EC8-2G

Il nuovo standard europeo per la progettazione sismica

EUCENTRE FOR YOUR SAFETY.

reluis

Materiali e tipologie costruttive

EN1998-1-2. Strutture in legno

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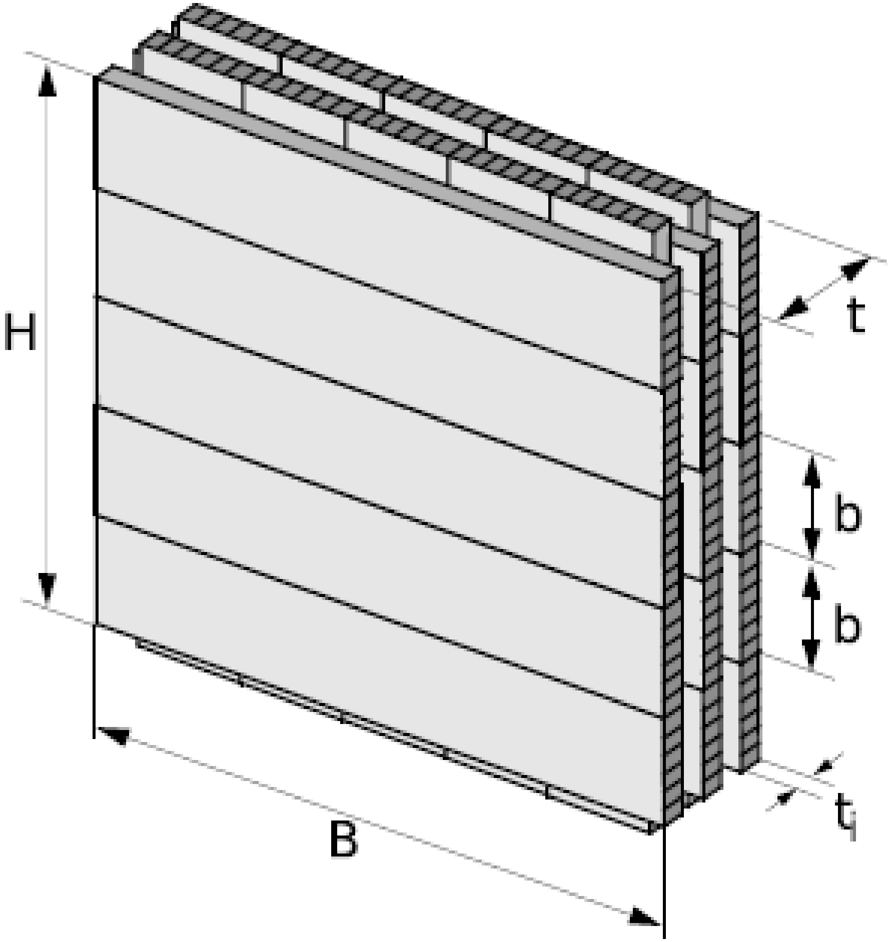
Pavia - 5 Giugno 2025

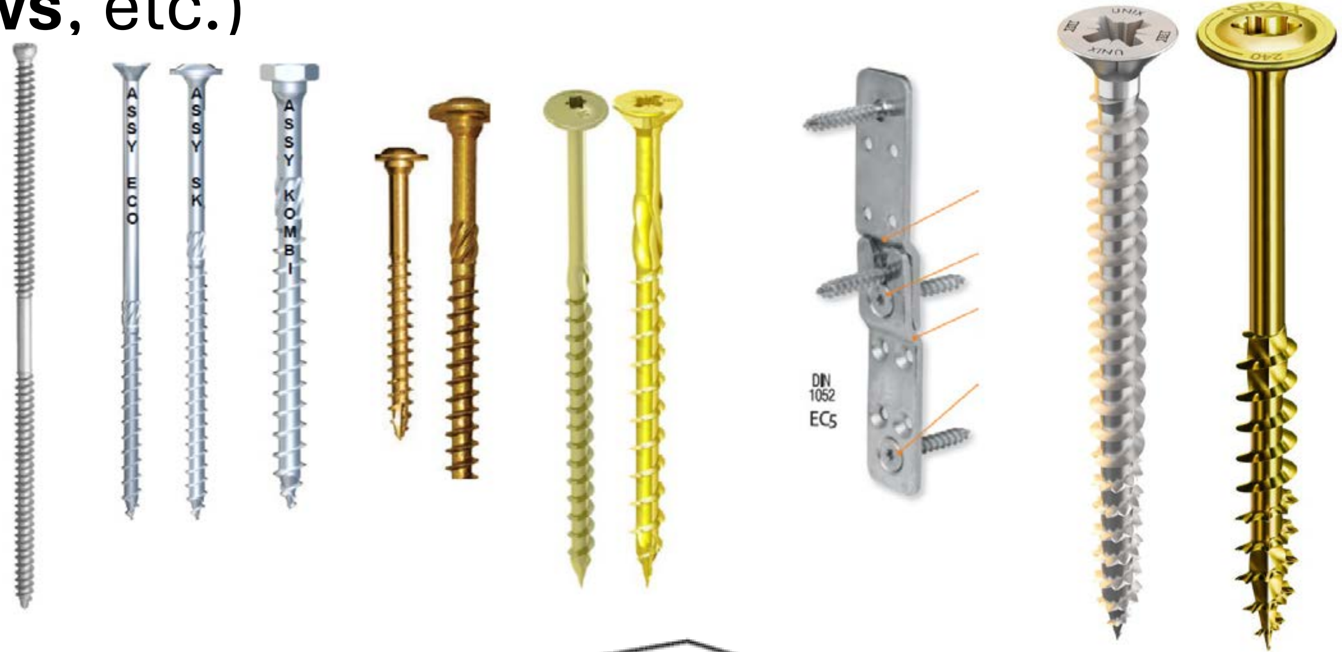
Outline

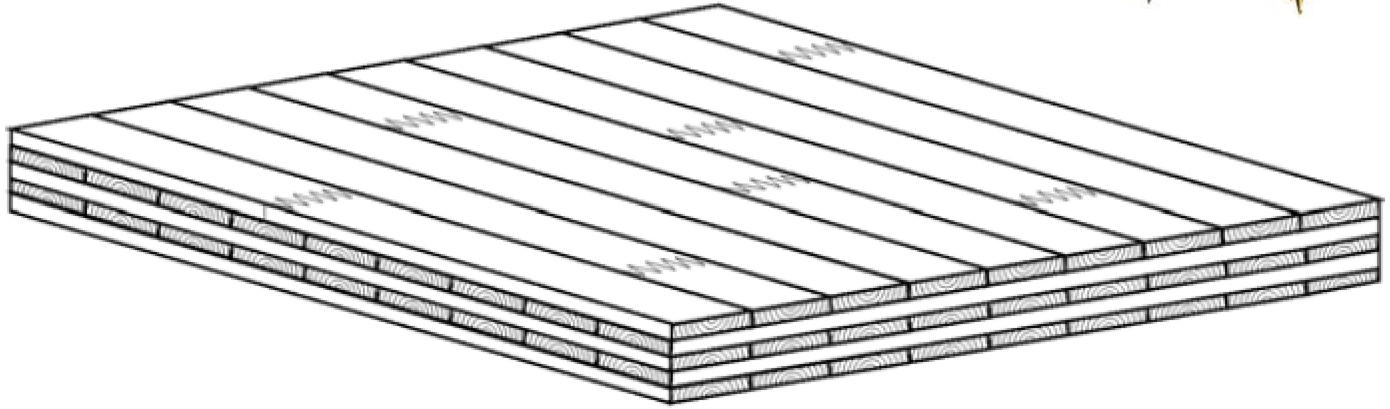
- Introduction:
  - On timber buildings in earthquake-prone regions
  - On timber structures in Eurocode 8
- Main updates of the Timber Chapter of EN1998-1-2:
  - A - Introduction of new wood-based panels
  - B - Revised definition of structural types
  - C - New safety format for seismic verifications at SD
  - D - New definition of behaviour factor q according to prEN1998-1-1
  - E - New ductility rules for dissipative zones
  - F - Capacity design and overstrength factors
  - G - Detailing rules for all structural types
- Conclusions and acknowledgements

Introduction on timber buildings in earthquake-prone regions

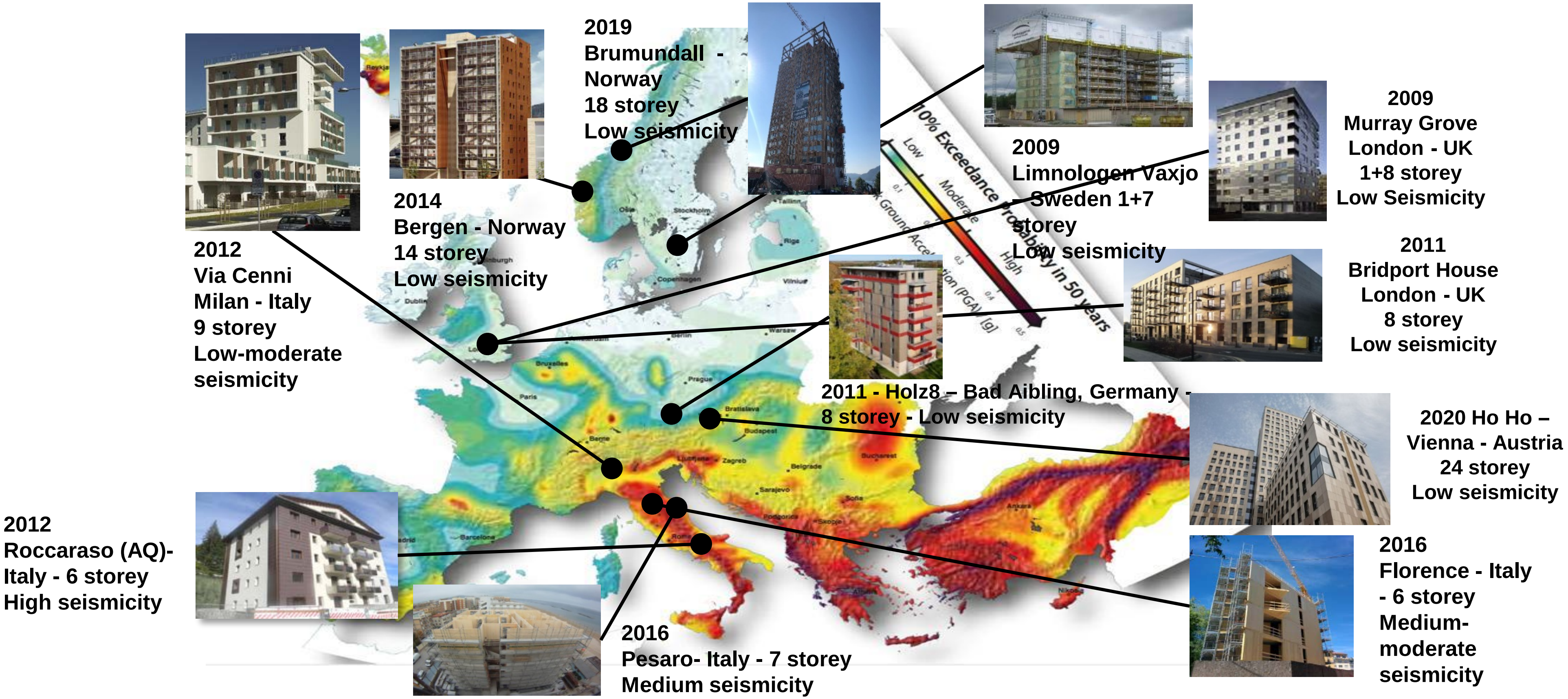
- Significant **evolution of timber buildings** since the 90's **due to new wood-based materials** (**CLT**, OSB, etc.) and **connection systems** (**self-drilling screws**, etc.)







- **Increase in size and height of timber buildings** also **in earthquake-prone regions**



Introduction on timber buildings in earthquake-prone regions

- **Significant research on seismic behaviour** carried out worldwide, demonstrating the overall **excellent seismic performance of timber buildings**


2006: 3 storey CLT – Kobe record 0.82 g (M 6.8) - Japan


2009: 6 storey Light-Frame – Northridge record 0.74g (M 7.2) - Japan


2007: 7 storey CLT – Kobe record 0.82 g (M 6.8) Japan


2011: 2 storey Log House– Montenegro record 0.50g - Portugal


2011: 3 storey Light Frame – Tohoku record 0.54g - Portugal


2011: 3 storey Light Frame – Montenegro record 0.50g - Portugal

Introduction on timber structures in Eurocode 8

Current version of EC8:

Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings (Chapter 8: Timber, **6** pp.)

Eurocode 8: Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings (no Chapter on timber buildings)

New generation of EC8:

Part 1-1: General rules and seismic action

Part 1-2: Rules for new buildings (Chapter 13 + Annex L: Timber, **42 + 6** pp.)

Revision of the whole standard

Added a new Chapter 10 and Annex C on timber (**26 + 3** pp.), previously missing

Introduction on timber structures in Eurocode 8

Current version of EC8:

Eurocode 8: Design of structures for earthquake resistance – Part 2: Bridges
(no Chapter on timber bridges)

No standard on cyclic testing of joints (for timber: EN12512 applies, however is out-of-date!)

New generation of EC8:

Revision of the whole standard

Added a new Annex C on timber bridges (**5** pp.) previously missing

New Part 1-101: Technical Specifications on Characterisation and qualification of structural components for seismic applications by means of cyclic tests

Added Section 6.8 on timber joints to update EN12512 (**3** pp.)

Main updates – Introduction of new wood-based panels

13.3.2 Material properties and detailing requirements


(1) The thickness of **cross laminated timber** (CLT) and glue-laminated timber (GL) panels should not be smaller than 54 mm.

….

(3) The sheathing material of panels in dissipative zones should satisfy a) to h):


….

d) **Oriented Strand Board (OSB)** sheathing should comply with EN 300, be at least 12 mm thick and have a characteristic density of at least 550 kg/m^3.



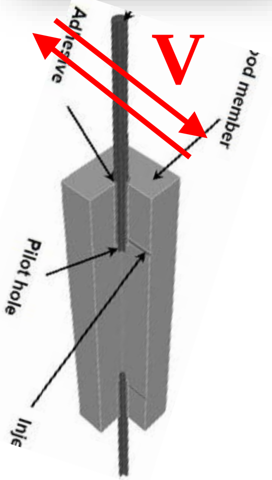
e) **Gypsum Fibre board (GFB)** sheathing should comply with EN 15283-2 and be at least 12 mm thick.

f) **Densified Laminated Wood (DLW)** sheathing should comply with type C4R of EN 61061-3-1:1998, Table 1, and have a characteristic density of at least 1300 kg/m^3.

….

(4) In dissipative zones, steel elements, metal fasteners and **bonded-in rods** in accordance with EN 1995-1-1:2024, 11.10, should satisfy a) to e):

….

c) **Bonded-in rods** should be either ribbed rods made of reinforcing steel with ductility class C in EN 1992-1-1:2022, Table 5.5, or threaded rods of strength class 4.6 and 5.6 in accordance with EN ISO 898-1. Threaded rods made of stainless steel in accordance with EN ISO 3506-1 should be of property classes 45 or 50.

Main updates – Revised definition of structural types

13.4.1 Structural types

(1) Buildings with a primary timber structure should be classified into the structural types defined in Table 13.1.

NOTE The drawings in Table 13.1 depict a part of a structure. Different number of storeys and structural layout may be used.

Table 13.1 — Timber structural types and examples of structures

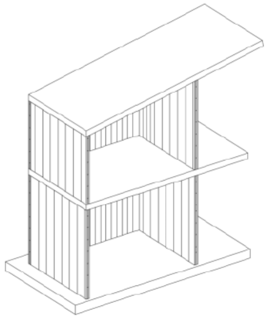
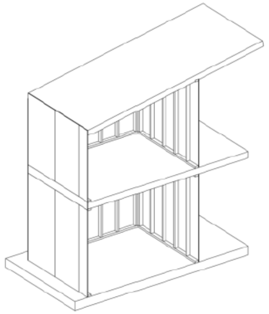
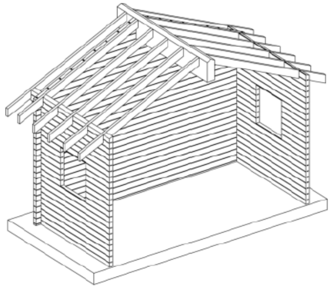
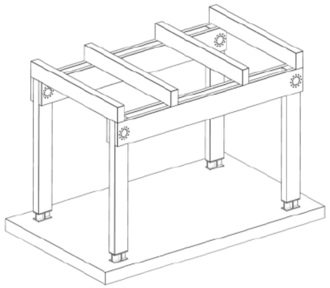
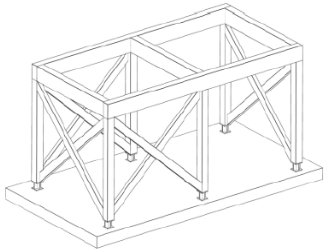
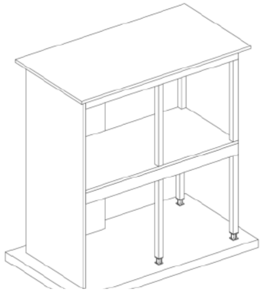
Examples of structural types	Timber structural types
	a) <u>Cross laminated timber (CLT) structures</u> [a] CLT structures are those where the primary seismic structure (see 3.1.24) is composed of shear walls made of cross laminated timber panels. Glulam, LVL or GLVL may be used as an alternative to CLT only in DC1 and DC2 design and for a seismicity index $S_\delta \leq 4{,}0$ [m/s²]. CLT structures should be designed according to 13.7.
	b) <u>Framed wall structures</u> Framed wall structures are those where the primary seismic structure is composed of shear walls made of timber frames to which a wood-based panel (e.g. plywood or OSB) or other type of sheathing material is connected. Framed wall structures should be designed according to 13.8. Framed wall structures can be classified as b1) or b2): b1) with fully anchored walls; b2) with non-fully anchored walls.
	c) <u>Log structures</u> Log structures are those where the primary seismic structure is composed by the superposition of rectangular or round solid or glulam timber elements ('logs'), prefabricated with carpentry connections at their ends and with upper and lower grooves. Log structures should be designed according to 13.9.

Table 13.1 — Timber structural types and examples of structures

Examples of structural types	Timber structural types
	d) <u>Moment-resisting frame structures</u> Moment-resisting frame structures are those where the primary seismic structure is composed of frames made of timber elements with semi-rigid (as defined in 3.1.33) moment-transmitting joints between the members, achieved with dowel-type connections. Moment-resisting frames structures should be designed according to 13.10.
	e) <u>Braced frame structures with dowel-type connections</u> Braced frame structures with dowel-type connections are those consisting of timber columns and beams, where the primary seismic structure is composed of timber diagonal bracings, with all pin-jointed dowel-type connections. Braced frame structures with dowel-type connections should be designed according to 13.11.
	f) <u>Vertical cantilever structures</u> [a] Vertical cantilever structures are those where the primary seismic structure is composed of vertically continuous cantilever glulam, LVL, GLVL or CLT walls or columns without any horizontal joints (wall on the left on the figure). Vertical cantilever structures should be designed according to 13.12.

[a] CLT structures can belong to category a) if the height of shear walls is equal to one interstorey height (platform frame construction – see 3.1.21); they belong to category f) if the height of shear walls is greater (balloon frame construction – see 3.1.2).

Main updates – New safety format for seismic verifications at SD

13.2.2 SAFETY VERIFICATIONS

(1) **For verifications of DC2 and DC3 design at SD Limit State, the design strength of dissipative zones should be calculated by Formula (13.1).**

$$F_{\mathrm{Rd,d}} = k_{\mathrm{deg}} k_{\mathrm{mod}} \frac{F_{\mathrm{Rk,d}}}{\gamma_M} \quad \text{with the limitation } k_{\mathrm{deg}} \leq 1 \qquad (13.1)$$

where:

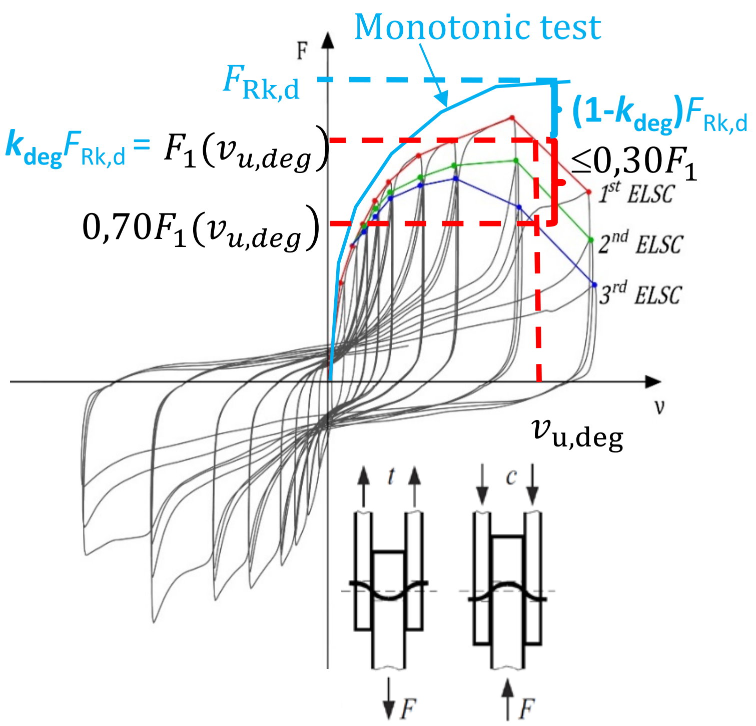
$F_{\mathrm{Rd,d}}$ is the design value of the strength of the dissipative zones;

k_{deg} is the strength reduction factor due to degradation under cyclic loading, given in **13.3.1(1)**

k_{mod} is the modification factor for duration of load and moisture content according to EN1995-1-1:2024, **5.1.3**, Table 5.4;

$F_{\mathrm{Rk,d}}$ is the characteristic value of the strength of the dissipative zones, according to EN1995-1-1:2024, clause **11**;

γ_{M} is the partial factor for a material property according to EN1995-1-1:2024, **4.5.2**, Table 4.3.



For DC2 and DC3 the values of γ_{M} are those given in EN1995-1-1:2024, 4.5.2, Table 4.3, for accidental situations (=1), unless the National Annex gives different values for use in a Country.

NOTE CEN/TS 1998-1-101 and EN 12512 can be used to determine the mechanical properties of a dissipative zone such as the strength reduction factor k_{deg} and the ductility μ.

$k_{\mathrm{deg}} = F_{1,\mathrm{cyclic}}(\mathrm{EN12512})/F_{\mathrm{monotonic}} = F_1(v_{\mathrm{u,deg}})/F_{\mathrm{Rk,d}}$

k_{deg}=0,8 when no experimental results are available

Main updates – New safety format for seismic verifications at SD

(2) **The design strength of the non-dissipative components of DC2 and DC3 design and of all members of DC1 design should be calculated as given by Formula (13.2).**

$$F_{\mathrm{Rd,nd}} = k_{\mathrm{mod}} \frac{F_{\mathrm{Rk,nd}}}{\gamma_{\mathrm{M}}} \quad (13.2)$$

where:

$F_{\mathrm{Rd,nd}}$ is the design value of the strength of the non-dissipative components;

$F_{\mathrm{Rk,nd}}$ is the characteristic value of the strength of the non-dissipative components, according to EN1995-1-1:2024, Clauses **8**, **11** and **12**;

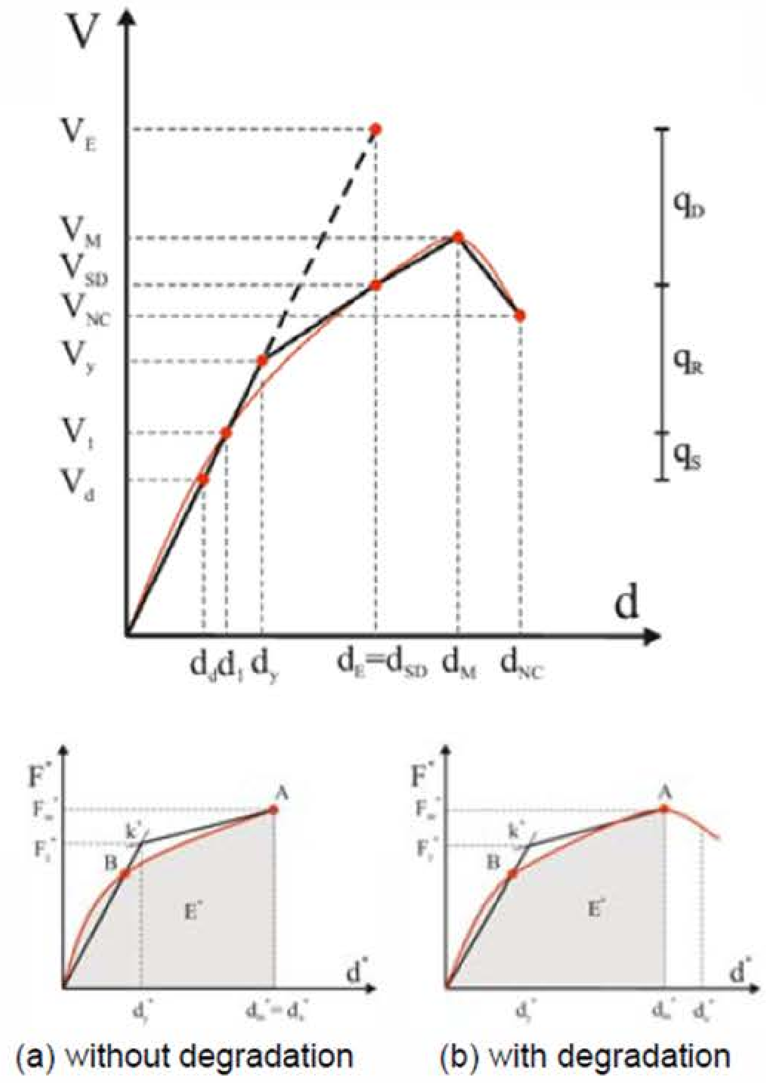
k_{mod} is the modification factor for duration of load and moisture content according to EN1995-1-1:2024, **5.1.3**, Table 5.4;

γ_{M} is the partial factor for a material property according to EN1995-1-1:2024, **4.5.2**, Table 4.3.

NOTE The values of γ_{M} are:

- **For DC1, those given by EN1995-1-1:2024, 4.5.2, Table 4.3, for persistent and transient situations (>1), unless the National Annex gives different values for use in a country;**
- **For DC2 and DC3, those given in EN1995-1-1:2024, 4.5.2, Table 4.3, for accidental situations (=1), unless the National Annex gives different values for use in a Country,**

Main updates – New definition of q according to prEN1998-1-1



$$\boxed{q = q_S \cdot q_R \cdot q_D = \frac{V_1}{V_d} \cdot \frac{V_{SD}}{V_1} \cdot \frac{V_E}{V_{SD}}}$$

(1) (2)

(1)
- q_s :Over-strength component introduced in the design phase (1.5 material independent)
- q_R :Over-strength component due to the redistribution of seismic action in redundant structures
- q_D :Deformation capacity and energy dissipation component

(2)
- V_d :Design base-shear
- V_1 :First plasticization base-shear
- V_{SD} :Significant Damage base-shear
- V_E :Elastic response base-shear

Main updates – New definition of q according to prEN1998-1-1

Table 13.2 — Default values of the behaviour factors q for buildings regular in elevation and maximum seismic action index S_δ for design in DC1

Structural type	Maximum S_δ for design in DC1 [m/s²]	DC1	DC2			DC3		
		q	q_D	q_R	q	q_D	q_R	q
a) Cross laminated timber (CLT) structures, any height H	4,0	1,5	1,2	1,3	2,3	1,4	1,5	3,2
b1) Framed wall structures, any height H With fully anchored walls	5,0	1,5	1,5	1,1	2,5	2,4	1,1	4,0
b2) Framed wall structures, any height H With non-fully anchored walls	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
c) Log structures $H \leq 9{,}0$ m	4,0	1,5	1,2	1,1	2,0	N/A	N/A	N/A
d) Log structures $H > 9{,}0$ m	4,0	1,5	1,0	1,1	1,65	N/A	N/A	N/A
d1) Moment-resisting frames, any height H Single storey	4,0	1,5	1,3	1,1	2,1	2,0	1,1	3,3
d2) Moment-resisting frames any height H Multi-storey, one-bay	4,0	1,5	1,3	1,2	2,3	2,0	1,2	3,6
d3) Moment-resisting frames any height H Multi-storey, multi-bay	4,0	1,5	1,3	1,3	2,5	2,0	1,3	3,9
e) Braced frame structures with dowel-type connections $H \leq 20$ m	4,0	1,5	1,3	1,0	2,0	N/A	N/A	N/A
f) Braced frame structures with dowel-type connections $H > 20$ m	4,0	1,5	1,0	1,0	1,5	N/A	N/A	N/A
g) Vertical cantilever structures $H \leq 12$ m	4,0	1,5	1,2	1,3	2,3	N/A	N/A	N/A
h) Vertical cantilever structures $H > 12$ m	4,0	1,5	1,0	1,3	2,0	N/A	N/A	N/A
i) Braced frame structures with carpentry connections and interacting masonry infills $H \leq 12$ m	4,0	1,5	1,3	1,1	2,0	N/A	N/A	N/A
j) Braced frame structures with carpentry connections and interacting masonry infills $H > 12$ m	4,0	1,5	1,0	1,1	1,65	N/A	N/A	N/A
k) Braced frame structures with carpentry connections, any height H	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
l) Two-pin and three-pin timber arches, three-pin timber frames and timber dome structures, any height H	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
m) Large span timber truss portal frame structures, any height H	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A

N/A: Not Applicable

Table 13.2 — Default values of the behaviour factors q for buildings regular in elevation and maximum seismic action index S_δ for design in DC1

Main updates – New ductility rules for dissipative zones

13.2 Basis of design

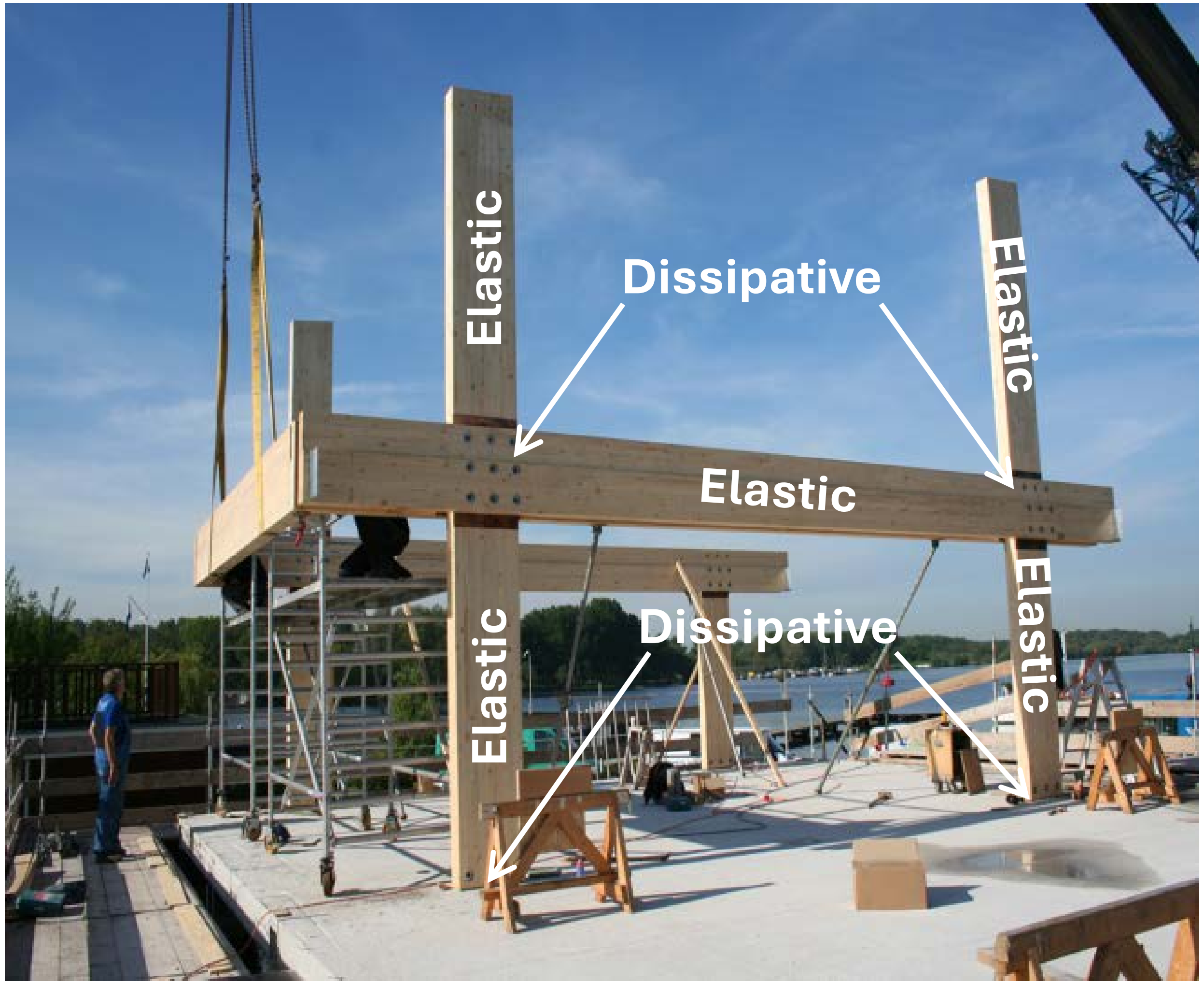
13.2.1 Design concepts

(6) In buildings designed in DC2 or DC3, **dissipative zones should be located either in** a) or in b):

a) the **joints and connections**;

b) **energy dissipation systems**.

(8) The energy dissipation in joints and connections should take place by flexural yielding of laterally loaded metal fasteners or of laterally loaded bonded-in rods (see 3.1.2), and **the timber members should remain in the elastic range**, at the exception of systems satisfying 13.4.2(7) and 3.7.2(12).



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Main updates – New ductility rules for dissipative zones

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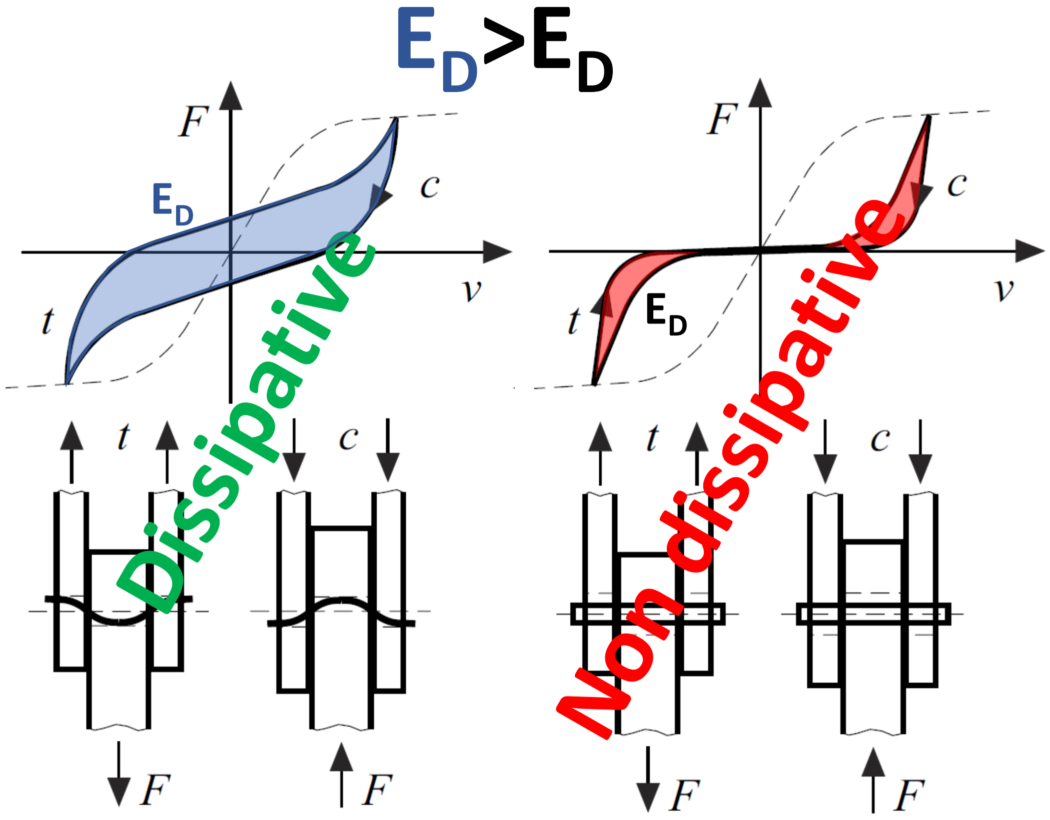
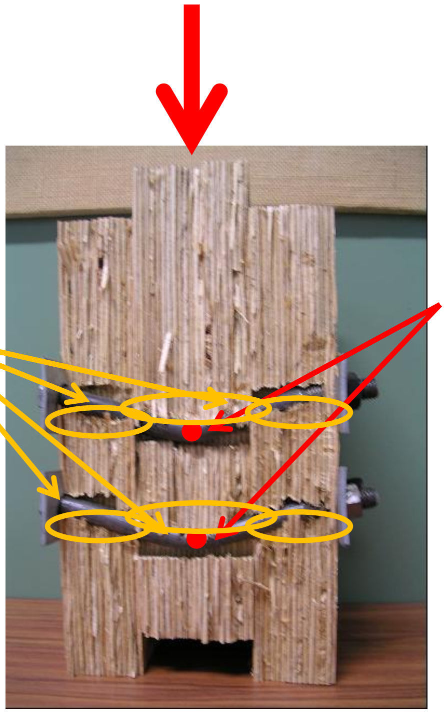
(8) The **energy dissipation in joints and connections should take place by flexural yielding of laterally loaded metal fasteners** or of laterally loaded bonded-in rods (see 3.1.2), and the timber members should remain in the elastic range, at the exception of systems satisfying 13.4.2(7) and 3.7.2(12).



Dissipative mechanism: fastener plasticization and timber plasticization in compression at the interface with the fastener

Non dissipative mechanism: timber plasticization in compression at the interface with the fastener, with fastener still in elastic phase

Crushing failure of timber parallel to grain at the interface with the metal fastener

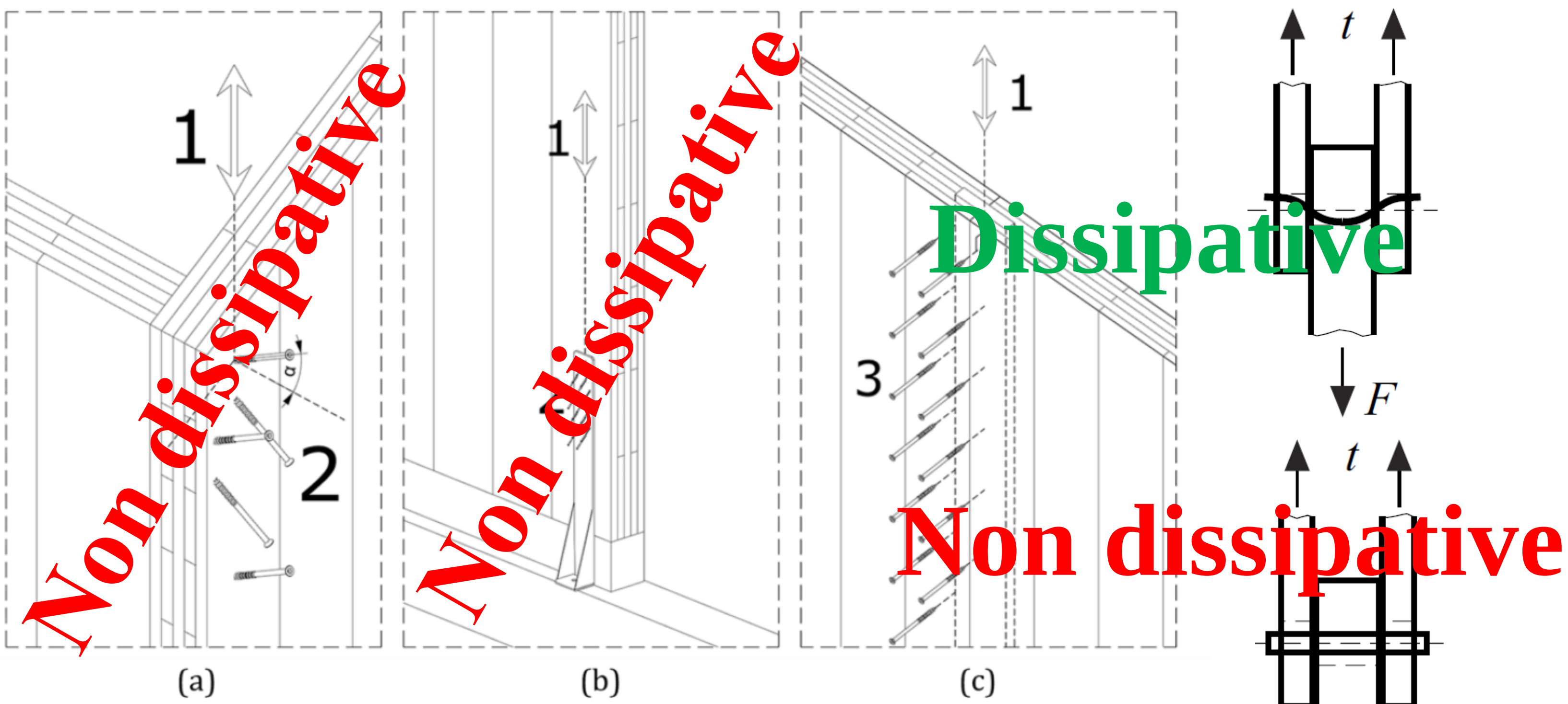


Plastic hinge formation in the laterally loaded metal fastener

(7) **Buildings in which all joints and connections are made with axially-loaded fasteners** and/or axially loaded bonded-in rods **should be designed to DC1.**

NOTE Axially-loaded fasteners and bonded-in rods cannot dissipate energy.

Main updates – New ductility rules for dissipative zones



Key

1 direction of the shear force being transferred

2 fasteners inclined with respect to the direction of the shear force being transferred

3 fasteners perpendicular to the direction of the shear force being transferred

Figure 13.1 — Examples of non-dissipative and dissipative connections: (a) and (b) connection with fasteners inserted inclined with respect to the direction of the shear force, transferring most of the load via axial resistance, which should not be considered as dissipative; (c) connection with fasteners inserted perpendicular to the direction of the shear force, transferring most of the action effect via shear resistance, which may be considered dissipative

Main updates – New ductility rules for dissipative zones

(3) Failure modes (a), (b) and (c) for dowel-type connection in single shear as given in EN 1995-1-1:2024, 11.2.3.2(1), and failure modes (a) and (b) for dowel-type connection in double shear as given in EN 1995-1-1:2024, 11.2.3.2(2), should be avoided in all dissipative zones by satisfying Formula (13.5). Failure modes (a) and (b) for dowel-type connection in multiple shear, as given in EN 1995-1-1:2024, 11.2.3.5, should be avoided in all dissipative zones by satisfying Formula (13.5).

$$\gamma_{Rd,d} F_{\mathrm{v,Rk,d}} \leq F_{\mathrm{v,Rk,nd}} \tag{13.5}$$

where:

$F_{\mathrm{v,Rk,d}}$ is the characteristic strength of the selected ductile failure mode providing energy dissipation, according to EN 1995-1-1:2024, **11.2.3.2**;

$F_{\mathrm{v,Rk,nd}}$ is the characteristic strength of the less ductile failure mode, according to EN 1995-1-1:2024, **11.2.3.2**;

$\gamma_{\mathrm{Rd,d}}$ is a partial factor equal to 1,2

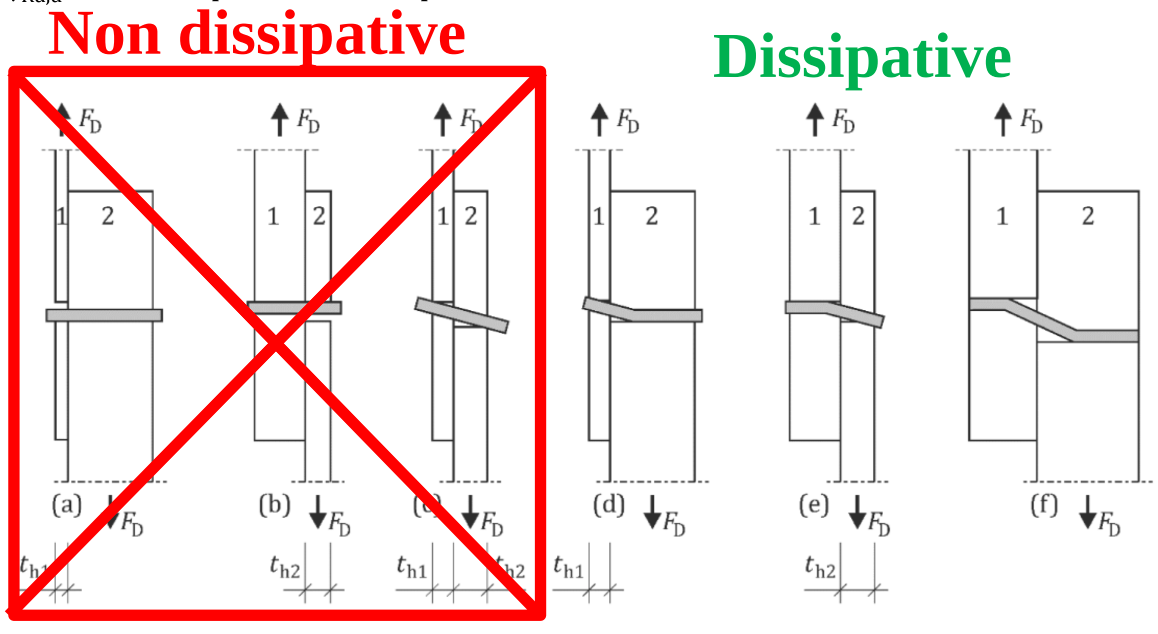


Figure 11.5 — Possible failure modes a) – f) for a fastener loaded in single shear

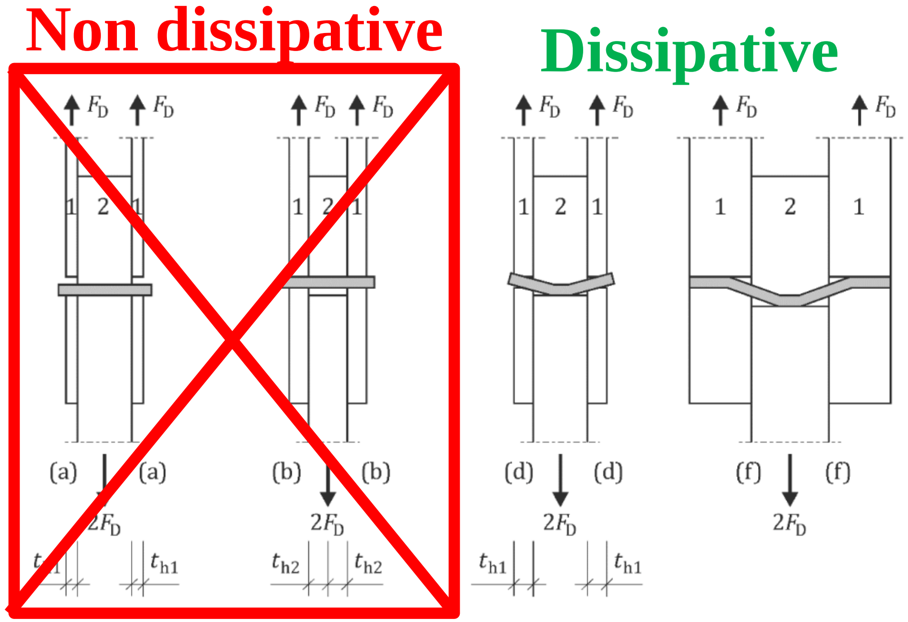
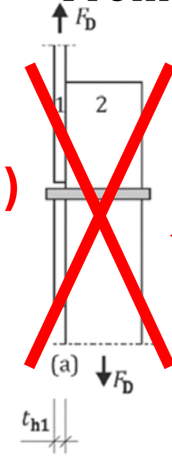


Figure 11.6 — Failure modes (a), (b), (d) and (f) for a fastener loaded in double shear

Main updates – New ductility rules for dissipative zones

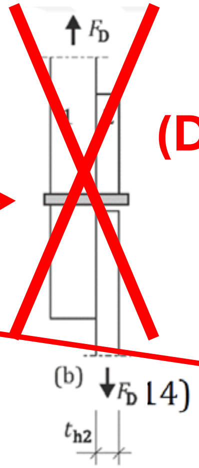
From the revised version of EN1995-1-1:2025 (currently in preparation in CEN/TC250/SC5 for FV):

$$1,2 \cdot \min\left(F_{v,k,(d)}; F_{v,k,(e)}; F_{v,k,(f)}\right) \le \min\left(F_{v,k,(a)}; F_{v,k,(b)}; F_{v,k,(c)}\right)$$

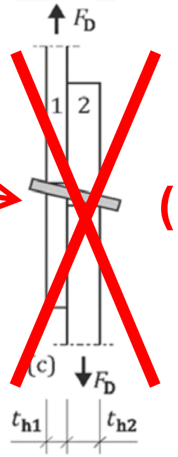


(DC1)

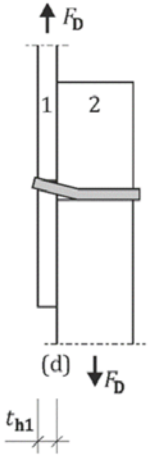
$$F_{D,k} = \min \begin{cases} f_{h,1,k}\, t_{h1}\, d & \text{mode (a)} \\ f_{h,2,k}\, t_{h2}\, d & \text{mode (b)} \\ \frac{f_{h,1,k}\, t_{h1}\, d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1 + \frac{t_{h2}}{t_{h1}} + \left(\frac{t_{h2}}{t_{h1}}\right)^2\right] + \beta^3\left(\frac{t_{h2}}{t_{h1}}\right)^2} - \beta\left(1 + \frac{t_{h2}}{t_{h1}}\right)\right] & \text{mode (c)} \\ 1{,}05\, \frac{f_{h,1,k}\, t_{h1}\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h1}^2}} - \beta\right] & \text{mode (d)} \\ 1{,}05\, \frac{f_{h,1,k}\, t_{h2}\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta) + \frac{4\beta(1+2\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h2}^2}} - \beta\right] & \text{mode (e)} \\ 1{,}15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,k}\, f_{h,1,k}\, d} & \text{mode (f)} \end{cases}$$



(DC1)



(DC1)



DC2

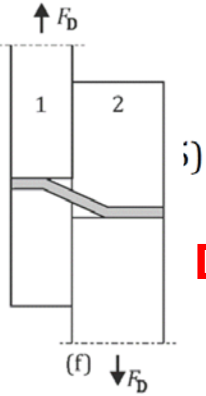
DC2

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}}$$

where

$f_{h,1,}$, $f_{h,2,k}$	are the characteristic embedment strengths of members 1 and 2, given in Ta
t_{h1}, $t_{h,2}$	are the embedment depths of members 1 and 2, see Figures 11.6 and 11.7;
$M_{y,k}$	is the characteristic yield moment given in Table 11.7;
d	is the diameter of the fastener.



DC3

Main updates – New ductility rules for dissipative zones

(4) Brittle failure modes like splitting, row shear, block shear, plug shear, and net tensile failure of wood, as defined in EN 1995-1-1:2024, 11.6, should be avoided in all dissipative zones by satisfying Formula (13.4).

NOTE Reinforcement can be used in a dissipative zone as a means to prevent brittle failure modes, see for example EN 1995-1-1:2024, 11.8.

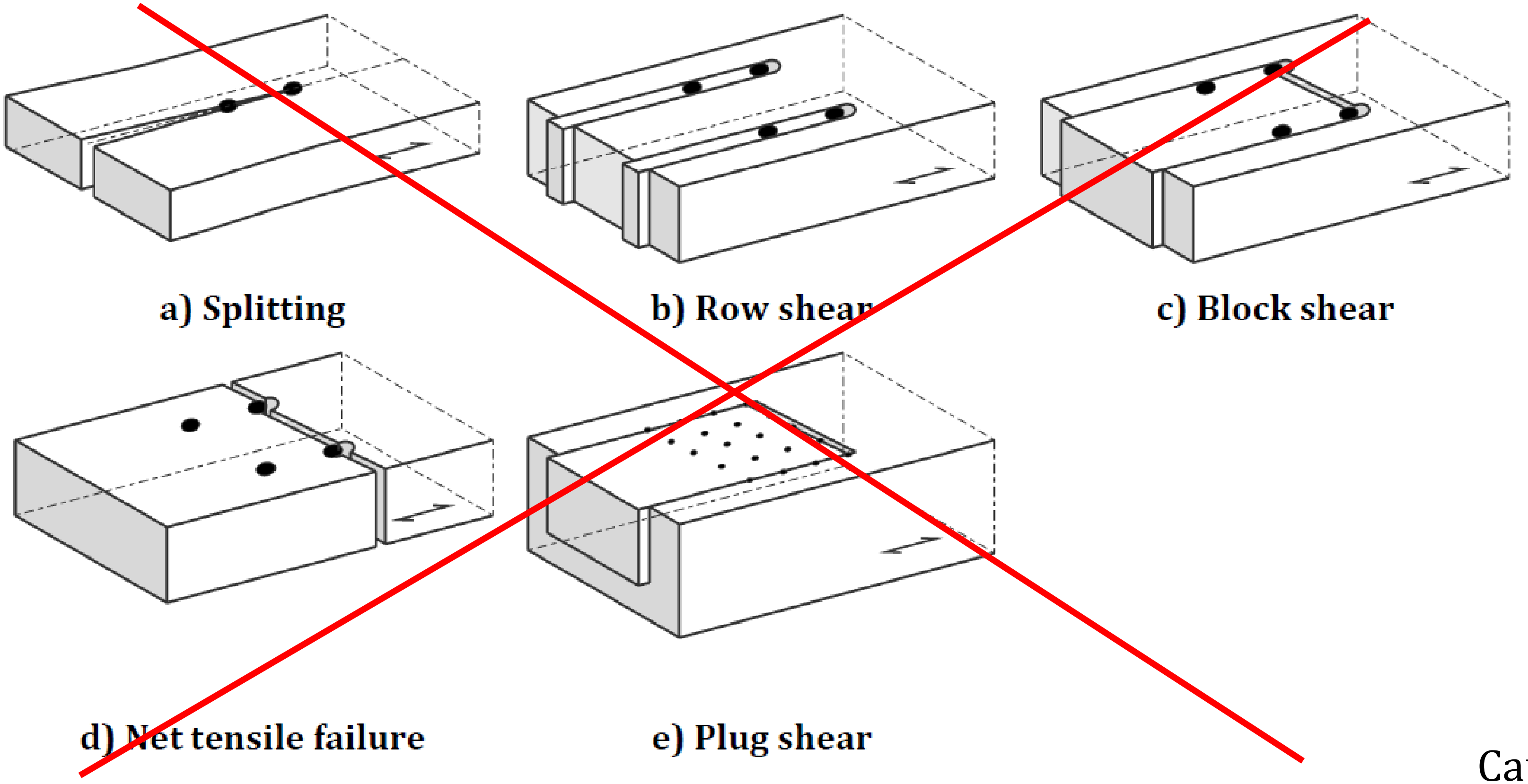
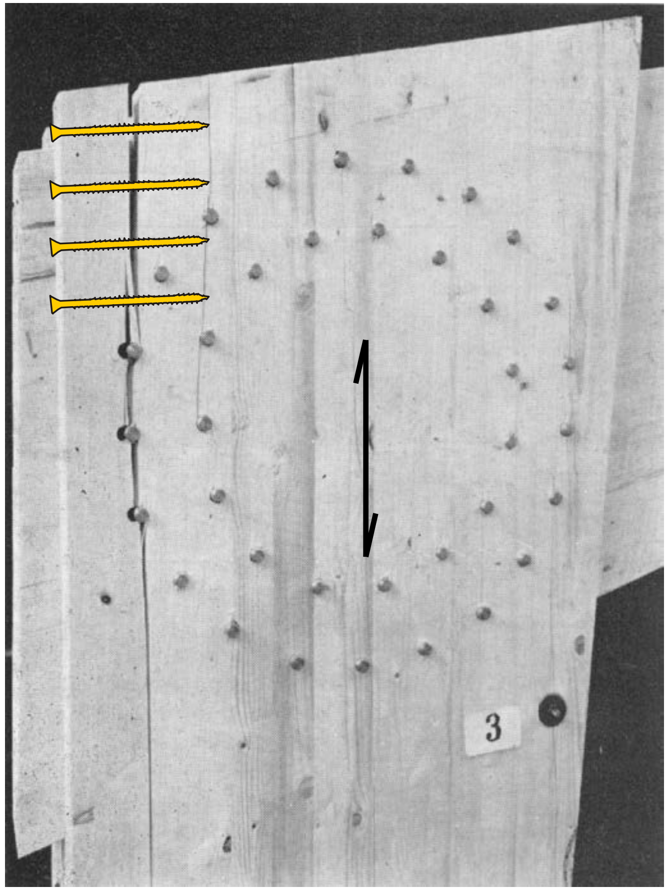


Figure 11.22 — Examples of brittle failure modes



Capacity design rule:

$$\boxed{\frac{\gamma_{\mathrm{Rd}}}{k_{\mathrm{deg}}} F_{\mathrm{Rd,d}} \leq F_{\mathrm{Rd,nd}} \qquad (13.4)}$$

For DC2 and DC3 design of the dissipative components (e.g. the beam-column joins):

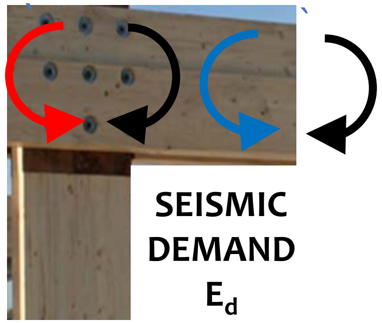
$$E_d(S_d) \leq F_{Rd,d} \ (= k_{deg} \, k_{mod} \, F_{Rk,d} \, / \, \gamma_M)$$

DISSIPATIVE (DUCTILE) JOINT

STRENGTH CAPACITY $F_{Rd,d}$

TIMBER BEAM MEMBER (NON DISSIPATIVE)

STRENGTH CAPACITY $F_{Rd,nd}$



SEISMIC DEMAND E_d

WHICH DEMAND?

$$\frac{\gamma_{Rd}}{k_{deg}} F_{Rd,d}$$

TIMBER COLUMN MEMBER (NON DISSIPATIVE)

13.4.3 Capacity design rules common to all dissipative structural types

(1) To ensure yielding of the dissipative zones, all non-dissipative members and connections in DC2 or DC3 structures should satisfy (2) to (6).

(2) For DC2 and DC3 design of the structural types in Table 13.2, the design strength $F_{Rd,nd}$ of the non-dissipative components should satisfy Formula (13.4).

$$\frac{\gamma_{Rd}}{k_{deg}} F_{Rd,d} \leq F_{Rd,nd} \left(= k_{mod} \frac{F_{Rk,nd}}{\gamma_M}\right) \qquad \text{with the limitation} \qquad k_{deg} \leq 1 \qquad (13.4)$$

where:

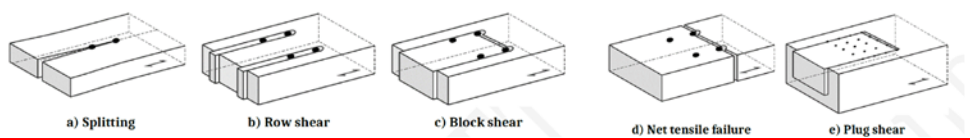
γ_{Rd} is the overstrength factor, given in Table 13.4;

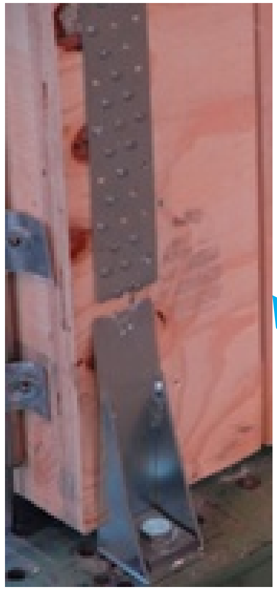
k_{deg} is the strength reduction factor defined in **13.3.1(1)**; **(0,8)**

$F_{Rd,d}$ is the design strength of the dissipative component, calculated according to **13.2.2(1)**;

$F_{Rd,nd}$ is the design strength of the non dissipative component, calculated according to **13.2.2(2)**.

Main updates – Capacity design and overstrength factor





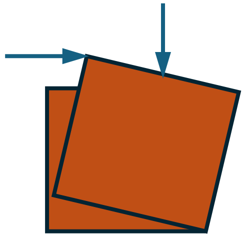
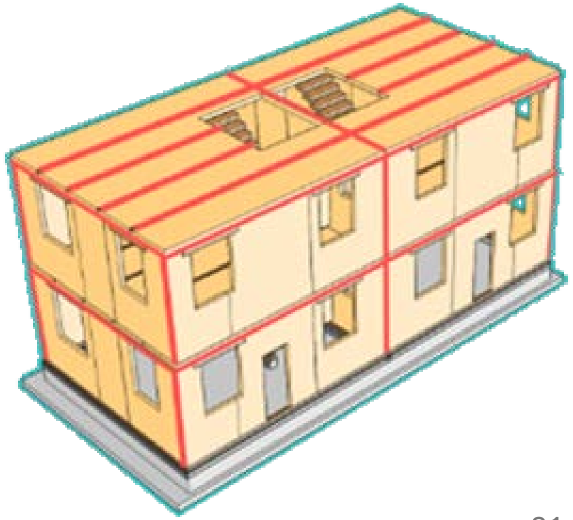


Table 13.4 — Values of the overstrength factors γ_{Rd} to be used in capacity design

Non-dissipative failure mode	Overstrength factor γ_{Rd}	Formula No.	
		Connection level	Wall and building level
Failure modes of timber	1,6[b]	(13.4)	Refer to the relevant structural type sections (from 13.7 to 13.14)
Failure of metal plates[a] in steel-to-timber or steel-to-foundation connections	1,6[b]	(13.4)	
Failure of anchor bolts connecting metal plates[a] to the foundation or of anchor bolts connecting two separate metal plates[a]	1,6[b]	(13.4)	
Failure of axially loaded timber-to-timber or timber-to-steel connections including axially-loaded bonded-in rods	1,6[b]	(13.4)	
Failure of laterally loaded timber-to-timber or timber-to-steel dowel-type connections	1,3	(13.4)	
Stabilising moment due to gravity loads in log shear walls	1,3	-	(13.19)

[a] 2D, 3D plates, connectors or any other equivalent element.

[b] For high ductility moment-resisting frames with expanded tube fasteners and Densified Laminated Wood (according to 13.10.3(2)) and log structures, the value of γ_{Rd} may be reduced to 1,3.

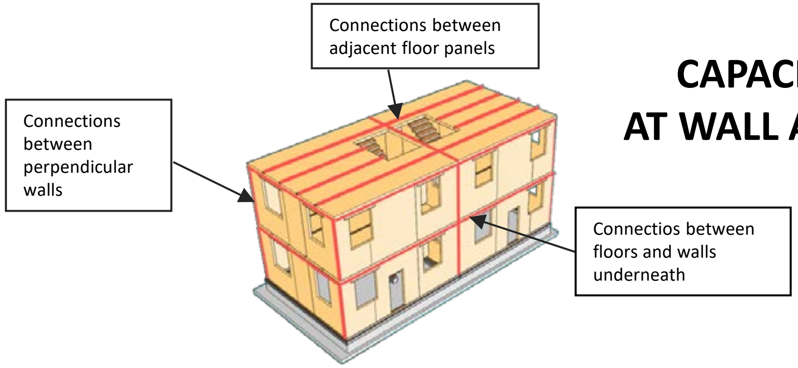


Main updates – Capacity design and overstrength factor

FOR EACH STRUCTURAL TYPE:

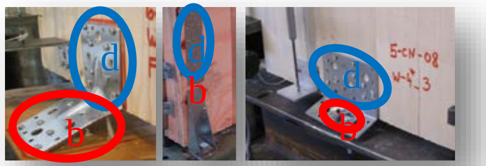
GENERAL RULES
General description of the structural components (walls, floors) and type of connections generally used

CAPACITY DESIGN RULES FOR DC2 AND DC3



CAPACITY DESIGN RULES AT WALL AND BUILDING LEVEL

CAPACITY DESIGN RULES AT CONNECTION AND 2D/3D NAILING PLATE LEVEL



Main updates – Capacity design and overstrength factor

For example, for CLT structures:

13.7 Rules for cross laminated timber (CLT) structures

13.7.1 General rules

(1) 13.7.1 should be applied to **platform-type structures** where **the primary seismic structure** is **made of CLT panels** according to 13.3.2(1). Glulam, LVL or GLVL as defined in 13.3.2(2) may be used as substitute of CLT in DC1 and DC2 provided that the seismic action index S_δ is not greater than 4,0 m/s^2.

(2) The **secondary structure** should be **made of** either **CLT panels**, or other types of solid wood panels as defined in 13.3.2(2). Post-and-beam members may also be used.

(3) The **connections of the walls to the foundation** should satisfy a) to f):

a) they should be **made by means of 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, etc.). These connectors should satisfy EN 1995-1-1:2024, Annex G;

b) they should **prevent uplift and sliding of the walls**;

c) **anchoring connections against overturning** should be **placed at wall ends**, and **adjacent to door openings** in wall panels. Anchoring connections against overturning should also be placed at opening ends when the wall is made by separate panel elements (i.e. wall segments connected with lintels and parapets), or when the ratio between the area of the opening and the area of the wall panel exceeds 0,50;

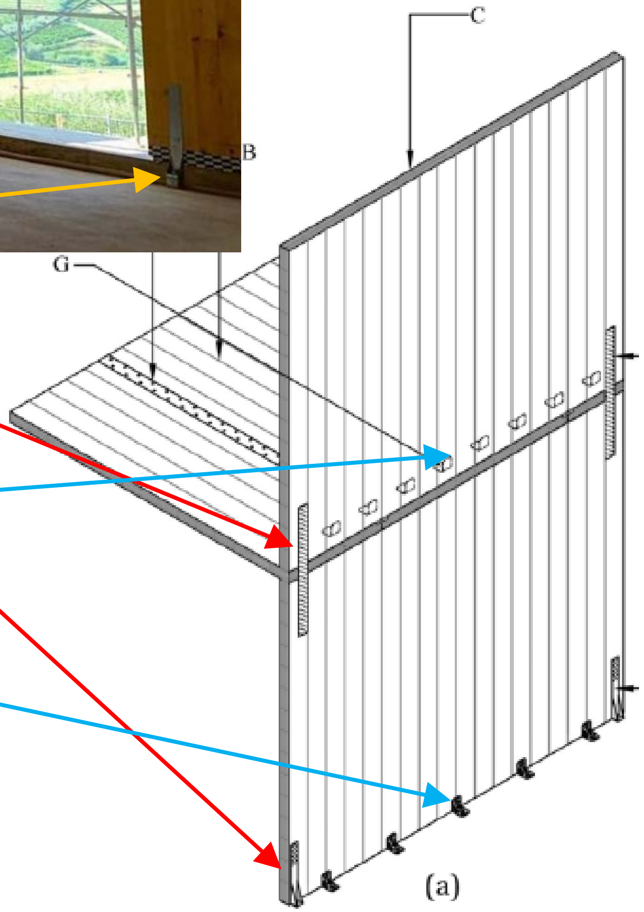
d) **shear connectors** (shear plates, angle brackets, anchoring bolts, nails and screws, etc.) should be **distributed uniformly along the wall width** (Figure 13.3);

e) shear connectors and anchoring connections against overturning should be **fixed to the CLT panels using metal fasteners** such as nails and screws, and to the foundation using elements such as anchor bolts;

f) connections should comply with EN 1998-1-1:2024, Annex G.

NOTE EN 1995-1-1:2024, Annex G, gives guidance for the design of 2D- and 3D-connectors.

(4) **All walls** should be **connected to walls at lower levels**, when present, or to foundations, **with 2D- and 3D-connectors** complying with (3)a) to f).



Main updates – Capacity design and overstrength factor

What are the dissipative zones in a CLT building?

Test of a 3-storey CLT building in Tsukuba, Japan (courtesy of Prof. Ceccotti – copyright CNR IVALSA):

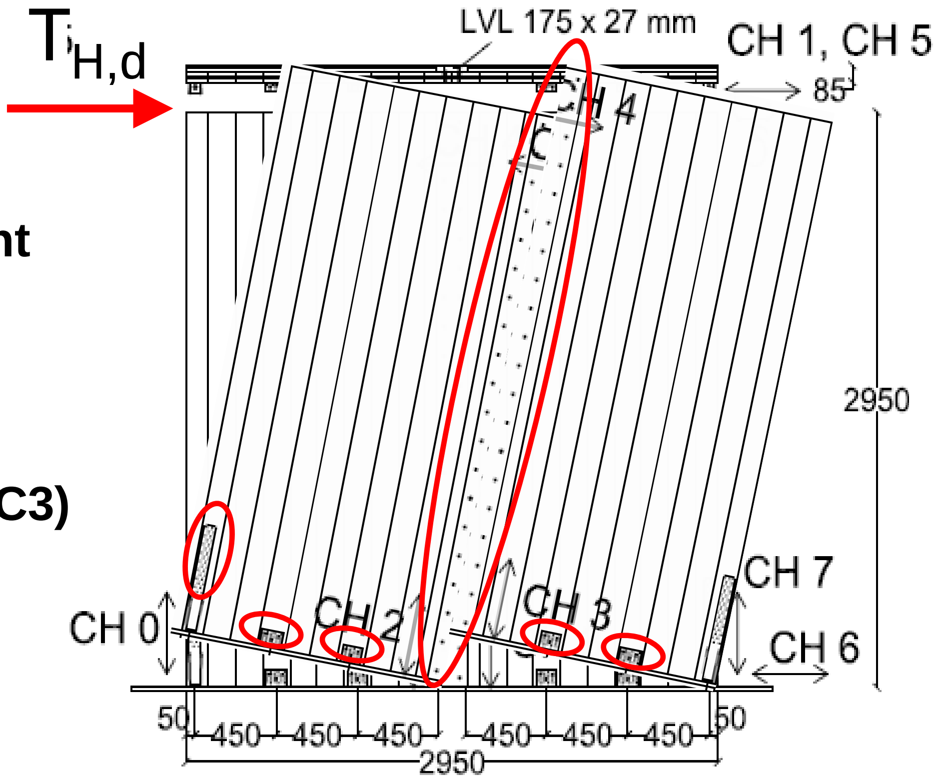
Dissipative mechanism: rocking of wall panels

(courtesy of Prof. Ceccotti – copyright CNR-Ivalsa)

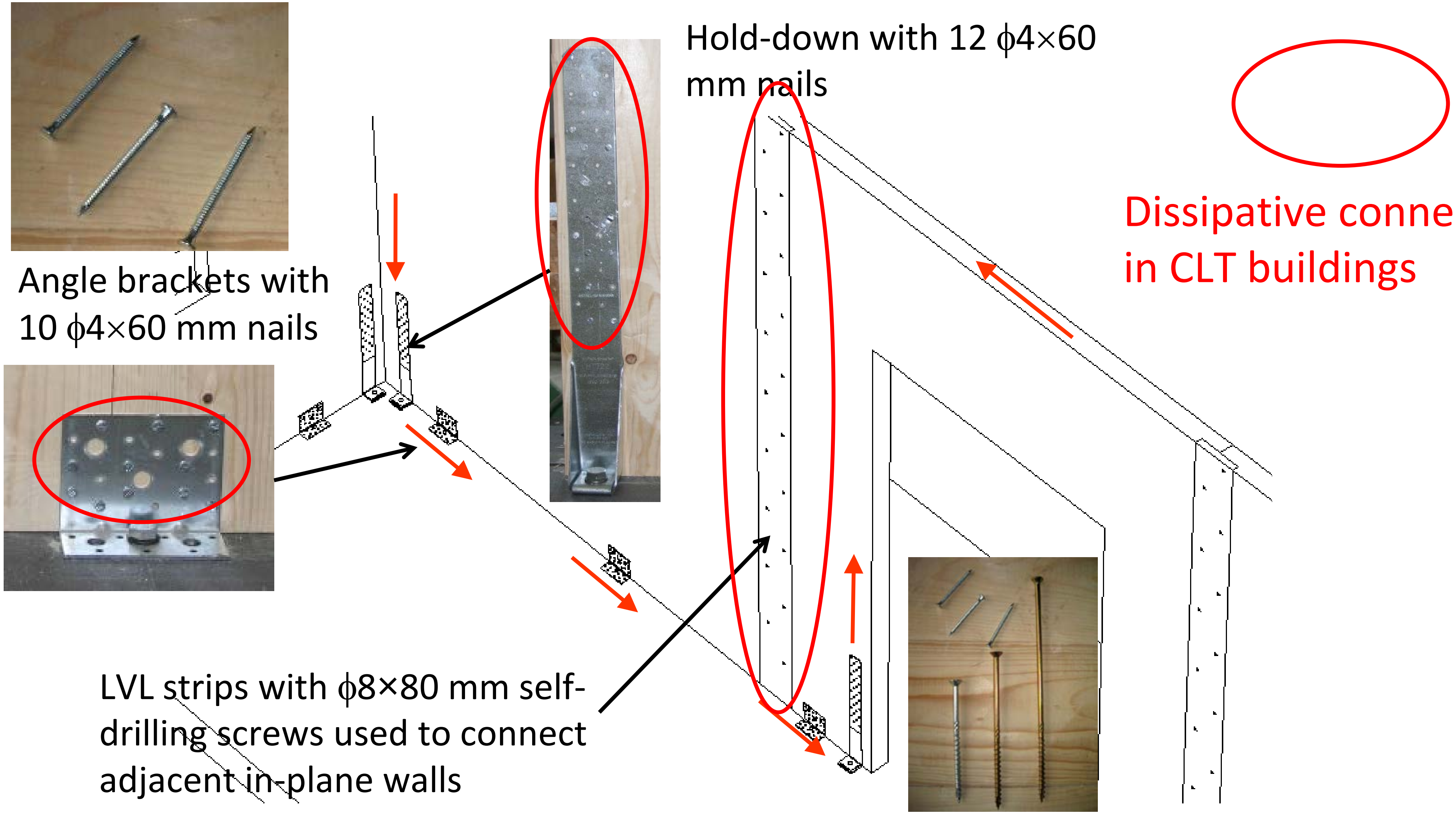
Dissipative mechanism of CLT buildings: rocking of wall panels

To enable the rocking of the wall panels, plasticization of the following connections must be attained:

- **Screw connection between adjacent in-plane wall panels (DC3)**
- **Nailed connections between metal connectors** (hold-downs, angle brackets) **and wall panels (DC2 & DC3)**



Main updates – Capacity design and overstrength factor

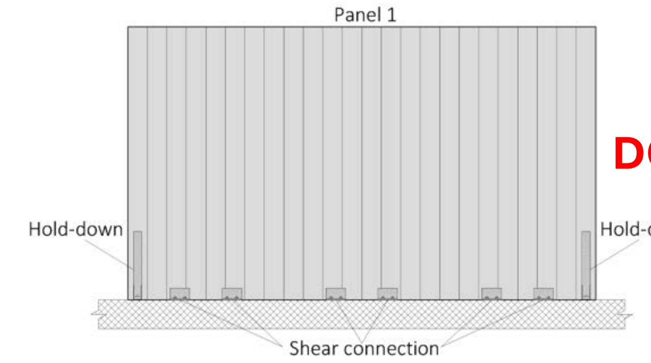


Main updates – Capacity design and overstrength factor

Wall panels in CLT buildings: monolithic (DC2) vs segmented (DC3)

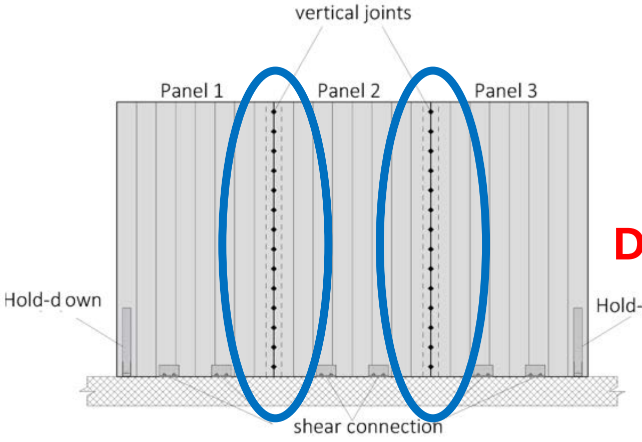
(5) **Wall panels** should **extend from one storey**, foundation included **to the next one**. Along their width, they should be **either made of a single element** ('monolithic wall', Figure 13.3(a)) **or composed of more than one panel**, which should measure in width not less than 0,25*hs* where *hs* is the interstorey height. Each of these panels should be connected to the other panels by means of vertical joints made with metal fasteners such as screws or nails ('segmented wall', Figure 13.3(b)).

Dissipative connections in DC3: the panel-to-panel screw connection



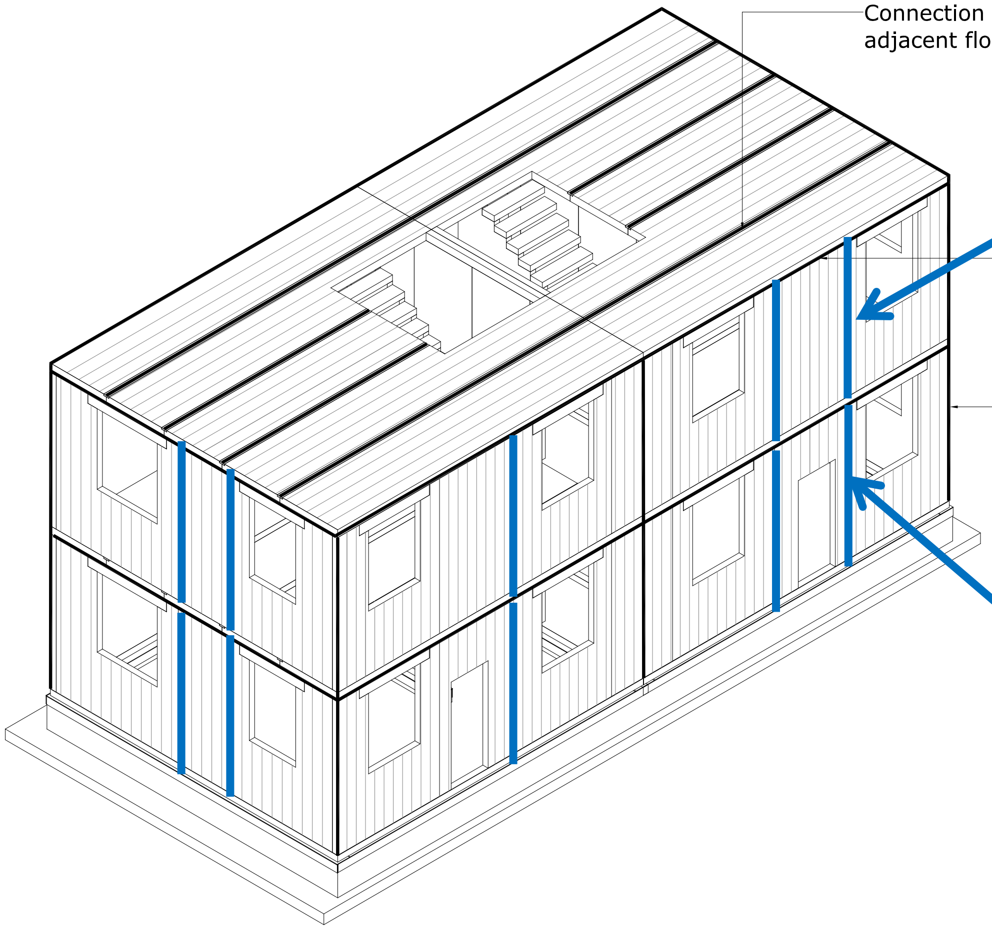
a) Single panel (Monolithic)

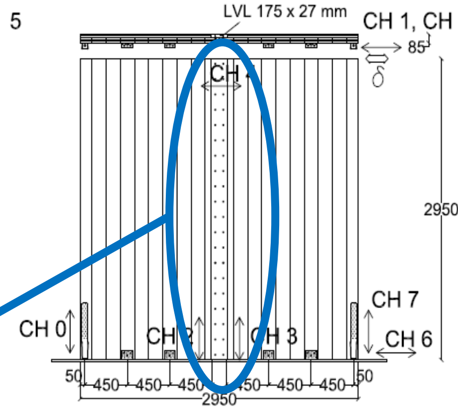
DC2



b) Multi-panel

DC3

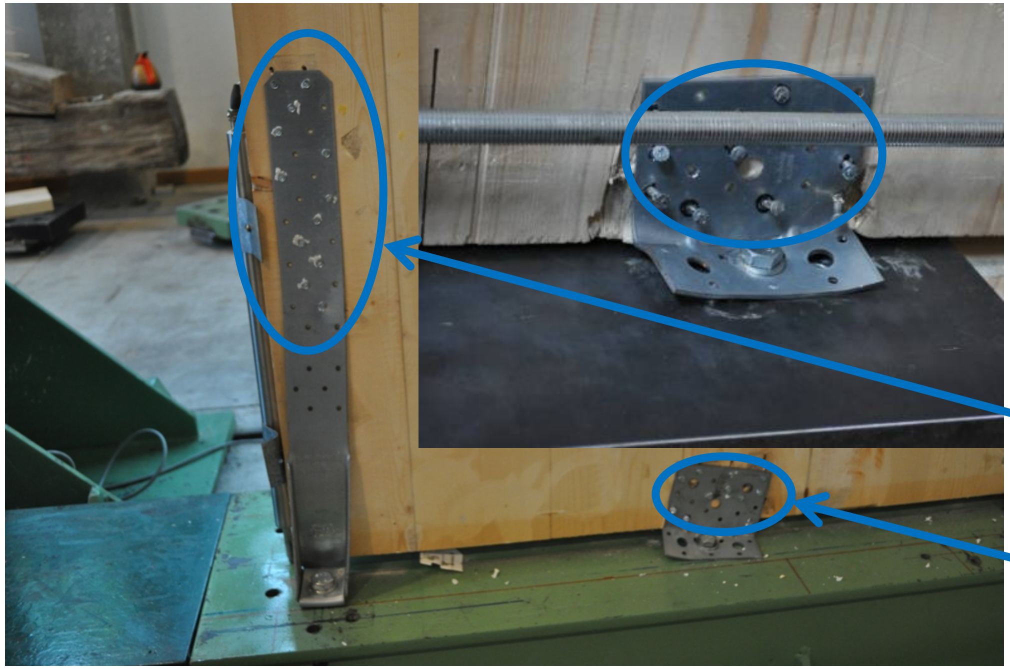


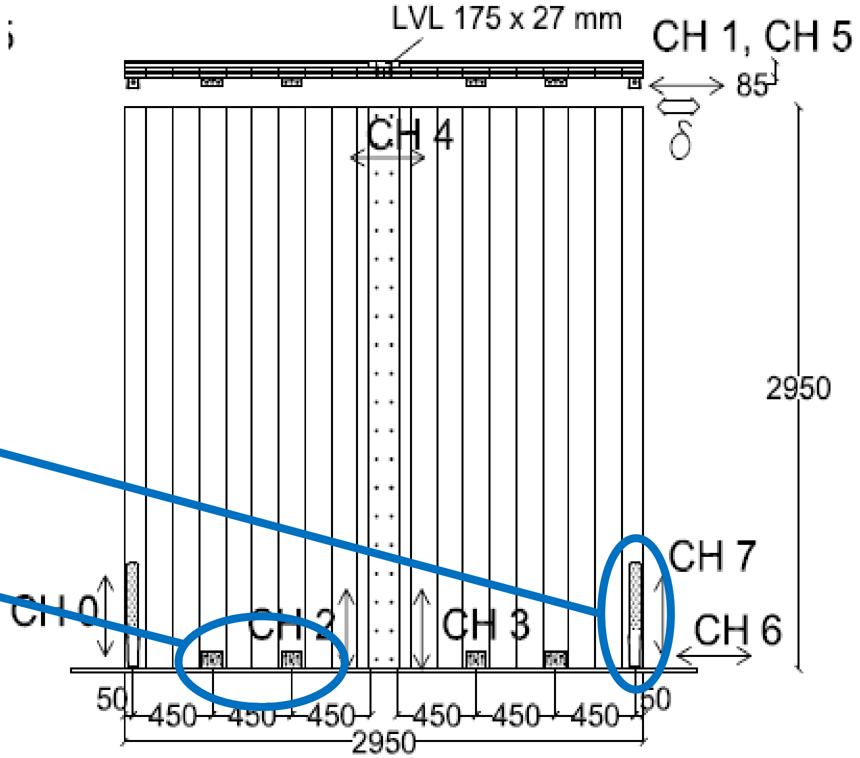


Main updates – Capacity design and overstrength factor

Dissipative zones (DC2 and DC3):

- **Nailed connections between angle brackets (shear connections) and wall panels;**
- **Nailed connections between hold-downs (tensile connections) and wall panels.**





Main updates – Capacity design and overstrength factor

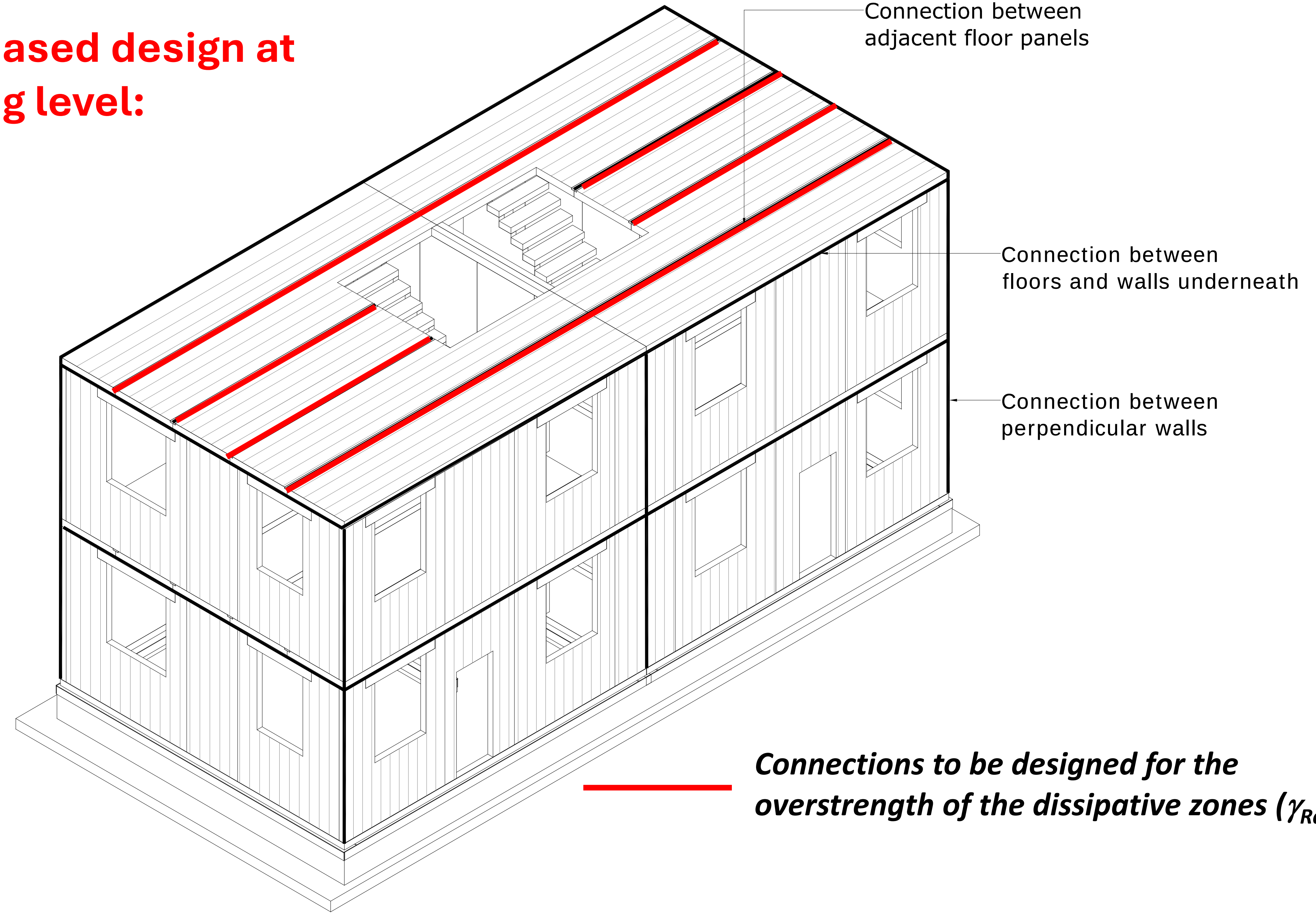
Capacity based design at the building level:

In order to ensure the development of cyclic yielding in the dissipative zones, **all other structural members and connections shall be designed with sufficient overstrength** so as to avoid anticipated brittle failure. This overstrength requirement applies especially to:

- **connections between adjacent floor panels** in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;

Main updates – Capacity design and overstrength factor

Capacity based design at the building level:



Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.3)

Main updates – Capacity design and overstrength factor

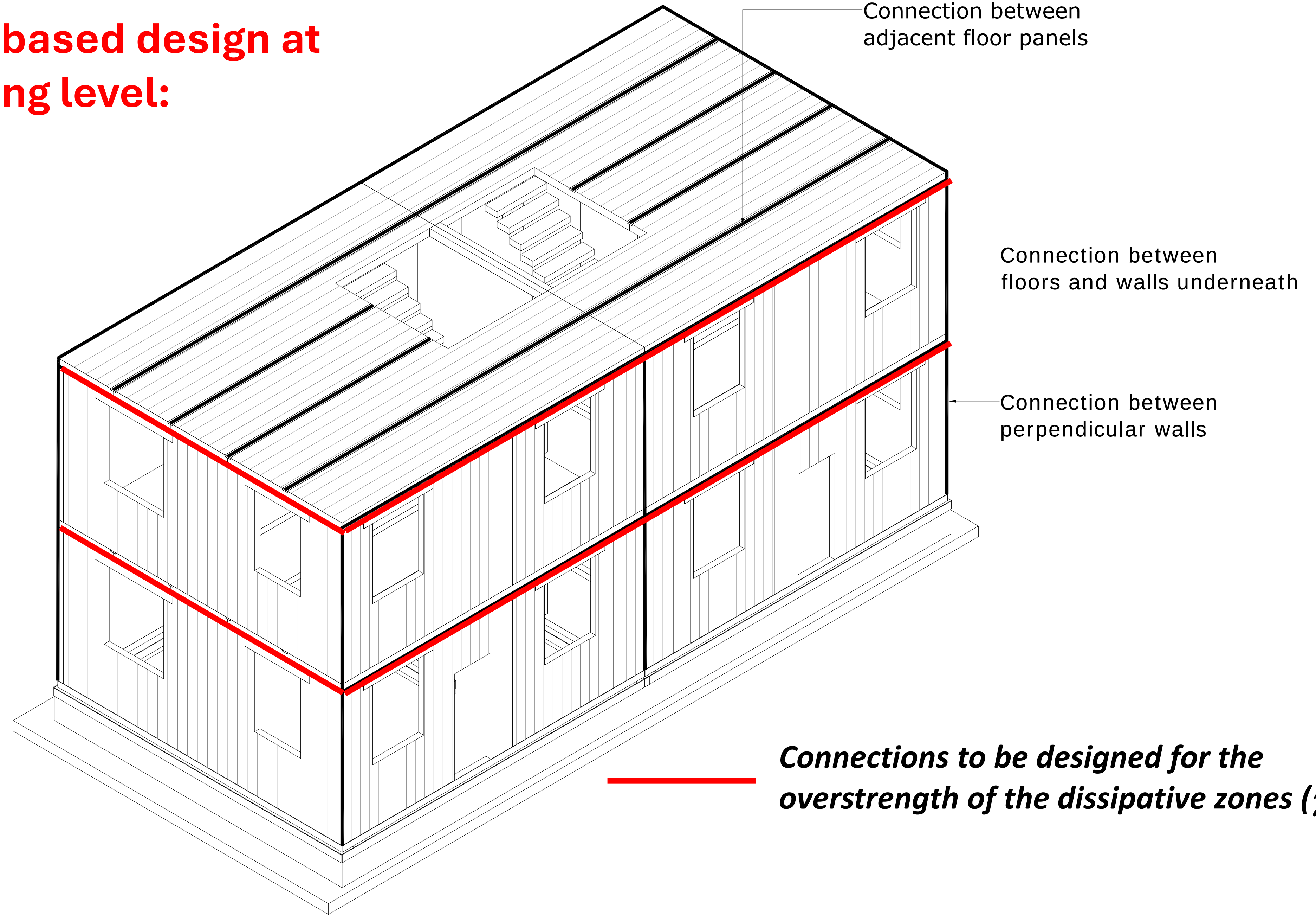
Capacity based design at the building level:

In order to ensure the development of cyclic yielding in the dissipative zones, **all other structural members and connections shall be designed with sufficient overstrength** so as to avoid anticipated brittle failure. This overstrength requirement applies especially to:

- **connections between adjacent floor panels** in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;
- **connections between floors and walls underneath** thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;

Main updates – Capacity design and overstrength factor

Capacity based design at the building level:



Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.3)

Main updates – Capacity design and overstrength factor

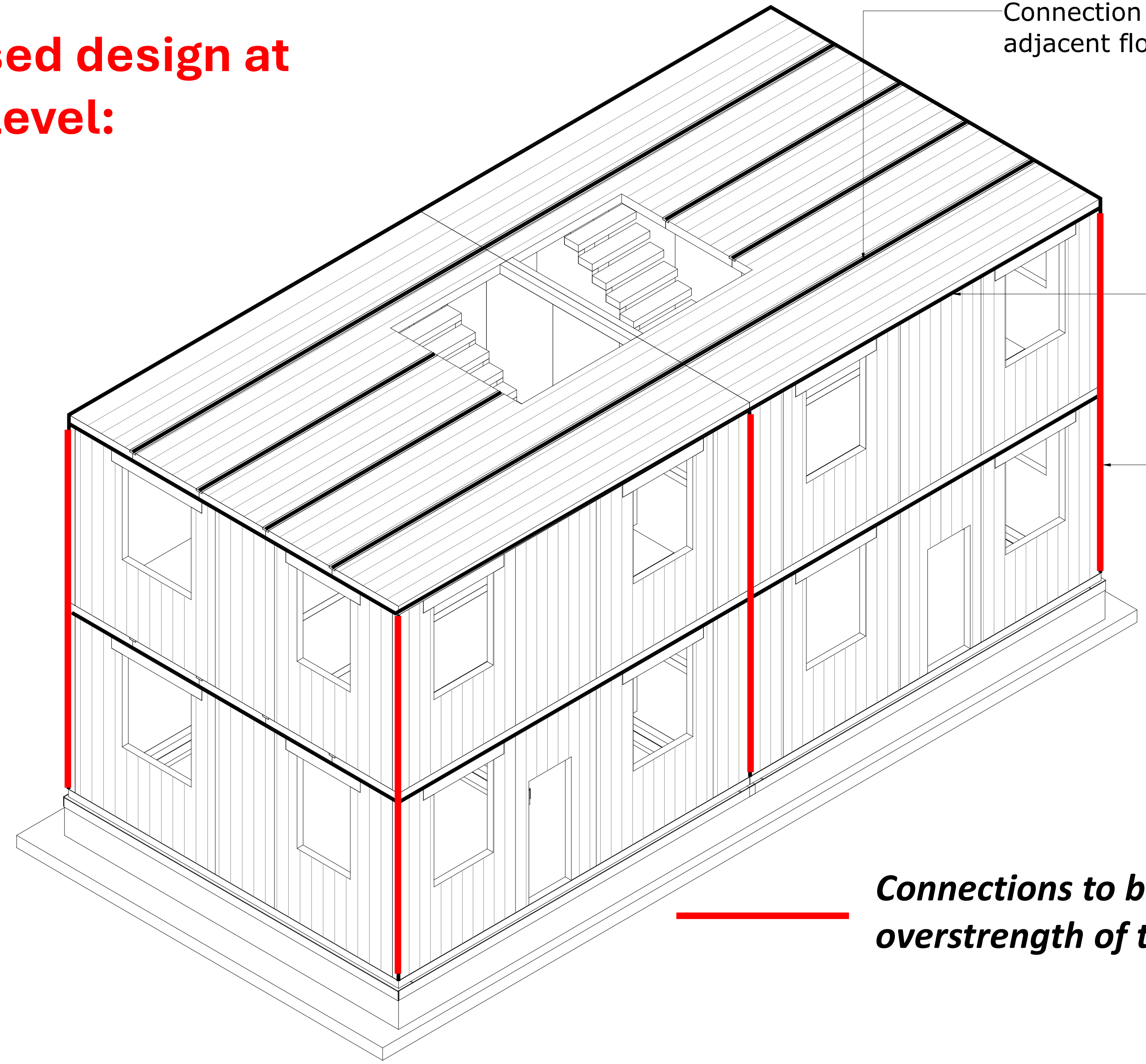
Capacity based design at the building level:

In order to ensure the development of cyclic yielding in the dissipative zones, **all other structural members and connections shall be designed with sufficient overstrength** so as to avoid anticipated brittle failure. This overstrength requirement applies especially to:

- **connections between adjacent floor panels** in order to limit at the greater possible extent the relative slip and to assure a rigid in-plane behaviour;
- **connections between floors and walls underneath** thus assuring that at each storey there is a rigid floor to which the walls are rigidly connected;
- **connections between perpendicular walls**, particularly at the building corners, so that the stability of the walls themself and of the structural box is always assured;

Main updates – Capacity design and overstrength factor

Capacity based design at the building level:



Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.3)

Main updates – Capacity design and overstrength factor

Capacity based design at the building and connection level:

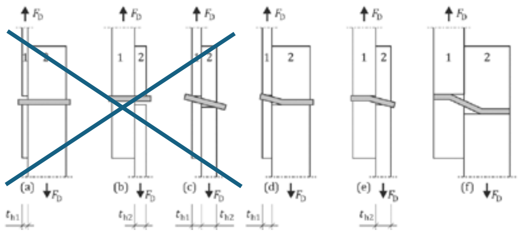
The overstrength must be applied also to:

- **Wall panels under in-plane vertical action due to the earthquake** and floor panels under diaphragm action due to the earthquake (γ_{Rd}=1.6);
- **Metal parts of hold-down and angle bracket connections** to avoid brittle tensile or shear failures (γ_{Rd}=1.6);
- **Connection of holddown and angle bracket to the foundation** or to lower wall panels (γ_{Rd}=1.6).

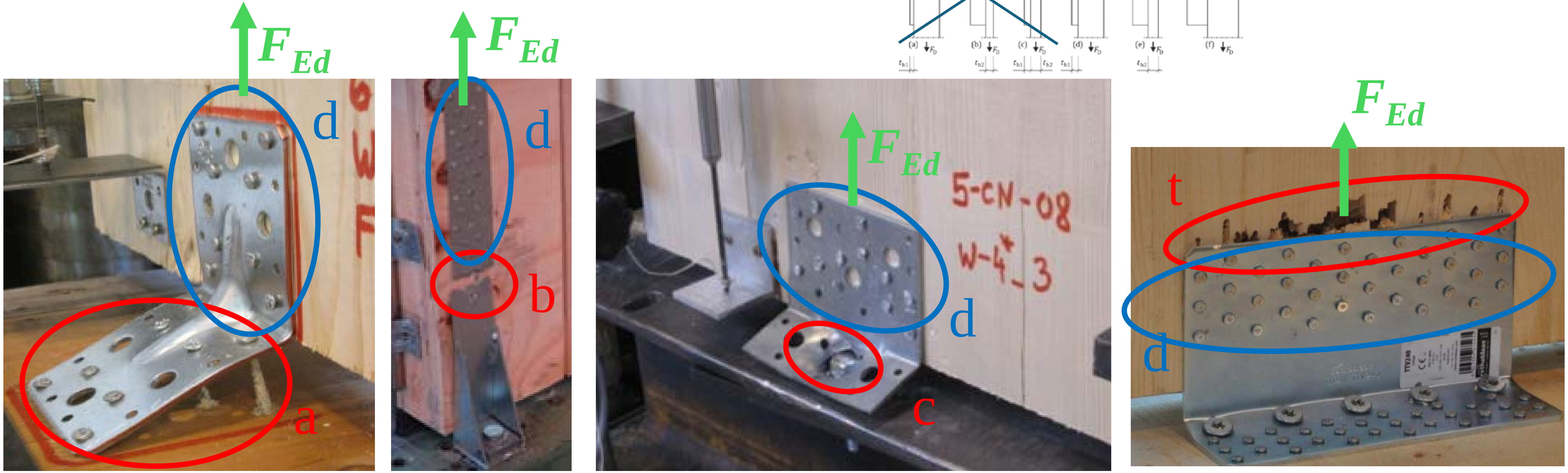


Main updates – Capacity design and overstrength factor

Capacity based design at the connection level:



d: ductile connection



$$F_{D,k} = \min \begin{cases} f_{h,1,k}\, t_{h1}\, d \\ f_{h,2,k}\, t_{h2}\, d \\ \frac{f_{h,1,k}\, t_{h1}\, d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\frac{t_{h2}}{t_{h1}}+\left(\frac{t_{h2}}{t_{h1}}\right)^2\right]+\beta^3\left(\frac{t_{h2}}{t_{h1}}\right)^2}-\beta\left(1+\frac{t_{h2}}{t_{h1}}\right)\right] \\ 1{,}05\,\frac{f_{h,1,k}\, t_{h1}\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h1}^2}}-\beta\right] \\ 1{,}05\,\frac{f_{h,1,k}\, t_{h2}\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h2}^2}}-\beta\right] \\ 1{,}15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y,k}\, f_{h,1,k}\, d} \end{cases}$$

1,6

0,8

$$F_{Ed} = \frac{\gamma_{Rd}}{k_{\deg}} \cdot F_{Rd,\text{nails in shear }(d)} \le \begin{cases} F_{Rd,\text{withdrawal screws }(a)} \\ F_{Rd,\text{metal plate in tension }(b)} \\ F_{Rd,\text{anchor bolt pull-through }(c)} \\ F_{Rd,\text{timber fracture in tension }(t)} \end{cases}$$

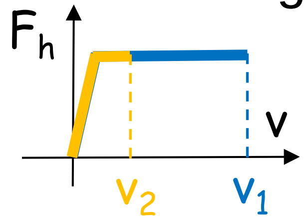
Main updates – Capacity design and overstrength factor

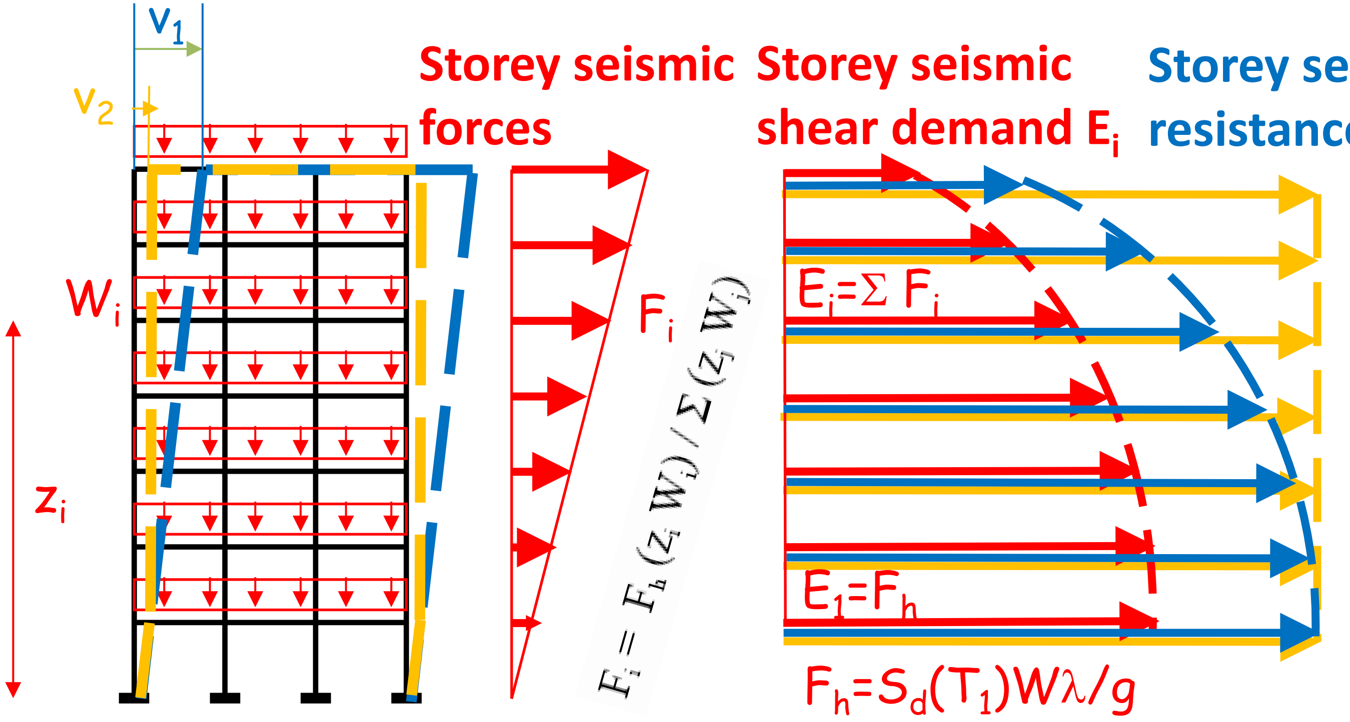
Capacity based design at the building level:

(9) The maximum storey overstrength ratio max($\Omega_{d,i}$) and the minimum storey overstrength ratio Ω_d, with Ω_d given by Formula (13.8), should satisfy Formula (13.14).

$$\frac{\max(\Omega_{d,i})}{\Omega_d} \leq 1{,}25 \qquad (13.14)$$

The seismic resistance of CLT walls should be higher at lower storeys and should decrease at higher storeys proportionally to the decrease of the storey seismic shear, thus leading to the simultaneous plasticization of the ductile connections in order to maximize the energy dissipation of the whole building.





E_i/R_i=constant at each storey i

~~**R_i=constant at each storey i**~~

Main updates – Detailing rules for all structural types

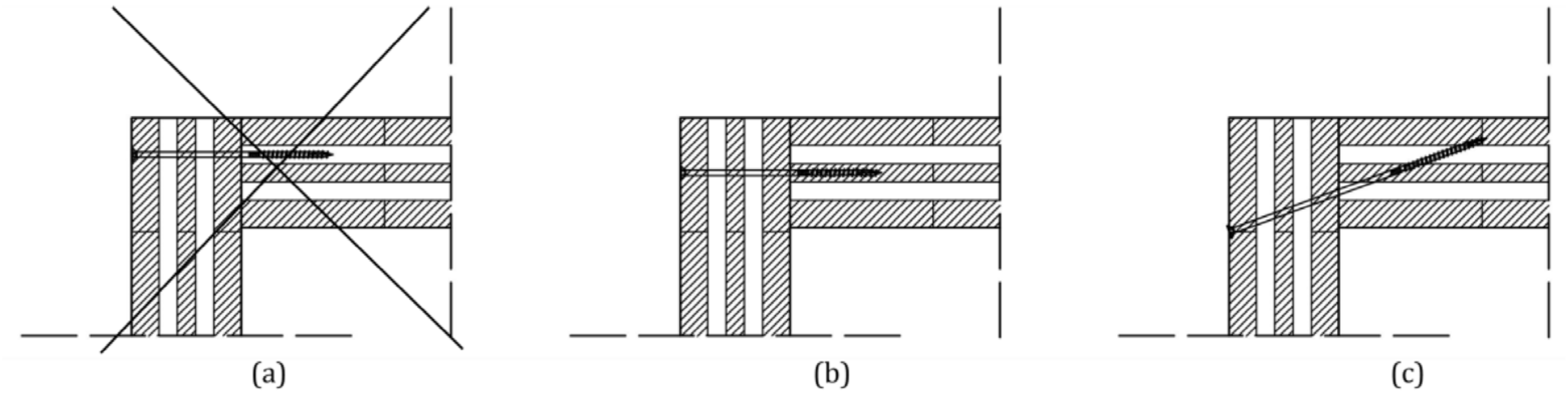
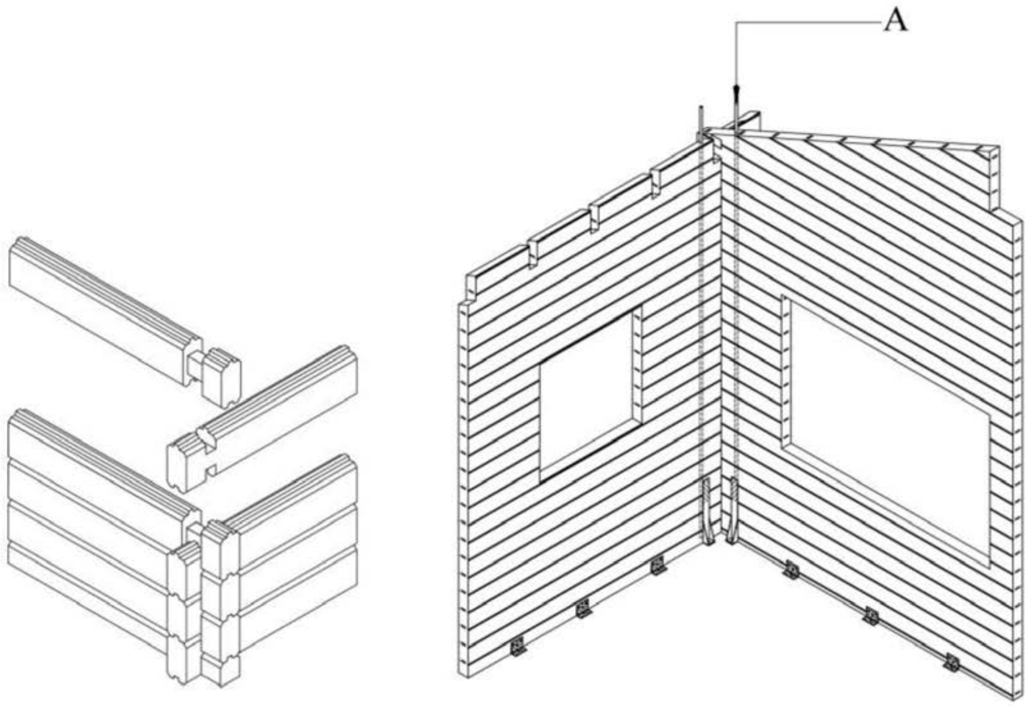


Figure 13.7 — CLT wall-to-wall connection: (a) wrong – screws inserted in layers with grain direction parallel to the screw axis; (b) correct, but difficult to achieve – screws inserted in layers with grain direction perpendicular to the screw axis; (c) correct – screws inserted inclined

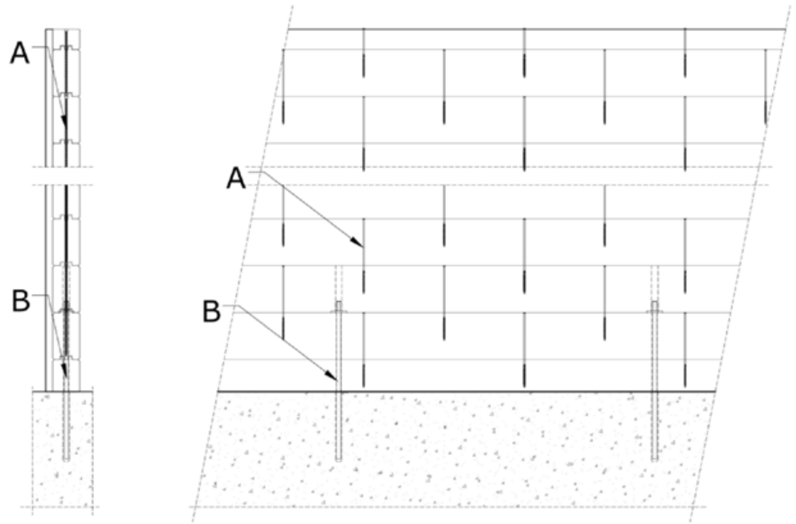
Main updates – Detailing rules for all structural types



Key

A possible steel rods as uplift restraint for timber logs

Figure 13.9 — Typical corner joint and connection details in log structures



Key

A connection between timber logs by means of self-tapping screws

B bolted connection to foundation

Figure 13.11 — Connection between timber logs by means of self-tapping screws (cross-section on the left and side view on the right)

Conclusions and acknowledgements

- The **timber chapter of EN 1998-1-2** represents a **step forward in the design of timber buildings** as it includes, for different structural types:
  - A **complete description** of the structural type with a sketch;
  - **Updated values of the q-factors** for DC2 and DC3 designs;
  - **Capacity design rules** at the global and local levels **and overstrength factors**;
- A **new safety format for design at SD** is also introduced.
- It is **acknowledged**:
  - The contribution of all members of Working Group **CEN/TC250/SC8/WG3**
  - The help of **Dr. Maurizio Follesa** in the preparation of this ppt presentation

THANK YOU VERY MUCH FOR YOUR ATTENTION!

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EC8-2G

Il nuovo standard europeo per la progettazione sismica

Materiali e tipologie costruttive

EN1998-1-2. Strutture in legno

Massimo Fragiacomo, Università degli Studi dell'Aquila

Pavia - 5 Giugno 2025