductility μ is defined as the ratio between the ultimate and the yielding displacement obtained from a cyclic test conducted according to EN 12512 [5] and prCEN/TS 1998-1-101. The required values of μ of dissipative connections are defined for each structural type both for DC2 and DC3 as listed in Table 2.

Table 2: Minimum required ductility μ as defined in EN 12512 and prCEN/TS 1998-1-1001 of dissipative zones tested accordingly.

Structural type	Dissipative subassembly/joint/ 2D or 3D connector/connection	Type of ductility	μ DC2	μ DC3
Cross laminated timber structures	Shear wall*	Displ.	1,5	2,5
	Hold-downs, tie-downs, foundation tie-downs, angle brackets, shear plates	Displ.	1,5	1,5
	Screwed wall panel-to-panel joints	Displ.	-	5,5
Framed wall structures	Shear wall*	Displ.	2,2	3,5
	Connection (nail/screw/staple)	Displ.	3,5	5,5
Log structures	Shear wall*	Displ.	1,4	-

*The values provided refer to the system ductility of the sub-assembly, taking into account the ductility of all the individual connections and components.

3.2 MATERIAL PROPERTIES

The new timber chapter provides specific provisions on material properties both for timber/timber-based products and steel elements and fasteners.

Specific thickness limits are proposed for timber elements, e.g.: cross laminated timber (CLT) and glued

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laminated timber (GL) panels which, should be thicker than 54 mm.

For use in dissipative zones, panels employed as sheathing elements have to meet specific thickness and density requirements. Additionally, steel elements, metal fasteners, and bonded-in rods must comply with defined ductility capacities and strength criteria to ensure proper energy dissipation. For all metal fasteners, (8) should be verified:

$$\frac{M_{y,95}}{M_{y,Rk}} \leq 1.4 \qquad (8)$$

where $M_{y,95}$ is the 95th percentile of the yield moment and $M_{y,Rk}$ is the declared characteristic values of the yield moment defined in EN 14592. For fasteners, the required ductility class is S2 and S3 for structures designed in DC2 and DC3 class, according to EN 14592:2022 [6], while for bonded-in rods the ductility class is C for ribbed rods in accordance with the EN 1992-1-1:2022 [7], Table 5.5 or threaded rods of strength class 4.6 and 5.6 in accordance with EN ISO 898-1 [8].

4 – STRUCTURAL TYPES, BEHAVIOUR FACTORS, CAPACITY DESIGN RULES AND LIMITS OF SEISMIC ACTION

4.1 STRUCTURAL TYPES

Buildings with a primary timber structure are classified according to their structural types. The most common structural types are presented in tabular format together with a sketch and a short description:

- Cross laminated timber (CLT) buildings.
- Framed wall structures.
- Log House buildings.
- Moment resisting frames.
- Post and beam timber buildings with vertical bracings made of timber trusses.
- Mixed structures made of timber framing and masonry infill resisting to the horizontal forces.
- Large span arches with two or three hinged joints.
- Large span trusses with nailed, screwed, doweled and bolted joints.
- Vertical cantilever systems made with structurally continuous glulam or CLT wall elements.

However, if a structural type is not included in the above list, it can still be used providing that: (i) it is designed to DC1 or (ii) the properties of its subassemblies and dissipative zones are assessed according to the aforementioned cyclic test procedure if the structural type is designed to DC2 or DC3.

4.2 BEHAVIOUR FACTORS

For timber buildings designed to DC2 or DC3 which are regular in elevation, the default values of the behaviour factor *q* are listed in the following Table 3.

For structures designed in accordance with the concept of low-dissipative structural behaviour (DC1), the behaviour factor q should be taken not greater than 1,5.

Table 3: Default values of the behaviour factors q (excerpt from prEN 1998-1-2:2024).

Structural type	Max. S_δ for design in DC1 [m/s^2]	Ductility class						
		DC1	DC2			DC3		
		q	q_D	q_R	q	q_D	q_R	q
Cross laminated timber (CLT) structures, any height *H*	4,0	1,5	1,2	1,3	2,3	1,4	1,5	3,2
Framed wall structures with fully anchored walls, any height *H*	5,0	1,5	1,5	1,1	2,5	2,4	1,1	4,0
Framed wall structures with non-fully anchored walls, any height *H*	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
Log structures, $H \leq 9$ m	4,0	1,5	1,2	1,1	2,0	N/A	N/A	N/A
Log structures, H>9 m	4,0	1,5	1,0	1,1	1,65	N/A	N/A	N/A

4 – RULES FOR CROSS LAMINATED TIMBER (CLT) AND FRAMED WALL STRUCTURES

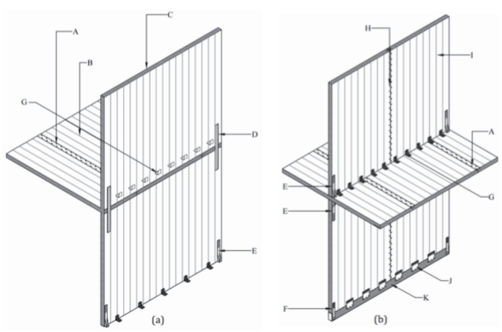
4.1 CROSS LAMINATED TIMBER STRUCTURES

General rules

The general rules for Cross-Laminated Timber (CLT) structures (Figure 1) that employ a platform-type system are reported in this Section. The shearwalls consist of

CLT shear wall panels and should be connected to the foundation and between each other through 2D or 3D connectors (e.g. hold-downs, foundation tie-downs, angle brackets, shear plates) and metal fasteners (e.g. anchoring bolts, nails and screws) in order to prevent the panel uplift and sliding. The anchoring connections against overturning should be placed at the wall ends and adjacent to the openings. Anchoring connections should be evenly distributed along the wall. The joints between orthogonal walls should be made with metal fasteners while floor and roof diaphragms can be made of CLT or other type of floor if in-plane resistance and stiffness are ensured.

The wall panels should be extended from one storey (or the foundation) to the next one along their width. The walls can be made of a single element ("single-panel") or composed of more than one panels ("multi-panel"). In the case of multi-panel configuration, each panel should be connected to adjacent panels through vertical joints using metal fasteners (e.g. screws or nails) and the width should be not less than 1/4 and 1/5 of the inter-storey height respectively for CLT and LVL/GLVL.



Keys

A	horizontal joint between CLT floor panels	*G*	angle bracket (in a) placed behind the wall, connecting to the floorplate)
B	CLT floor panel	*H*	vertical CLT panel joint
C	single piece CLT wall panel	*I*	CLT wall panel
D	tie-down	*J*	shear plate
E	hold-down	*K*	concrete base beam connected to the foundation and waterproof layer
F	foundation tie-down		

Figure 1. Walls and floors in CLT structures: (a) single-panel and (b) multi-panel [4]

Verifications

The dissipative zones for CLT structures designed for a medium ductility class (DC2) and composed of single-panel shearwalls are represented by (i) the shear connections between the CLT panel and the foundation (or the floor underneath) adopted to limit the sliding of the shearwalls and (ii) the mechanical anchors (e.g. hold-downs) used to prevent the rocking of the wall segments. For multi-panel shearwalls, vertical joints may be regarded as either dissipative or not dissipative. The CLT panels, the floor-to-floor and the floor-to-wall connections as well as the joints between the orthogonal walls are considered as non-dissipative and, for this reason, have to be over protected according to (9) as proposed by Casagrande et al. [9]:

$$F_{\mathrm{Rd,nd}} \geq \frac{\gamma_{\mathrm{Rd}}}{k_{\mathrm{deg}}} \Omega_{\mathrm{d}} F_{\mathrm{Ed,nd}} + F_{\mathrm{Ed,G}} \tag{9}$$

where $F_{\mathrm{Rd,nd}}$ is the design resistance of the non-dissipative component, k_{deg} is the strength reduction factor, $F_{\mathrm{Ed,nd}}$ and $F_{\mathrm{Ed,G}}$ are the design forces on the dissipative components due to the seismic and non-seismic action, respectively, and Ω_{d} is the structure overstrength ratio in the considered direction calculated as (10):

$$\Omega_{\mathrm{d}} = \min(\Omega_{\mathrm{d,i}}) \tag{10}$$

where $\Omega_{\mathrm{d,i}}$ is overstrength ratio at the i-th storey, in the considered direction. When a uni-directional behaviour of mechanical anchors (i.e. shear-connections, hold-downs) can be assumed, $\Omega_{\mathrm{d,i}}$ can be calculated according to (11):

$$\Omega_{\mathrm{d,i}} = \min\left(\frac{\sum_{j=1}^{N}|V_{\mathrm{Rd,a,i,j}}|}{\sum_{j=1}^{N}|V_{\mathrm{Ed,a,i,j}}|};\frac{\sum_{j=1}^{N}|M_{\mathrm{Rd,rock,i,j}}|}{\sum_{j=1}^{N}|M_{\mathrm{Ed,E,i,j}}|}\right) \tag{11}$$

where $V_{\mathrm{Rd,a,i,j}}$ is the shear resistance of the j-th shearwall at the i-th storey related to the shear connections, $M_{\mathrm{Rd,rock,i,j}}$ is the rocking moment resistance of the j-th shearwall at the i-th storey including the stabilizing effect of the gravitational load, $V_{\mathrm{Ed,a,i,j}}$ is the design shear load on the j-th shearwall at the i-th storey due to the seismic action and $M_{\mathrm{Ed,rock,i,j}}$ is the design rocking moment on the j-th shearwall at the i-th storey due to the seismic action. As an alternative to the rigorous approach expressed in (10), the structure overstrength ratio Ω_{d}can be assumed to be not greater than 1.2.

CLT structures designed for a high ductility class (DC3) shall consist of multi-panel shearwalls composed of panels with height-width aspect ratios not greater than 4:1 and not smaller than 1:1. Dissipative zones are those defined for DC2 for single-panel shearwalls (i.e. shear connections and hold-downs) and by vertical joints used to connect wall panels to one another in segmented multipanel walls (Table 4).

According to Casagrande et al. [9], when a uni-directional behaviour of shear connections can be adopted (i.e. shear behaviour along the horizontal direction), in order to ensure that *i)* the yielding of vertical joints occurs before the hold-down yielding and *ii)* a couple-panel (CP) kinematic mode of the shewarwall is reached (Masroor et

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al., 2020 [10]), the conditions expressed in (12) and (13) have to be satisfied:

$$F_{\mathrm{Rd,hd}} \geq 1.1 F_{\mathrm{Rd,c}} \frac{K_{\mathrm{SLS,\ anc}}}{K_{\mathrm{SLS,\ con}}} \qquad (12)$$

$$F_{\mathrm{Rd,hd}} \geq \max\left(1.1 F_{\mathrm{Rd,c}} \frac{K_{\mathrm{SLS,\ anc}}}{K_{\mathrm{SLS,\ con}}}; 1.1 n_{\mathrm{vj}} F_{\mathrm{Rd,c}} - \frac{N_{\mathrm{Ed}}}{m_{\mathrm{lp}}}\right) \qquad (13)$$

where $F_{\mathrm{Rd,hd}}$ is the design resistance of the hold-down, $F_{\mathrm{Rd,c}}$ is the design resistance of a single connection in the vertical joint, $K_{\mathrm{SLS,\ anc}}$ is the stiffness of the hold-down, $K_{\mathrm{SLS,\ con}}$ is the stiffness of a single connection in the vertical joint, n_{vj} is the number of connections in a vertical joint, N_{Ed} is the total compressive load on the shearwalls and m_{lp} is the number of panels in the shearwall.

Like for DC2 design, non dissipative components have to be protected by satisfying (11). For DC3, $\Omega_{\mathrm{d,i}}$ can be calculated according to (14):

$$\Omega_{\mathrm{d,i}} = \min\left(\frac{\sum_{j=1}^{N} |M_{\mathrm{Rd,rock,i,j}}|}{\sum_{j=1}^{N} |M_{\mathrm{Ed,E,i,j}}|}\right) \qquad (14)$$

where $M_{\mathrm{Rd,rock,i,j}}$ is the rocking moment resistance of the j-th shearwall at the i-th storey for a CP kinematic mode assuming that all vertical joints and hold-down yield and including the stabilizing effect of the gravitational load.

In order to establish a hierarchy of yielding between the rocking and the sliding failure mode of the shearwalls, the condition expressed by (15) has to be satisfied:

$$F_{\mathrm{Rd,s}} \geq 1.1 \frac{M_{\mathrm{Rd,rock}}}{M_{\mathrm{Ed,E}}} F_{\mathrm{Ed,E,s}} \qquad (15)$$

where $F_{\mathrm{Rd,s}}$ is the design resistance of the shear connection, $M_{\mathrm{Rd,rock}}$ is the rocking moment resistance of the shearwall for a CP kinematic mode and $F_{\mathrm{Ed,E,s}}$ is the design shear load on the shear connections due to the seismic action.

Table 4: Dissipative and non-dissipative zones in CLT structures

Ductility Class	**Shearwall type**	**Dissipative components**	**Non dissipative components**
DC2	Single-panel	Shear connections, Hold-down	CLT panels, floor-to-floor connection, floor-to-wall connections, jonst between ortoghonal walls
	Multi-panel	Shear connections, Hold-down, vertical joints (*)	CLT panels, floor-to-floor connection, floor-to-wall connections, jonst between ortoghonal walls, vertical joints (*)
DC3	Multi-panel	Vertical joint, Shear connections, Hold-down	CLT panels, floor-to-floor connection, floor-to-wall connections, jonst between ortoghonal walls

**In DC2 vertical joints can be either dissipative or non-dissipative*

4.2 FRAMED WALL STRUCTURES

General rules

Framed wall structures (Figure 2) employ a platform-type system, where the primary seismic-resistant system consists of shearwalls made of timber frames to which a wood-based panel (e.g. plywood or OSB) or other type of sheathing material is connected. The frames should be composed by vertical studs uniformly distributed with a bottom and top plate. The sheathing material should be attached to the frame on one or both sides using screws, nails, or staples. Joints between walls and foundations, or between stacked walls, should be made by means of 2D or 3D connectors (e.g., hold-downs, angle brackets, tie-downs) and/or shear connections (e.g., anchor bolts, nails, screws) to prevent overturning and sliding (fully anchored walls). Wall panel heights should be the same as the inter-story height and perpendicular walls should be connected by fastening two vertical studs together with nails, screws, or bolts.

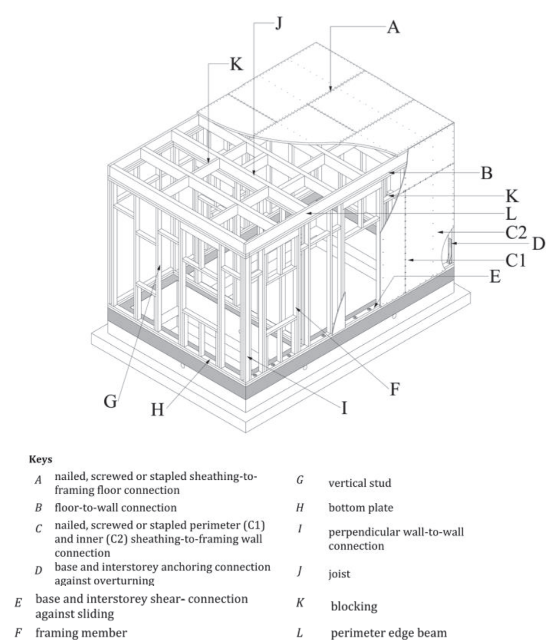

Figure 2. Framed wall structures [4]

Verifications

Framed wall structures designed according to a medium ductility class (DC2) should dissipate energy in (*i)* the sheathing-to-framing connections in the walls, (*ii)* the shear connections used to prevent the sliding of the walls and (*iii)* the connections designed to anchor the outer studs and prevent the rocking of the wall (i.e. hold-downs). All wooden frame and floor members as well as all other types of connections (e.g. floor-to-wall connections, sheathing-to-framing connections in the floor elements. etc.) should be regarded as non-dissipative and over protected according to (9) and (10) (Table 5).

Table 5: Dissipative and non-dissipative zones in Framed wall structures

Ductility Class	Sheathing panels	Fasteners in sheathing-to-framing connections	Dissipative components	Non dissipative components
DC2	Particleboard, Plywood, Fibreboard, OSB, Gypsum Fibre board (GFB), Densified Veneer Laminated Wood (DLW), Multi-layered solid wood panel (SWP-C), LVL-C	Screws, nails, stapes	Shething-to-framing connections, shear connections, Hold-downs	Wooden members, floor-to-floor connection, floor-to-wall connections, jonst between ortoghonal walls
DC3	OSB, plywood	Nails	Shething-to-framing connections	CLT panels, floor-to-floor connection, floor-to-wall connections, jonst between ortoghonal walls, shear connections, hold-downs

The overstrength ratio $\Omega_{d,i}$ can be calculated as expressed in (18) according to Casagrande et al. [9]:

$$\Omega_{d,i}=\min\left(\frac{\sum_{j=1}^{N}|V_{Rd,sh,i,j}|}{\sum_{j=1}^{N}|V_{Ed,sh,i,j}|};\frac{\sum_{j=1}^{N}|V_{Rd,a,i,j}|}{\sum_{j=1}^{N}|V_{Ed,a,i,j}|};\frac{\sum_{j=1}^{N}|M_{Rd,rock,i,j}|}{\sum_{j=1}^{N}|M_{Ed,E,i,j}|}\right) \quad (16)$$

where $V_{Rd,sh,i,j}$ is the shear resistance of the *j*-th shearwall at the *i*-th storey related to the sheathing-to-framing connection, $V_{Rd,a,i,j}$ is the shear resistance of the *j*-th shearwall at the *i*-th storey related to the shear connections, $M_{Rd,rock,i,j}$ is the rocking moment resistance of the j-th shearwall at the *i*-th storey including the stabilizing effect of the gravitational load, $V_{Ed,a,i,j}$ is the design shear load on the *j*-th shearwall at the *i*-th storey due to the seismic action and $M_{Ed,rock,i,j}$ is the design rocking moment on the *j*-th shearwall at the *i*-th storey due to the seismic action. Like for CLT structures, the structure overstrength ratio Ω_d can be assumed to be not greater than 1.2.

In DC3, framed walls should be sheathed only with plywood or OSB panels connected to the wooden frame only with nails. As a results, staples can be used to dissipate energy in DC2 only. Moreover, dissipative zones in DC3 are represented only by the sheathing-to-framing connections in the framed walls. Unlike for DC2, shear connections and hold-downs in DC3 have to be regarded as non-dissipative.

The overstrength ratio $\Omega_{d,i}$ can be calculated according to (17):

$$\Omega_d=\min(\Omega_{d,i})=\min\left(\frac{V_{Rd,LLRS,i}}{V_{Ed,E,LLRS,i}}\right) \quad (17)$$

It should be noted that only fully anchored walls can be used in DC2 and in DC3.

5 – PROVISIONS FOR NON-LINEAR STATIC ANALYSES OF TIMBER BUIDINGS

The second generation of Eurocodes allows the seismic design of new timber structures by adopting the Displacement-based approach (DBA). In particular, DBA for timber structures requires the verification of significant-damage (SD) and near-collapse (NC) limit states, in terms of local deformations of the dissipative and non-dissipative components through the N2-method. The development of the non-linear capacity curve of the whole structure is done by applying the Annex L rules which defines (*i)* the force-deformation relationships of dissipative timber components and (*ii)* the resistance of non-dissipative timber components for use in non-linear analysis.

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5.1 - DISSIPATIVE COMPONENTS

A trilinear load-deformation curve may be used as an approximation to model the behavior of dissipative zones in the timber structures as shown in Figure 3.

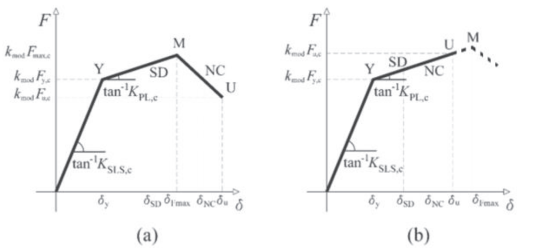

Figure 3. Derivation of the trilinear (a) and bilinear (b) load-deformation mean curve of dissipative zones in timber structures made with metal plate connectors and 3D-connectors [4]

The SD and the NC points should be obtained on the trilinear (or bilinear) curve at the deformation values SD and NC expressed in (18) and (19).

$$\delta_{SD}=\frac{1}{\gamma_{Rd,SD}}\cdot\left[\delta_y+\alpha_{SD,\vartheta}\cdot(\delta_u-\delta_y)\right] \quad (18)$$

$$\delta_{NC}=\frac{1}{\gamma_{Rd,NC}}\cdot\delta_u \quad (19)$$

where δ_{SD} is the deformation resistance at SD limit state, δ_{NC} is the deformation resistance at NC limit state, $\gamma_{Rd,SD}$ and $\gamma_{Rd,NC}$ are the partial factors on resistance at SD and NC limit states calculated according to (23) accounting for the total logarithmic standard deviation of the resistance model and $\alpha_{SD,\vartheta}$ is a percentage of the plastic part at the ultimate deformation (equal to 0.5).

5.2 - NON-DISSIPATIVE COMPONENTS

The design resistance of non-dissipative timber components for the verification of SD should be calculated using (20):

$$V_{Rd,b}=k_{mod}\cdot k_{mean}\frac{V_{Rk,b}}{\gamma_{Rd,LS}} \quad (20)$$

where $V_{Rd,b}$ is the design value of strength of the non-dissipative component, $V_{Rk,b}$ is the characteristic value of strength of the non-dissipative component, k_{mod} is the modification factor for duration of load and moisture content, k_{mean} is the ratio between the mean and the characteristic strength of the non-dissipative component and $\gamma_{Rd,LS}$ is the partial factor on resistance at the referred limit state (SD or NC) as reported in Section 5.3.

5.3 - PARTIAL FACTORS

New partial factors for seismic design/assessment are introduced in the second generation of Eurocode 8 [1]. For timber structures, they are valid only for new structures and they are calculated as expressed in (21):

$$\gamma_{Rd,LS}=e^{\left(\alpha_R^*\cdot\beta_{t,LS,CC}\cdot\sigma_{lnR}\right)} \quad (21)$$

where α_R^* is a constant corrected resistance sensitivity factor equal to 0.85, $\beta_{t,LS,CC}$ is the target reliability index at a specific limit state (equal to 1.60 for consequence class 2 and SD limit state) and σ_{lnR} is the total logarithmic standard deviation accounting for the model error and the variability in resistance resulting from uncertainty in the basic variables. The values of σ_{lnR} are reported in Table 6.

Table 6: Total logarithmic standard deviation σ_{lnR} of the resistance model

Type of product	Non-dissipative component and brittle failure	Dissipative component and ductile failure
Solid timber	0,26	-
Glulam, CLT	0,17	-
Wood - based panels	0,17	-
LVL, GLVL	0,14	-
Metal plate connectors and 3D-connectors	0,05	0,05
Connections, other than metal plate connectors and 3D-connectors, with laterally loaded metal fasteners with side members of wood and wood - based panels	0,19	0,19
Connections, other than metal plate connectors and 3D-connectors, with laterally loaded metal fasteners with side members of steel	0,10	0,10
Connections, other than metal plate connectors and 3D-connectors, with axially loaded fasteners	0,10	-

6 – CONCLUSIONS

This paper summarizes the most significant changes of the new chapter on timber structures of Eurocode 8 – part 1-2. New provisions for wood-based products and new principles for seismic design have been introduced, promoting the importance of capacity-based design at connection, wall and building levels. Overstrength factors and behaviour factors for dissipative structures have also been updated. New design provisions for cross-laminated timber and framed wall structures have been presented, defining the seismic design criteria and reporting the main formulas for seismic verifications. New rules for the non-linear design of timber structures have been introduced the new Annex L dedicated to the definition of the non-linear properties of the timber connections.

7 – ACKNOWLEDGEMENTS

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