Second Generation of Eurocode 8

EN 1998-1-2: New buildings

Timber Buildings

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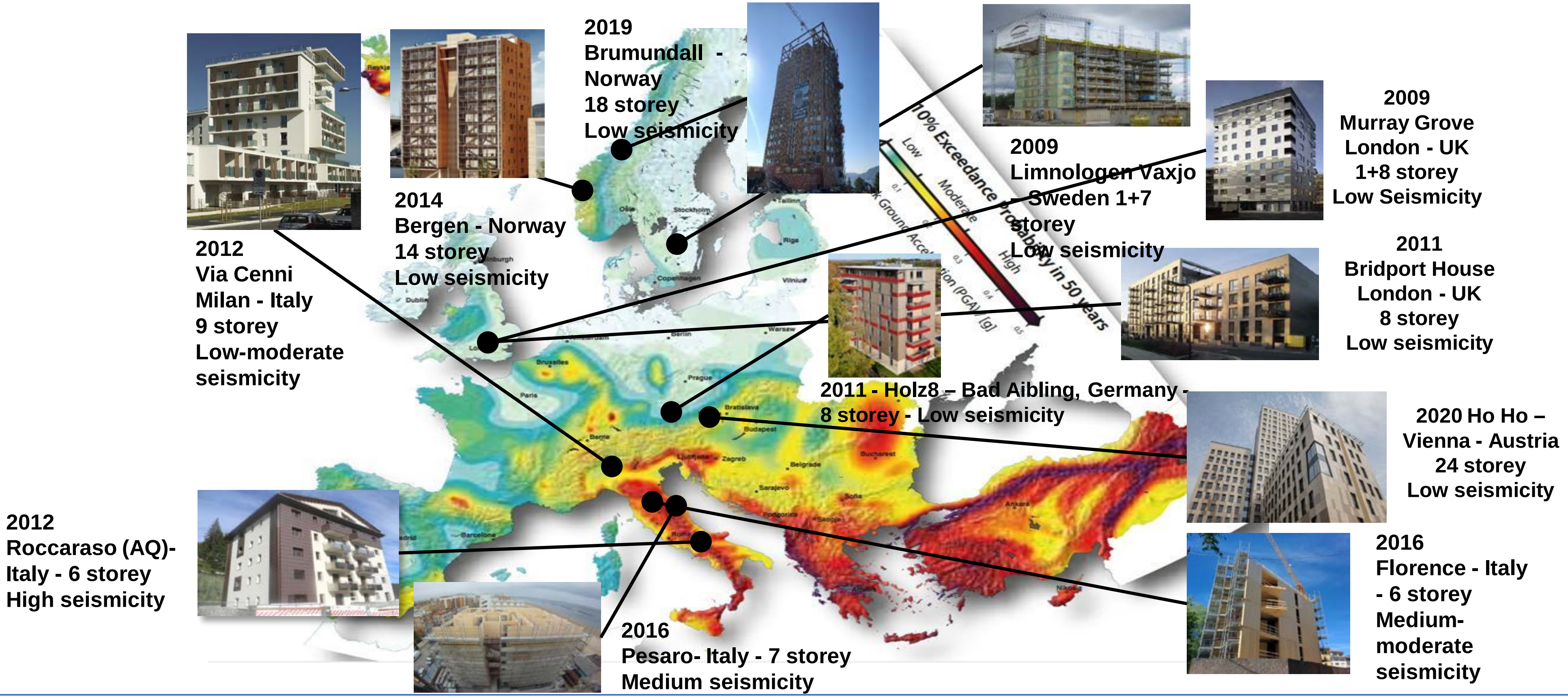
INTRODUCTION

- 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

- **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

- However, **very little** (only 6 pages) or no information at all **in current regulations on seismic design of** new (EN1998-1) and existing (EN1998-3) **timber buildings**, respectively

- **Implementation of extended new rules** reflecting the state-of-the-art **for seismic design of** new (EN1998-1-2) and existing (EN1998-3) **timber buildings, in the second generation of Eurocode 8.** More specifically:
  - **Chapter 13** and **Annex L of EN1998-1-2**: Specific rules for timber buildings (41 + 5 pages)
  - **Section 6.8 of prCEN/TS 1998-1-101** « Characterisation and qualification of structural components for seismic applications by means of cyclic tests »: Timber buildings (3 pages)
  - **Chapter 10** and **Annex C of EN1998-3**: Specific rules for timber buildings (26 + 3 pages)

THE NEW CHAPTER 13 OF EN1998-1-2 – MAIN UPDATES:

- A - Introduction of new wood-based panels
- B - Revised definition of structural types
- C - New safety format for seismic verifications
- 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

A – INTRODUCTION OF NEW WOOD-BASED PANELS

13.3.2 MATERIAL PROPERTIES

(1) The thickness of **cross laminated timber (CLT)** and glue-laminated timber (glulam or GLT) panels should be not smaller than 60 mm.

….

(4) The sheathing material 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 (GF)** sheathing should comply with EN 15283-2, be at least 12 mm thick and have a characteristic density of at least 1000 kg/m^3.

f) **Densified Veneer Wood** sheathing should comply with EN 61061-3-1 and have a characteristic density of at least 1200 kg/m^3.

….

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

Table 13.1 — Timber structural types and examples of structures

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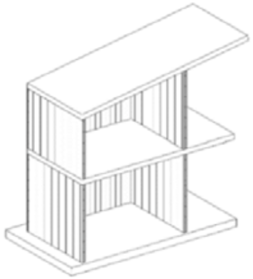
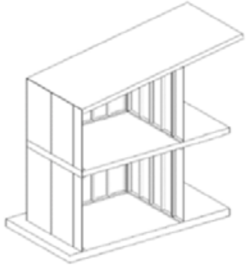
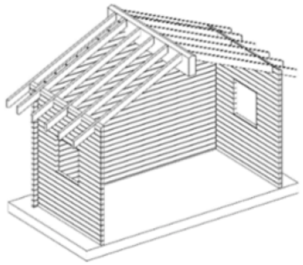
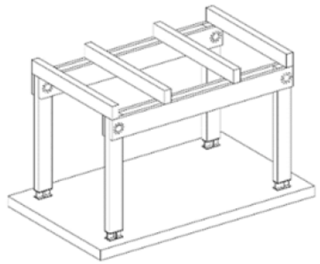
Examples of structural types*	Timber structural types
	a) <u>Cross laminated timber (CLT) structures**</u> CLT structures are those where the primary structure (see **3.1.23**) is composed of shear walls made of cross laminated timber panels according to **13.3.2(1)**. Glulam, LVL or GLVL may be used 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 structure is composed of framed 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**. b1) With fully anchored walls. b2) With non-fully anchored walls.
	c) <u>Log structures</u> Log structures are those where the primary 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**.
	d) <u>Moment-resisting frame structures</u> Moment-resisting frame structures are those where the primary structure is composed of frames made by timber elements with semi-rigid (as defined in **3.1.28**) moment-transmitting joints between the members, achieved with mechanical fasteners. Moment-resisting frames structures should be designed according to **13.10**.

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

CLT structures can fall into either category a) or f) depending on whether the shear walls have heights equal to one inter-storey height (platform frame construction – see **3.1.20) or more (balloon frame construction – see **3.1.2**).

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CLT structures can fall into either category a) or f) depending on whether the shear walls have heights equal to one inter-storey height (platform frame construction – see **3.1.20) or more (balloon frame construction – see **3.1.2**).

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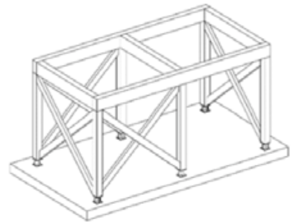
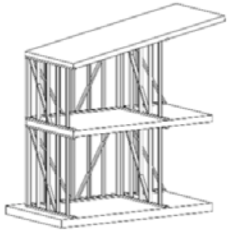
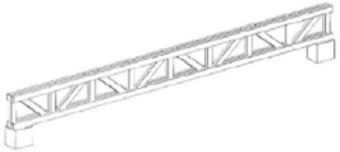
	e) <u>Braced frame structures with dowel-type connections</u> Braced frame structures with dowel-type connections are those consisting of timber columns and beams, where the primary structure is composed of timber diagonal bracings, with all pin-jointed dowel-type connections (as defined in **3.1.9**). Braced frame structures with dowel-type connections should be designed according to **13.11**.
	f) <u>Vertical cantilever structures**</u> Vertical cantilever structures are those where the primary structure is composed of vertically continuous cantilever glulam, LVL, GLVL or CLT walls or columns without any horizontal joints. Vertical cantilever structures should be designed according to **13.12**.
	g) <u>Braced frame structures with carpentry connections and interacting masonry infill</u> Braced frame structures with carpentry connections are those consisting of timber columns and beams, where the primary structure is composed of vertical timber bracing with compression-only carpentry connections and interacting masonry infill. Braced frame structures with carpentry connections and interacting masonry infill should be designed according to **13.13**.

Table 13.1 — Timber structural types and examples of structures

Examples of structural types*	Timber structural types
	h) <u>Braced frame structures with carpentry connections</u> Braced frame structures with carpentry connections are those consisting of timber columns and beams, where the primary structure is composed of vertical timber bracing with compression-only carpentry connections. Braced frame structures with carpentry connections should be designed according to **13.14** as non-dissipative systems.
	i) <u>Two-pin and three-pin arches, three-pin frames and dome structures.</u> Structures composed by two-pin and three-pin timber arches, three-pin timber frames and timber dome structures should be designed as non-dissipative systems.
	j) <u>Large span timber truss portal frame structures.</u> Large span truss portal frame structures are those consisting of timber trusses with semi-rigid moment-transmitting joints between the chords and the columns, forming a moment frame. Large span truss portal frame structures should be designed as non-dissipative systems.

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

CLT structures can fall into either category a) or f) depending on whether the shear walls have heights equal to one inter-storey height (platform frame construction – see **3.1.20) or more (balloon frame construction – see **3.1.2**).

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

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).**

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

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)**

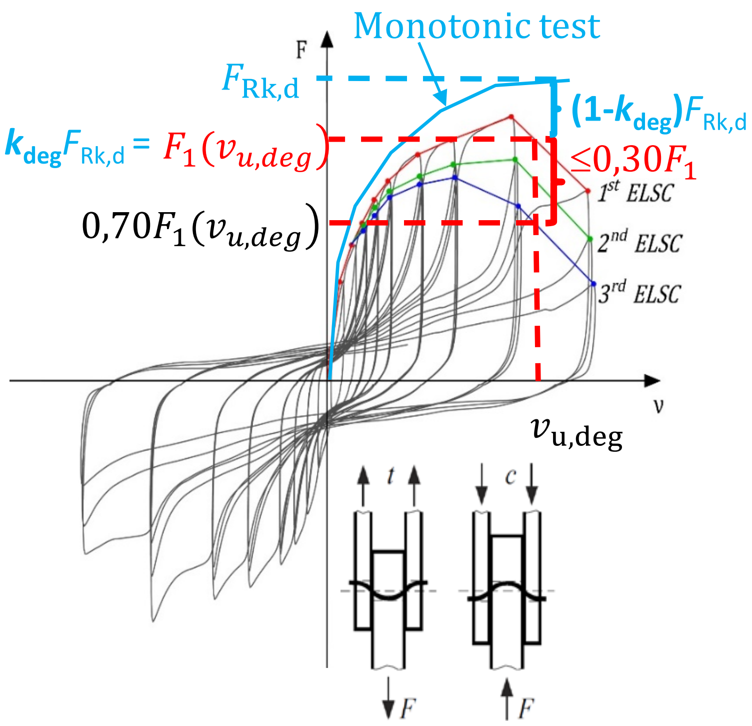
k_{mod} is the modification factor for duration of load and moisture content according to prEN1995-1-1:2021, **5.1.3**, Table 5.1;

$F_{\mathrm{Rk,d}}$ is the characteristic value of the strength of the dissipative zones, according to prEN1995-1-1:2021, **11**;

γ_M is the partial factor for a material property according to prEN1995-1-1:2021, **4.5.2.2**, Table 4.6.

For DC2 and DC3 the values of γ_M are those given in prEN1995-1-1:2021, 4.5.2.2, Table 4.6, for accidental situations (=1), unless the National Annex gives different values for use in a Country.

NOTE 3 prCEN/TS 1998-1-101 can be used to determine the mechanical properties of a dissipative zone such as the strength reduction factor k_{deg} and the ductility μ in accordance with EN 12512.



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

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

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

13.2.2 SAFETY VERIFICATIONS

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

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

where:

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

k_{mod} is the modification factor for duration of load and moisture content according to prEN1995-1-1:2021, **5.1.3**, Table 5.1;

$F_{\mathrm{Rk,b}}$ is the characteristic value of the strength of the non-dissipative components, according to prEN1995-1-1:2021, **8**, **11** and **12**;

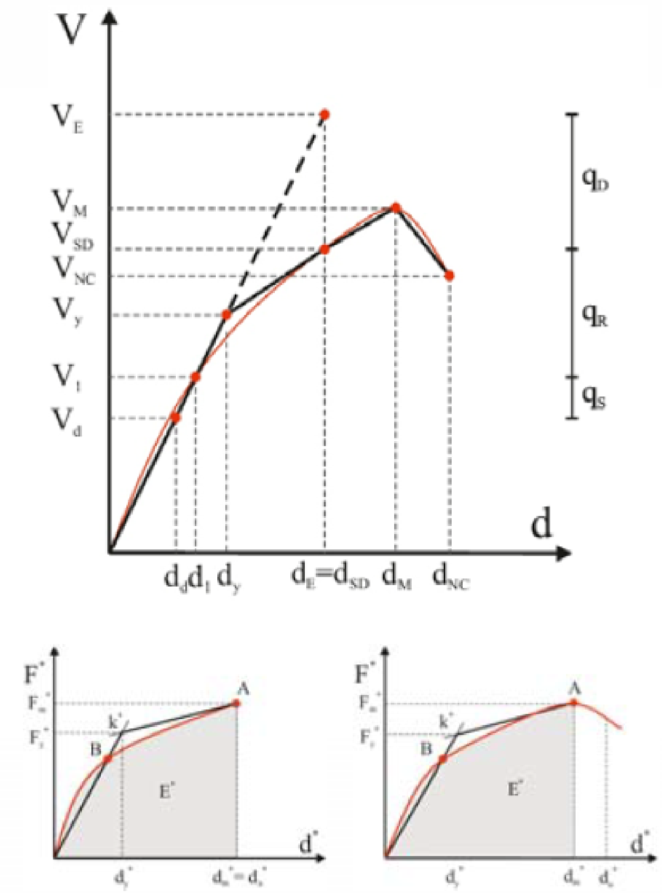
γ_{M} is the partial factor for a material property according to prEN1995-1-1:2021, **4.5.2.2**, Table 4.6.

NOTE The values of γ_{M} are:

- **For DC1, those given by prEN1995-1-1:2021, 4.5.2.2, Table 4.6, for persistent and transient situations (>1)**
- **For DC2 and DC3, those given in prEN1995-1-1:2021, 4.5.2.2, Table 4.6, for accidental situations (=1),**

unless the National Annex gives different values for use in a Country,

D – NEW DEFINITION OF BEHAVIOUR FACTOR q ACCORDING TO prEN1998-1-1



(a) without degradation (b) with degradation

$$q = q_S \cdot q_R \cdot q_D = \underbrace{\frac{V_1}{V_d}}_{(1)} \cdot \underbrace{\frac{V_{SD}}{V_1} \cdot \frac{V_E}{V_{SD}}}_{(2)}$$

(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

D – NEW DEFINITION OF BEHAVIOUR FACTOR q ACCORDING TO prEN1998-1-1

13.4.2 Behaviour factors

Table 13.2 — Default values of the behaviour factors *q* for buildings regular in elevation with maximum values of the 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	4,0	1,5	1,2	1,3	2,3	1,4	1,5	3,2
b) Framed wall structures								
b1) With fully anchored walls	5,0	1,5	1,5	1,1	2,5	2,4	1,1	4,0
b2) With non-fully anchored walls	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
c) Log structures	4,0	1,5	1,2	1,1	2,0	N/A	N/A	N/A
d) Moment-resisting frames structures								
d1) Single storey	4,0	1,5	1,3	1,1	2,1	2,0	1,1	3,3
d2) Multi-storey, one-bay	4,0	1,5	1,3	1,2	2,3	2,0	1,2	3,6
d3) Multi-storey, multi-bay	4,0	1,5	1,3	1,3	2,5	2,0	1,3	3,9

N/A: Not Applicable

NOTE Conditions to design structural types h), i) and j) for S_δ greater than 3 m/s² are given in **13.4.4(3)**.

Table 13.2 — Default values of the behaviour factors *q* for buildings regular in elevation with maximum values of the 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
e) Braced frame structures with dowel-type connections	4,0	1,5	1,3	1,0	2,0	N/A	N/A	N/A
f) Vertical cantilever structures	4,0	1,5	1,2	1,3	2,3	N/A	N/A	N/A
g) Braced frame structures with carpentry connections and interacting masonry infills	4,0	1,5	1,3	1,1	2,0	N/A	N/A	N/A
h) Braced frame structures with carpentry connections	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
i) Two-pin and three-pin timber arches, three-pin timber frames and timber dome structures	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A
j) Large span timber truss portal frame structures.	3,0	1,5	N/A	N/A	N/A	N/A	N/A	N/A

N/A: Not Applicable

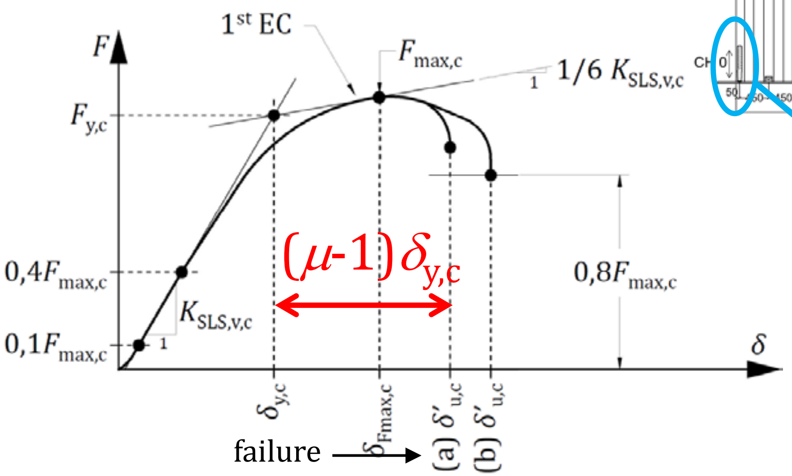
NOTE Conditions to design structural types h), i) and j) for S_δ greater than 3 m/s² are given in **13.4.4(3)**.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(4) The dissipative zones, specified for each structural type designed in DC2 and DC3, should attain a ductility not smaller than the values in Table 13.3 in cyclic tests performed according to EN 12512.

(5) For each structural type designed in DC2 and DC3, **(4)** should be fulfilled by at least one type of dissipative sub-assembly/joint/2D- or 3D-connector/connection in Table 13.3.

$$\boxed{\mu=\delta_{u,c}/\delta_{y,c}}$$



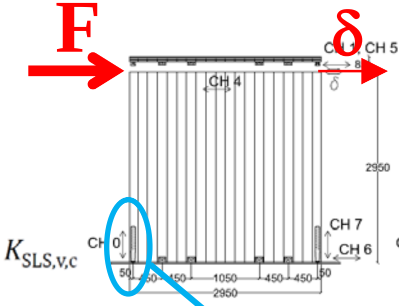


Table 13.3 — Minimum required ductility μ as defined in EN 12512 of dissipative zones tested accordingly

Structural type	Dissipative sub-assembly/joint/2D-or 3D connector/connection	Type of ductility	μ DC2	μ DC3
a) Cross laminated timber structures	Shear wall*	Displacement	1,5	2,5
	Hold-downs, tie-downs, foundation tie-downs, angle brackets, shear plates	Displacement	1,5	1,5
	Screwed wall panel-to-panel joints	Displacement	-	5,5
b) Framed wall structures	Shear wall*	Displacement	2,2	3,5
	Connection (nail/screw/staple)	Displacement	3,5	5,5
c) Log structures	Shear wall*	Displacement	1,4	-
d) Moment-resisting frames	Portal Frame*	Displacement	2,0	3,0
	Beam-column joint	Rotational	4,0	7,0
e) Braced frame structures with dowel-type connections	Braced Frame*	Displacement	1,4	-
f) Vertical cantilever structures	Shear wall*	Displacement	2,0	-
g) Braced frame structures with carpentry connections and interacting masonry infill	Shear wall*	Displacement	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.

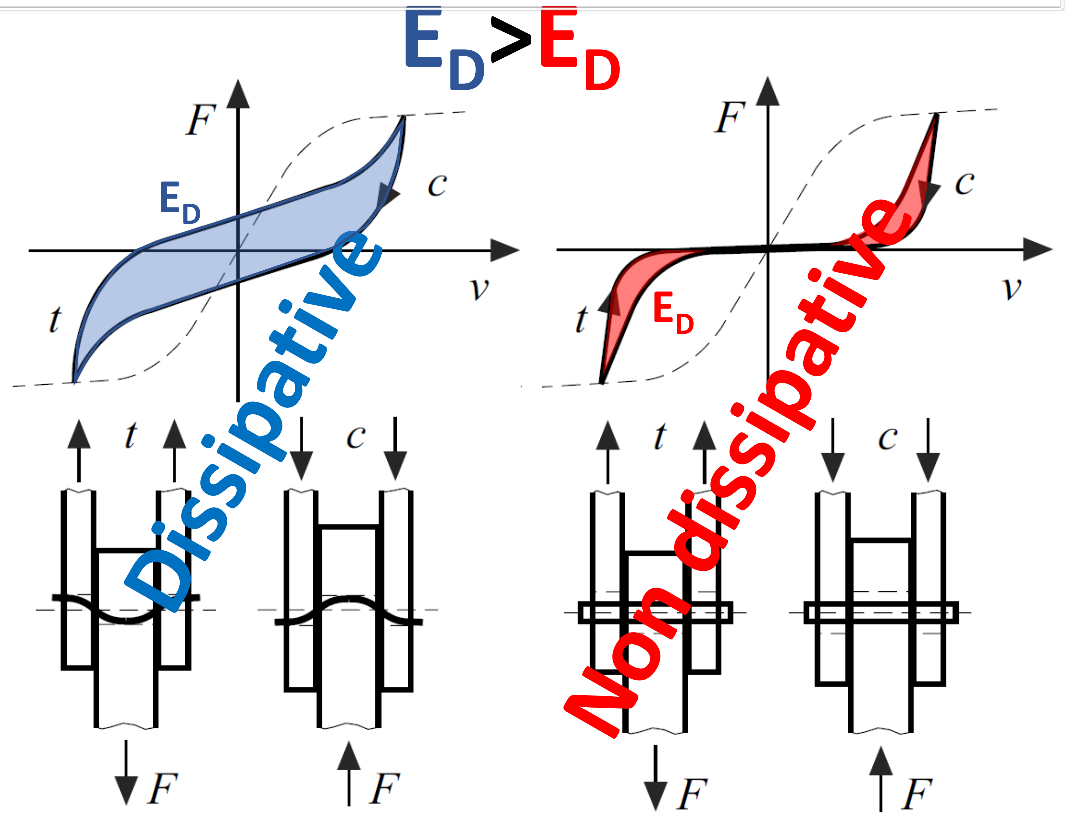
E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

13.2 Basis of Design

13.2.1 Design concepts

(5) For dissipative structural behaviour (design to DC2 and DC3), **dissipative zones should be located in joints and connections or** outside the joints and connections **in purposely developed energy dissipation systems.**

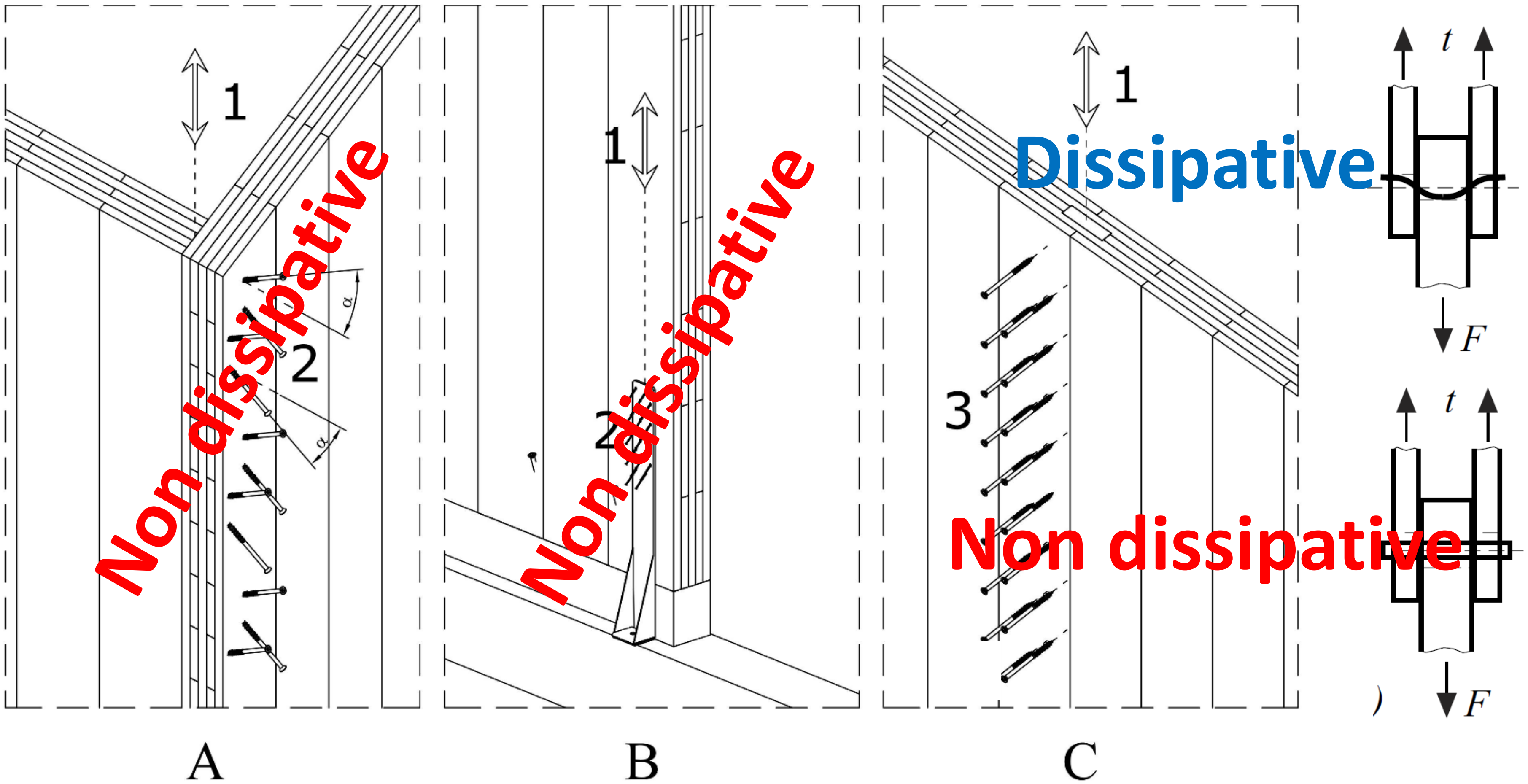
(6) If dissipative zones are located in joints and connections, **the energy dissipation should take place by flexural yielding of metal fasteners,** whereas the **timber members themselves should be designed to remain elastic.**



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

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES



KEY: A and B: Connections 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: Connections inserted perpendicular with respect to the direction of the shear force, transferring most of the load via shear resistance, which may be considered dissipative.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(7) For each structural type designed in DC2 and DC3, the **default values of the behaviour factor *q* may be taken from Table 13.2** in all of the following cases a) to e), **provided that (8) and (9) are also satis**fied:………

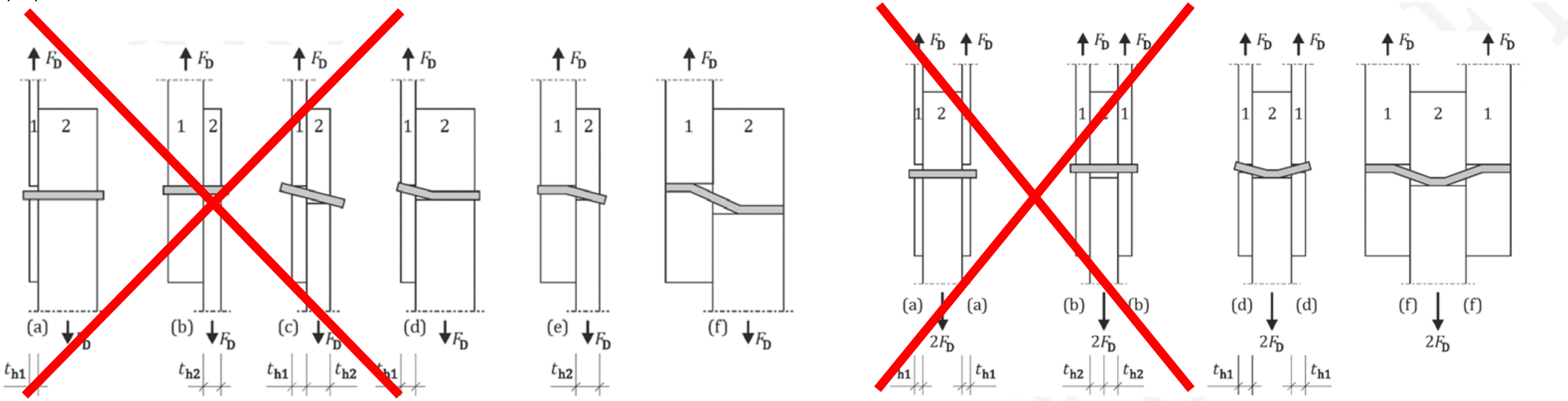
(8) **Failure modes (a), (b) and (c) for fasteners in single shear** as given in prEN1995-1-1:2021, **11.3.2(1)**, **and failure modes (a) and (b) for fasteners in double shear** as given in prEN1995-1-1:2021, **11.3.2(2)**, **should be avoided in the dissipative zones** by satisfying Formula (13.5). Failure modes (a) and (b) for fasteners in multiple shear as given in prEN1995-1-1:2021, **11.3.5**, should be avoided in the dissipative zones by satisfying Formula (13.5).

$$1{,}2\, F_{\mathrm{v,Rk,d}} \leq F_{\mathrm{v,Rk,nd}} \tag{13.5}$$

where:

$F_{\mathrm{v,Rk,d}}$ is the characteristic strength of the selected ductile failure mode providing energy dissipation, according to prEN1995-1-1:2021, **11.3.2**;

$F_{\mathrm{v,Rk,nd}}$ is the characteristic strength of the less ductile failure mode, according to prEN1995-1-1:2021, **11.3.2**;



E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

From prEN1995-1-1:2021 (currently under discussion in CEN/TC250/SC5):

4.5.2 Design value of a resistance

(1) The design resistance R_d (load-carrying capacity) shall be calculated from Formula 4.7:

$$R_d = k_{mod} \frac{R_k}{\gamma_R} \quad (4.7)$$

where

k_{mod} is the modification factor accounting for the effect of the duration of load and moisture content, see 5.1.3;

R_k is the characteristic resistance (load-carrying capacity);

γ_R is the partial factor for resistance.

11.2.3 Lateral resistance of a fastener per shear plane

11.2.3.1 General

(1) The characteristic lateral resistance per shear plane $F_{v,d}$ of a single fastener should be determined by:

$$F_{v,k} = F_{D,k} + F_{rp,k} \quad (11.9)$$

where

$F_{D,k}$ is the characteristic dowel-effect contribution per shear plane according to 11.3.2;

$F_{rp,k}$ is the characteristic rope-effect contribution determined according to 11.2.3.6;

(2) The lateral design load-carrying capacity per staple per shear plane should be considered as equivalent to that of two nails with the staple leg diameter.

(3) The characteristic rope-effect contribution per shear plane per fastener $F_{rp,k}$ should be taken from:

$$F_{rp,,k} = min \begin{cases} k_{rp,1} F_{ax,t,k} \\ k_{rp,2} F_{D,k} \end{cases} \quad (11.16)$$

with

$$F_{ax,t,k} = min \begin{cases} F_{pull,k} \\ F_{w,k} \\ F_{t,k} \end{cases} \quad (11.17)$$

where

$k_{rp,1}$ is the factor for the rope effect, see Table 11.9;

$k_{rp,2}$ is the limitation factor for the rope effect, see Table 11.10;

$F_{D,k}$ is the design dowel effect contribution determined with 11.2.3.2;

$F_{ax,k}$ is the characteristic axial capacity of the fasteners according to 11.2.2.1(1);

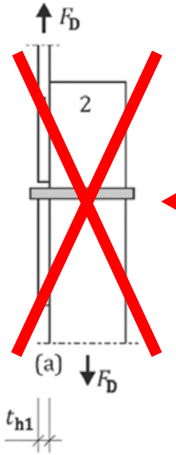
$F_{pull,k}$ is the characteristic head pull-through resistance, see 11.2.2;

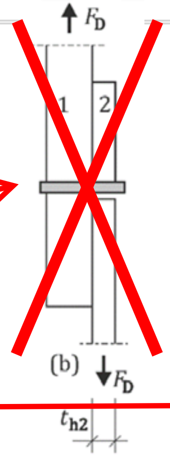
$F_{w,k}$ is the characteristic withdrawal resistance, see 11.2.2.3;

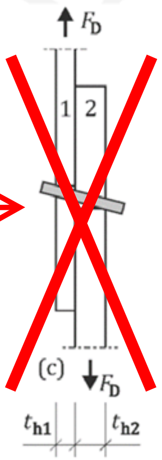
$F_{t,k}$ is the characteristic tensile resistance, see 11.2.2.4.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

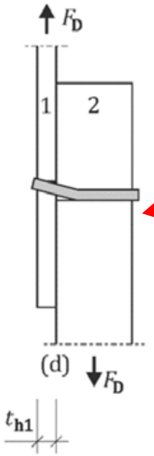
From prEN1995-1-1:2021 (currently under discussion in CEN/TC250/SC5):

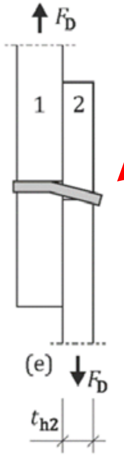


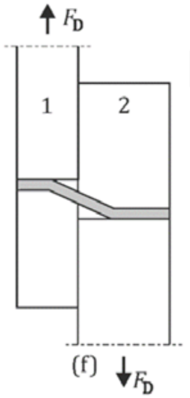




$$F_{D,k} = \min \begin{cases} f_{h,1,k}\, t_{h1}\, d & (a) \\ f_{h,2,k}\, t_{h2}\, d & (b) \\ \dfrac{f_{h,1,k}\, t_{h1}\, d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\dfrac{t_{h2}}{t_{h1}}+\left(\dfrac{t_{h2}}{t_{h1}}\right)^2\right]+\beta^3\left(\dfrac{t_{h2}}{t_{h1}}\right)^2}-\beta\left(1+\dfrac{t_{h2}}{t_{h1}}\right)\right] & (c) \\ 1{,}05\,\dfrac{f_{h,1,k}\, t_{h1}\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\dfrac{4\beta(2+\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h1}^2}}-\beta\right] & (d) \\ 1{,}05\,\dfrac{f_{h,1,k}\, t_{h2}\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\dfrac{4\beta(1+2\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h2}^2}}-\beta\right] & (e) \\ 1{,}15\sqrt{\dfrac{2\beta}{1+\beta}}\sqrt{2M_{y,k}\, f_{h,1,k}\, d} & (f) \end{cases} \quad (11.10)$$







with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \quad (11.11)$$

where

$f_{h,1,k}$, $f_{h,2,k}$	are the characteristic embedment strengths of members 1 and 2 given in Table 11.7;
t_{h1}, t_{h2}	are the embedment depths of members 1 and 2, see Figures 11.5 and 11.6;
$M_{y,k}$	is the characteristic yield moment given in Table 11.8;
d	is the diameter of the fastener.

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

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(9) Brittle failure modes like splitting, row shear, block shear, plug shear, and net tensile failure of wood in the connection regions, as defined in prEN1995-1-1:2021, **11.6**, should be avoided by satisfying condition given by Formula (13.4).

NOTE Reinforcement can be used in a dissipative zone as a means to prevent brittle failure modes, see for example prEN1995-1-1:2021, **11.8**.

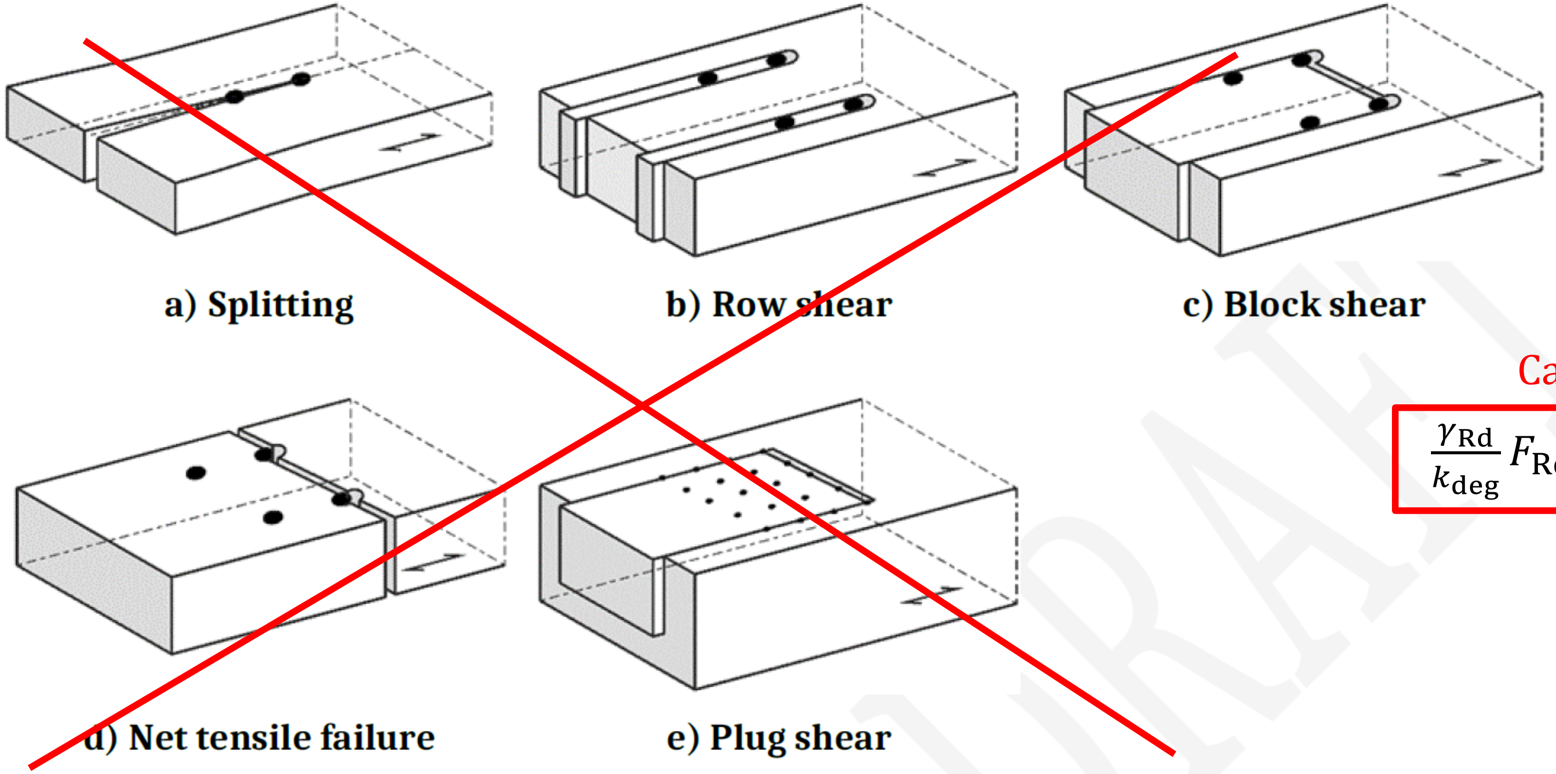


Figure 11.22 — Examples of brittle failure modes

Capacity design rule:

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

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

(1) P To ensure yielding of the dissipative zones, all non-dissipative members and connections in DC2 or DC3 structures shall be capacity designed according to **(2)** and either **(4)** or **(5)**.

(2) For DC2 and DC3 design of the structural types in Table 13.2, the design strength $F_{\mathrm{Rd,b}}$ of the brittle components should satisfy Formula (13.4).

$$\frac{\gamma_{\mathrm{Rd}}}{k_{\mathrm{deg}}} F_{\mathrm{Rd,d}} \leq F_{\mathrm{Rd,b}} \quad \text{with the limitation } k_{\mathrm{deg}} \leq 1 \qquad (13.4)$$

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)**, for which the value given in **13.3.1(1)** should be used; **(0,8)**

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

$F_{\mathrm{Rd,b}}$ is the design strength of the brittle component, calculated according to **13.2.2(2)**.

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

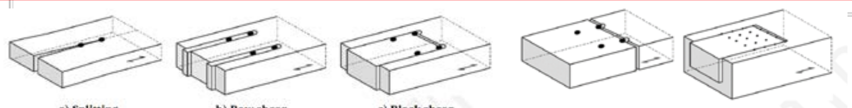


Table 13.4 — Values of the overstrength factors γ_{Rd} to be used in capacity design

Capacity design at	Brittle/non-dissipative failure mode	Overstrength factor γ_{Rd}	Formula No.
Connection and 2D- or 3D-connector level	Failure of timber by fracture in tension (net tensile failure, prEN1995-1-1:2021, **11.6.8**), failure by row shear, block shear or plug shear (prEN1995-1-1:2021, **11.6.5, 11.6.6, 11.6.7**), failure by splitting (prEN1995-1-1:2021, **11.4.4.2**)	1,6*	(13.4)
	Tensile (1993-1-1:2005, **8.2.3**) and shear (1993-1-1:2005, **8.2.6**) failure of the steel plates (angle brackets, hold-downs, tie-downs, etc.), tensile (1993-1-8:2005, **5.7**) and pull-through failure of anchor bolts or screws	1,6*	(13.4)
	Tensile (1992-4:2018, **7.2.1** and 1998-1-1, Annex G) and shear (1992-4:2018, **7.2.2** and 1998-1-1, Annex G) of headed and post-installed fasteners between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc) and reinforced concrete member.	1,6*	(13.4)
	Axial (prEN1995-1-1:2021, **11.2**) failures of dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members (head pull-through, withdrawal, tensile or buckling failure of the fastener, prEN1995-1-1:2021, **11.2.2, 11.2.3, 11.2.4, 11.2.5**)	1,6*	(13.4)
	Failure in shear (prEN1995-1-1:2021, **11.3**) of dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members	1,3	(13.4)

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

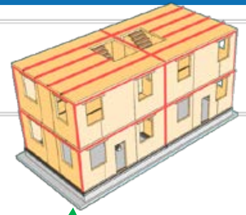


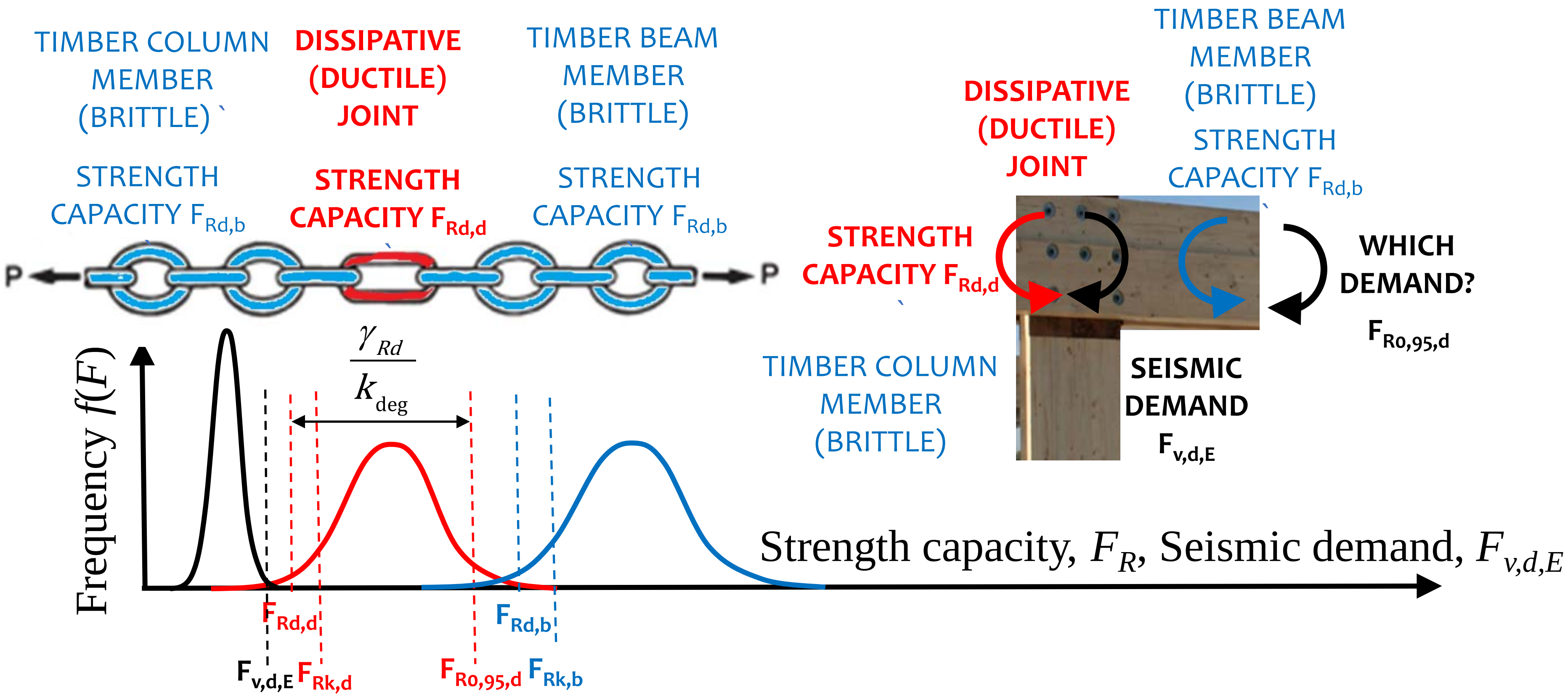
Table 13.4 — Values of the overstrength factors γ_{Rd} to be used in capacity design

Capacity design at	Brittle/non-dissipative failure mode	Overstrength factor γ_{Rd}	Formula No.
Wall and building level	Failure of timber members (prEN1995-1-1:2021, **8**)	1,6*	(13.7)
	Axial (prEN1995-1-1:2021, **11.2**) failures (head pull-through, withdrawal, tensile or compression failure of the fastener, prEN1995-1-1:2021, **11.2.2, 11.2.3, 11.2.4, 11.2.5**) of: - dowel-type joints between adjacent floor panels; floors and supporting walls underneath; orthogonal walls, including the ones at the building corners; - dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members	1,6*	(13.7)
	Failure in shear (prEN1995-1-1:2021, **11.3**) of: - dowel-type joints between adjacent floor panels; floors and supporting walls underneath; orthogonal walls, including the ones at the building corners; - dowel-type connections between metal plates (e.g. hold-downs, tie-downs, angle brackets, etc.) and timber members.	1,3	(13.7)
	Stabilising moment due to gravity loads in log shear walls.	1,3	(13.19)

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

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

13.4.3 Capacity design rules



$$F_{Rd,d} = k_{deg} k_{mod} \frac{F_{Rk,d}}{\gamma_m} \geq F_{v,d,E}$$

$$F_{Rd,b} = k_{mod} \frac{F_{Rk,b}}{\gamma_m} \geq F_{R0.95,d} = \frac{\gamma_{Rd}}{k_{deg}} F_{Rd,d} = \gamma_{Rd} \cdot k_{mod} \frac{F_{Rk,d}}{\gamma_m}$$

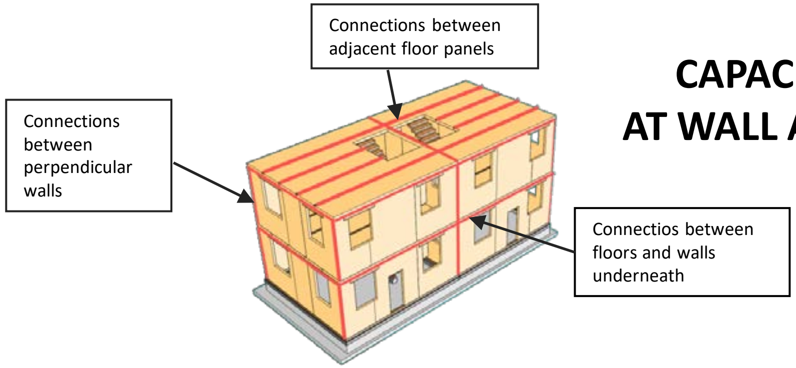
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EACH STRUCTURAL TYPE

GENERAL RULES

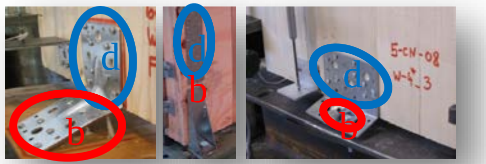
General description of the structural components (walls, floors) and type of connections generally used

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



FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

General rules – connections:

- The **joint of the walls to the foundation** should comply with conditions a) to f):

a) It 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.).

b) It should **prevent uplift and sliding of the walls**.

c) **Anchoring connections against overturning** (hold-downs or foundation tie-downs) should be **placed at wall ends, adjacent to door openings in wall panels, and at opening ends** either: 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 window opening and the area of the wall panel exceeds 0,50.

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

e) Shear **connections** and anchoring connections against overturning should be **connected to the CLT panels using metal fasteners** such as nails and screws, **and to the foundation using anchor bolts**.

f) Fastening to foundation should comply with prEN 1998-1-1:2022, Annex G

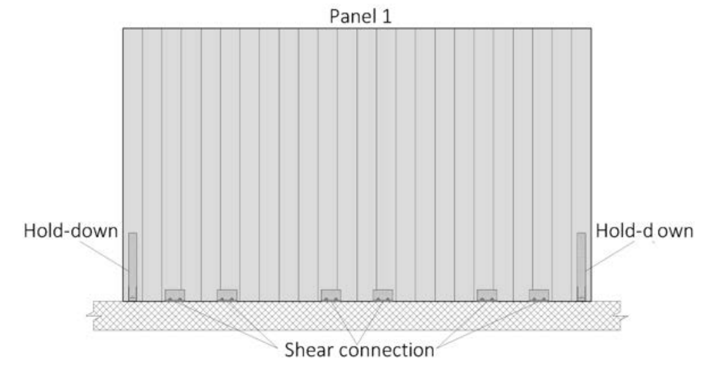
Hold-down

Angle bracket

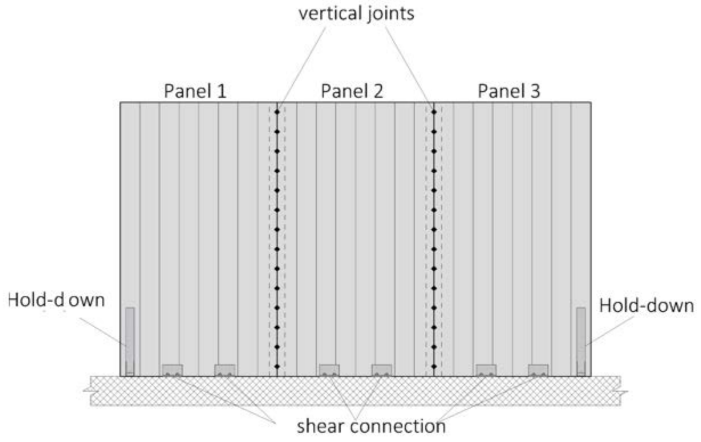
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

Wall panels: monolithic (DC2) vs segmented (DC3)

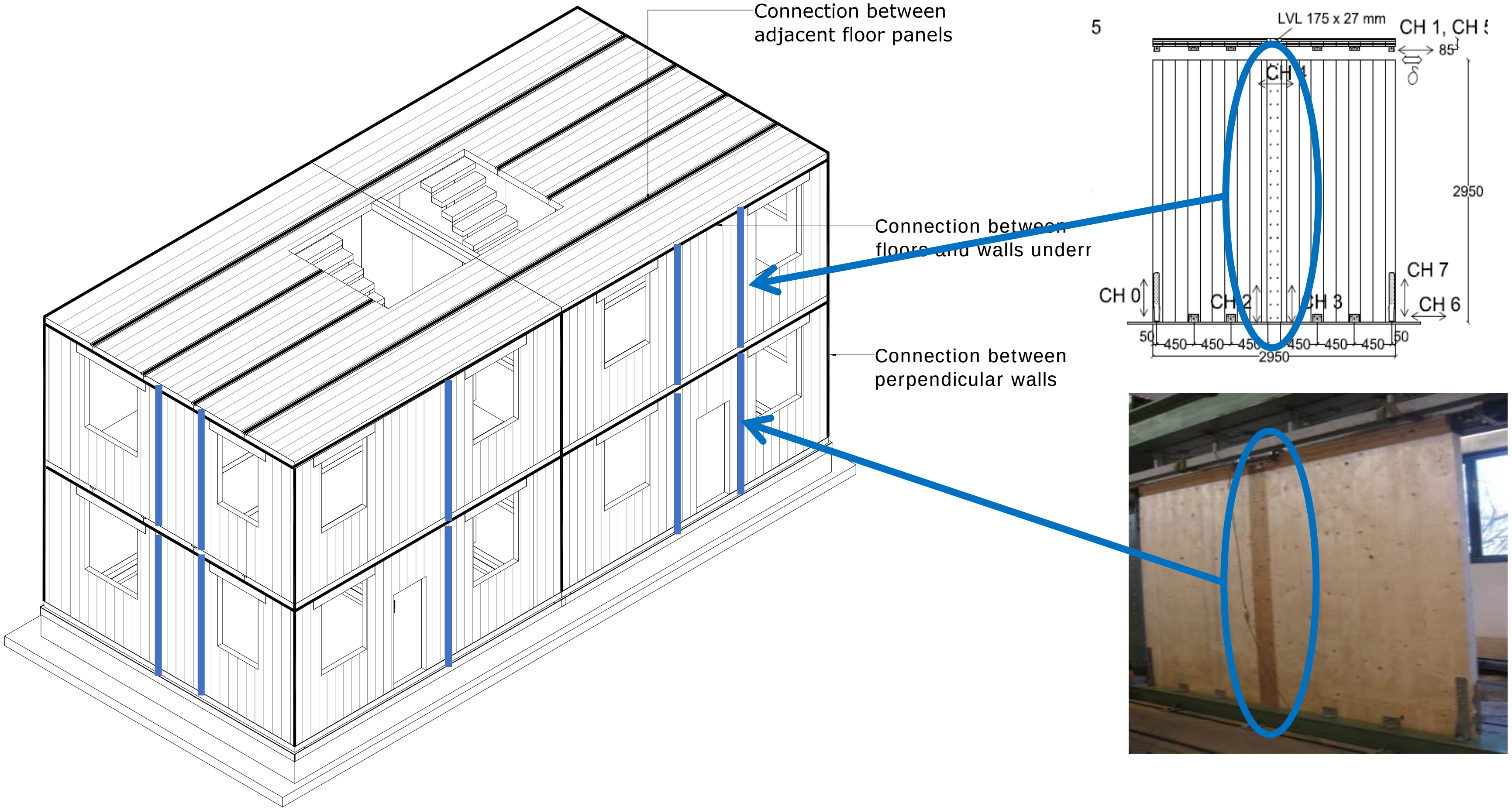


a) Single panel (Monolithic)



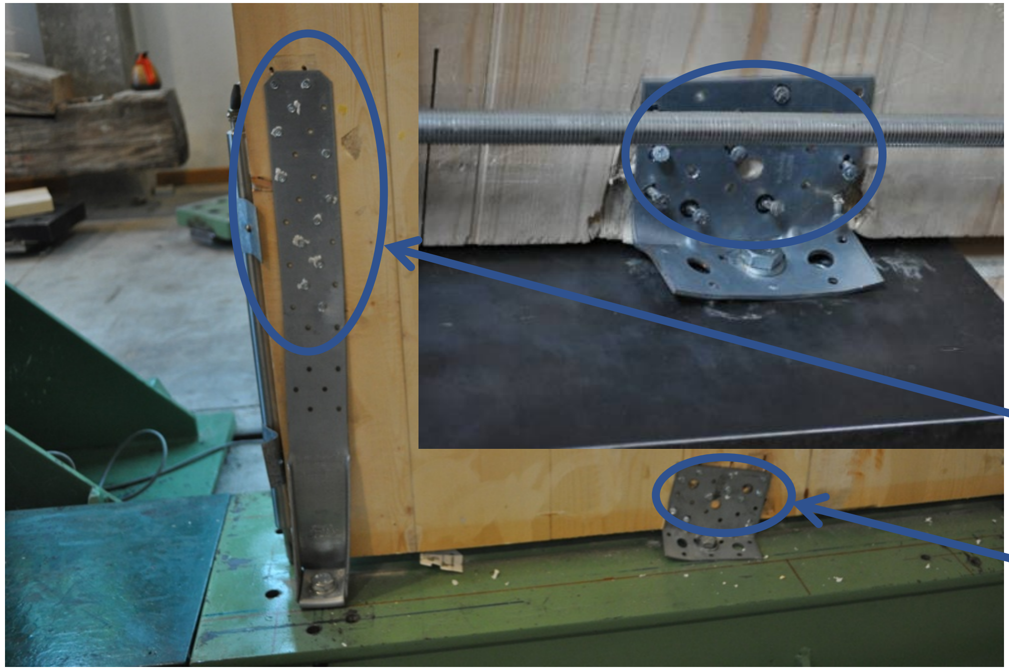
b) Multi-panel

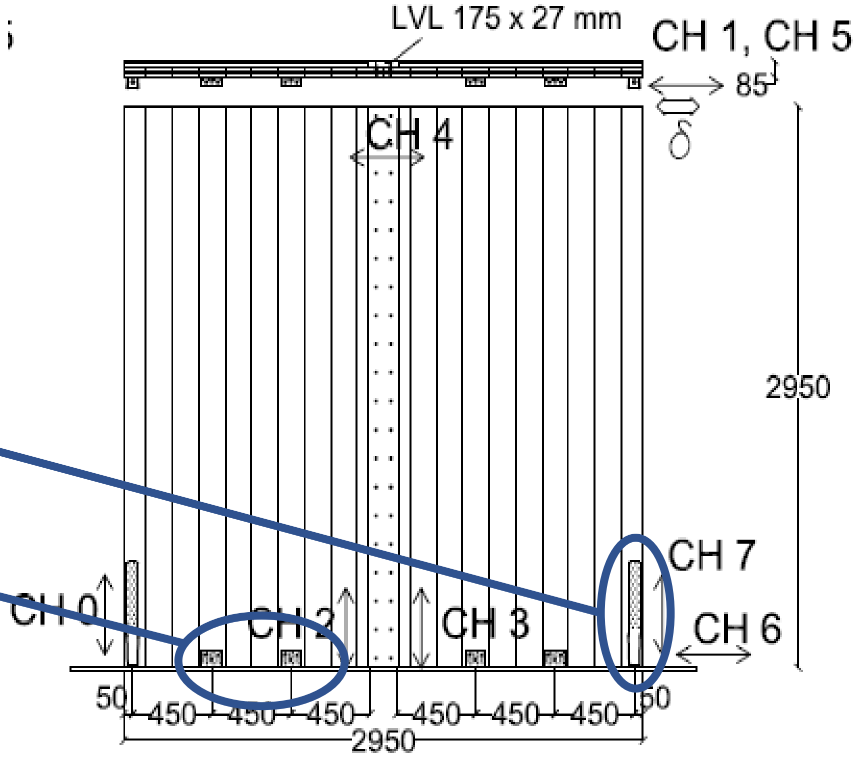
Dissipative zones (DC3):



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.**



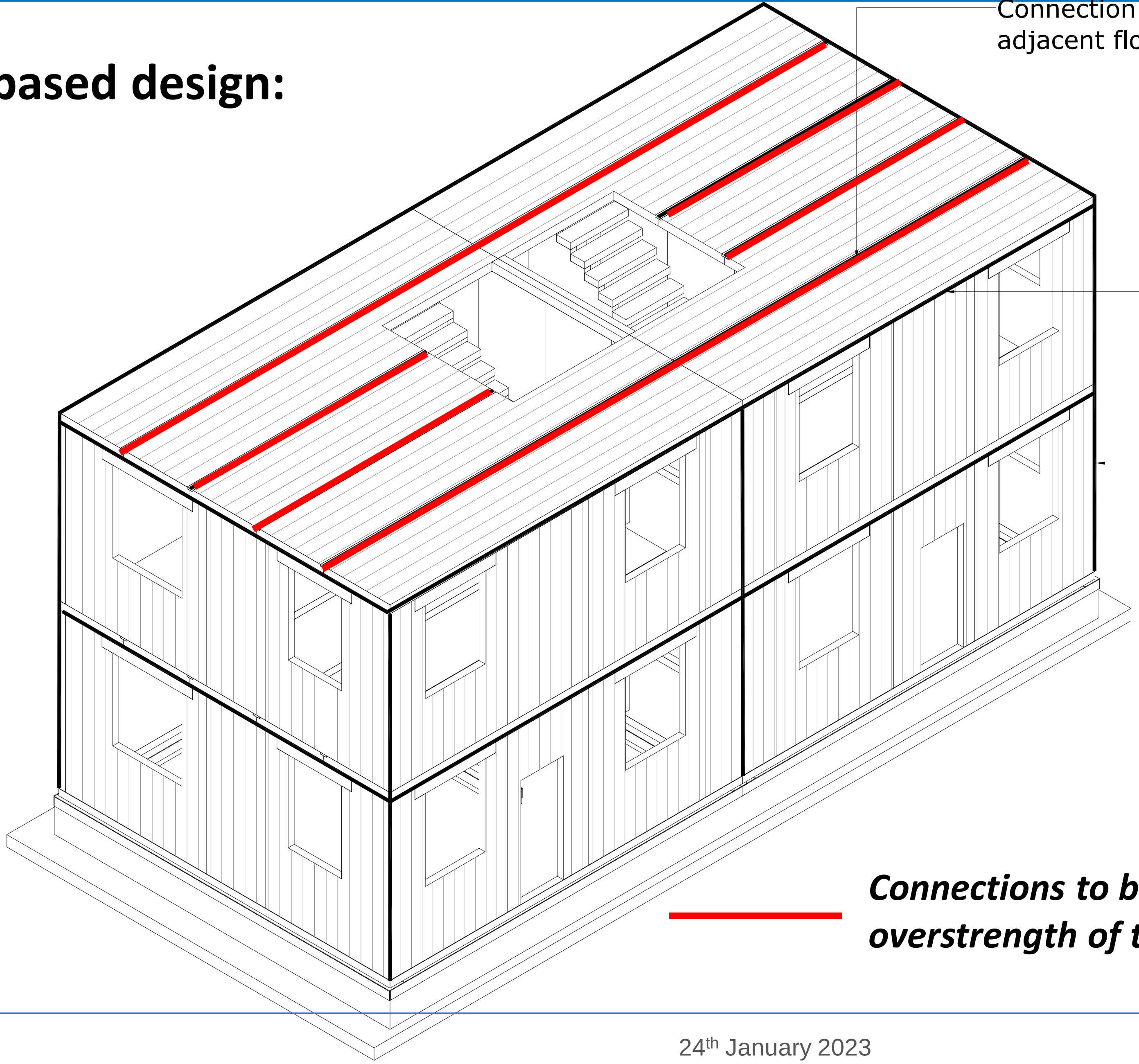


Capacity based design:

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;

Capacity based design:



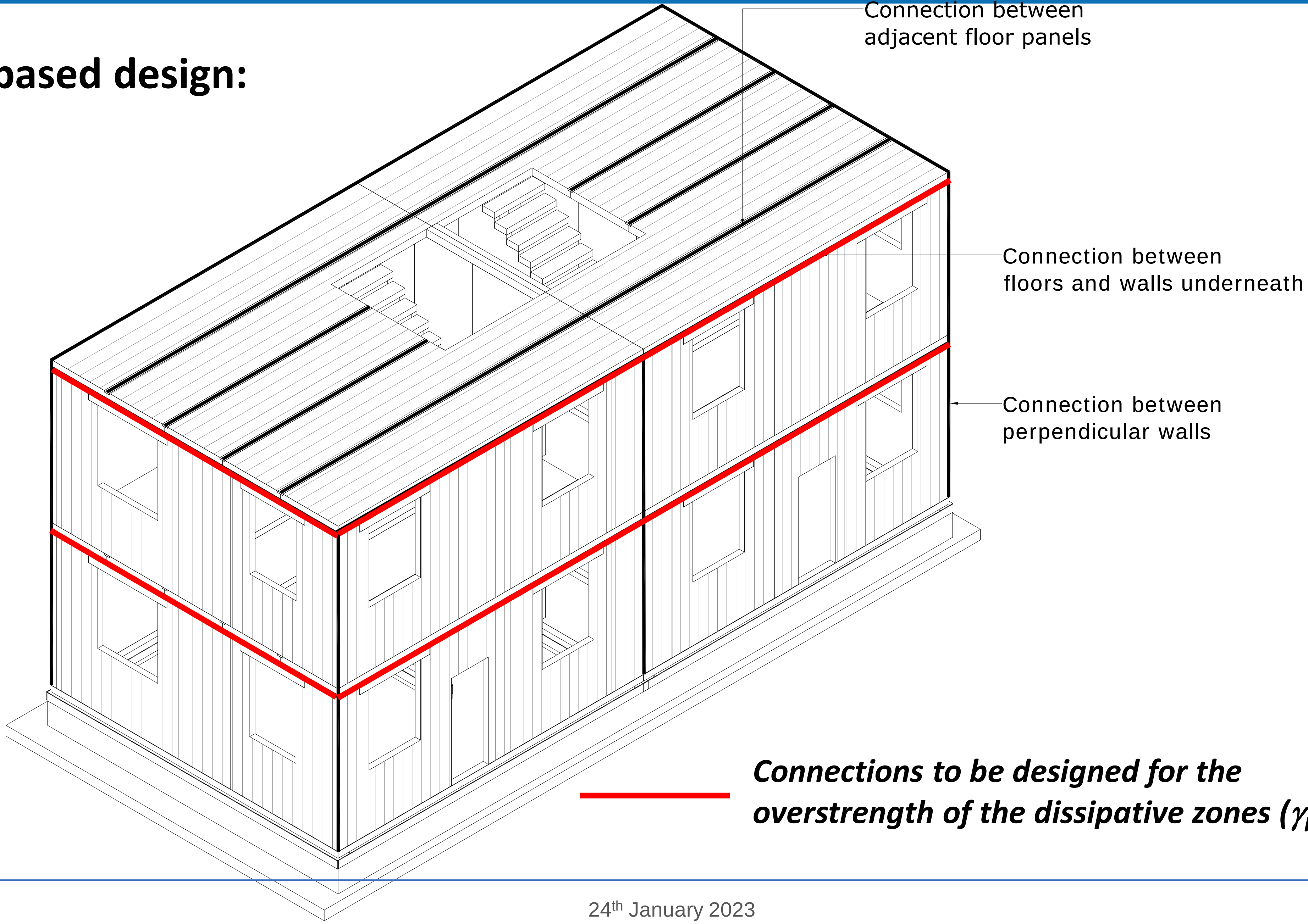
Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.3)

Capacity based design:

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;

Capacity based design:



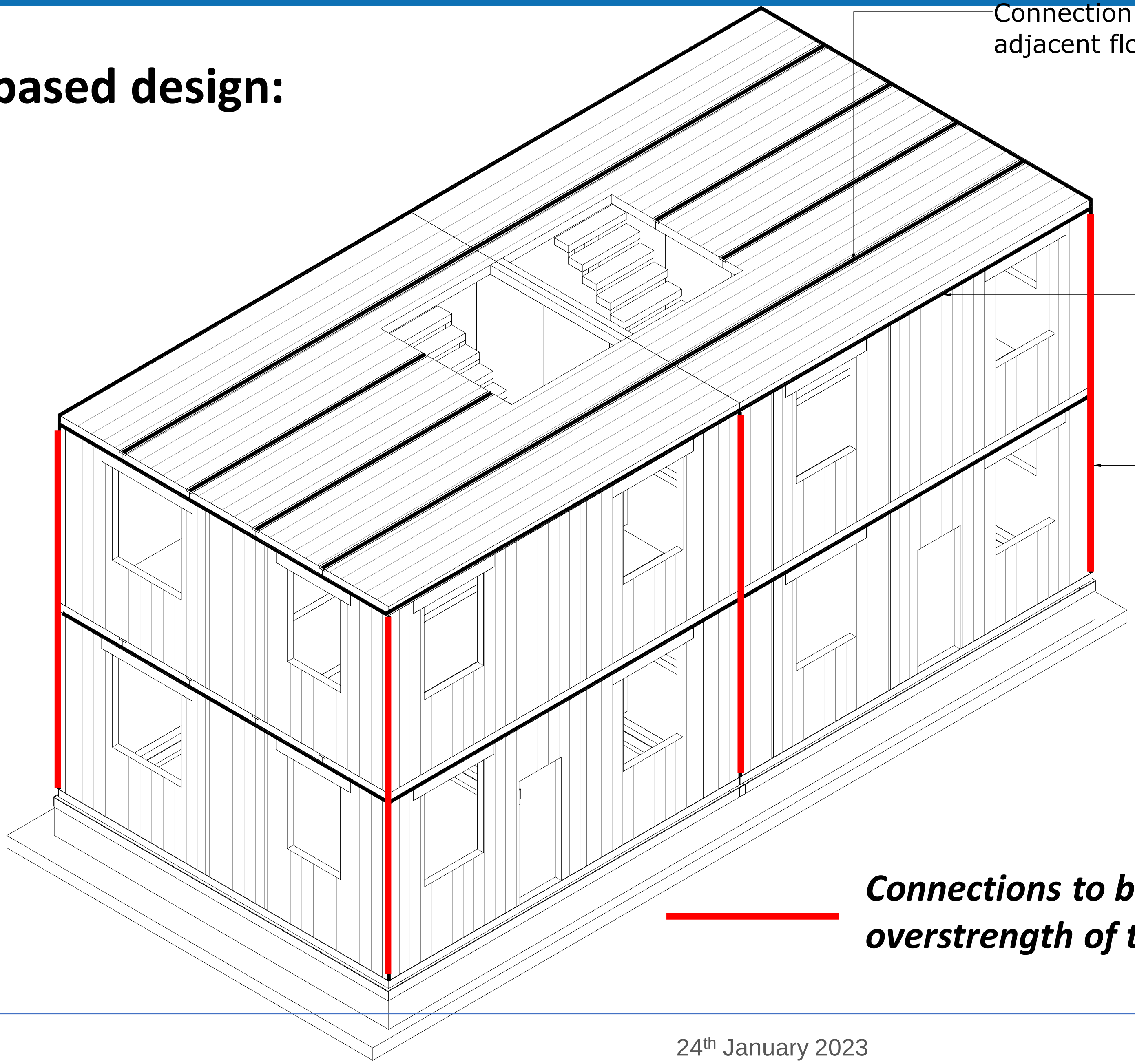
Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.3)

Capacity based design:

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;

Capacity based design:



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

The overstrength must be applied also to:

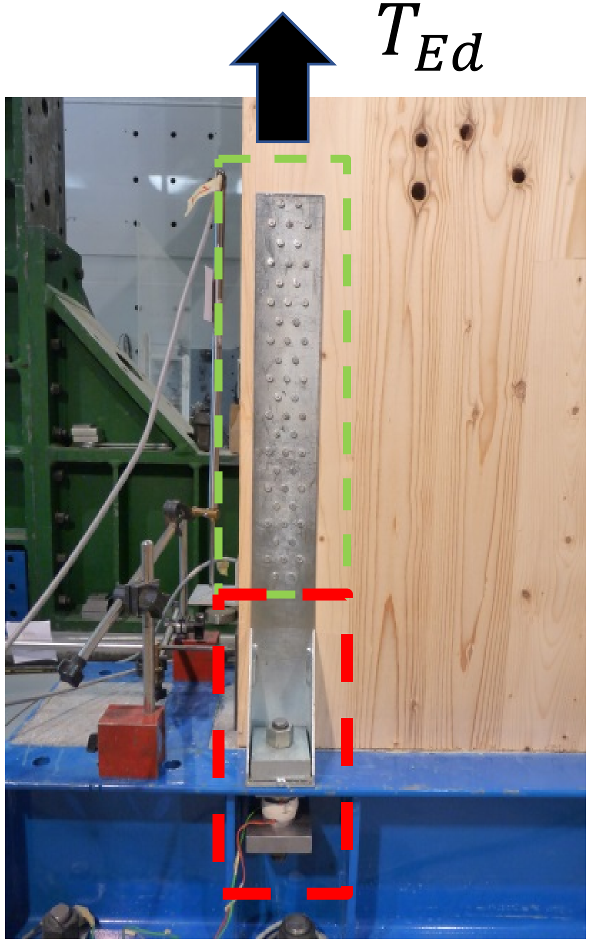
- Wall panels under in-plane vertical action due to the earthquake and floor panels under diaphragm action due to the earthquake;
- Metal parts of hold-down and angle bracket connections to avoid brittle tensile or shear failures;
- Connection of holddown and angle bracket to the foundation or to lower wall panels



F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

Capacity based design for CLT structures: connection level

T_{Ed}



$$\frac{\gamma_{Rd}}{k_{deg}} \cdot T_{\mathrm{Rd},nails} \leq \begin{cases} F_{\mathrm{Rd,b},steel-plate} \\ F_{\mathrm{Rd,b},anchor-bolt} \\ F_{\mathrm{Rd,b},timber} \end{cases}$$

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

Capacity based design for CLT structures: wall level

$$F_{\mathrm{Rd,b}} \geq \frac{\gamma_{\mathrm{Rd}}}{k_{\mathrm{deg}}} \Omega_{\mathrm{d}} F_{\mathrm{Ed,E}} + F_{\mathrm{Ed,G}}$$

$F_{\mathrm{Rd,b}}$ is the design strength of the non-dissipative joint or structural member from Formula (13.2);
γ_{Rd} is the overstrength factor, given in Table 13.4;
k_{deg} is the strength reduction factor defined in **13.3.1(1)**, for which the value given in **13.3.1(1)** should be used;
$F_{\mathrm{Ed,E}}$ is the action effect in the non-dissipative joint or member due to the design seismic action;
$F_{\mathrm{Ed,G}}$ is the action effect in the non-dissipative joint or member due to the non-seismic actions in the design seismic situation;
Ω_{d} is the minimum value of all overstrength ratios $\Omega_{\mathrm{d,i}}$, calculated by Formula (13.8);

$$\boxed{\Omega_{\mathrm{d}} = \min \Omega_{\mathrm{d,i}}} \quad (13.8)$$

$\Omega_{\mathrm{d,i}}$ is the overstrength ratio at the i^{th} storey calculated by Formula (13.9);

$$\boxed{\Omega_{\mathrm{d,i}} = V_{\mathrm{Rd,LLRS,i}} / V_{\mathrm{Ed,E,LLRS,i}}} \quad (13.9)$$

$V_{\mathrm{Rd,LLRS,i}}$ is the design lateral strength of the primary structure at the i^{th} storey;
$V_{\mathrm{Ed,E,LLRS,i}}$ is the design global shear of the i^{th} storey due to the seismic action.

NOTE The design forces in the non-dissipative joints or structural elements can be taken not greater than the forces determined using a behaviour factor q equal to 1.

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

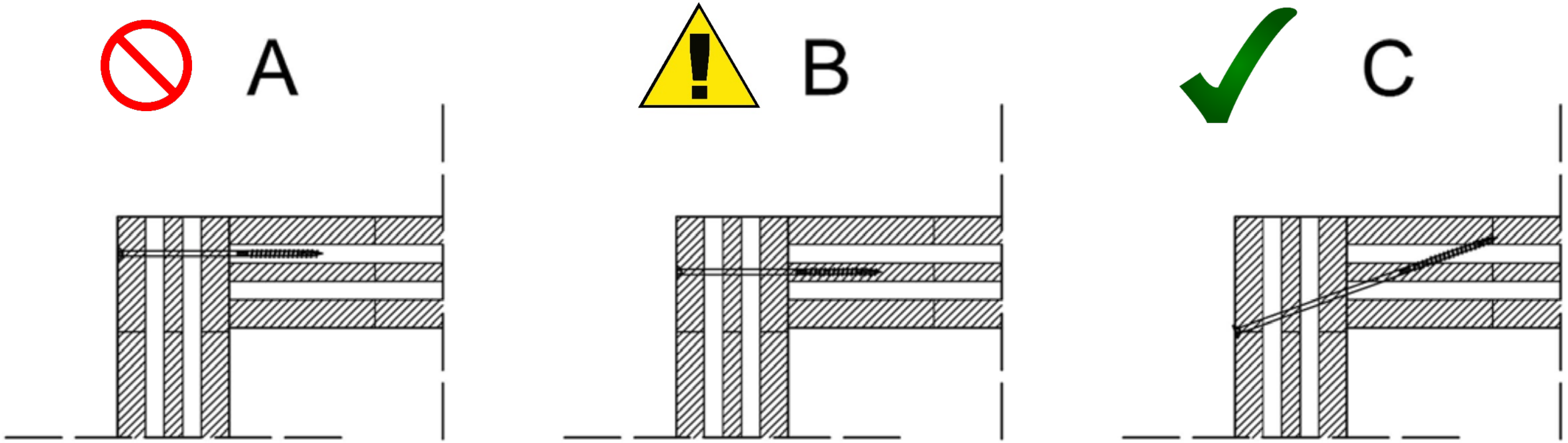
Capacity based design for CLT structures: building level

13.7.3 Verification in DC3

(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 verify Formula (13.14).

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

G – DETAILING RULES FOR ALL STRUCTURAL TYPES - CLT

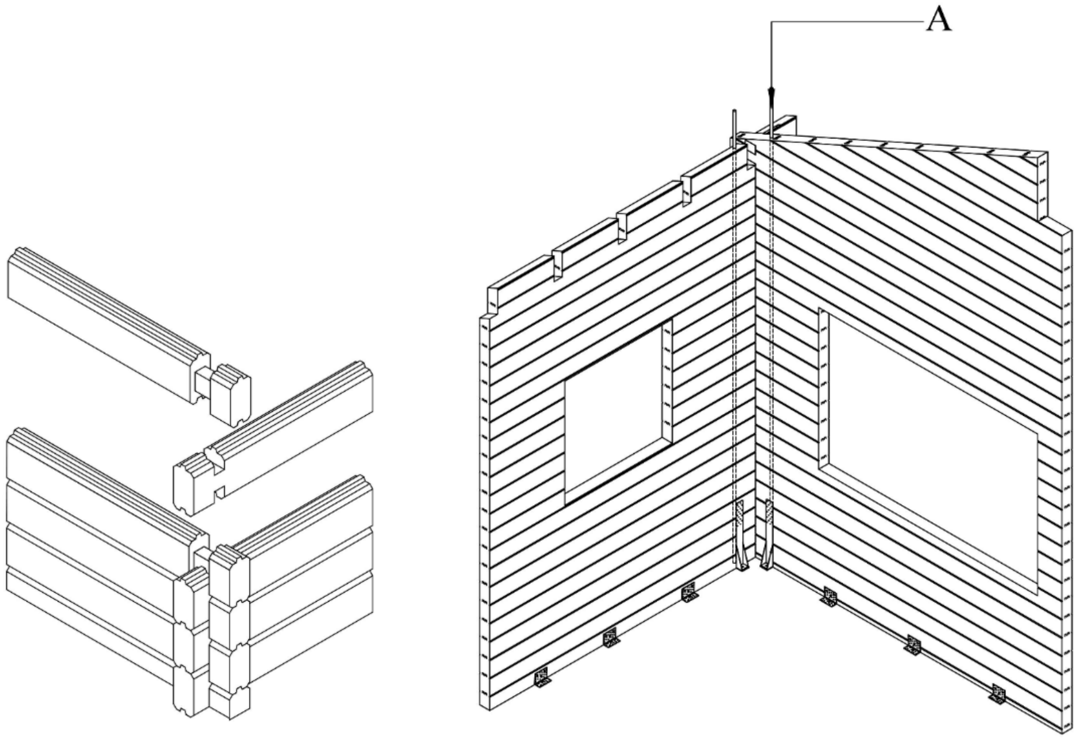


Key

A Wrong - screws inserted in layers with grain direction parallel to the screw axis.
B Right but difficult to achieve - screws inserted in layers with grain direction perpendicular to the screw axis.
C Right - screws inserted inclined.

Figure 13.7 — CLT wall-to-wall connection

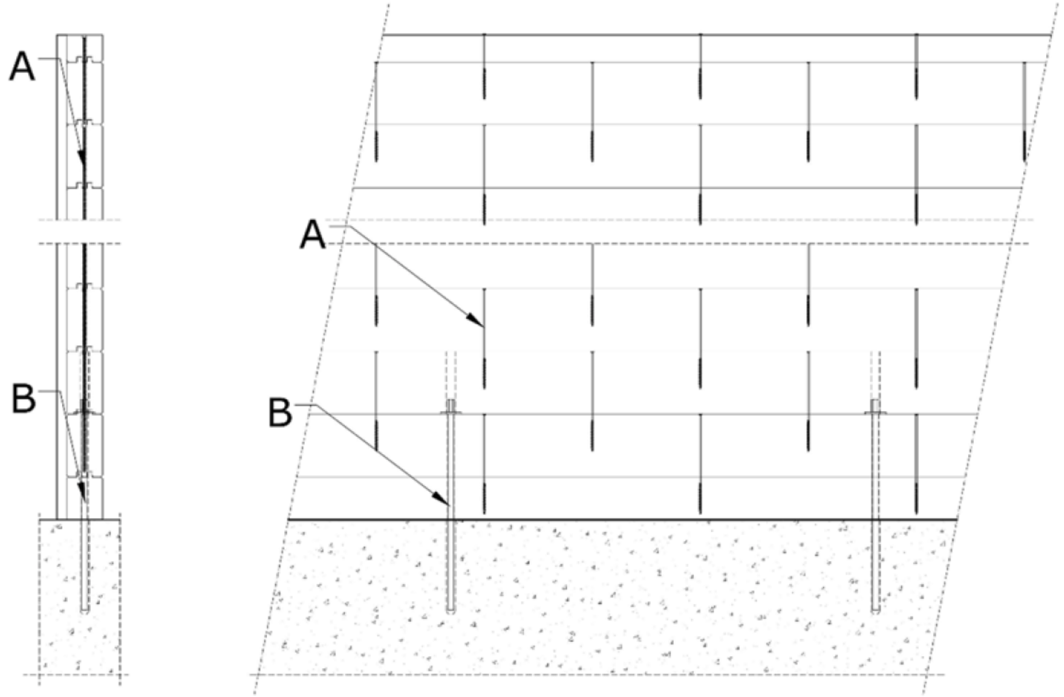
G – DETAILING RULES FOR ALL STRUCTURAL TYPES – LOG STRUCTURES



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 (right: side view; left: cross-section).

Figure 13.11 — Connection between timber logs by means of self-tapping screws

ACKNOWLEGMENTS

The contribution of all members of Working Group **CEN/TC250/SC8/WG3** in the revision of former Chapter 8 of EN 1998-1 (now Chapter 13 of EN1998-1-2) is gratefully acknowledged.

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Thank you very much for your attention!

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