reluis

Rete dei Laboratori Universitari di Ingegneria Sismica e Strutturale

SETTIMANA NAZIONALE DELLA PROTEZIONE CIVILE

PROMOSSA DA PROTEZIONE CIVILE Presidenza del Consiglio dei Ministri Dipartimento della Protezione Civile

6 / 13 OTTOBRE 2024

#settimanadiPC

SAIE Bologna, 9/12 ottobre 2024

SCUOLA DI INGEGNERIA STRUTTURALE – RELUIS

Bologna, 9-11 ottobre 2024

COMPORTAMENTO SISMICO DELLE STRUTTURE DI LEGNO ALLA LUCE DELLO SVILUPPO DELLA NUOVA EN1998-1-2

Prof. Ing. Massimo Fragiacomo, Università degli Studi dell'Aquila

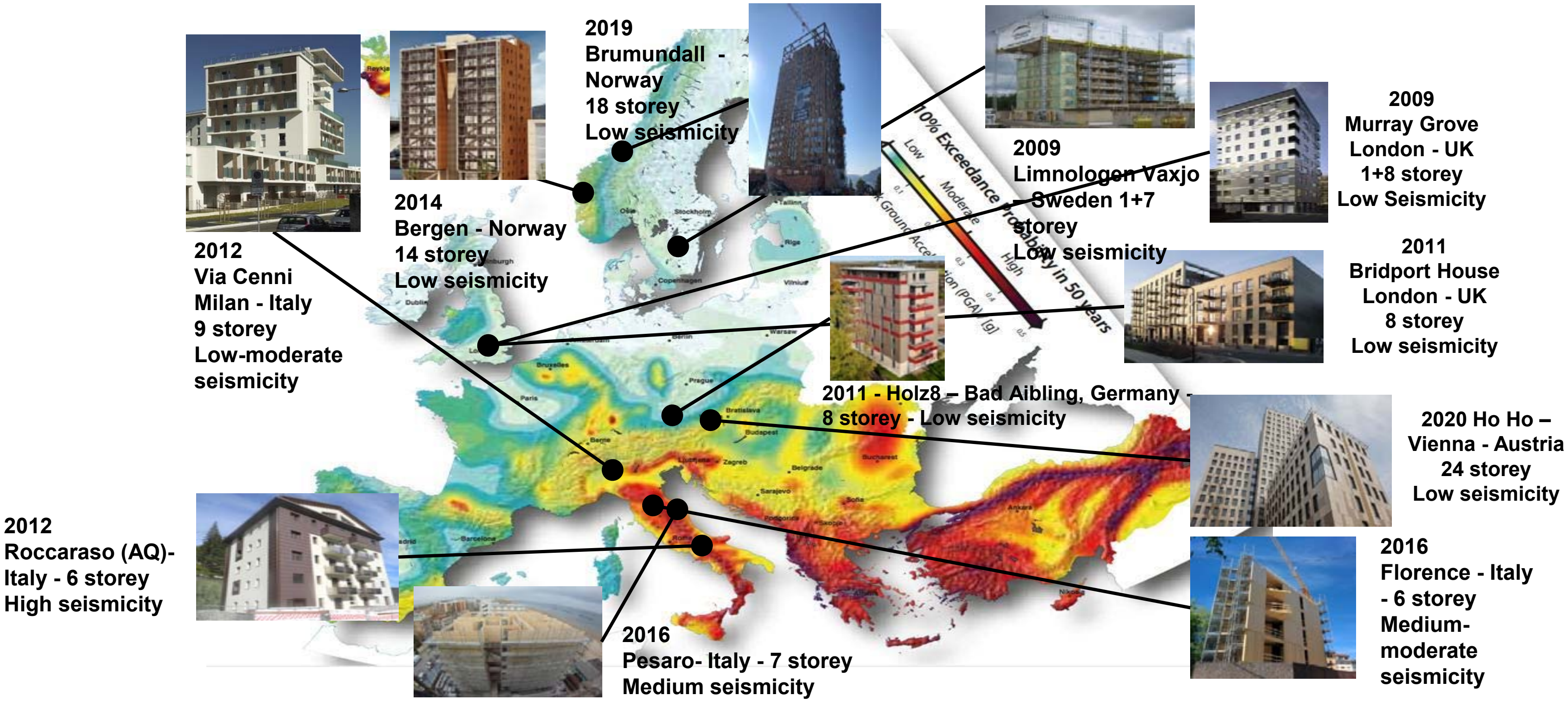
OBIETTIVI:

- **Introdurre le modifiche alla nuova generazione dell'Eurocodice 8 – progettazione antisismica delle strutture, con riferimento agli edifici nuovi in legno, in particolare edifici a pannelli di compensato di tavole (XLAM - CLT)**

- **Confrontare tali modifiche con la normativa attuale nazionale (NTC 2018)**

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**

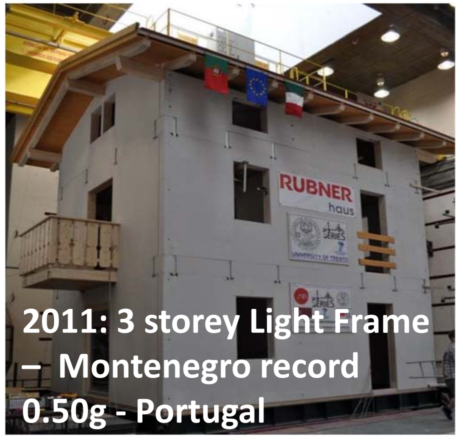

2006: 3 storey CLT – Kobe record 0.82 g (M 6.8) - Japan


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

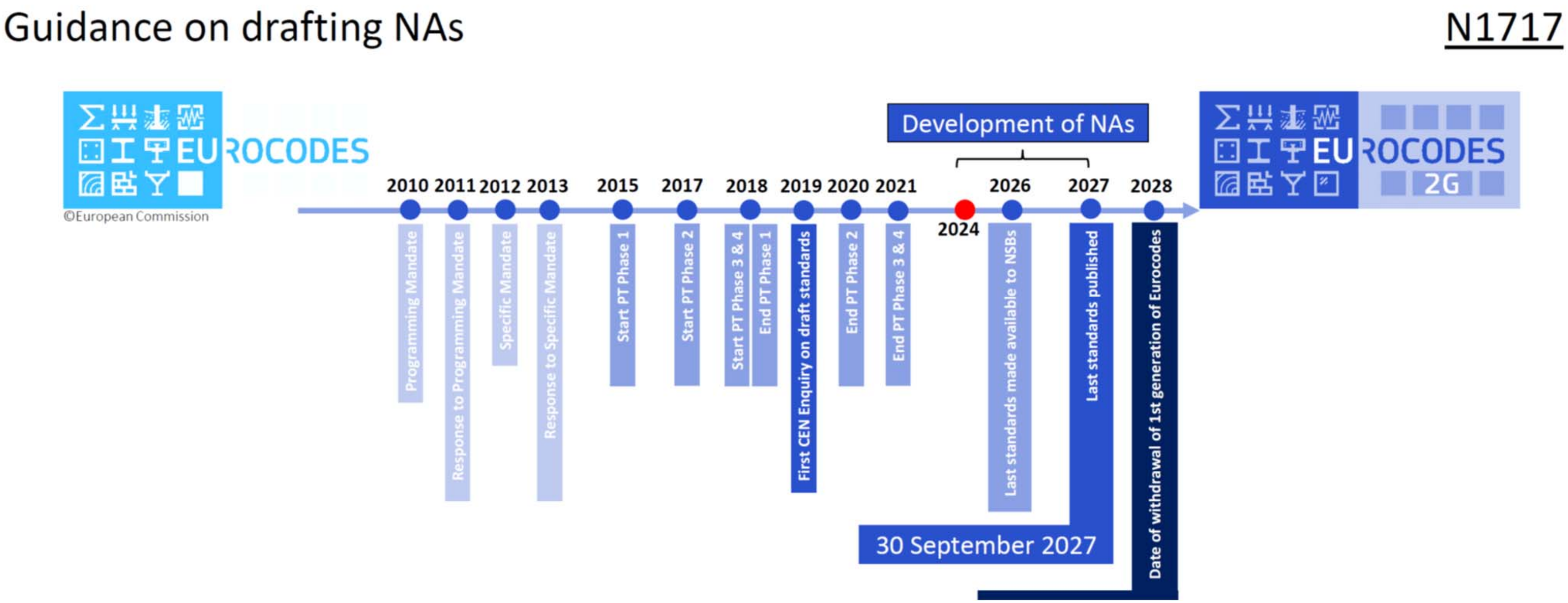

2011: 3 storey Light Frame – Tohoku record 0.54g - Portugal


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

INTRODUZIONE

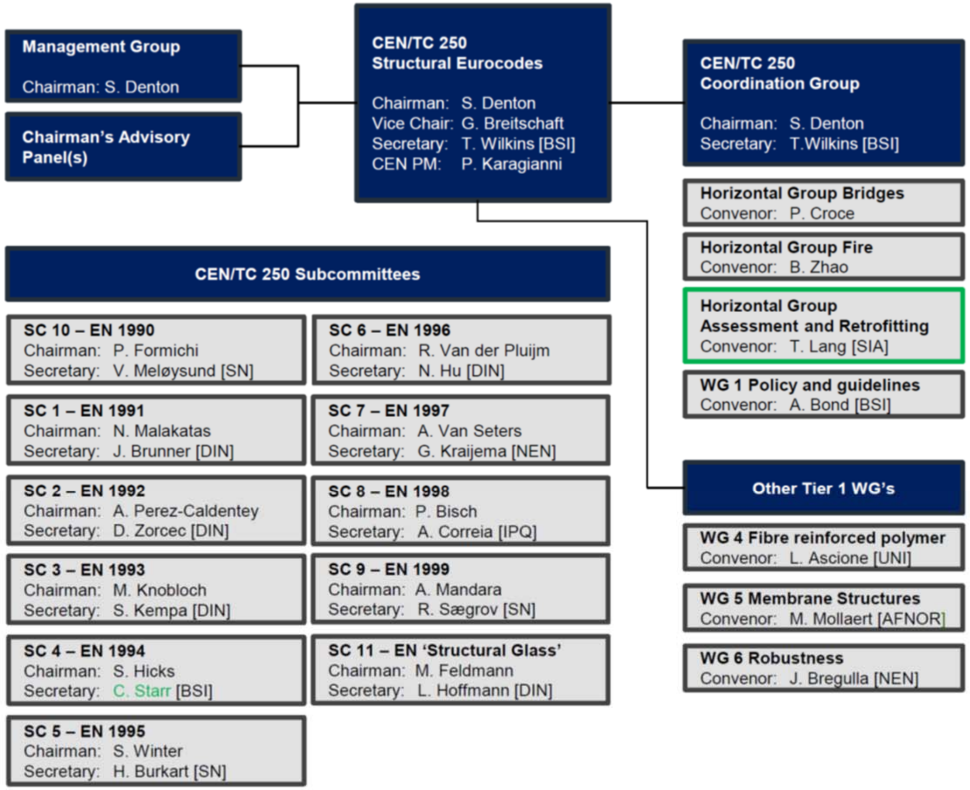
Revisione degli Eurocodici:

Un processo iniziato 14 anni fa per tutte le diverse parti, che si dovrebbe concludere nel 2028…



INTRODUZIONE

7.1 CEN/TC 250 structure



INTRODUCTION

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, **41 + 5** pp.)

Revision of the whole standard

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

INTRODUCTION

Current version of EC8:

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

New generation of EC8:

Revision of the whole standard

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

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

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.)

INTRODUCTION

Current version of EC5 – Design of timber structures:

EN 1995-1-1: General rules and rules for buildings
EN 1995-1-2: Structural fire design
EN 1995-2: Bridges

New generation of EC5:

Revision of the whole standards

New standards:

EN 1995-3: Execution

CEN/TS 19103:2021 - Structural design of timber-concrete composite structures – Common rules and rules for buildings

INTRODUCTION

Norme Tecniche sulle Costruzioni - NTC 2018:

Capitolo 4: Costruzioni civili e industriali
Capitolo 4.4: Costruzioni di legno
Progettazione di strutture in legno in condizioni statiche

Capitolo 7: Progettazione per azioni sismiche
Capitolo 7.7: Costruzioni in legno
Progettazione antisismica di nuovi edifici in legno

Capitolo 8: Costruzioni esistenti
Non ci sono indicazioni sulle strutture di legno – Qualche indicazione nella Circolare Applicativa (C8.5.4.3, C8.7.1.4)

Capitolo 11: Materiali e prodotti per uso strutturale
11.7: Materiali e prodotti a base di legno
Indicazioni sulle proprietà meccaniche ed i controlli di accettazione

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 (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.

....

NTC 2018:

Per l'utilizzo nelle pareti di taglio e nei diaframmi orizzontali, i pannelli strutturali di rivestimento devono rispettare le seguenti condizioni:

a) i pannelli di particelle (UNI EN 312) devono avere uno spessore non inferiore a 13 mm e massa volumica caratteristica in accordo a UNI EN 12369-1);

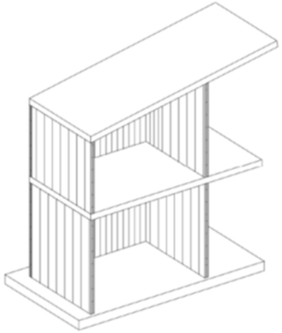
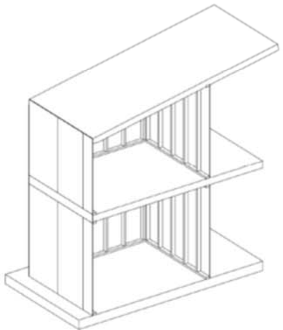
b) i pannelli di compensato (UNI EN 636) devono avere spessore non inferiore a 9 mm;

c) i pannelli di OSB (UNI EN 300) devono avere spessore non inferiore ai 12 mm se disposti a coppia, non inferiore a 15 mm se disposti singolarmente.

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

	a) <u>Cross laminated timber (CLT) structures**</u> CLT structures are those where the primary seismic 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/s^2]. 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 ~~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. Framed wall structures can be classified as b1) or b2): b1) With fully anchored walls; b2) With non-fully anchored walls.

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

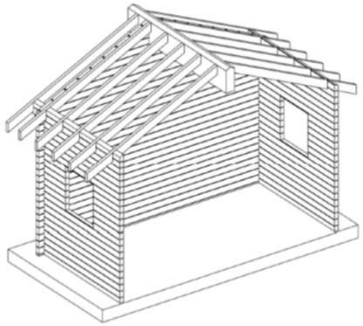
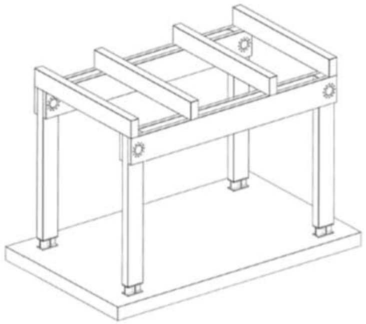
CLT structures can fall into either category a) or f) depending on whether the shear walls have heights equal to one inter-storey height (platform frame construction – see **3.1.20) or more (balloon frame construction – see **3.1.2**).

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

Table 13.1 — Timber structural types and examples of structures

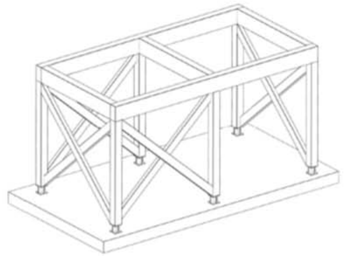
Examples of structural types*	Timber structural types
	c) Log structures Log structures are those where the primary seismic structure is composed by the superposition of rectangular or round solid or glulam timber elements ('logs'), prefabricated with carpentry connections at their ends and with upper and lower grooves. Log structures should be designed according to 13.9.
	d) Moment-resisting frame structures Moment-resisting frame structures are those where the primary seismic structure is composed of frames made of ~~by~~ timber elements with semi-rigid (as defined in 3.1.30 ~~28~~) moment-transmitting joints between the members, achieved with mechanical fasteners. Moment-resisting frames structures should be designed according to 13.10.

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

Table 13.1 — Timber structural types and examples of structures

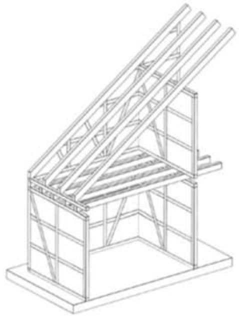
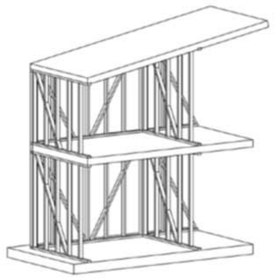
Examples of structural types*	Timber structural types
	e) Braced frame structures with dowel-type connections Braced frame structures with dowel-type connections are those consisting of timber columns and beams, where the primary seismic structure is composed of timber diagonal bracings, with all pin-jointed dowel-type connections (as defined in 3.1.11 ~~9~~). Braced frame structures with dowel-type connections should be designed according to 13.11.
	f) Vertical cantilever structures** 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. Vertical cantilever structures should be designed according to 13.12.
*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**).	

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

Table 13.1 — Timber structural types and examples of structures

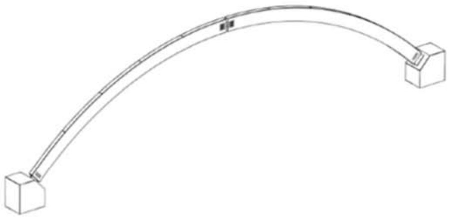
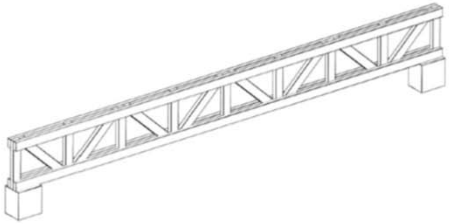
Examples of structural types*	Timber structural types
	g) <u>Braced frame structures with carpentry connections and interacting masonry infill</u> Braced frame structures with carpentry connections are those consisting of timber columns and beams, where the primary seismic structure is composed of diagonal ~~vertical~~ timber bracing with compression-only carpentry connections (as defined in 3.1.4) and interacting masonry infill. Braced frame structures with carpentry connections and interacting masonry infill should be designed according to 13.13.
	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 seismic structure is composed of diagonal ~~vertical~~ timber bracing with compression-only carpentry connections (as defined in 3.1.4). Braced frame structures with carpentry connections should be designed according to 13.14 as ~~non-~~low dissipative systems.

B – REVISED DEFINITION OF STRUCTURAL TYPES

13.4.1 Structural types

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

Table 13.1 — Timber structural types and examples of structures

Examples of structural types*	Timber structural types
	i) Two-pin and three-pin arches, three-pin frames and dome structures Structures composed by two-pin and three-pin timber arches, three-pin timber frames and timber dome structures should be designed as low ~~non-~~dissipative systems.
	j) Large span timber truss portal frame structures Large span truss portal frame structures are those consisting of timber trusses with semi-rigid (as defined in 3.1.30) moment-transmitting joints between the chords and the columns, forming a moment frame. Large span truss portal frame structures should be designed as low ~~non-~~dissipative systems.

B – REVISED DEFINITION OF STRUCTURAL TYPES

NTC 2018:

Costruzioni di legno (§ 7.7.3)
Pannelli di parete a telaio leggero chiodati con diaframmi incollati, collegati mediante chiodi, viti e bulloni Strutture reticolari iperstatiche con giunti chiodati
Portali iperstatici con mezzi di unione a gambo cilindrico
Pannelli di parete a telaio leggero chiodati con diaframmi chiodati, collegati mediante chiodi, viti e bulloni.
Pannelli di tavole incollate a strati incrociati, collegati mediante chiodi, viti, bulloni
Strutture reticolari con collegamenti a mezzo di chiodi, viti, bulloni o spinotti
Strutture cosiddette miste, con intelaiatura (sismo-resistente) in legno e tamponature non portanti
Strutture isostatiche in genere, compresi portali isostatici con mezzi di unione a gambo cilindrico, e altre tipologie strutturali

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:

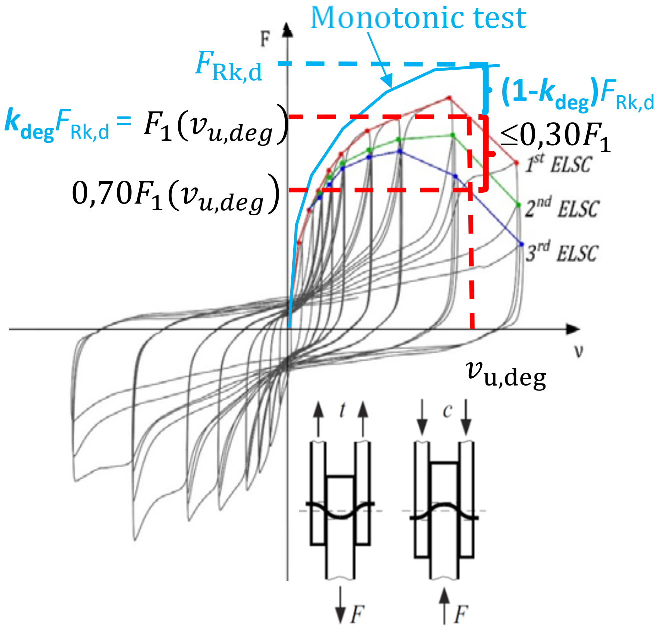
$F_{\mathrm{Rd,d}}$ is the design value of the strength of the dissipative zones;

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

NTC 18	Current EC8	New generation of EC8
ND	DCL	-> DC1 (non dissipative)
CD "B"	DCM	-> DC2 (medium dissipative)
CD "A"	DCH	-> DC3 (highly dissipative)

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).**

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

where:

$F_{\mathrm{Rd,nd}}$ 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;

$F_{\mathrm{Rk,nd}}$ 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,

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

prEN1995-1-1:2023

Table 4.3 (NDP) — Partial factor γ_M for materials in fundamental design situations

Group / Subgroup / Product	Abbreviation	Partial factor[a]
Solid wood based	**SWB**	
Structural lumber	SL	$\gamma_M = 1{,}30$
Structural timber	ST	
Structural finger jointed timber	FST	
Parallel laminated timber	PL	
Glued solid timber	GST	$\gamma_M = 1{,}25$
Glued laminated timber	GL	
Block glued glulam	BGL	
Single layered solid wood panel	SWP-P	
Cross layered timber	CL	
Cross laminated timber	CLT	$\gamma_M = 1{,}25$
Multi-layered solid wood panel	SWP-C	
Veneer-based	**VB**	
Laminated veneer lumber	LVL	
LVL with parallel veneers	LVL-P	$\gamma_M = 1{,}20$
LVL with crossband veneers	LVL-C	
Glued laminated veneer lumber	GLVL	
GLVL with parallel veneers	GLVL-P	$\gamma_M = 1{,}20$
GLVL with crossband veneers	GLVL-C	
Plywood	PW	$\gamma_M = 1{,}20$

Table 4.4 (NDP) — Partial factors γ_M and γ_R for connections in fundamental design situations

Connection type	Partial factor
Connections with dowel-type fasteners and connectors	$\gamma_R = 1{,}30$
Carpentry connections	$\gamma_R = 1{,}30$
Bond line failure	$\gamma_M = 1{,}30$
Steel design resistance based on an (semi-) empirical analysis with a ductile failure mode	$\gamma_{M1} = 1{,}10$
Steel design resistance of cross-sections in tension to fracture	$\gamma_{M2} = 1{,}25$
NOTE 1 Partial factors γ_M for connections with punched metal plate fasteners are given in <v2>Annex H, H.3. NOTE 2 Partial factors for prestressing steel elements are given in <v2>Annex L, L.3.	

Table 4.5 (NDP) — Partial factors γ_M and γ_R for accidental design situations

Material or connection type	Partial factor
All materials and connections	$\gamma_M = \gamma_R = 1{,}00$

Table 5.4 — Values of k_{mod}

Material		Service class	Load-duration of action				
			Permanent	Long-term	Medium-term	Short-term	Instantaneous
Structural timber (ST)		1 and 2	0,60	0,70	0,80	0,90	1,10
		3	0,55	0,60	0,70	0,80	1,00
		4	0,50	0,55	0,65	0,70	0,90
Structural finger-jointed timber (FST)		1 and 2	0,60	0,70	0,80	0,90	1,10
Glued laminated timber (GL)		1 and 2	0,60	0,70	0,80	0,90	1,10
		3[a]	0,55	0,60	0,70	0,80	1,00
Block glued glulam (BGL)		1 and 2	0,60	0,70	0,80	0,90	1,10
Glued solid timber (GST)		1 and 2	0,60	0,70	0,80	0,90	1,10
Cross laminated timber (CLT)		1 and 2	0,60	0,70	0,80	0,90	1,10
Laminated veneer lumber (LVL)		1 and 2	0,60	0,70	0,80	0,90	1,10
		3	0,55	0,60	0,70	0,80	1,00
Glued laminated veneer lumber (GLVL)		1 and 2	0,60	0,70	0,80	0,90	1,10
Solid wood panel (SWP)	SWP/1S or SWP/2S	1	0,60	0,70	0,80	0,90	1,10
	SWP/2S	1 and 2	0,60	0,70	0,80	0,90	1,10
Plywood (PW)	Type EN 636-1S or -2S or -3S	1	0,60	0,70	0,80	0,90	1,10
	Type EN 636-2S or -3S	2	0,60	0,70	0,80	0,90	1,10
	Type EN 636-3S	3	0,55	0,60	0,70	0,80	1,00

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

NTC 2018:

7.7.6. VERIFICHE DI SICUREZZA

I valori di resistenza degli elementi di legno fanno riferimento a carichi di tipo “istantaneo”, nelle condizioni di servizio assunte per la struttura.

Per la verifica di strutture progettate in conformità al concetto di comportamento strutturale dissipativo (classe di duttilità CD ”A” o CD ”B”), può considerarsi valido quanto riportato nelle verifiche di resistenza (RES) del § 7.3.6.1. quando siano soddisfatti i requisiti di cui al § 7.7.3 per le zone dissipative (anche sulla base di apposite prove sperimentali) e la resistenza del materiale sia opportunamente ridotta del 20% per tener conto del degrado per deformazioni cicliche.

7.3.6.1 ELEMENTI STRUTTURALI (ST)

VERIFICHE DI RESISTENZA (RES)

La resistenza dei materiali può essere ridotta per tener conto del degrado per deformazioni cicliche, giustificandolo sulla base di apposite prove sperimentali. In tal caso, ai coefficienti parziali di sicurezza sui materiali γ_M si attribuiscono i valori precisati nel Cap. 4 per le situazioni eccezionali.

$F_{\mathrm{Rd}} = k_{\mathrm{mod}} \frac{F_{\mathrm{Rk}}}{\gamma_{\mathrm{M}}}$ con γ_M>1 scelto dalla Colonna A della Tabella 4.4.III, per strutture dissipative e non dissipative.

In alternativa, SOLO per strutture dissipative (progettate in CD “A” o “B”):

$F_{\mathrm{Rd}} = k_{\mathrm{mod}} \frac{0.8 F_{\mathrm{Rk}}}{\gamma_{\mathrm{M}}}$ con γ_M=1 (combinazione eccezionale in Tab. 4.4.III)

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

NTC 2018:

Il coefficiente γ_M è valutato secondo la colonna A della tabella 4.4.III. Si possono assumere i valori riportati nella colonna B della stessa tabella, per produzioni continuative di elementi o strutture, soggette a controllo continuativo del materiale dal quale risulti un coefficiente di variazione (rapporto tra scarto quadratico medio e valor medio) della resistenza non superiore al 15%. Le sud-dette produzioni devono essere inserite in un sistema di qualità di cui al § 11.7.

Tab. 4.4.III - *Coefficienti parziali γ_M per le proprietà dei materiali*

Stati limite ultimi	Colonna A γ_M	Colonna B γ_M
combinazioni fondamentali		
legno massiccio	1,50	1,45
legno lamellare incollato	1,45	1,35
pannelli di tavole incollate a strati incrociati	1,45	1,35
pannelli di particelle o di fibre	1,50	1,40
LVL, compensato, pannelli di scaglie orientate	1,40	1,30
unioni	1,50	1,40
combinazioni eccezionali	1,00	1,00
	Per i materiali non compresi nella Tabella si potrà fare riferimento ai pertinenti valori riportati nei riferimenti tecnici di comprovata validità indicati nel Capitolo 12, nel rispetto dei livelli di sicurezza delle presenti norme.	

C – NEW SAFETY FORMAT FOR SEISMIC VERIFICATIONS

NTC 2018:

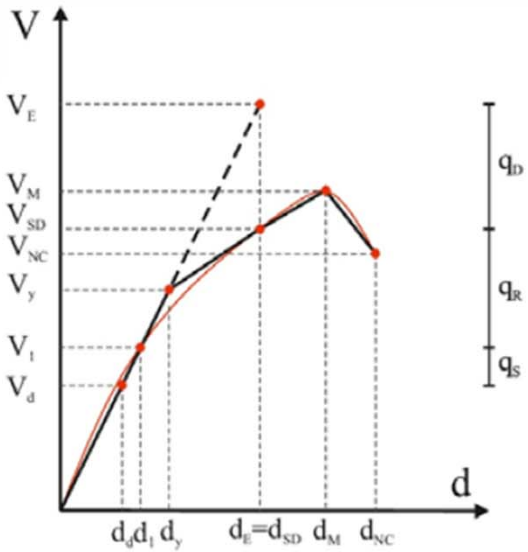
Tab. 4.4.IV -*Valori di* k_{mod} *per legno e prodotti strutturali a base di legno*

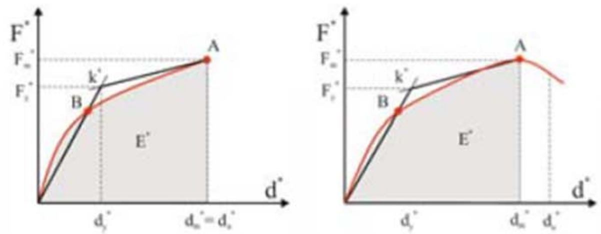
Materiale	Riferimento		Classe di servizio	Classe di durata del carico: Permanente	Lunga	Media	Breve	Istantanea
Legno massiccio	UNI EN 14081-1		1	0,60	0,70	0,80	0,90	1,10
Legno lamellare incollato (*)	UNI EN 14080		2	0,60	0,70	0,80	0,90	1,10
LVL	UNI EN 14374, UNI EN 14279		3	0,50	0,55	0,65	0,70	0,90
Compensato	UNI EN 636:2015		1	0,60	0,70	0,80	0,90	1,10
			2	0,60	0,70	0,80	0,90	1,10
			3	0,50	0,55	0,65	0,70	0,90
Pannello di scaglie orientate (OSB)	UNI EN 300:2006	OSB/2	1	0,30	0,45	0,65	0,85	1,10
		OSB/3 – OSB/4	1	0,40	0,50	0,70	0,90	1,10
			2	0,30	0,40	0,55	0,70	0,90
Pannello di particelle (truciolare)	UNI EN 312 :2010	Parti 4, 5	1	0,30	0,45	0,65	0,85	1,10
		Parte 5	2	0,20	0,30	0,45	0,60	0,80
		Parti 6, 7	1	0,40	0,50	0,70	0,90	1,10
		Parte 7	2	0,30	0,40	0,55	0,70	0,90
Pannello di fibre, pannelli duri	UNI EN 622-2:2005	HB.LA, HB.HLA 1 o 2	1	0,30	0,45	0,65	0,85	1,10
		HB.HLA 1 o 2	2	0,20	0,30	0,45	0,60	0,80
Pannello di fibre, pannelli semiduri	UNI EN 622-3:2005	MBH.LA1 o 2	1	0,20	0,40	0,60	0,80	1,10
		MBH.HLS1 o 2	1	0,20	0,40	0,60	0,80	1,10
			2	-	-	-	0,45	0,80
Pannello di fibra di legno, ottenuto per via secca (MDF)	UNI EN 622-5:2010	MDF.LA, MDF.HLS	1	0,20	0,40	0,60	0,80	1,10
		MDF.HLS	2	-	-	-	0,45	0,80

Per i materiali non compresi nella Tabella si potrà fare riferimento ai pertinenti valori riportati nei riferimenti tecnici di comprovata validità indicati nel Capitolo 12, nel rispetto dei livelli di sicurezza delle presenti norme.

(*) I valori indicati si possono adottare anche per i pannelli di tavole incollate a strati incrociati, ma limitatamente alle classi di servizio 1 e 2.

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)

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

(2)

- $\boldsymbol{V_d}$:Design base-shear
- $\boldsymbol{V_1}$:First plasticization base-shear
- $\boldsymbol{V_{SD}}$:Significant Damage base-shear
- $\boldsymbol{V_E}$:Elastic response base-shear

D – NEW DEFINITION OF BEHAVIOUR FACTOR 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]	Ductility class						
		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
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
d) 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
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
e) Vertical cantilever structures $H \leq 12$ m	4,0	1,5	1,2	1,3	2,3	N/A	N/A	N/A
Vertical cantilever structures $H > 12$ m	4,0	1,5	1,0	1,3	2,0	N/A	N/A	N/A
f) 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
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
h) 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
i) 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
j) 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

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2,5

-

3,0

5,0

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

13.2 Basis of Design

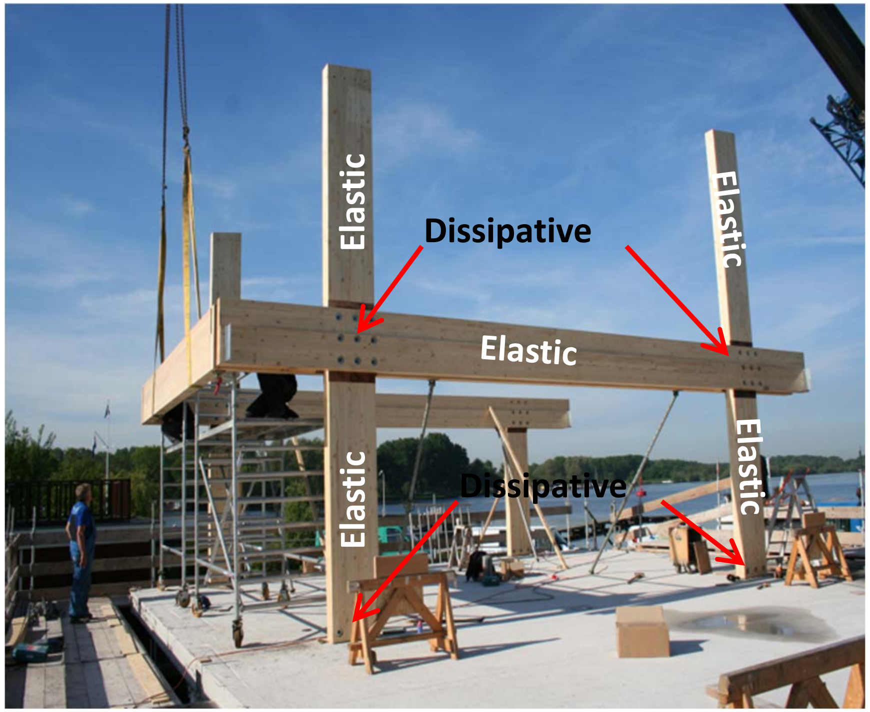
13.2.1 Design concepts

(6) In buildings designed in DC2 or DC3**, dissipative zones should be located in 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.

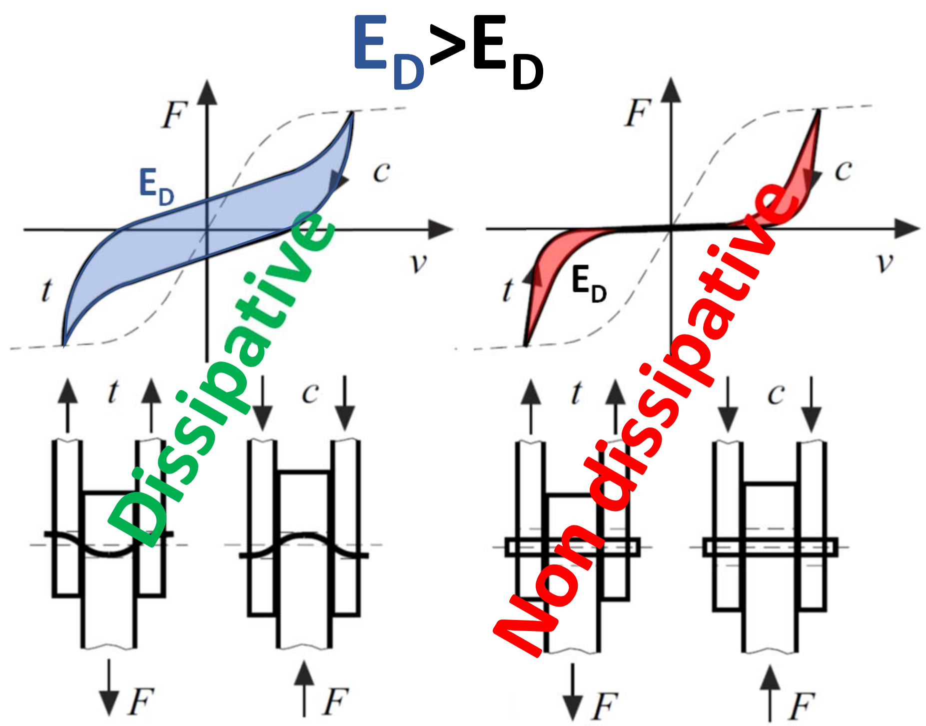


E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(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.

(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**.

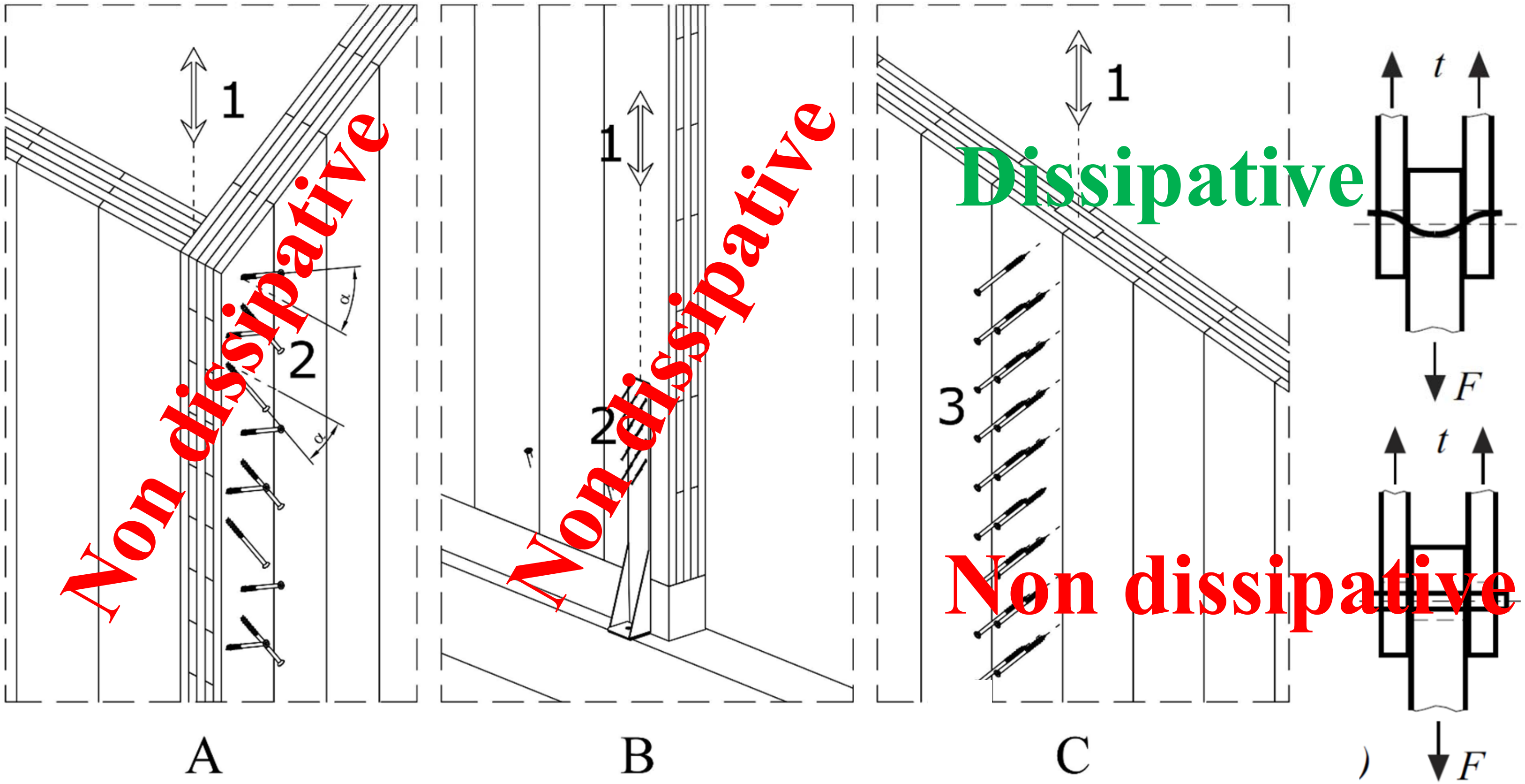
NOTE Axially-loaded fasteners and bonded-in rods cannot dissipate energy



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

13.2 Basis of Design

13.2.1 Design concepts

(10) If dissipative zones are purposely developed in energy dissipation systems, both the timber members and the connections should be regarded as behaving elastically and should be capacity designed according to EN 1998-1-1:2024, 4.5.2(2), in relation to the resistance of the energy dissipation systems. They should satisfy EN 1998-1-1:2024, 6.8, and prEN 1998-1-2:2022, 9 and Annex D, but not 13.4.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

13.4.2 Behaviour factors

(1)For timber buildings designed to DC2 or DC3 which are regular in elevation in accordance with 4.4.4.2, the default values of the behaviour factor q should be taken from Table 13.2 provided that either a) or b) or c) is satisfied:

a) the structural type of the building is one in the list in (2), while (3) and (4) are also satisfied;

b) the dissipative connections satisfy (5), while (4) is also satisfied;

c) cyclic tests satisfying (6) are performed on dissipative connections or subassemblies.

(2) For the structural types and ductility classes in a) and b) and c) and d) below, the default values of the behaviour factor q from Table 13.2 may be used:
a) in DC2, for all structural types in Table 13.2;
b) in DC3, for CLT systems with segmented walls according to 13.7.3;
c) in DC3, for framed wall systems according to 13.8.3 with dissipative sheathing-to-framing nailed connections;
d) in DC3, for moment-resisting frames according to 13.10.3, where a ductile mechanism with at least two flexural plastic hinges is formed in the dowel-type connection of the dissipative zones.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(3) Failure modes (a), (b) and (c) for dowel-type connection in single shear as given in prEN 1995-1-1:2023, 11.2.3.2(1), and failure modes (a) and (b) for dowel-type connection in double shear as given in prEN 1995-1-1:2023, 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 prEN 1995-1-1:2023, 11.2.3.5, should be avoided in all dissipative zones by satisfying Formula (13.5).

$$\gamma_{Rd,d} F_{\mathrm{v,Rk,d}} \leq F_{\mathrm{v,Rk,nd}} \qquad (13.5)$$

where:

$F_{\mathrm{v,Rk,d}}$ is the characteristic strength of the selected ductile failure mode providing energy dissipation, according to EN 1995-1-1:2021, **11.3.2**;

$F_{\mathrm{v,Rk,nd}}$ is the characteristic strength of the less ductile failure mode, according to EN 1995-1-1:2021, **11.3.2**;

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

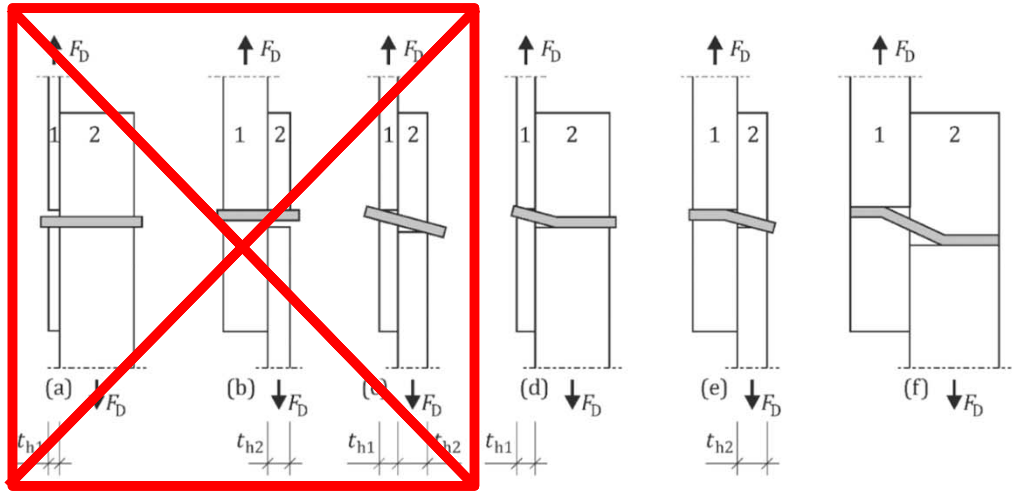


Figure 11.5 — Possible failure modes a) – f) for a fastener loaded in single shear

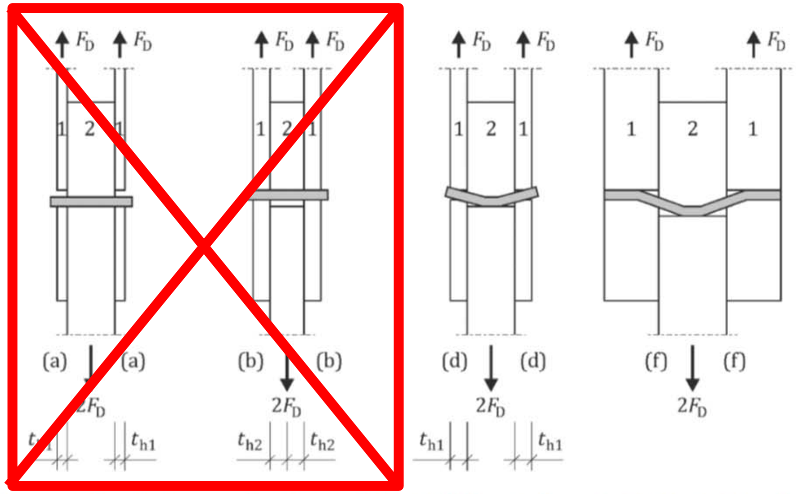


Figure 11.6 — Failure modes (a), (b), (d) and (f) for a fastener loaded in double shear

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From the revised version of prEN1995-1-1 (currently under discussion in CEN/TC250/SC5):

11.2.3 Lateral resistance of a fastener per shear plane

11.2.3.1 General

(1) The design lateral resistance per shear plane $F_{v,d}$ of a single fastener should be taken as follows:

$$F_{v,d} = F_{D,d} + F_{rp,d} \tag{11.13}$$

where

- $F_{D,d}$ is the design dowel-effect contribution per shear plane according to <v2>11.2.3.2;
- $F_{rp,d}$ is the design rope-effect contribution determined according to <v2>11.2.3.8.

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

11.2.3.2 General method for dowel-effect contribution of a single fastener per shear plane

(1) For a connection with a fastener loaded in single shear, the six failure modes (a) to (f) shown in <v2>Figure 11.6 shall be considered when determining the characteristic dowel-effect contribution. For definition of the embedment depth t_h, see <v2>Figure 11.1.

NOTE Timber members and metal plates are both represented by a specific characteristic embedment strength.

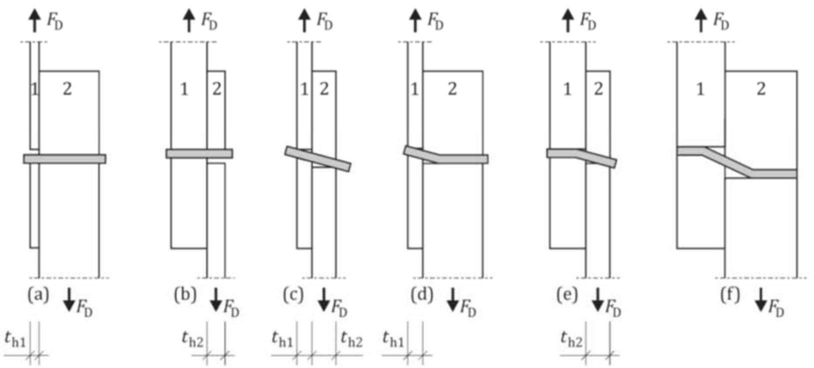


Figure 11.6 — Possible failure modes a) – f) for a fastener loaded in single shear

$$F_{D,k} = \min \begin{cases} f_{h,1,k}\, t_{h1}\, d & \text{mode (a)} \\ f_{h,2,k}\, t_{h2}\, d & \text{mode (b)} \\ \frac{f_{h,1,k}\, t_{h1}\, d}{1+\beta}\left[\sqrt{\beta + 2\beta^2\left[1+\frac{t_{h2}}{t_{h1}}+\left(\frac{t_{h2}}{t_{h1}}\right)^2\right]+\beta^3\left(\frac{t_{h2}}{t_{h1}}\right)^2} - \beta\left(1+\frac{t_{h2}}{t_{h1}}\right)\right] & \text{mode (c)} \\ 1{,}05\,\frac{f_{h,1,k}\, t_{h1}\, d}{2+\beta}\left[\sqrt{2\beta(1+\beta)+\frac{4\beta(2+\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h1}^2}} - \beta\right] & \text{mode (d)} \\ 1{,}05\,\frac{f_{h,1,k}\, t_{h2}\, d}{1+2\beta}\left[\sqrt{2\beta^2(1+\beta)+\frac{4\beta(1+2\beta)M_{y,k}}{f_{h,1,k}\, d\, t_{h2}^2}} - \beta\right] & \text{mode (e)} \\ 1{,}15\sqrt{\frac{2\beta}{1+\beta}}\sqrt{2\, M_{y,k}\, f_{h,1,k}\, d} & \text{mode (f)} \end{cases} \tag{11.14}$$

with

$$\beta = \frac{f_{h,2,k}}{f_{h,1,k}} \tag{11.15}$$

where

- $f_{h,1,k} f_{h,2,k}$ are the characteristic embedment strengths of members 1 and 2, given in <v2>Table 11.6;
- t_{h1}, t_{h2} are the embedment depths of members 1 and 2, see <v2>Figures 11.5 and 11.6;
- $M_{y,k}$ is the characteristic yield moment given in <v2>Table 11.7;
- d is the diameter of the fastener.

11.2.3.8 Rope-effect contribution

(1) The rope-effect may be taken into account if gaps are avoided between adjacent members and axial forces occur in the deformed fastener.

(2) For failure modes (a) and (b) in <v2>Figure 11.6 and <v2>Figure 11.7, no rope-effect contribution shall be accounted for.

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

$$F_{rp,d} = \min \begin{cases} k_{rp,1}\, F_{ax,t,d} \\ k_{rp,2} F_{D,d} \end{cases} \tag{11.19}$$

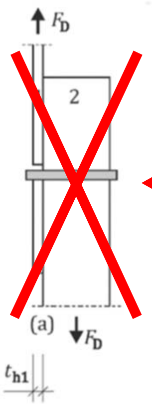
where

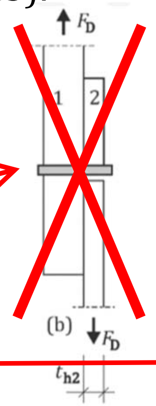
- $k_{rp,1}$ is the factor for the rope-effect, see <v2>Table 11.8;
- $F_{ax,t,d}$ is the design tensile resistance, see <v2>11.2.2.1;
- $k_{rp,2}$ is the limitation factor for the rope-effect, see <v2>Table 11.9;
- $F_{D,k}$ is the characteristic dowel-effect contribution determined with <v2>11.2.3.2.

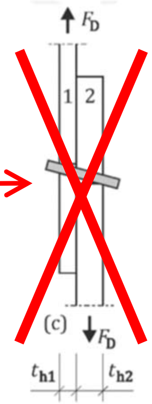
E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

From the revised version of prEN1995-1-1 (currently under discussion in CEN/TC250/SC5):

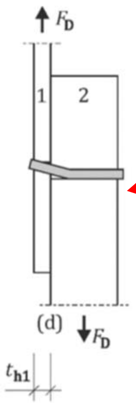
$$1,2 \cdot \min\left(F_{v,k,(d)};F_{v,k,(e)};F_{v,k,(f)}\right) \leq \min\left(F_{v,k,(a)};F_{v,k,(b)};F_{v,k,(c)}\right)$$



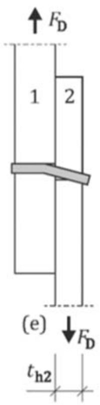




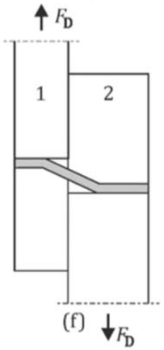
DC2



DC2



DC3



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

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.

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

(4) Brittle failure modes like splitting, row shear, block shear, plug shear, and net tensile failure of wood in the connection regions, as defined in prEN 1995-1-1 (under development), 11.6 (under development), 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 prEN 1995-1-1:2023, 11.8.

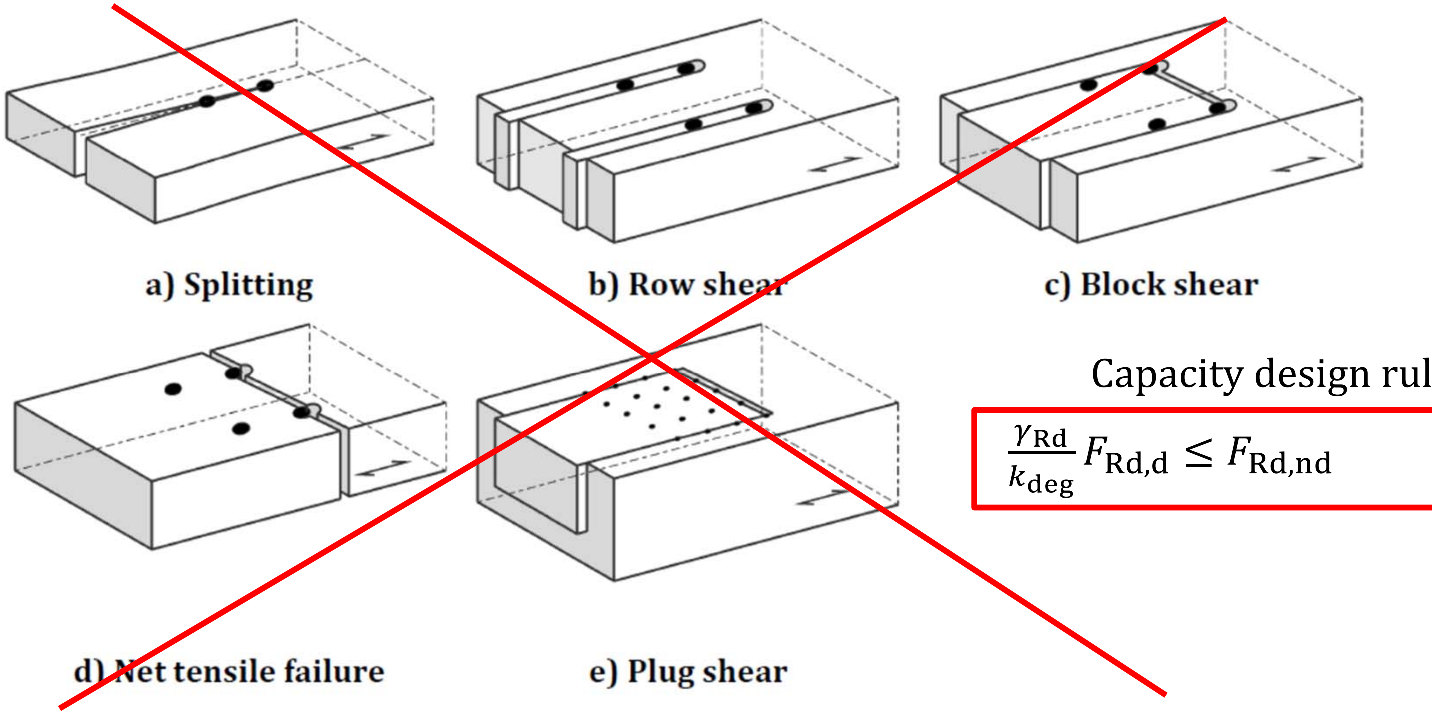


Figure 11.22 — Examples of brittle failure modes

Capacity design rule:

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

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

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Le zone dissipative devono essere localizzate, in accordo al meccanismo di collasso duttile globale prescelto, in alcuni dei collegamenti o in elementi specificatamente progettati; le membrature lignee devono essere considerate a comportamento elastico, salvo che non siano adottati per gli elementi strutturali provvedimenti tali da soddisfare i requisiti di duttilità di cui al § 7.7.3.

Le disposizioni di cui al precedente capoverso possono considerarsi soddisfatte nelle zone dissipative di ogni tipologia strutturale se si rispettano le seguenti prescrizioni:

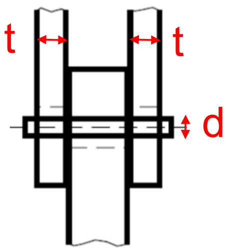
a) i collegamenti legno-legno o legno-acciaio sono realizzati con perni o con chiodi presentanti diametro *d* non maggiore di 12 mm ed uno spessore delle membrature lignee collegate non minore di 10*d*;

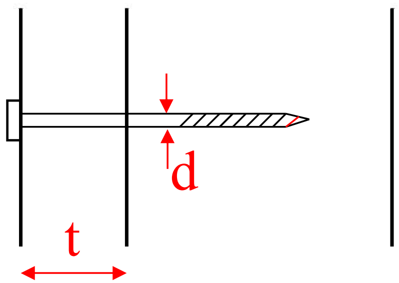
b) nelle pareti e nei diaframmi con telaio in legno, il diametro *d* dei chiodi non è superiore a 3,1 mm e il materiale di rivestimento strutturale è di legno o di materiale da esso derivato, con uno spessore minimo pari a 4*d*.

Qualora alcune o tutte le precedenti prescrizioni non siano rispettate, ma sia almeno assicurato lo spessore minimo degli elementi collegati pari, rispettivamente, a 8*d* per il caso a) e a 3*d* per il caso b), le zone dissipative saranno da considerare in classe di duttilità CD "B".

CD "B"
$t \geq 8d$
$d \leq 12$ mm

CD "A"
$t \geq 10d$
$d \leq 12$ mm





CD "A"
$t \geq 4d$
$d \leq 3{,}1$ mm

CD "B"
$t \geq 3d$
$d \leq 3{,}1$ mm

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

NTC 2018:

Queste limitazioni sullo spessore del pannello portano alla formazione di una o piu' cerniere plastiche nel connettore:

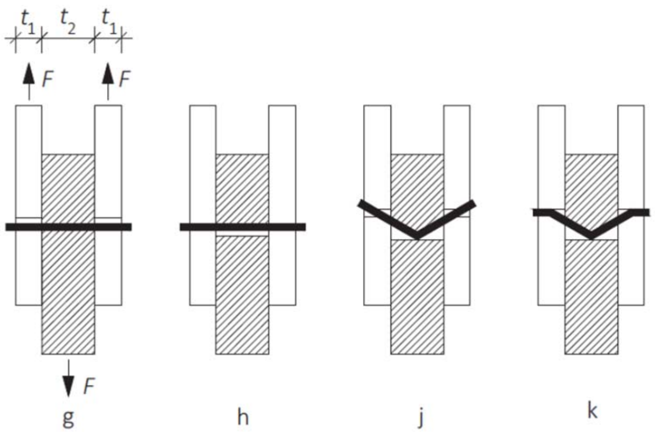


Figure E1-10 Possible failure modes of a timber-to-timber joint with two shear planes.

Failure mode g: Embedment strength of the side members is reached:

$$F_{v,Rk} = f_{h,1,k} \cdot t_1 \cdot d \qquad \text{(E1-1)}$$

Failure mode h: Embedment strength of the middle member is reached:

$$F_{v,Rk} = 0.5 \cdot f_{h,2,k} \cdot t_2 \cdot d \qquad \text{(E1-2)}$$

Failure mode j: Embedment strength in all members is reached and a plastic hinge is formed:

$$F_{v,Rk} = 1.05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left[\sqrt{2 \cdot \beta \cdot (1+\beta) + \frac{4 \cdot \beta \cdot (2+\beta) \cdot M_{y,Rk}}{f_{h,1,k} \cdot t_1^2 \cdot d}} - \beta\right] + \frac{F_{ax,Rk}}{4} \qquad \text{(E1-3)}$$

Failure mode k: Embedment strength in all members is reached and three plastic hinges are formed:

$$F_{v,Rk} = 1.15 \cdot \sqrt{\frac{2 \cdot \beta}{1+\beta}} \cdot \sqrt{2 \cdot M_{y,Rk} \cdot f_{h,1,k} \cdot d} + \frac{F_{ax,Rk}}{4} \qquad \text{(E1-4)}$$

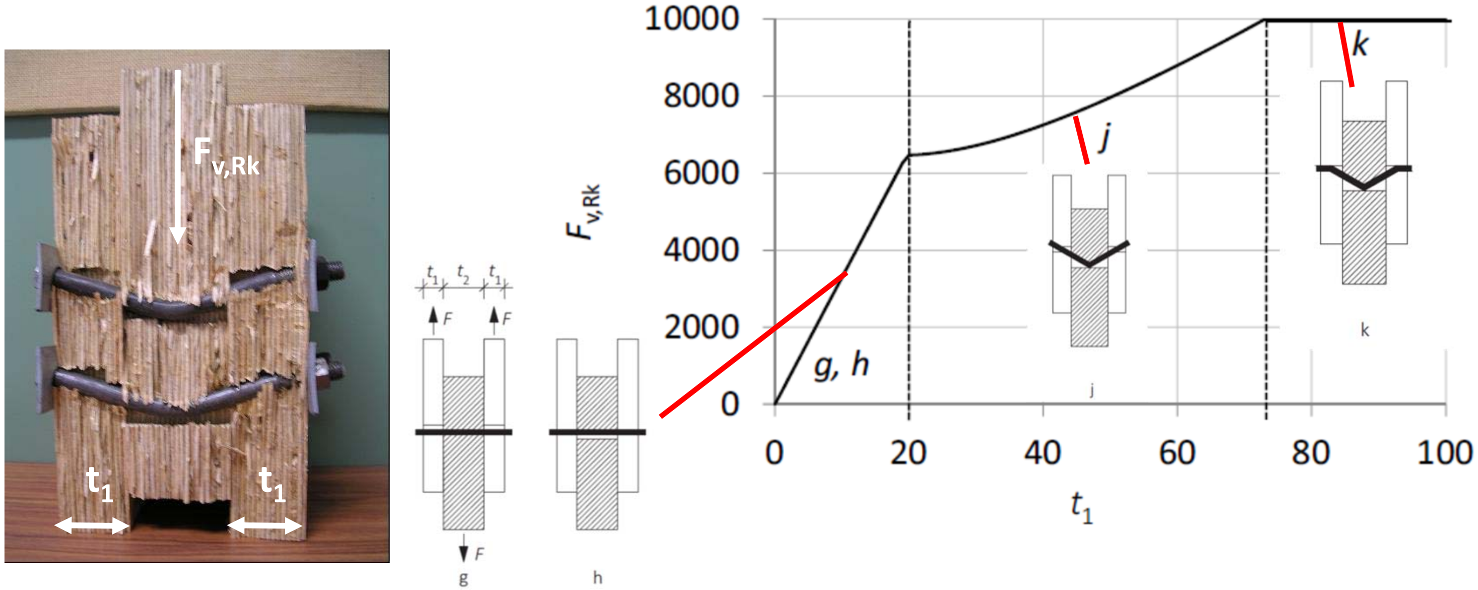
where

- $F_{v,Rk}$ Characteristic capacity per fastener and shear plane
- t_i Member thickness, i = 1 or 2
- $f_{h,i,k}$ Characteristic embedment strength in timber member i
- d Fastener diameter
- $M_{y,Rk}$ Characteristic yield moment of fastener
- $F_{ax,Rk}$ Characteristic withdrawal capacity of fastener
- β Ratio of embedment strength values, $\beta = f_{h,2,k}/f_{h,1,k}$

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

NTC 2018:

Diagrammando la resistenza caratteristica a taglio del connettore in funzione dello spessore del pannello utilizzando le formule di Johanssen dell'EC5, si ottiene il seguente diagramma:



All'aumentare dello spessore t_1 del pannello si passa dai modi di rottura g,h con solo rifollamento del legno (non dissipativo) a quello j con una cerniera plastica (dissipativo – CD "B") e k con tre ceniere plastiche nel connettore metallico (molto dissipativo – CD "A").

E – NEW DUCTILITY RULES FOR DISSIPATIVE ZONES

NTC 2018:

In alternativa alle prescrizioni di cui sopra, per le zone dissipative di classe CD “B”, i collegamenti meccanici a gambo cilindrico possono essere progettati per garantire lo sviluppo di almeno una cerniera plastica nel gambo dei connettori metallici in accordo ai meccanismi di collasso riportati nelle normative e documenti tecnici di comprovata validità di cui al Capitolo 12. Particolare attenzione dovrà essere rivolta a impedire rotture fragili tipo fessure da spacco longitudinale, espulsione di tasselli di legno, rotture a taglio e a trazione del materiale base.

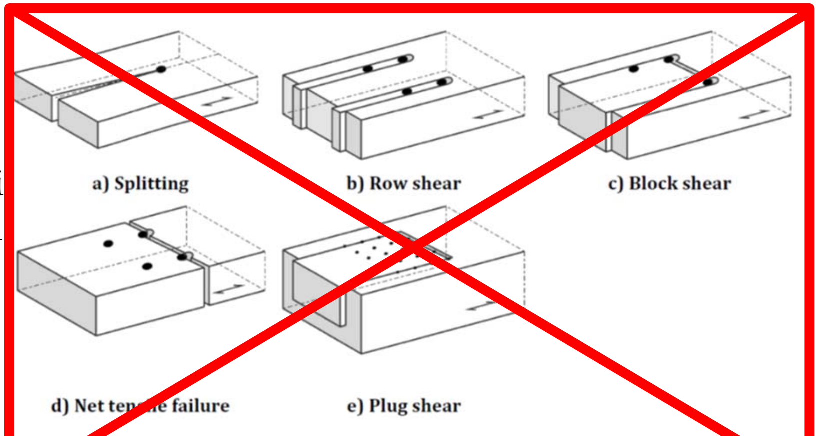


Figure 11.22 — Examples of brittle failure modes

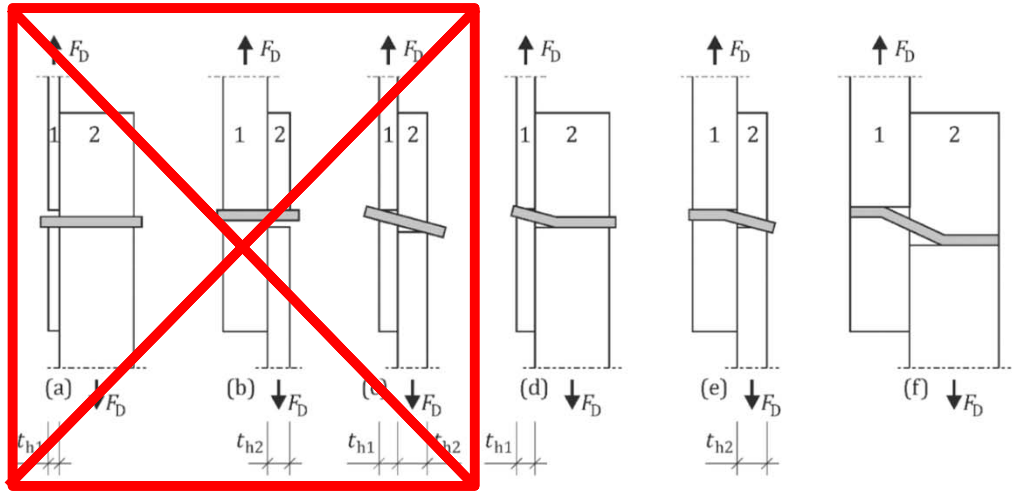


Figure 11.5 — Possible failure modes a) – f) for a fastener loaded in single shear

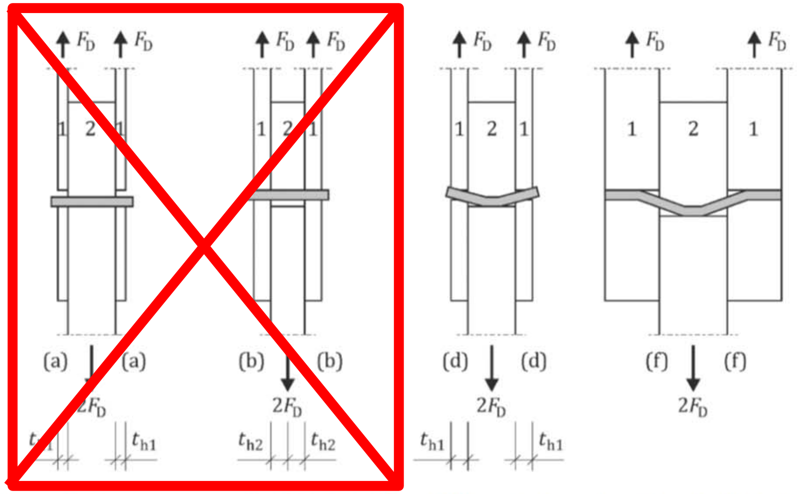


Figure 11.6 — Failure modes (a), (b), (d) and (f) for a fastener loaded in double shear

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

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 $F_{\mathrm{Rd,nd}}$ of the non-dissipative components should satisfy Formula (13.4).

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

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);** **(0,8)**

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

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

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

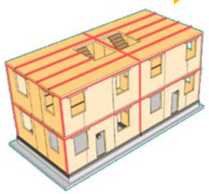
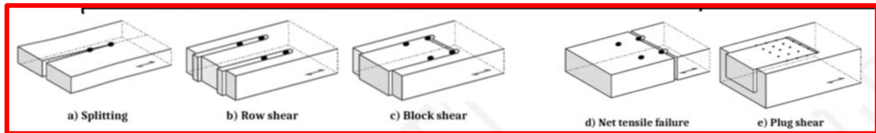


Table 13.4 — Values of the overstrength factors γ_{Rd} to be used in capacity design



		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 ~~dowel-type~~ 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.

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

NTC 2018:

Ai fini dell'applicazione dei criteri della progettazione in capacità, per assicurare la plasticizzazione delle zone dissipative (i collegamenti prescelti e/o gli elementi specificatamente progettati), queste devono possedere una capacità almeno pari alla domanda mentre le componenti non dissipative (gli altri collegamenti e gli elementi strutturali) adiacenti, debbono possedere una capacità pari alla capacità della zona dissipativa amplificata del fattore di sovraresistenza γ_{Rd}, di cui alla Tab. 7.2.I; valori inferiori del fattore di sovraresistenza ed in ogni caso maggiori o uguali a 1,3 per CD "A" e a 1,1 per CD "B" devono essere giustificati sulla base di idonee evidenze teorico-sperimentali.

Tab. 7.2.I - *Fattori di sovraresistenza γ_{Rd} (fra parentesi quadre è indicato il numero dell'equazione corrispondente)*

Tipologia strutturale	Elementi strutturali	Progettazione in capacità	γ_{Rd}	
			CD"A"	CD"B"
Legno	Collegamenti		1,60	1,30

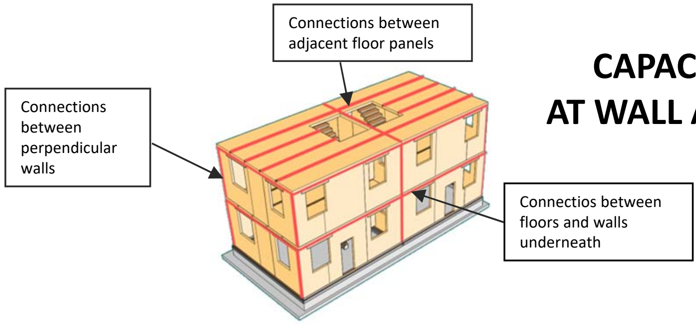
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EACH STRUCTURAL TYPE

GENERAL RULES

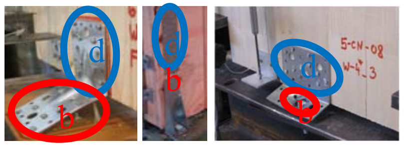
General description of the structural components (walls, floors) and type of connections generally used

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



F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

13.7 Rules for cross laminated timber (CLT) structures
13.7.1 General rules

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

a) They should be **made by means of 2D- or 3D-connectors** (e.g. hold-downs, foundation tiedowns, angle brackets, shear plates) **and metal fasteners** (e.g. anchoring bolts, nails and screws, etc.). These connectors should satisfy EN1995-1-1:2024, Annex I.

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 also comply with prEN 1998-1-1:2022, Annex G.

Hold-down

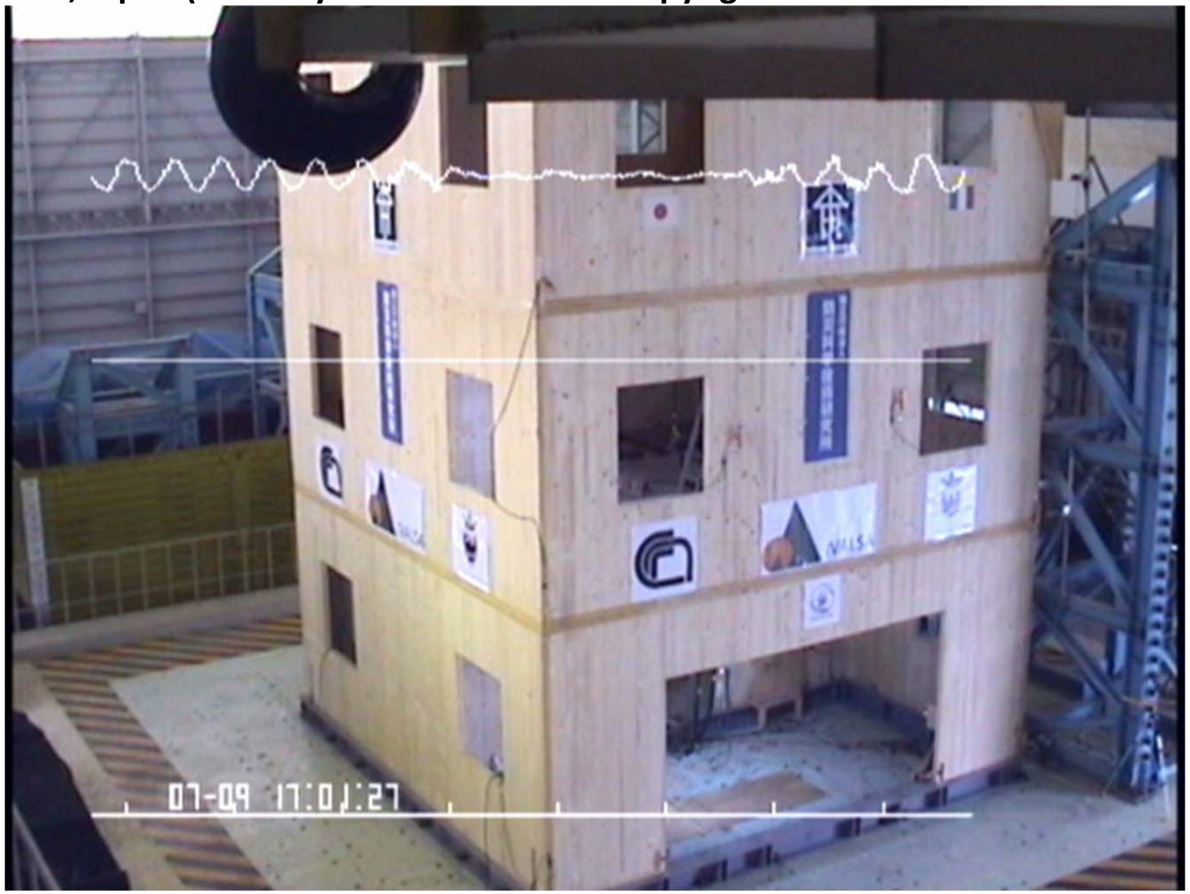
Angle bracket

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

What are the dissipative zones in a Xlam buildings?

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

Dissipative mechanism: rocking of wall panels

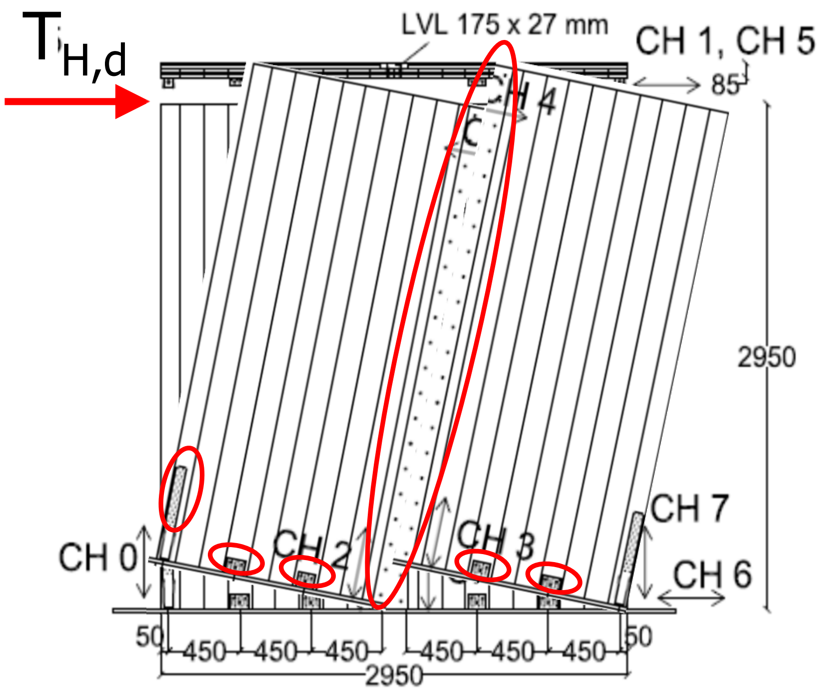


F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

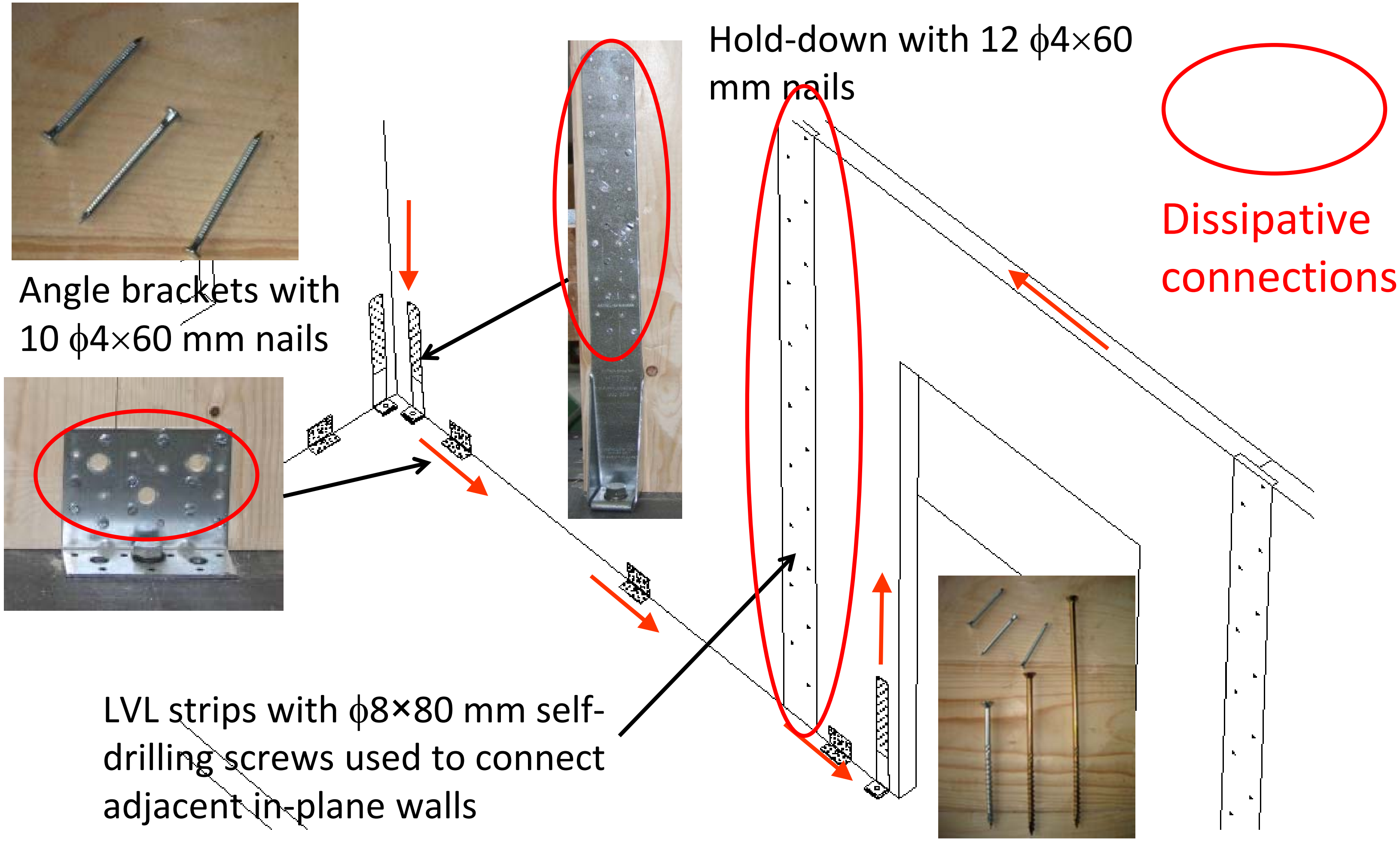
Dissipative mechanism: rocking of wall panels

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

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



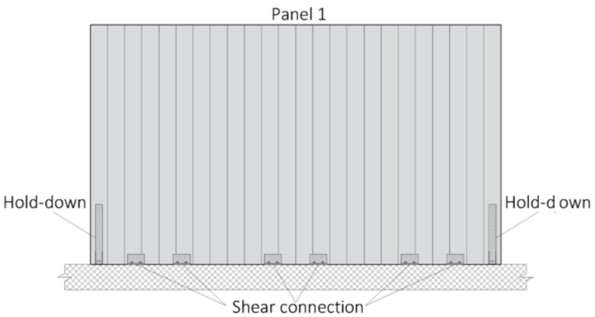
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS



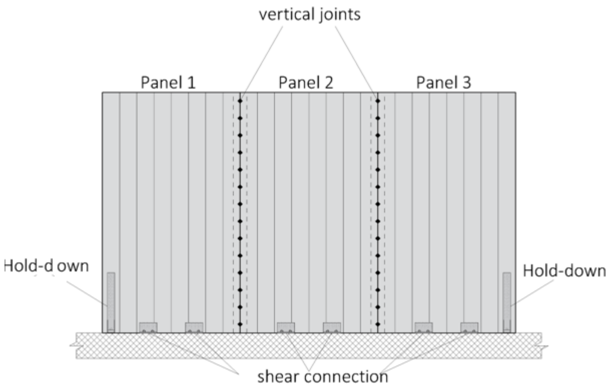
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

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

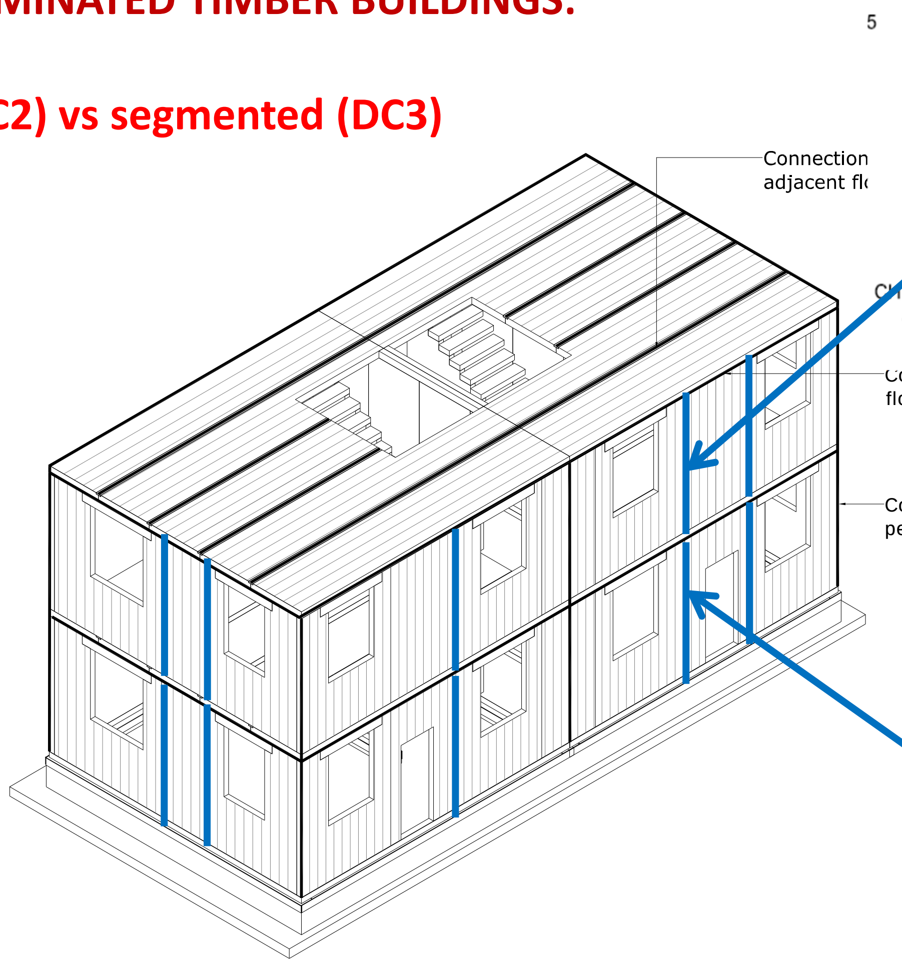


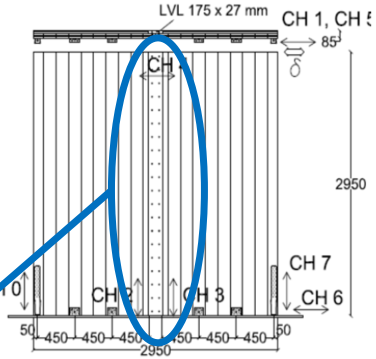
a) Single panel (Monolithic)



b) Multi-panel

Dissipative zones (DC3):



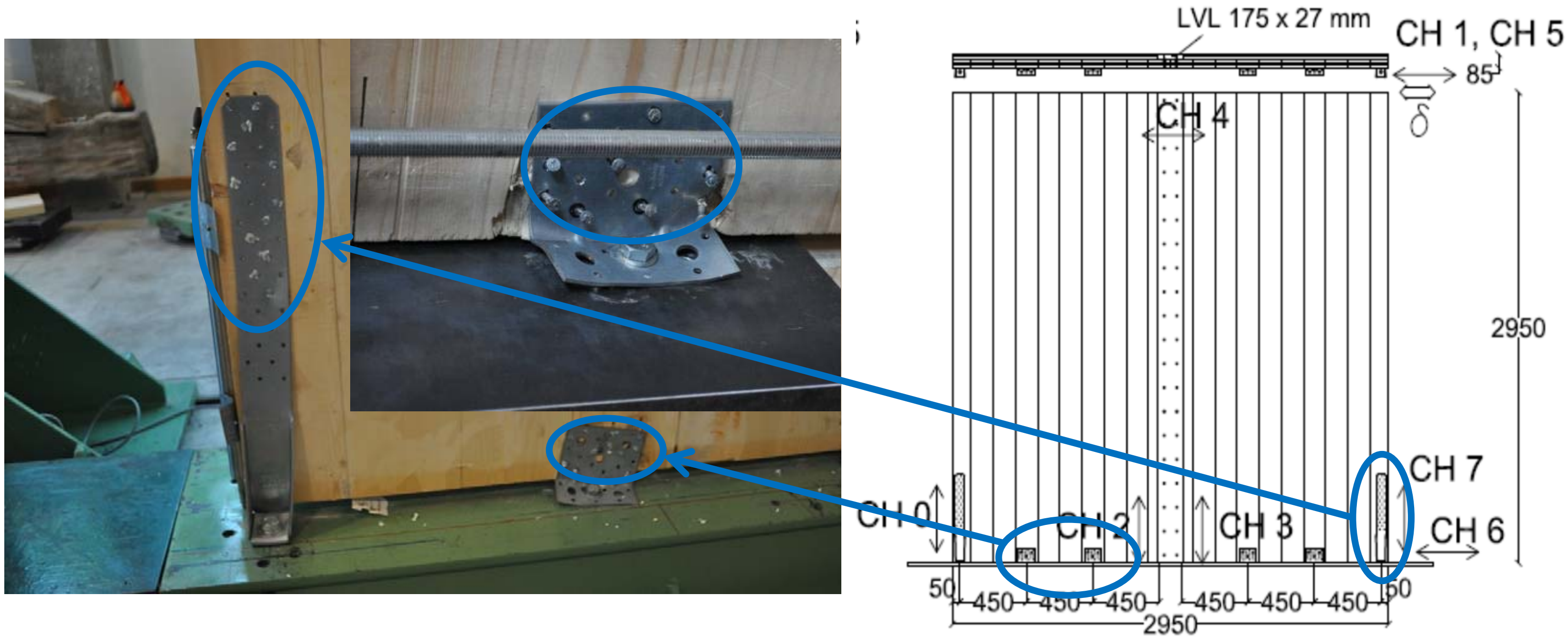


F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

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.**



F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

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;

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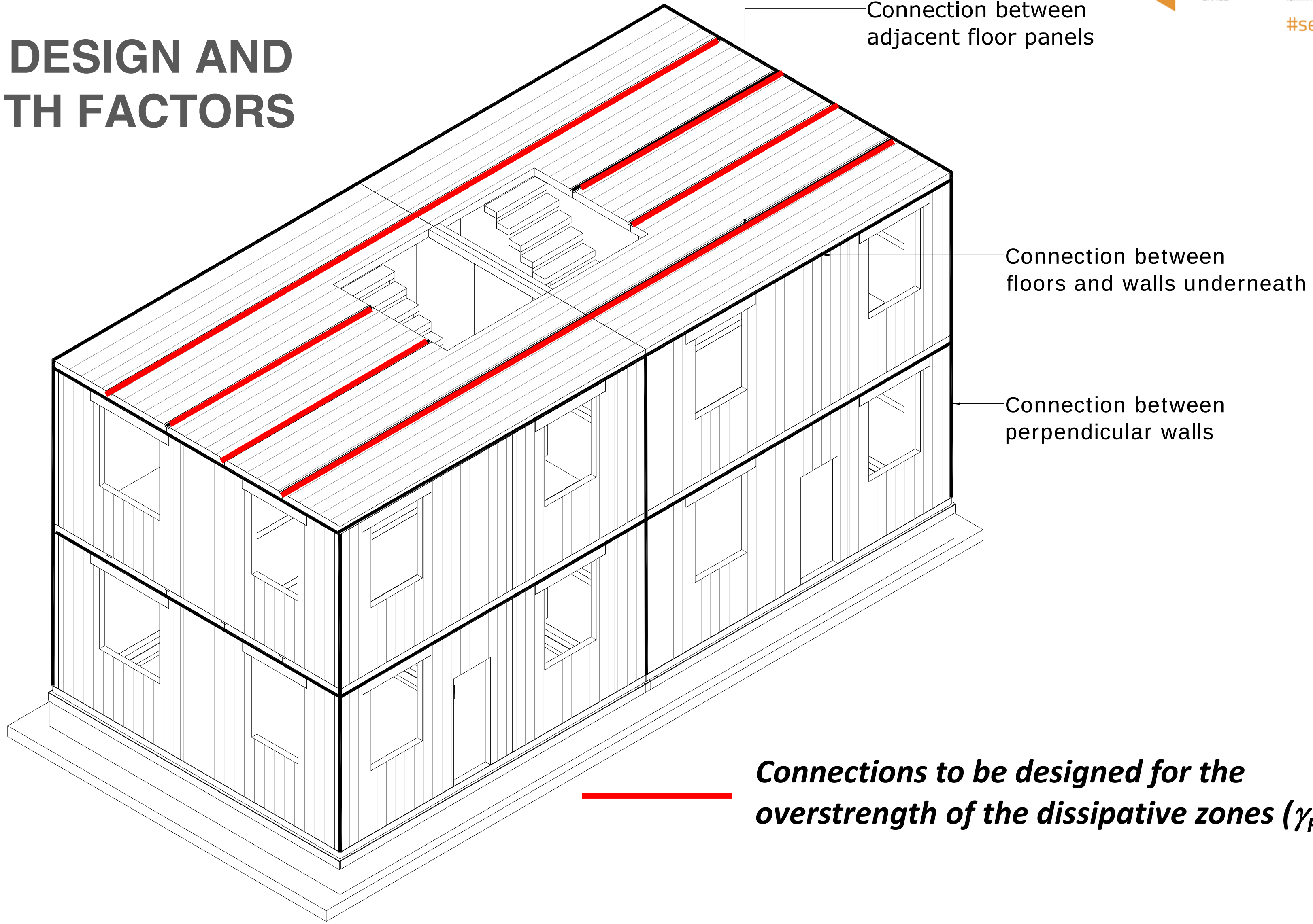
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS



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

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

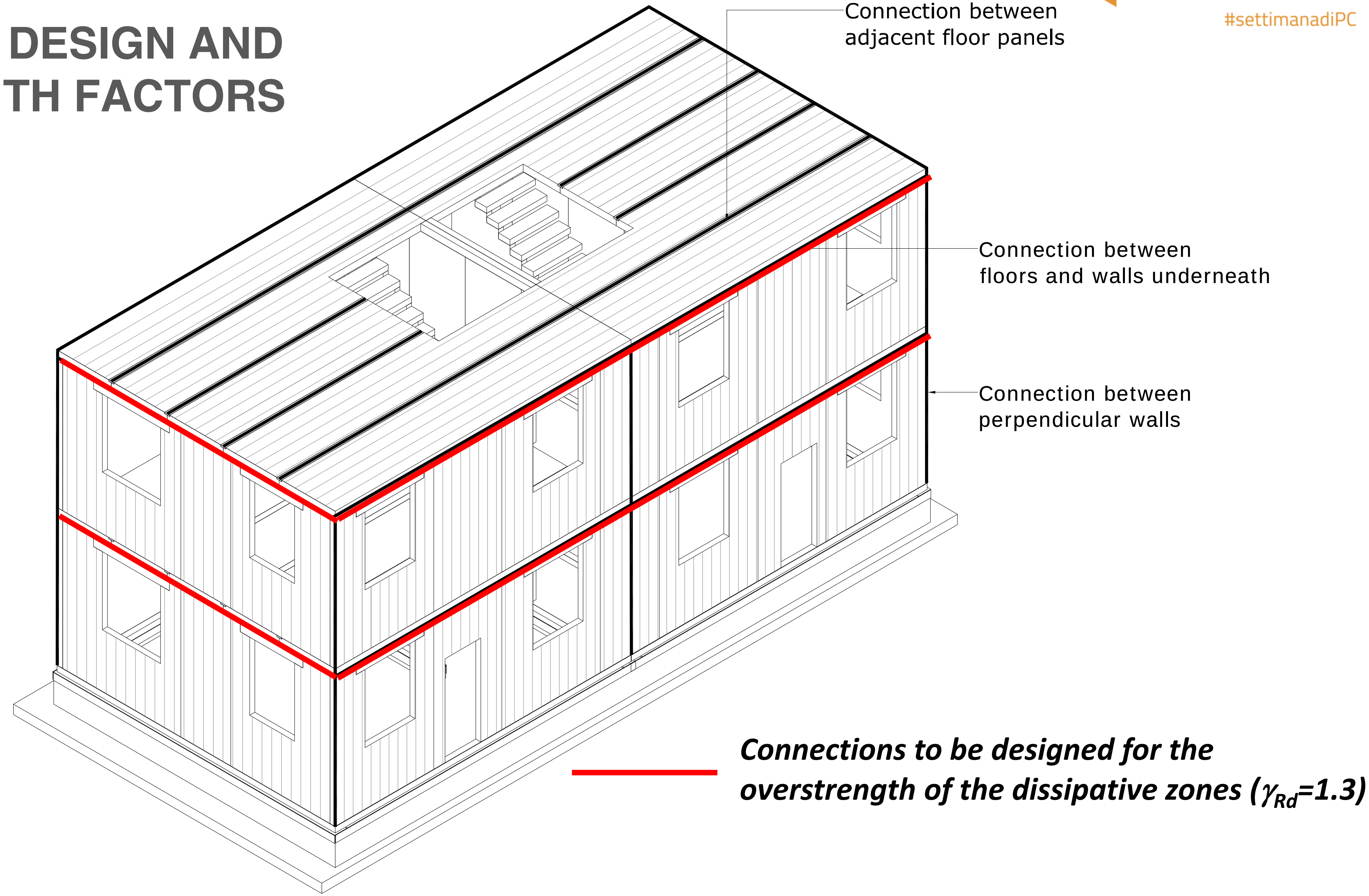
FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

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;

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS



F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

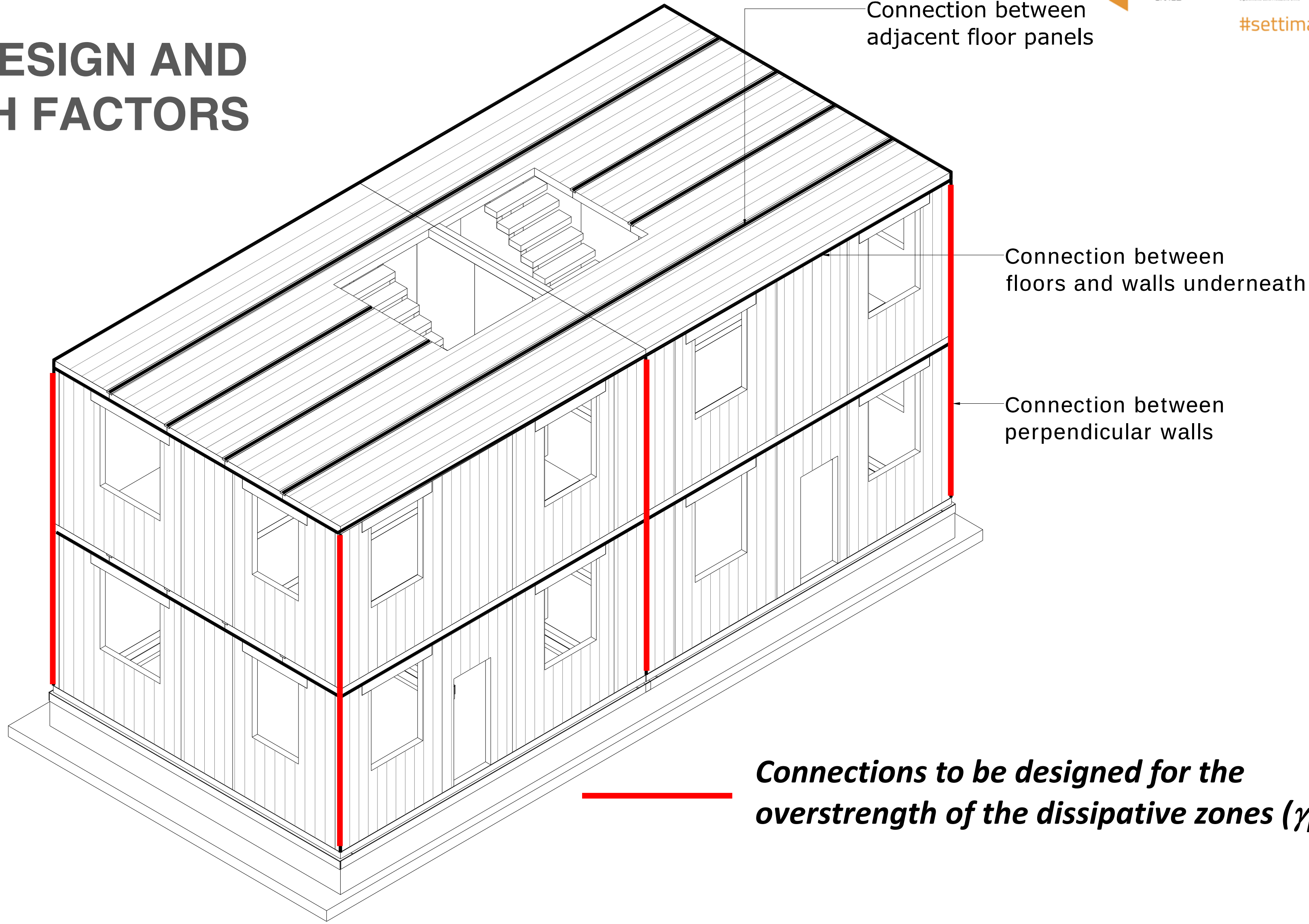
FOR EXAMPLE, FOR CROSS-LAMINATED TIMBER BUILDINGS:

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;

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS



Connections to be designed for the overstrength of the dissipative zones (γ_{Rd}=1.6)

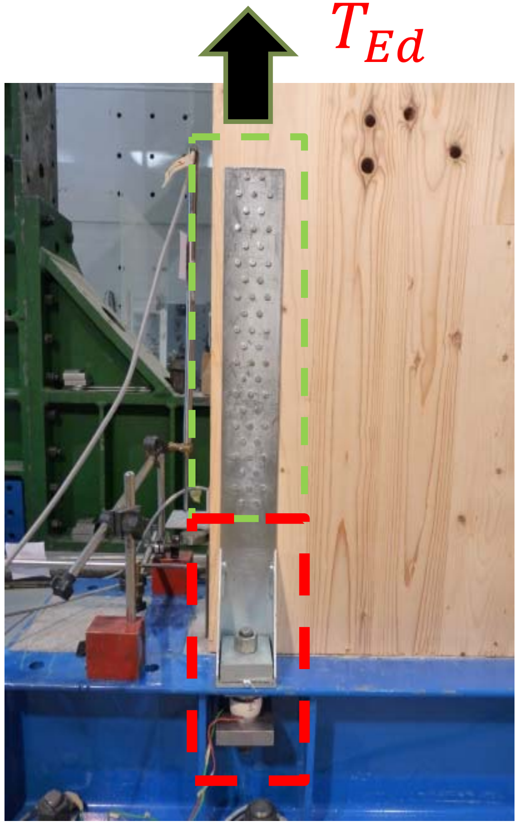
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

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).

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

Capacity based design for CLT structures: connection level



T_{Ed}

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

F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

Capacity based design for CLT structures: building level

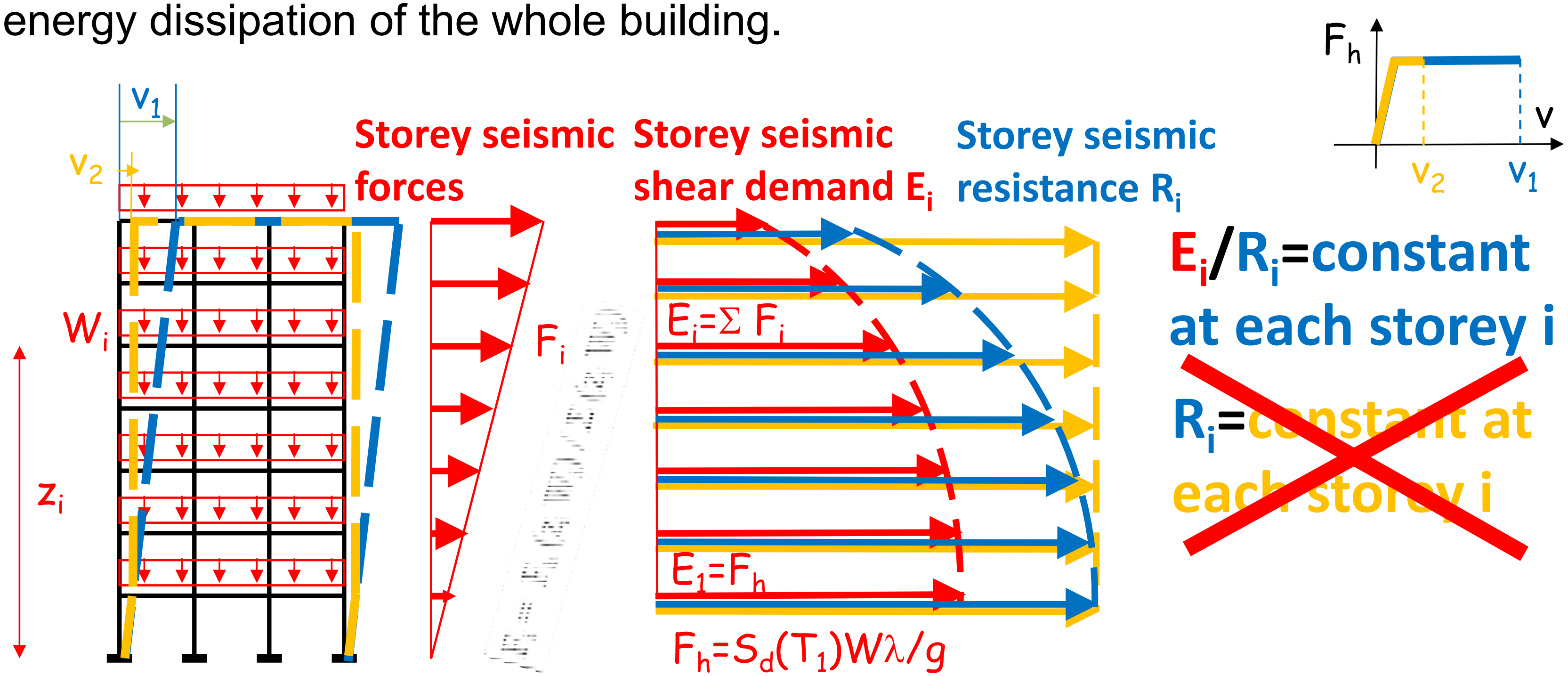
(9) The maximum storey overstrength ratio max($\Omega_{d,i}$) and the minimum storey overstrength ratio Ω_d, with Ω_d given by Formula (13.8), should ~~verify~~satisfy Formula (13.14).

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

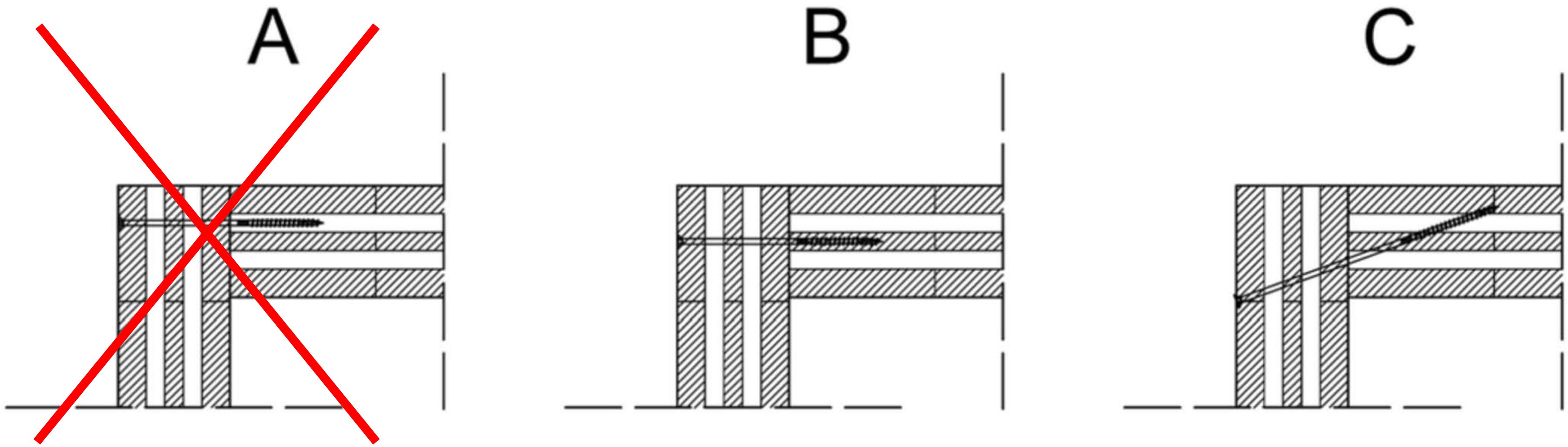
F – CAPACITY DESIGN AND OVERSTRENGTH FACTORS

Capacity based design for CLT structures: building level

The seismic resistance of Xlam 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.



G – DETAILING RULES FOR ALL STRUCTURAL TYPES - CLT

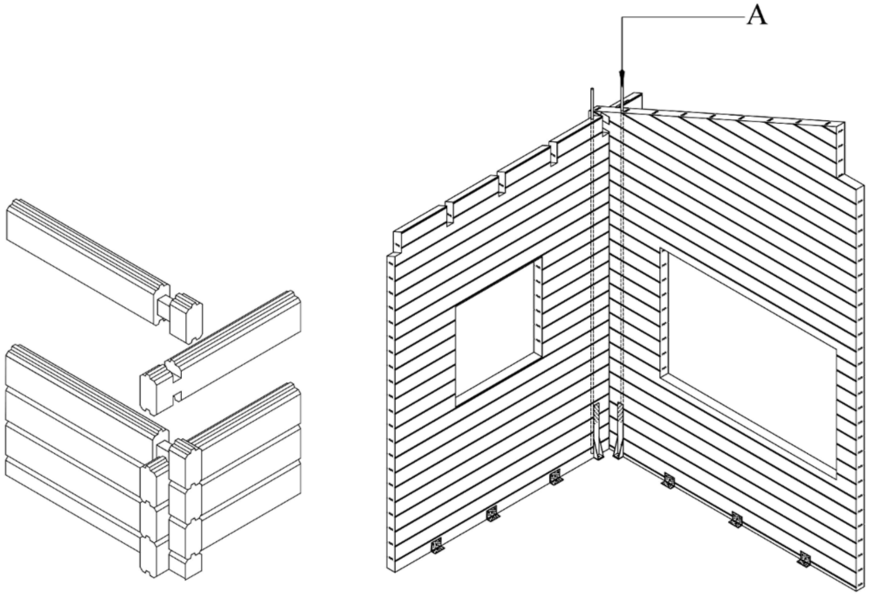


Key

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

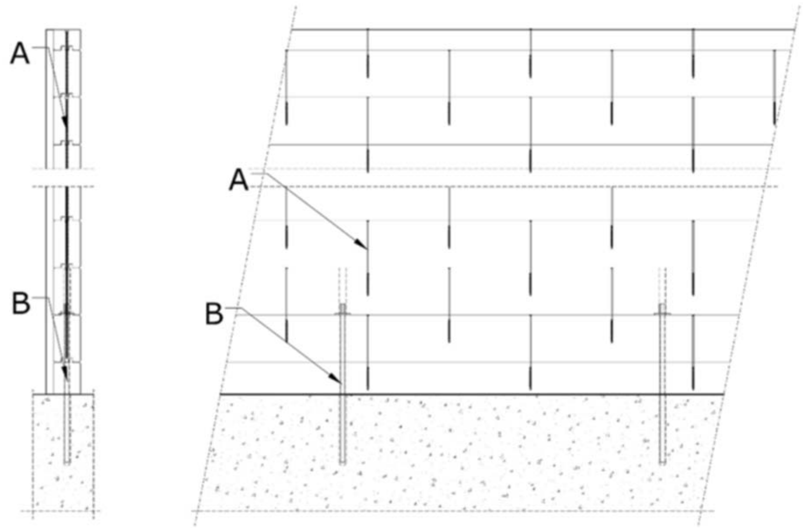
G – DETAILING RULES FOR ALL STRUCTURAL TYPES – LOG STRUCTURES



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

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SCUOLA DI INGEGNERIA STRUTTURALE – RELUIS

Bologna, 9-11 ottobre 2024

COMPORTAMENTO SISMICO DELLE STRUTTURE DI LEGNO ALLA LUCE DELLO SVILUPPO DELLA NUOVA EN1998-1-2

Prof. Ing. Massimo Fragiacomo, Università degli Studi dell'Aquila